Lack of Maintenance Compromises Tunnel Structural Safety

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ABSTRACT: *The Chekka tunnels represent a critical section of the Northern Lebanon Highway. The approximately 350-m long twin tunnels are the most important tunnels in the country. They are used by regular and freight traffic and are a vital link to the capital, Beirut. The tunnels were completed in the early 1970s on the eve of the Lebanon civil war. Given the state of affairs in the country for about two decades of strife, the tunnels were not maintained nor did they undergo any regular inspection. This unique condition of near-abandonment continued up until recently, when the first signs of distress were documented. These consisted of severe and extensive cracking, particularly in the portal approach of the southbound tunnel, along with significant deterioration of the tunnel lining and radial section joints. The danger to traffic and, potentially, to the structure itself was evident. In this paper, the results of an extensive site inspection and evaluation program are presented. The underlying causes of the observed damage are analyzed and discussed using a modeling approach of the progressive loading conditions. Remedial measures are discussed along with a summary of the recently completed repair and rehabilitation works.*

KEYWORDS: Tunnels, maintenance, structural failure, finite element analysis

SITE LOCATION: IJGCH Geographic Database (requires Google Earth)

ACCOMPANYING DATA: Supplemental Photos & Digital Data

INTRODUCTION

The Chekka Twin Tunnels represent a critical section of the "Northern Lebanon Highway" and are the most important such structures in the country. They are used by regular and freight traffic and are a vital link to the capital, Beirut. They are an intrinsic part of the highway network linking Lebanon to Syria, its neighbor to the north and northeast, beyond which lies Turkey at the gates of eastern and western Europe. The coordinates of the tunnel portals are as follows: Southern portals: 34^{o} 17' 0.8" N 35^{o} 41' 58.32" E; Northern portals: 34^{o} 17' 7.92"N 35^{o} 42' 10.92" E.

The tunnels were completed in the early 1970s on the eve of the Lebanon civil war. They consist of two twin or parallel tunnel structures, each 14.3 m wide and 10 m high. The northbound tunnel is 367 m long, while the southbound tunnel is 385 m. The tunnels were advanced through the mountain side by drilling and blasting. The tunnels were lined using a slip-formed concrete lining with 0.1 m wide construction joints at approximately 10 m spacing.

The approach to the northbound tunnel is defined by an approximately 35-m long, concrete, open-skeletal frame extending in advance of the tunnel entrance. The southbound tunnel has no approach structure at the entrance. The southbound tunnel exit is defined by an approximately 35-m long reinforced concrete tunnel portal structure, followed by a 35-m open-skeletal concrete frame.

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PROBLEM DESCRIPTION: PRELIMINARY OBSERVATIONS

For a period of 25 years following the completion of the tunnels, no maintenance or repair works were implemented. As a result, the condition of the tunnels deteriorated significantly. The absence of preventive or corrective maintenance is mainly attributed to the two decades-long civil strife and wars which ravaged Lebanon. As the reconstruction effort started in the mid-nineties, other priorities took precedence over the road network and associated tunnels and bridges. This was true of the Chekka tunnel, until its structural safety was compromised and remedial action could no longer be deferred.

The main causes of alarm were significant signs of structural distress along the 35-m long reinforced concrete portal structure, at the exit of the southbound tunnel. The clearest and most critical among these was a major near-horizontal crack (crack width > 5 mm) running all along the exposed side of the structure (Fig. 1), along with a lateral offset of the portal wall on the order of 15 cm, with the greatest movement occurring at the crack elevation (Fig. 2). The distress signs had become severe enough that they could be clearly seen from the road by northbound drivers/passengers. It was only then, due to growing public fears and apprehension at using the tunnels, that the priorities were shifted to address the potential failure of the structures.

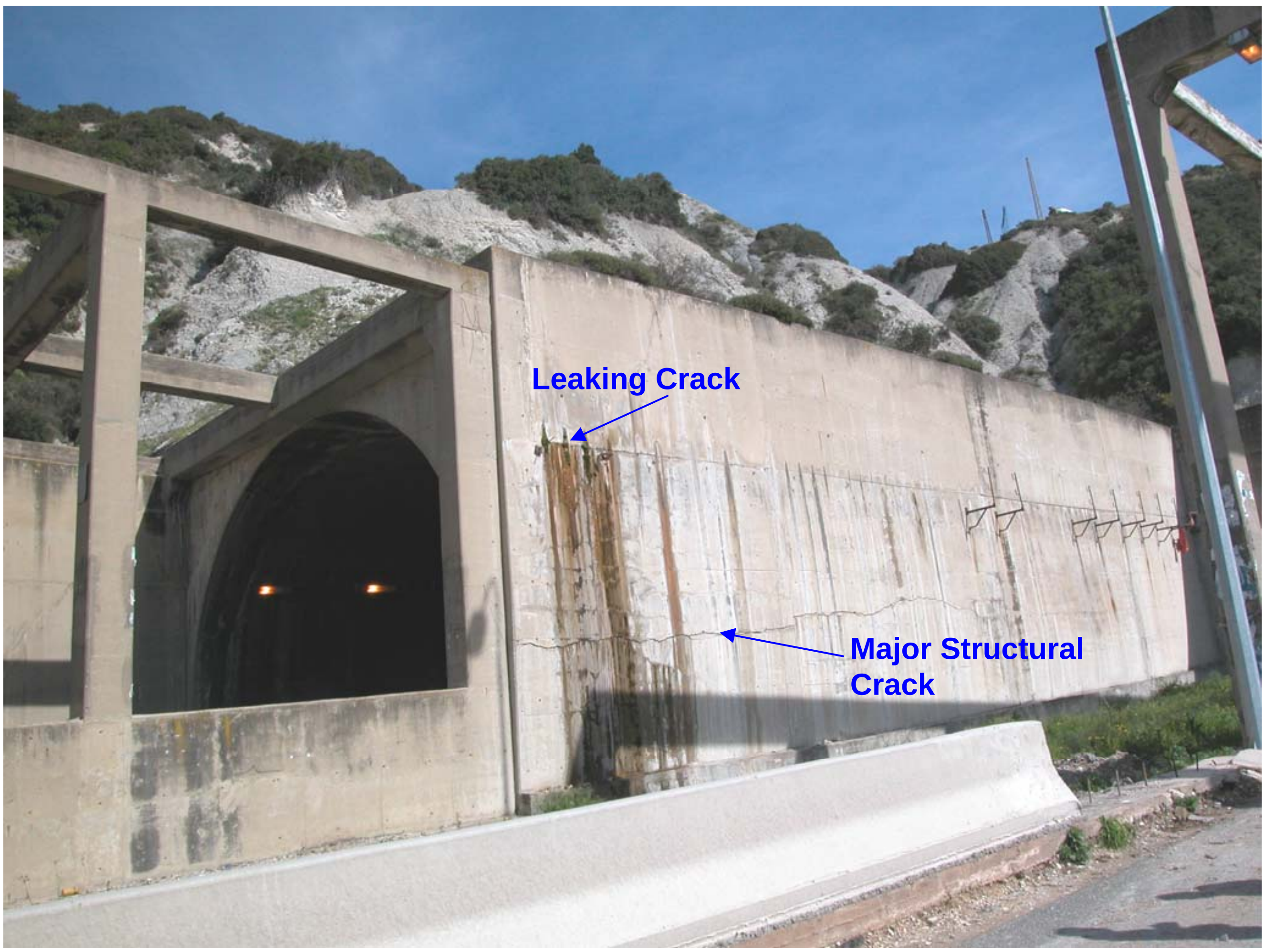

Figure 1. Southbound Tunnel Portal - View of External Distress.

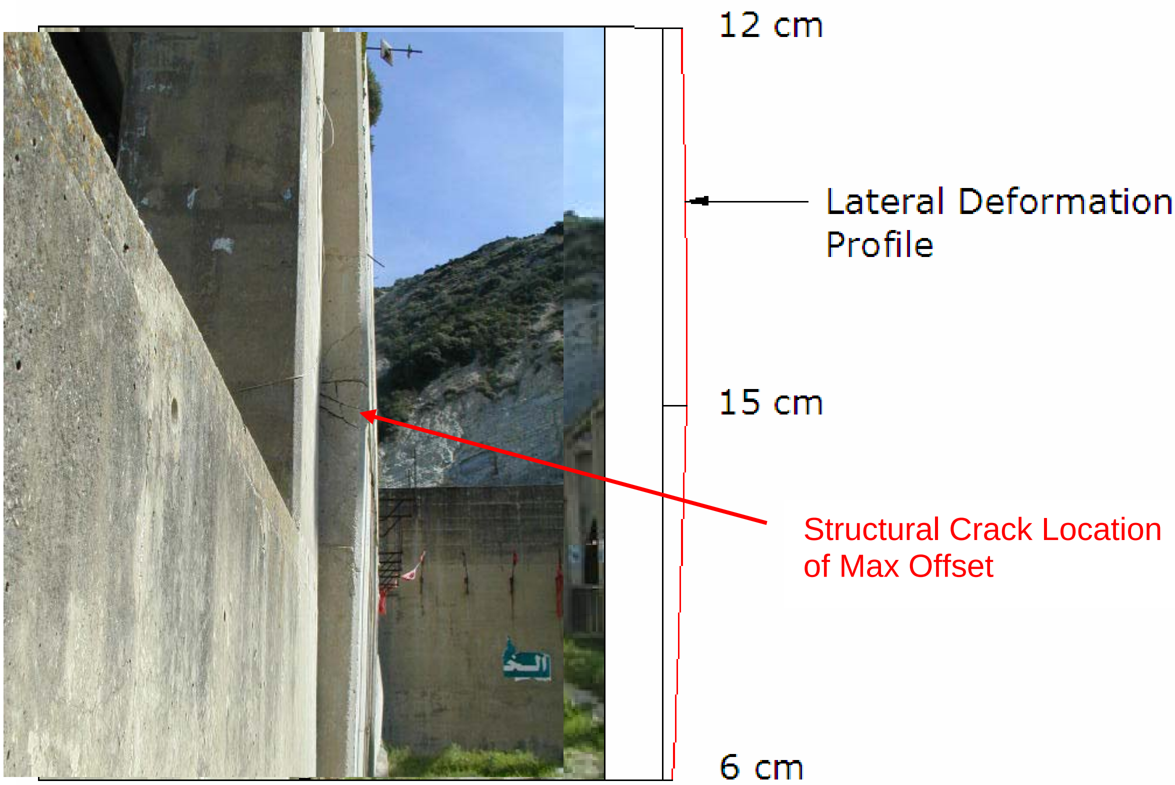

Figure 2. Close-up View of the Outward Movement/bulging of the Eastern Portal Wall.

The initial scope of the geotechnical-structural program was to assess/identify the problem(s) and to establish the need for, and nature of, any remediation measures to be implemented to mitigate any hazards to public safety. As a result of this initial phase of our involvement, different types of distress were identified along with their potential causes, and a further in-depth site investigation and assessment program was outlined. In essence, the affected tunnel structure suffered from two major types of distress which were "coupled" with respect to their effects on the tunnel structure:

1. Loading-related damage, which appears as significant deformations and/or major cracks.

2. Poor drainage provisions, which led to water leakage, additional loading, and consequently steel corrosion and durability-related problems for the concrete structures and tunnel lining.

The major crack along the 35-m long portal structure at the southbound tunnel exit (shown in Fig. 1 and Fig. 2) which initiated the engineering intervention, was attributed to increased differential loading. This possible cause for the structural damage was identified following the observation of a relatively large slumped mass of soil loading the western side of the portal structure of the southbound tunnel (Fig. 3). The slumped soil mass was due to weathering of the friable chalky limestone rock on the adjacent exposed slopes and subsequent instability and mass transport. The differential loading of one side of the portal frame structure by the slumped soil mass, along with other contributing factors, could have led to the observed cracking of the eastern wall of the portal along with the associated lateral deformations of up to 15 cm (Fig. 1 and Fig. 2). Additional pictures of this particular tunnel segment are provided in the online database that accompanies this paper.

In addition to the differential loading on one side of the portal structure due to accumulation of eroded materials and mass movement, the tunnel structures clearly suffered from poor drainage. Here again, time and lack of maintenance and repair caused the failure of all the water control provisions described in subsequent sections of this paper. The “natural” flow paths from the surrounding hills lead to the upper platform along the tunnel portal as seen in Fig. 4. The tunnels exhibit signs of water leakage along their entire lengths, which resulted in, and are associated with, weakened concrete lining and significant carbonation depths. Plots of carbonation of concrete, f'_c, and mapping of leaking cracks/joints are provided in the online database that accompanies this paper.

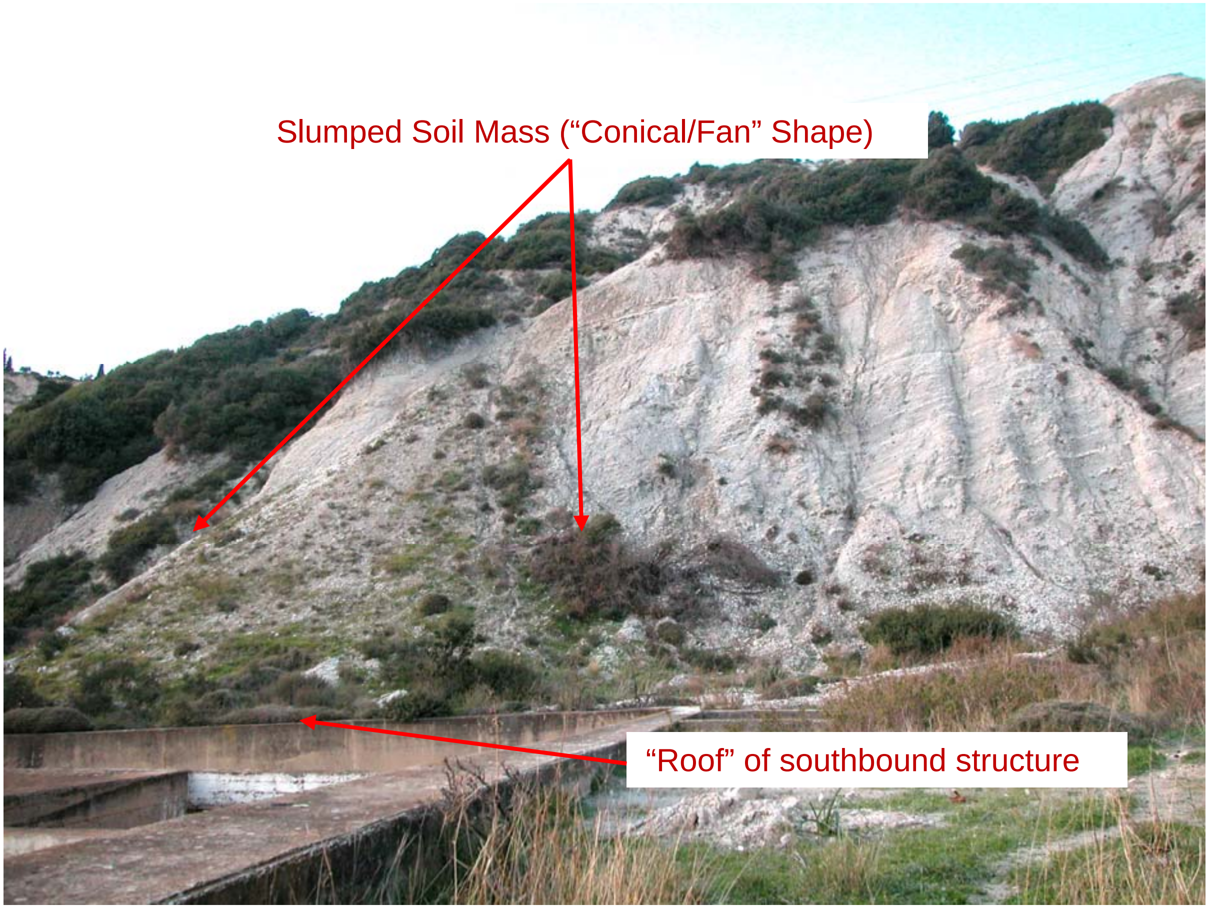

Figure 3. Slumped Soil Mass Loading the Top of the Southbound Tunnel Exit Structure.


Figure 4. Direct Surface Flow (arrows) Towards South Facing Tunnel Exit/entrance Structures -View From Top.

Furthermore, we observed water leakage and severe water-related damage and spalling in the lining of all the construction joints along the entire length of both tunnels. Moreover, and possibly as a result of time, erosion, settlement, and lack of maintenance and repair, the area above the tunnel entrance/exit was flattened and lost all sloped gravity drainage provisions away from the sensitive structures. In fact, this area acts as a natural depression basin, collecting surface water runoff and causing additional loading on the tunnel structures. Furthermore, all the structural drainage provisions such as sumps and channels specifically designed and constructed in that area (as detailed later in the paper) were not operating as intended.

SITE ASSESSMENT AND CHARACTERIZATION

General

Based on the initial observations presented earlier, an extensive site and structural assessment and characterization program was initiated as a clear priority. The assessment effort included a number of elements; some associated with site-scale features including topography, geology, surface water hydrology and geotechnical characterization, others concentrated on the tunnel structure and the concrete frames and lining.

The design and implementation of the assessment program was complicated by the fact that the as-built or design plans/drawings were not available, as they were destroyed or misplaced throughout the war years. After a thorough search and attempts to contact engineers and engineering firms associated with the tunnel construction project, a few drawings were secured, which provided an indication as to the general characteristics of the important structures. Basic information such as lining thickness, portal dimensions, reinforcement, joint details, drainage provisions, had to be re-established and/or checked.

In the following sections, some of the important results of the site assessment program are summarized.

Topographic Survey

No comprehensive topographic maps or surveys of the site (prior and/or post-execution of the tunnel project) were available. A topographic map of the area was produced from different sources by superimposing existing maps to aerial photographs in order to develop the adopted contour map with 5 meter intervals and from which digital terrain models of the area were extracted. The resolution of the extracted digital terrain models (DTM) was improved by relying on available high precision satellite imagery along with a large number of on-the-ground survey points.

Geological Setting

The dominant geologic formation in the area is the Senonian Limestone (C6 – in reference to Dubertret, 1955). The C6 referred to locally as the "Chekka Marl" is a chalky marl and chalky marly limestone formation with nodules and bands of chert. The reported thickness of the Chekka Marl ranges between 100 and 500 m. Lithologically, the Chekka Formation consists of gray to white, chalky to marly limestone rocks, alternating with white to light gray marl beds. Phosphate nodules are present in the chalky facies, with chert bands occurring in the upper levels of this formation. Organic material is also abundant in the Chekka Formation. The rock contains intercalated beds of white chalky limestone of uniform thickness, ranging from 20 to 50 cm, and unconsolidated grayish-white marl. Typical differential weathering is clearly seen on an outcrop scale, as the friable marl layers are eroded faster than the harder chalky beds. Typical pictures of the unit are provided in the online database that accompanies this paper.

The Chekka Formation is considered as an important aquiclude, since in the north of Lebanon, the Chekka strata form the impervious substratum of the younger Miocene aquifer. Moreover, landslides and slumps are noticeable features within this formation and these are present extensively in the area near the tunnel, along the outcrops. The outcropping rocks exposed in the tunnel portals area are differentially-weathered white-gray chalky-marly-limestones, variably jointed, fractured and fissured. The rock layers range in thickness from about 10 cm up to 1 m. Oxidation marks and friable weathered material are common along the exposed surfaces. The marl layers have a general dip ranging between 21º and 28º to the northwest and they strike northeastwards (Fig. 5). Several joint sets were identified and measured; one set exhibited an attitude of 318º/87º, another major set strikes at 96º dipping by 77º (96 º/77 º). The marly layers have a general dip ranging between 21º and 28º to the northwest and they strike northeastwards (Fig. 5). Structurally, the study area is affected by the general tectonic activity of the north Lebanon region. Given the high degree of fracturing, and since the C6 is the only outcropping formation in the area, no clearly discernable faults could be identified in the tunnels zone.

Based on the topographic and geological surveys, it is clear that the prevailing strata are prone to erosion and mass wasting, and that significant slopes are evident in the vicinity of the tunnels with adverse dips identified towards the tunnel exit and entrance points.

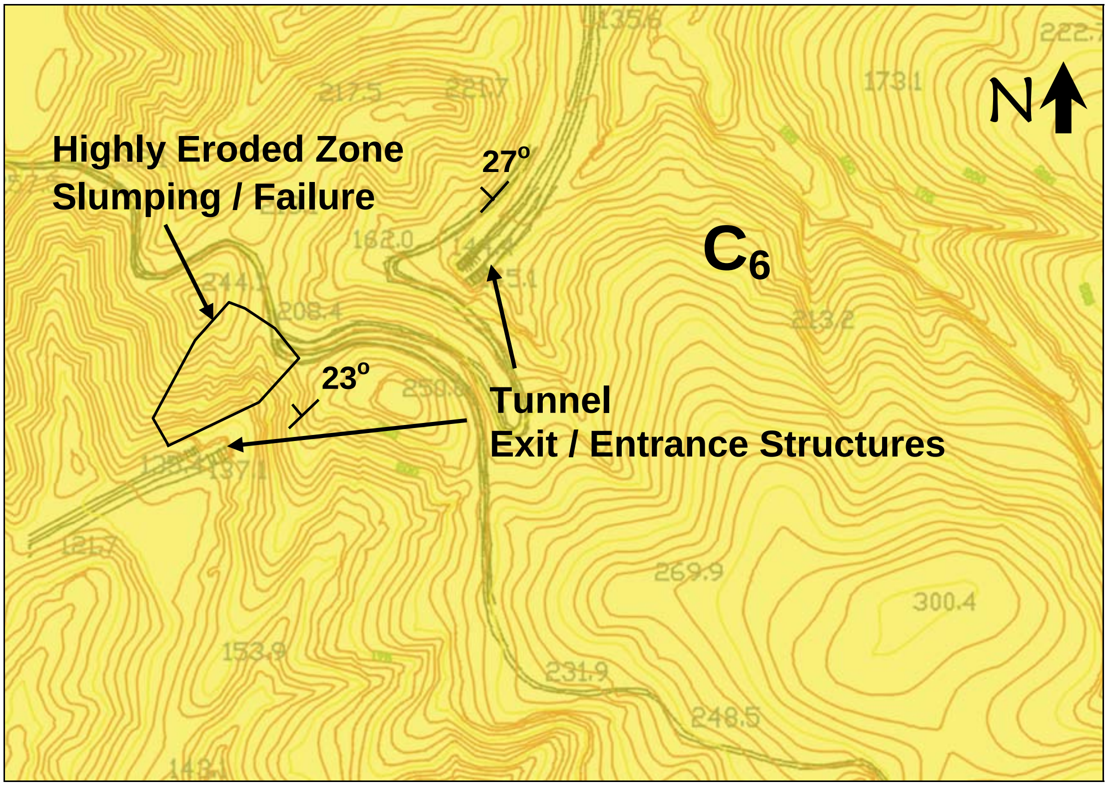

Figure 5. Geology of Area: The Only Outcropping Unit is the Chekka Formation C6.

Hydrological and Hydro-Geological Assessment

Given the vulnerable geological setting and associated topography, a study of the potential surface and subsurface water flow regimes was necessary. Annual precipitation data for the area was collected and a comprehensive study was initiated to identify the problematic drainage zones at and near the site (Fig. 6). Existing drainage structures/installations incorporated in the original construction of the tunnels and associated earthworks were identified and characterized in the absence of any as-built documentation; they consist mainly of the following:

1. Surface water runoff collecting above the tunnel/portal structures was channeled (through grading, which was rendered ineffective due to the lack of maintenance and gradual change of grades caused by settlement and erosion) towards a storage sump with a capacity of approximately 100 m^3. The sump was provided with a V-shaped weir at its entrance. Vegetation (including a 5 m high tree) occupied the majority of the sump. Pictures of these features are provided in the online database that accompanies this paper.

2. A surface storm water channel, running parallel to the outer tunnel structures, particularly along the reinforced concrete portal of the southbound tunnel.

3. Internal graded side channels within the tunnels. These elements run along the entire tunnel length and are intended to collect water from behind the tunnel lining through weep holes provided at regular intervals (~1 m) and channel drainage out of the structure by gravity. The absence of any maintenance of these key features, led to the blockage of the weep holes and, as is evident from the water damage patterns within the tunnels, flow was directed elsewhere developing weaknesses within the lining, particularly at the construction joints.

The water flow regimes and patterns at and around the site area as described earlier, affect the stability and performance of the tunnels and associated structures in a number of ways:

1. The resulting erosion and mass movement, slumping and slope instability increase the loading on the structural frames and tunnel walls.

2. The Chekka Marl layers act as low permeability layers, and affect the internal water drainage patterns by directing the flow at the marl/chalk layer interfaces along the stratification dip towards the tunnel and portal walls.

3. Improper surface drainage conditions led to significant ponding, and infiltration increasing the seeps through tunnel walls.

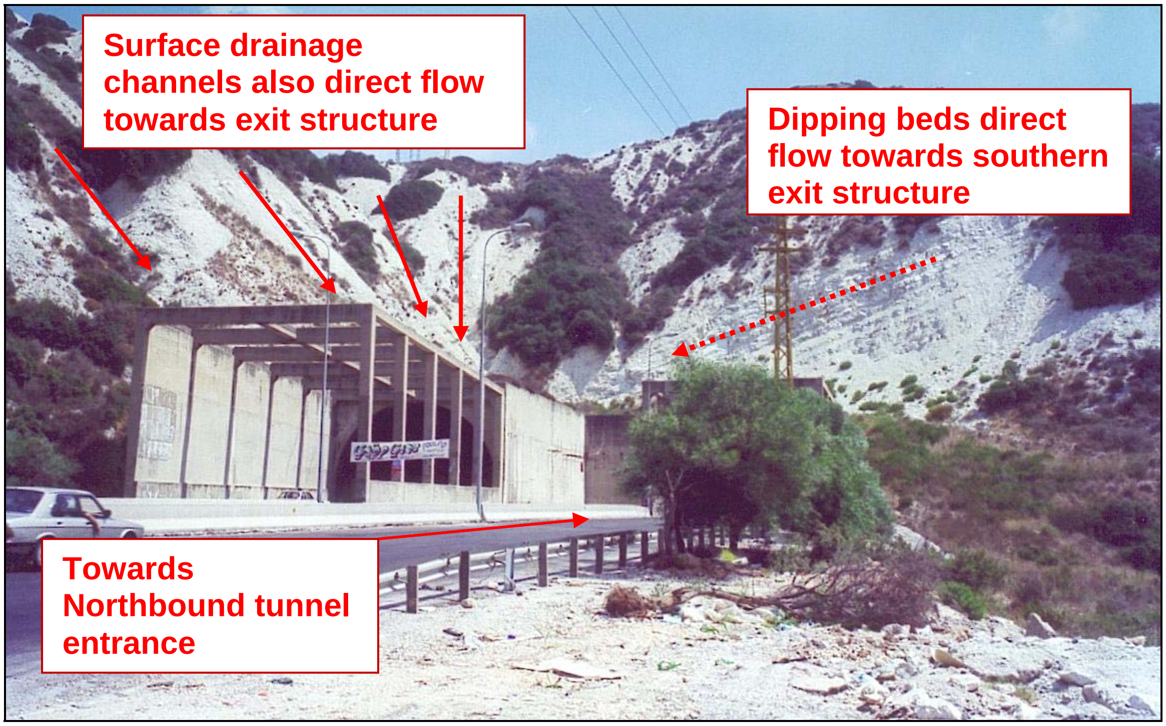

Figure 6. Dipping Beds and Surface Drainage Channels Increase Flow Towards Southbound Tunnel Exit Structure and "Sump" Area Above Tunnels.

Geotechnical Site Exploration

The geotechnical site exploration campaign was initiated to complement the information and data obtained and developed in the assessment phases described earlier, namely the topographic, geological, hydrological and hydrogeological study-area characteristics. The objective of the site investigation was to determine/quantify to the extent possible, the subsurface soil/rock profile and establish the engineering characteristics of the strata used in the subsequent analyses of the tunnel and the associated structures.

Field Exploration and In Situ Tests

A total of six boreholes, approximately 10 m deep, were drilled in addition to a number of shallow pits. The locations of the boreholes were restricted to accessible areas and are indicated in Fig. 7. The exploration pits complemented the boreholes and were located primarily above the southern entrance/exit points.

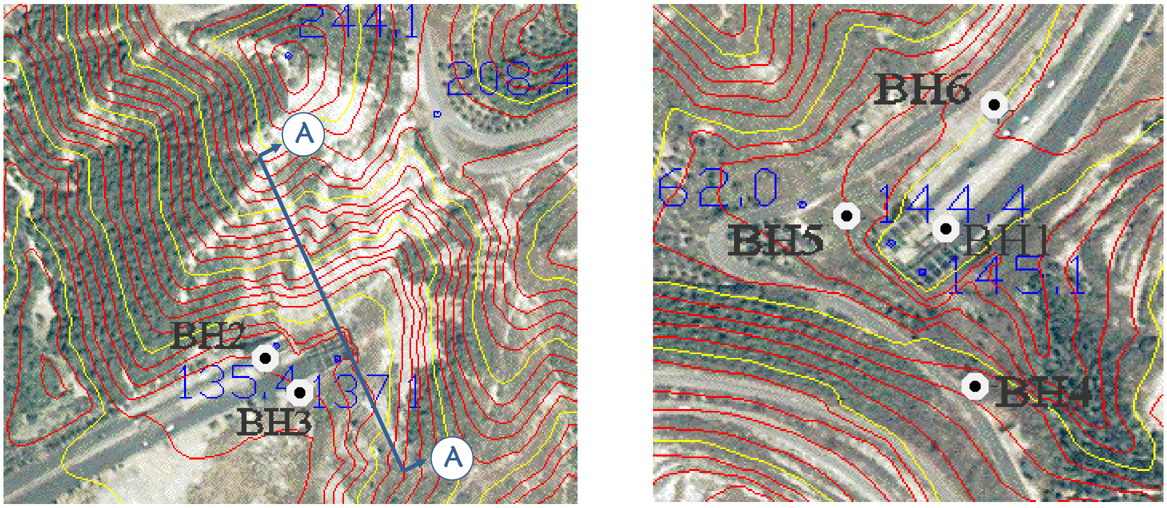

Figure 7. Location of Exploration Boreholes BH-1 Through BH-6.

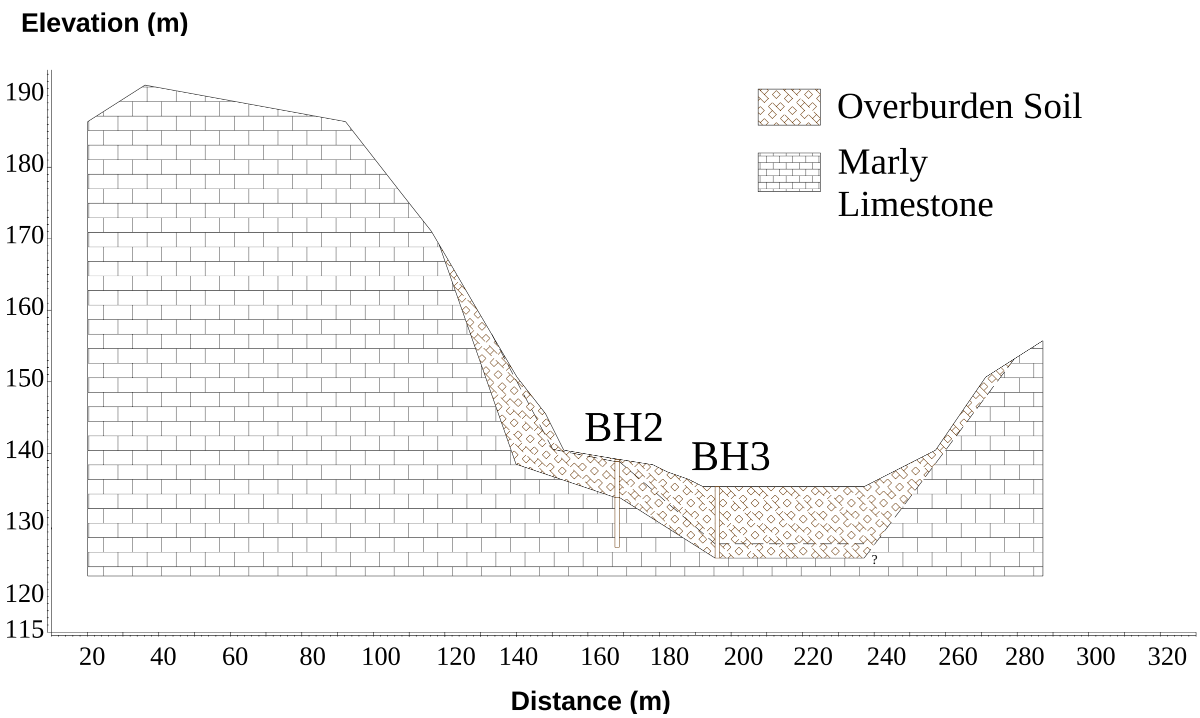

Figure 8. Transverse Section Across South tunnel (A-A in Fig. 7). Tunnel Portal Between Two Hills.

Soil samples were extracted whenever possible, mostly in relative loose and unconsolidated sediments, using a split-spoon standard penetration test sampler. Continuous rock cores were obtained as drilling proceeded into the more coherent rock, using a double core barrel. Standpipe piezometers were installed in all the boreholes to monitor the groundwater conditions. In addition, Menard stress-controlled pressuremeter tests were performed according to ASTM D4719 in dedicated boreholes (BH-1 and BH-2). Pressuremeter tests were also conducted on the slumped material above the southbound tunnel exit structure. Finally, the in-situ hydraulic conductivity was measured using “lefranc” packer tests and a gross estimate was established at ~ $6*10^{-7}$ m/s. This value may be misleadingly low as the hydraulic conductivity of the rock is controlled by the pattern and density of structural discontinuities, such as joints and fractures.

The strata identified from the boreholes advanced on site, consist mainly of chalky and fractured marlstone overlain by different thicknesses of “overburden soil” material ranging from ~ 1.5 to 9 m. The material described as “overburden soil” is basically the product of weathering, erosion and local landslides and mass movements. The standard penetration value in the soil layers consistently exceeded ~50 for penetrations less than 4 to 7 cm, due to the relatively large particles within these materials. For depths advanced in the rock, the total core recovery (TCR) ranged from 12 to 44 %, while the rock quality designation (RQD) ranged from 0 to 28 %. Based on the borehole logs, a profile along a transverse section across the southbound tunnel was derived for the area close to the exit structure and is presented in Fig. 8. The Menard deformation modulus was determined from the pressuremeter test results. The Menard Modulus values (E_m) varied from ~31000 to ~87600 kPa.

The water table levels were monitored in the installed piezometers and readings were taken periodically. The water level was less than 30 cm from the top of boreholes for BH-1, BH-2 and BH-6, and at a depth of about 8 m in borehole BH-3.

Laboratory Tests

Unconfined compression tests were conducted on rock cores according to ASTM D2166. Some of the samples were submerged in water for 48 hours prior to testing in order to explore the effect of increased water content on the strength of the marlstone/rock samples. The elastic deformation modulus values obtained from the laboratory tests on the cores extracted from the material above the southbound tunnel cross-section are relatively low (from about 5800 to 15000 kPa), which was expected given their higher degree of weathering. The results of the unconfined compression tests showed a significant decrease in strength for the wetted samples. The average unconfined compression strength values from samples from BH-1 and BH-5 for the marly limestone layers are shown in Table 1. The unconfined compressive strengths above the southbound tunnel cross-section suggested an average value of q_u ~19.5 MPa for the natural condition samples, and a value of q_u ~7.0 MPa for the wetted samples. Typical results from BH1 samples are presented in Fig. 9.

In addition, rock absorption and the moisture content of the wetted rock samples were measured according to the provisions of ASTM C97 and ASTM D2216 respectively. The absorption capacity of the rock samples ranged from 16.8% to 27%. The moisture content of the wetted samples ranged from 8.25% to 10.51%.

Tests on the "overburden soil" from the same boreholes yielded liquid limits ranging from 35 to 76 for and plastic limits of 27 to 42 (ASTM D4318). The overburden samples classified as either "ML - low plasticity silt" or "MH - high plasticity silt" based on the Unified Soil Classification System (USCS).

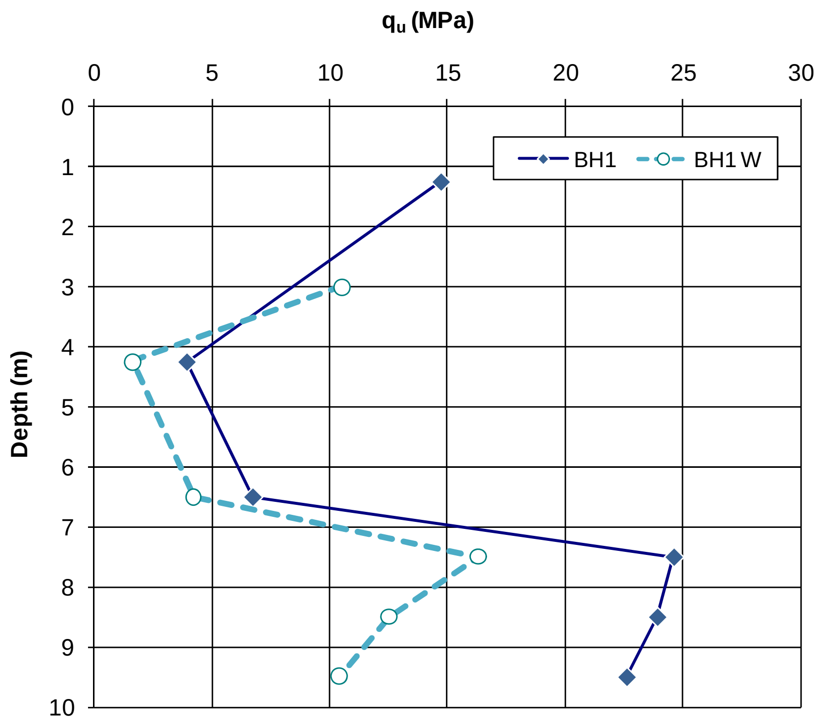

Figure 9. Unconfined Compressive Strength Results From BH1: Natural Water Content and Wetted Samples (w).

Table 1. Representative Sample of Unconfined Compressive Strength Results

BH	Depth (m)	«Natural Condition» $q_{u(ave)}$ (MPa)	Wetted Sample $q_{u(ave)}$ (MPa)
BH-1	< 6	8.4	5.4
BH-1	> 6	23.7	13.0
BH-2	5.5-10	10.0	3.3
BH-5	5-8.5	17.7	8.6

Subsurface Model and Material Parameters used in the Analyses

Based on all the laboratory and field results obtained, and given the need to establish strata characteristics for the analysis of the structural response of the outer southbound tunnel portal, three different soil/rock types or materials were considered representative of the encountered conditions; they are listed below and their respective properties summarized in Table-2:

1. "Soil 1", is characteristic of the slumped mass above the southbound tunnel cross-section (Fig. 3). This soil type is characterized by relatively low values of the Menard modulus, dry density and unconfined compressive strength.

2. "Soil 2" is typical of the soft marlstone layers. This material is typical of materials in a band 4 to 6 m thick as evidenced in boreholes BH-1, BH-2, and BH-5.

3. Finally, the third type is the intact marly-limestone rock of the Chekka Formation.

Table 2. Recommended Parameters For Use in the Structural Assessment Analyses

Material	Wet Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Menard Modulus (MPa)	Cohesion (kPa)	Friction angle (degrees)
Soil 1	18	20	10	40	30
Soil 2	20	22	35	60	25
Intact rock	20	22	100	200	35

Structural Assessment

The structural exploration and assessment program was essential in establishing the actual geometries of the elements, their reinforcement detailing and current conditions. Such information was critical in building the analysis models needed. The primary objectives of this phase of the work were to:

1. Prepare "as-built" drawings.

2. Evaluate the service performance conditions.

3. Identify all locations of distress in the form of cracks, fractures, excessive deflections, water infiltration, or misalignment of structural elements.
4. Determine convenient locations for in-situ carbonation tests, concrete sampling (coring) and for removing concrete covers to verify size and quantity of reinforcing bars. Typical plots of carbonation depths and concrete f'_c, are provided in the online database that accompanies this paper.

The main observations documented in the structural assessment phase are summarized as follows:

1. A major longitudinal structural crack was clearly evident on the outside face of the eastern wall of the tunnel portal frame in the last 35 m span of the exit side of the southbound tunnel (Fig. 1). Signs of water leakage through a crack in the eastern wall were evident at the junction between the tunnel portal and the 4.72 m high retaining parapet. This pattern of damage was indicative of a number of problems with the tunnel portals, which are typically identified as potentially problematic areas in a number of situations (Rothfuss et al., 1995).

2. Cracks ranging in width from 1 mm to more than 3 mm were spread throughout the entrance and exit frames and throughout the tunnel lining itself. Spalling concrete at the locations of construction joints is due to corrosion of the steel reinforcement. Efflorescence was caused by water leakage and infiltration.

3. Close inspection of the top of the portal tunnel frame structures on the southern side of both the northbound and southbound tunnels revealed the absence of any waterproofing provisions to protect the portal reinforced concrete structural elements from water ingress.

4. All the drawings obtained from various sources regarding the structural framing proved to be unreliable, particularly with reference to conditions at the south entrance/exit points and in the vicinity of the 35 m heavily damaged concrete portal.

FINITE ELEMENT ANALYSES: ESTABLISHING THE CAUSE OF STRUCTURAL DISTRESS

The structural elements that suffered most significantly include the tunnel lining and the 35 m projecting portal of the southbound tunnel. The structural model and analysis results related to the latter are presented below. The goal of this phase of the work was to establish/confirm the likely causes of the observed distress and as such, allow for a reasoned approach to the repair and rehabilitation stage of the project.

The analysis was concerned with two distinct conditions: The first describing the situation (loads and moments) at the end-of-construction, is referred to as “Stage 1”; while “Stage 2” relates to the effect of increased loading due to slumping of the weathered soils atop the section as a result of mass movement and the absence of maintenance.

The finite element (FE) analyses conducted were two-dimensional plane strain analyses, using the PLAXIS® V.8 software (PLAXIS Manual, 2006). The adopted methodology incorporated the interaction between the tunnel structure and the soil elements rather than performing separate analyses for each. The soils and rocks were modeled as elasto-plastic materials and the concept of staged loading was adopted to accurately model the loading sequence the tunnel cross-section was subjected to, from construction up to the current state.

Preparation of the Model

A representative section was considered at approximately half the length of the affected section. Two topographic contour maps were used in the analyses. The first represents the topographic conditions after the completion of the project in the 1970s and the second represents the current conditions. The subsurface strata and material parameters were determined/assumed based on the results of the site investigation process. The geometry of the southbound portal and the available steel reinforcement were determined from the structural assessment program. Fig. 10 is a representation of the geometry of the portal used in the finite element model.

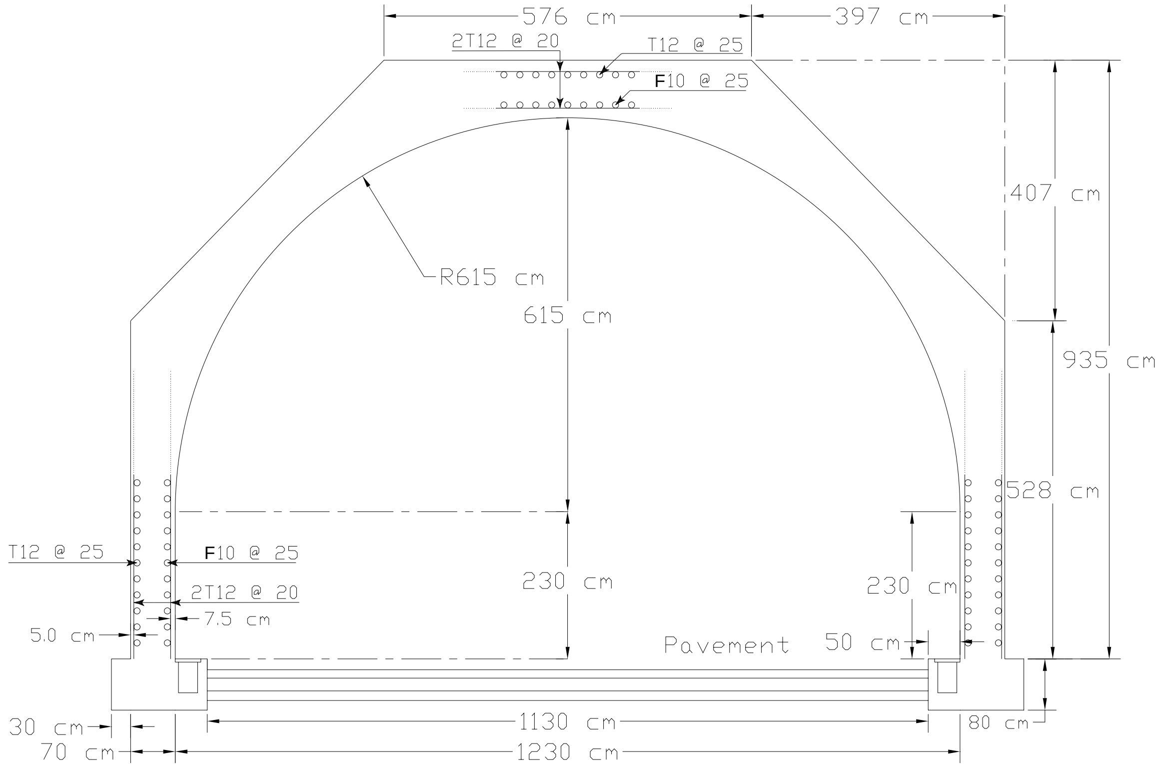

Figure 10. Geometry of the 35-m Long Extension of the Southbound Tunnel Portal.

Fig.11 is a detail view of the finite element mesh near the tunnel portal, showing the soil and rock strata as described in Table 2. The intact rock is shown in light blue, while the slumping soil mass is green. The olive green color represents the engineered backfill soil at the end of construction. It is important to note here that the slumping soil mass was only activated for Stage 2.

Results and Discussion

The results obtained from the finite element model analyses were compared with the structural capacities of the sections. The ultimate moment capacity of the tunnel wall section was calculated based on the basic assumptions of pure bending of a slender beam. The ultimate moment capacity of the available section differs with the section dimensions. The section width varies from 70 to ~ 130 cm and the associated nominal bending moment capacity ranges from 16.4 to 30.7 Ton-m/m. The structural assessment was based primarily on the values of computed shears and bending moments. The results of the analysis conducted at Stage 1 conditions suggest the following:

1. The outer wall (eastern wall of the southbound tunnel exit structure) experienced relatively larger moments and deformations than the other segments of the tunnel section.

2. The values of the shear forces and bending moments are within the capacities of the various sections of the frame.

For conditions representing the current loading, Stage 2, the analyses yielded the deformed mesh shown in Fig. 12. The FE results for Stage 2 suggest the following:

1. The maximum bending moment values calculated are much larger than the ultimate moment capacity of the critical wall section. This implies that the additional loading due to the slumped soil accumulated on top of the tunnel section is indeed a major contributing cause of the observed failure of the portal wall section.

2. The values of the shear forces are within the section capacities.

Table 3 presents the bending moments calculated from the FE analyses and the ultimate moment section capacity at the position of the observed crack ~ 2.5 m above the footing level (Fig. 10).

Table 3. Summary of Calculated Bending Moment Values at Stages 1 and 2

Stage	**Calculated Bending Moment (Ton-m/m)**	**Ultimate Section Bending Moment Capacity (Ton-m./m)**
Stage-1	11.6	16.4
Stage-2	31.9	16.4

Conclusions - Southbound Tunnel Exit Portal Structure

The use of a finite element modeling approach allowed the authors to fully capture the soil-structure interaction of the southbound tunnel exit structure and the surrounding material. From the results of the analyses it is clear that the as-built structure was subjected to significant bending moments particularly along its outer or eastern wall that exceeded the structural bending capacity of the available sections. The shear and axial load capacity of the same sections were not exceeded. This result is reasonable given that the loading on the section was non-symmetric as shown in Fig. 10. The numerical model correctly predicted the location and relative extent of the structural distress at the subsequent stage where the load due to accumulated and slumped material was added. Cracking could also be present at other inaccessible locations such as the outside tension face of the backfilled western wall of the portal. This result of the FE analyses was very important in that it led to the provisions of repair and support measures along the "un-exposed" western wall, as discussed in subsequent section.
Based on the above, it becomes clear that original design of the structural system did not consider significant loads atop the frame, and presumed that sufficient and regular maintenance would remove any slumped or accumulated materials.

REPAIR AND REHABILITATION RECOMMENDATIONS

As a result of the work presented in this paper, remedial measures were formulated, designed and gradually implemented. Details are presented in the online database that accompanies this paper. They included:

1. Controlling surface water flow and improving drainage around structure area.

2. Removal of the slumped mass on top of the southbound tunnel exit structure and the provision of long-term stability and erosion control measures to eliminate the possibility of recurrence. These provisions included the installation of rock bolts/nails with a high strength steel wire mesh facing (Fig. 13 and Fig. 14). A program for regular inspection and maintenance of the structures was also developed.

3. Repair and rehabilitation of the distressed structural elements including jacketing and repair of the eastern outer wall and the provision of passive nails/rock bolts on the western wall, tying into the parent intact rock, thus reducing pressures on the wall and lateral deformations.

4. Repair and sealing of all cracks and fissures in the original concrete tunnel lining, followed by placement of a sprayed 4-mm thick waterproofing membrane and a protective layer of fiber-reinforced shorcrete.

Aspects of the repair and rehabilitation measures implemented are presented in the following sections.

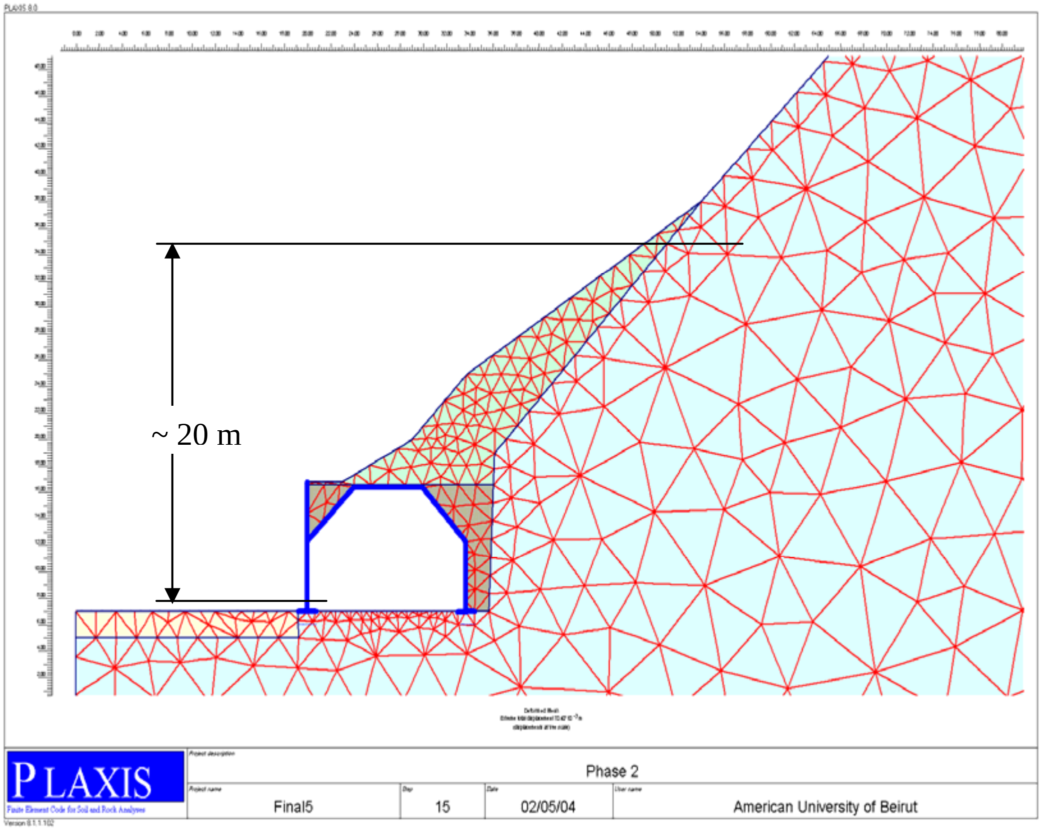

Figure 11. Finite Element Model Showing the Various Strata and the Loading Sequences – Stages 1 and 2.

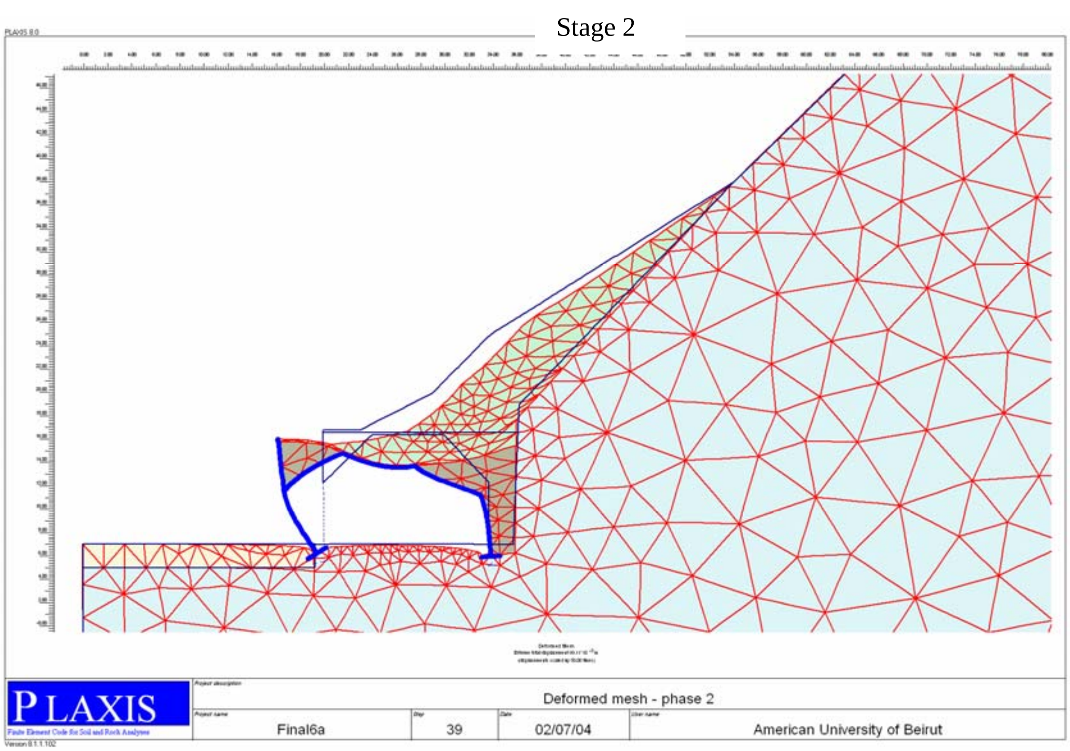

Figure 12. Deformed Mesh for Stage-2 (Deformations Are Scaled up 50 times).

Stage 2

Figure- 13. Installation of the Nails/bolts Along the Western Slopes.

Figure 14. Completed Rock Bolts/nails and Wire Mesh Installation Along the Western Slope.

Surface Water Control and Drainage

Given the importance of controlling the ingress of water at and around the site, a number of control and capture provisions were designed and constructed:

1. A water runoff interception scheme, with particular emphasis on the slopes contributing to the flow patterns towards the southbound tunnel exit. A schematic of the layout of the system is shown in Fig. 15. The interceptor trenches and drainage channels are concrete-lined and feed into controlled sumps and outlets, moving surface water away from the portals and vulnerable tunnel sections.

2. The surface water drainage provisions were re-designed to incorporate the already existing drainage structures that were not in service conditions. Surface run-off is collected by directing the rainwater by adequate leveling the area above the exit/entrance structures, into concrete-lined channels, which carry the water into the existing sump for storage and subsequent evacuation by gravity outside the project area. Fig. 16 shows the remediated area above the tunnels, with the v-notched entrance to the sump and peripheral drainage channels.

3. To mitigate water seeping into the rock and collecting at or around the tunnel itself. Specifications for the repair and rehabilitation of the existing scheme were provided and implemented. The specifications included work on the construction joints and the provision of perimeter drainage geo-elements along those locations. Water collected through the weep-holes and/or the newly constructed perimeter drains is then gravity-driven within the existing side channels.

4. Water proofing was provided through a projected/sprayed membrane. The sprayable membrane selected was a MASTERSEAL 345. The membrane was applied using an automated, computer controlled robotic arm, which insured thorough coverage/thickness and uniform application (Fig. 17).

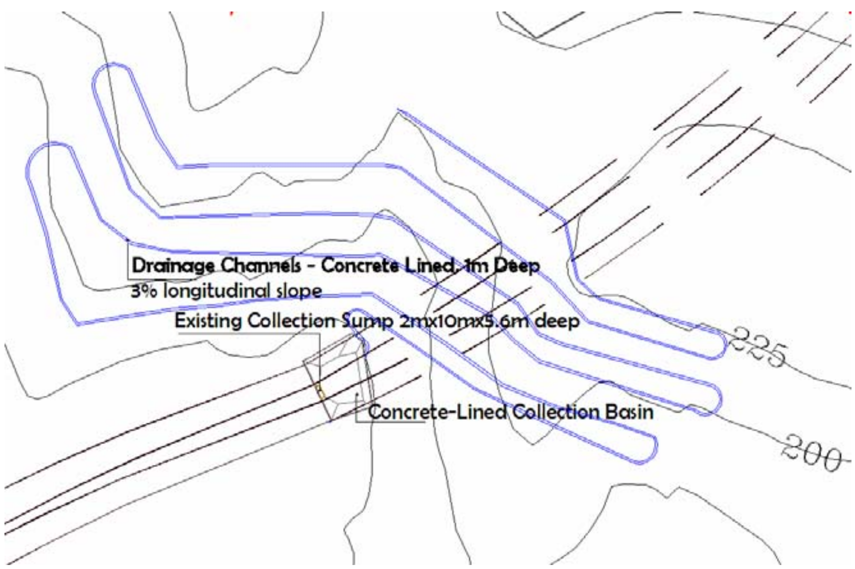

Figure 15. Schematic Plan of the Drainage/interception Channels (Trace of Tunnels Shown as Dashed Lines).

Repair and Strengthening of the Southbound Tunnel Portal Structure

In addition to some of the provisions presented earlier regarding the removal of the accumulated soils which were loading the southbound tunnel portal structure and the associated maintenance program, the following repair measures were designed and implemented:

1. The major structural cracks on the outside face of the eastern wall of the portal frame were repaired by "crack-injection" using low viscosity, two-component epoxy resin. The injection was completed after thorough cleaning and preparation of the affected areas and the provision of injection ports to ensure thorough filling of the cracks. Similar treatments were applied to all cracks and damaged areas in the tunnel concrete lining (Fig. 15).

2. The eastern wall of the southbound tunnel portal was further strengthened after the accumulated weathered material above it was removed and the crack treatment/repair completed. Strengthening was achieved by connecting a 15-cm reinforced concrete jacket to the existing wall.

3. Whereas the distress signs were evident along the eastern projecting portal wall only, the FEM analyses suggested that cracks could have developed along the inaccessible outside tension face of the backfilled western wall of the portal. As such, strengthening provisions for that element were taken. Jacketing was ruled out given that the outer surface is backfilled and cannot be exposed. Instead, the installation of passive nails/rock bolts at elevations of grade plus 2 m and grade plus 4 m was recommended, tying the wall to the intact rock and reducing the stresses on the wall itself.

Figure 16. Regraded and Remediated Area Above the Portals and Southern Entrance/exit of Tunnels.

Figure 17. Installation of the Sprayed Membrane. Also Shown Are the Repaired Cracks in Original Lining.

SUMMARY AND CONCLUSIONS

In this paper the case study of the Chekka tunnels including the results of a comprehensive site, geological, hydrological and geotechnical and structural assessment are presented. The structural distress and service failures in the Chekka tunnels provide a very interesting and telling example of the importance of even the most minimal maintenance. The sometimes forced abandonment of important infrastructure assets for long durations may lead to structure-threatening distress and damage, which are very costly to repair.

In summary, the tunnel exit/entrance points are vulnerable and susceptible to slope instabilities and surface water-induced erosion and loading as water seeped along the rock beds dipping towards the structural frames. Whereas certain provisions aimed at addressing these problems were considered in the original design, a number of construction omissions, the most important of which is the incomplete backfilling along the eastern wall of the southbound tunnel portal, compounded the effect of the unbalanced and differential loading as weathered materials and slope instabilities accumulated over the western side of the exposed portal slab. However, the absence of maintenance and repair for nearly 25 years is still the single most critical factor contributing to the observed structural distress and problems, namely:

1. Major structural cracking and failure in the eastern wall of the southbound tunnel portal frame.

2. Major distress and cracking in the tunnel concrete lining and along all joints.

3. Failure of the limited drainage provisions in the zone atop the southern portal structures as well as within the tunnels themselves.

Thorough analyses were conducted on a model of the southern portal structures using finite elements. The analyses were run for loading conditions representative of both the end-of-construction and current stages. The analyses performed for the current loading conditions indicated that the resulting bending moments on various sections of the southbound tunnel portal frame largely exceeded their capacity. This resulted in the observed deformations and structural failure.

A comprehensive set or repair and rehabilitation provisions addressing both the structural stability of the tunnels and the water control provisions were presented.

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