ADDIS ABABA INSTITUTE OF TECHNOLOGY [AAiT]
SCHOOL OF CIVIL and ENVIRONMENTAL ENGINEERING
[M.Sc. Degree in Railway-Civil Engineering]

DESIGN AND CONSTRUCTION OF TUNNELS IN ETHIOPIA: Lessons Learned from the Case of Gilgel Gibe III Hydropower Tunnels

By

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A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the Requirement of the Degree of Master of Science in Railway-Civil Engineering

Advisor

Henok Fikre (Dr-Ing.)

April 16, 2015

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ABSTRACT

Tunnels are long linear undertakings with few opportunities to perform the work at more than one location. Tunnels are also a series of repetitive operations each of which usually must be finished before the next can be started. Tunnel engineering makes possible many vital underwater and underground facilities. Unique design and construction techniques are involved because of the necessity of protecting the constructors and users of these facilities from alien environments.

In Ethiopia, tunnelling has been recently practiced only for one of the utility tunnels, i.e., hydropower tunnels. The purpose of this research is to evaluate the design and construction approaches of the GG III hydropower tunnels, and to generate lessons which could be applied for proper design and construction for future tunnels in Ethiopia. To achieve the objectives, efforts were made to evaluate the design and construction approaches, including literature review, geological, hydrogeological and geotechnical investigation reports, design reports, method statements, various pictures and drawings, challenges and remedial work reports, site visit and interviews.

In this research the author has found design and construction problems related to the applied rock mass classification system. The problems in the design include: using inappropriate GSI values which lead to uneconomical primary support design and inconsideration of stand-up time (one of the design parameters). The problems in the construction include: using incorrect construction procedures of NATM, installation of improper ground excavation supports, and no installation of instrumentations and drainage holes (in the diversion tunnels). Due to these problems, rock and shotcrete wedges failure occurred at various locations of power and diversion tunnels, and a tunnel section collapsed in the central diversion tunnel. A worker has been lost due to the rock wedges failure in the manifold of the right power tunnel. Due to the collapsed tunnel section, the unexpected quantity and various types of rock supports, ground improvement materials, various construction techniques, a construction of additional 63.25m access tunnel (adit), etc. are used in the remedial works. The remedial works took almost nine months, from May 2007 to February 2008. As a result of this collapsed section, the construction cost and the construction time of the project obviously increased.

Due to lack of proper design of the primary supports in the project, the author has performed the proper design of ground excavation primary supports using up-to-date Q-support design method.

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List of Notations

Abbreviation	Description
1. GIR	geotechnical interpretative report
2. σ_c	uniaxial compressive strength of the intact rock elements
3. σ_{cm}	rock mass strength
4. p_o	in-situ stress
5. RMR	Rock mass rating
6. Q	rock mass quality
7. TBM	tunnel boring machine
8. DS TBM	double shield TBM
9. EPB TBM	earth pressure balanced TBM
10. RQD	rock quality designation
11. GSI	geological strength index
12. GPa	Giga Pascal
13. MPa	mega Pascal
14. kPa	kilo Pascal
15. ESR	excavation support ratio
16. MW	mega watt
17. kW	kilo Watt
18. kN	kilo Newton
19. U-T	unweathered trachyte
20. SW-T	slightly weathered trachyte
21. Br. T	trachytic breccia
22. ITA	international tunnelling association
23. R3	medium strong rock
24. R4	strong rock
25. R5	very strong rock
26. E	modulus of elasticity of intact rock
27. ν	poisson's ratio
28. D_e	equivalent dimension
29. @	rock bolt spacing
30. γ_r	unit weight of rock mass
31. JRC	joint roughness coefficient
32. JCS	joint compressive strength
33. GG III HPTs	Gigel gibe III hydropower tunnels
34. OM	operational method
35. SCL	Sprayed Concrete Lining

CHAPTER 1. INTRODUCTION

1.1. Background of tunnelling

A tunnel is an underground passageway, completely enclosed except for openings or egress, commonly at each end. Tunnels are long linear undertakings with few opportunities to perform the work at more than one location. Tunnels are also a series of repetitive operations each of which usually must be finished before the next can be started.

Tunnel engineering makes possible many vital underwater and underground facilities. Unique design and construction techniques are involved because of the necessity of protecting the constructors and users of these facilities from alien environments.

The World's oldest underwater tunnel is assumed to be the "Terelek kaya tüneli" under Kizil River, a south of the towns of Boyabat and Duragan in Turkey, estimated to have been built more than 5000 years ago, it is assumed to have had a defense purpose[7].

In (BC2180-2160) Babylon (Iraq) built a tunnel underpassing the Euphrates River for foot. It was 900m long from the Royal Palace to the Jupiter Shrine (a temple) [7].

On 10th of January 1863, the first metro in the world (London metro 7.6km long) started its operation, 9.5 million people took this metro in the first year. The tunnel section is 5.18m high and 8.69m wide. Now, the London Metro has 12 lines, with a total length of 408km [7].

The City with the longest metro in the world -Shanghai. By 2010-04-20, 11 metro lines have been built, with 266 stations and a total operation length of 410km (excluding Maglev train), Shanghai has the longest metro in the world surpassing London [7].

The highest tunnel in the world, Fenghuoshan Tunnel, was built on the rail line from Tibet to Qinghai China. The tunnel is 1338m long and 4905m in altitude [7].

Till the end of 2007, China has built more than 10,000 tunnels with a total length of about 7000km, including 4673 road tunnels with a total length of 2555.5km, 6213 rail tunnels with a total length of 3511km (up to Year 1999)[7].

Tunnelling is most efficient in a country that has mountainous terrains like Ethiopia. In Ethiopia, tunnelling has been practiced recently only for one of the utility tunnels, i.e., hydropower tunnels. In Ethiopia, there are four old roadway tunnels along the road line from Addis Ababa to Mekele Town. The tunnels are very short in length (the longer one is592meters). They were built by Italian during the invasion of Ethiopia.

The hydropower tunnels are namely: Gigel Gibe I (about 9.3km), Gigel Gibe II (26km), Fincha (5.2km), Beles (20km) and Gilgel Gibe III (about 5.242km).

Currently, there is a cut-and-cover railway tunnel in Addis Ababa city around Arada St. George Church which is the first railway line in the city and it is under construction.

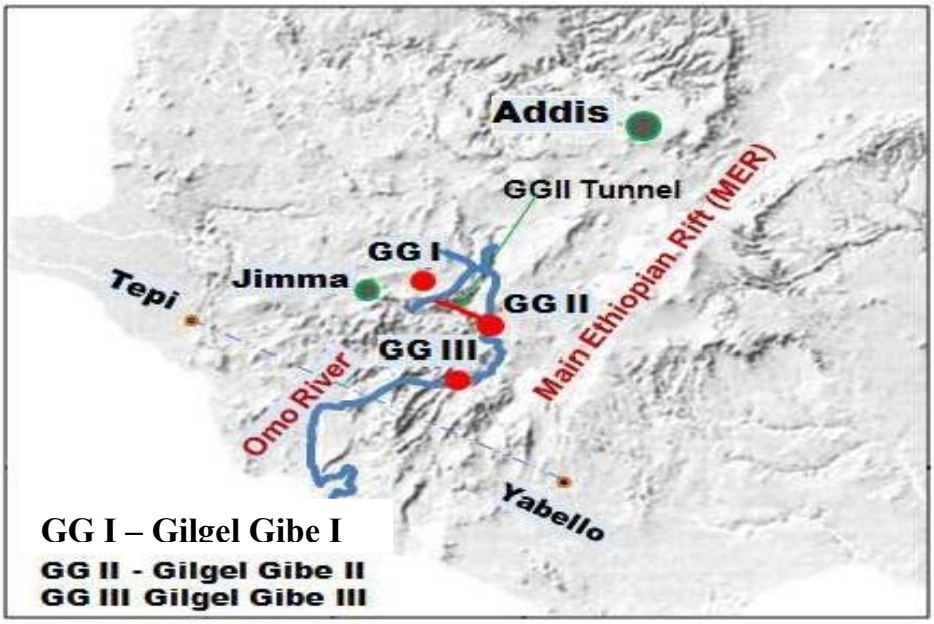


Figure 1.1 Location Map of Gibe I, II and III [17]

1.2. Statement of the Problem

Ethiopia is a large country and its topography is mostly mountainous or highlands. The country's population is almost 90 million and its economy is mainly dependent on agriculture. In addition, Ethiopia has national and international parks as well as historical or special cultural areas. For resourceful land use and worthy environmental purposes, it is a greater opportunity to practice underground or tunnel constructions.

Tunnelling is one of the important engineering technologies which can be applied for the purposes of: Transportation, Utility, Mining, military, storage, etc…Tunnelling has been recently practiced in Ethiopia only for one of the utility tunnels, i.e., hydropower tunnels. Therefore, it is a must to learn from the past hydropower tunnelling practices to develop proper design and construction for future tunnels in Ethiopia.

1.3. Objectives of the Study

A) General objective

The objective of this research is to evaluate the design and construction approach implemented in hydropower tunnels of Gilgel Gibe III, and to generate lessons which could be applied for proper design and construction for future tunnels in Ethiopia.

B) Specific objectives

This research will have the following specific objectives:

- To evaluate the design parameters and construction methods or techniques implemented in Gilgel Gibe III Hydropower tunnels;
- To assess challenges encountered in the case of Gilgel gibe III hydropower tunnel constructions with regard to geotechnical parameters, support requirements and construction procedures.
- To recommend future tunnel design and construction considerations based on the lessons learned from Gilgel Gibe III hydropower tunnelling practices.

1.4. Research Methodology

The research will involve the following:

- Literature review: this includes review of current tunnel design and construction reports and publications.
- Evaluation of the overall topographic, geological, hydrogeological and geotechnical condition of the project area including major rock/soil types, hydrological (sub-surface hydrology) of the areas.
- Any potential tunnel construction challenges in relation to excavation methods, support methods and ground improvement mechanisms. This will include field observations of the Gilgel gibe III hydropower tunnels.
- Detailed evaluation of the challenges encountered during the design and construction of the Gilgel Gibe III hydropower tunnels. This includes evaluation of existing reports, interview of professionals who were involved in the design and construction of the tunnels.

1.5. Scope of the research

This research will briefly review current design and construction methods as well as hydropower tunnels in Ethiopia. Basically, this research will carry out the evaluation of the design and construction approach implemented in hydropower tunnels of the Gilgel Gibe III, and to generate lessons which could be applied for proper design and construction for future tunnels in Ethiopia.

CHAPTER 2. LITERATURE REVIEW

2.1. Current Design and Construction Methods of Tunnels

Tunnelling is unique when compared to other types of civil construction. Tunnels are long linear undertakings with few opportunities to perform the work at more than one location. Tunnels are also a series of repetitive operations each of which usually must be finished before the next can be started. This uniqueness and the linear, repetitive nature of the work must be understood by the planners and builders of tunnel projects to control and manage the project to a successful conclusion.
The geology encountered determines the tunnelling methods that will be used, the speed that the tunnel can be constructed and the types of specialized equipment that are required.

2.1.1. General Approaches

The general approach to the design of tunnels includes site investigations, ground probings and in-situ monitoring, as well as the analysis of stresses and deformations [1].

a) General procedure in designing a tunnel

Planning a tunnelling project requires the interdependent participation of the following disciplines, at a minimum [1]:

- Geology.
- Geotechnical engineering.
- Excavation technology, e.g. machine tunnelling.
- Design of the supporting structural elements, including long-term behaviour of materials.
- Contract principles and law.

Although the experts in each of these disciplines may be responsible only for their specific area of knowledge, the decision on the main design features should be the outcome of the cooperative integration of all the disciplines.

The basic documents for tunnel design should include or cover [1,2, 3]:

- The geological report presenting the results of the geological and geophysical survey.
- The hydrogeological report.
- The geotechnical report on site investigations, including the interpretation of the results of site and laboratory tests with respect to the tunnelling process, soil and rock classification, etc.,
- Information on line, cross-section, drainage, and structural elements affecting later use of the tunnel.
- Plans for and a description of the projected excavation or driving procedure, including the different cross-sections related to different ground conditions.

- Design documents for the types of excavation methods and tunnel supports likely to be applied, considering, e.g. excavation advance and face support (types and number of anchors, shotcrete strength, closure length, etc.).
- The program for the in-situ monitoring of the tunnel by field measurements.
- The analysis of stresses and deformations (for unlined tunnels as well as for single-or double-lined tunnels), and the dimensioning of the tunnel support for intermediate phases and final linings.
- The design for waterproofing or drainage.
- Structural documents for the final design of the tunnel project, including the detailing.
- During and after excavation, reports on the field measurements and interpretation of their results with respect to the response of the ground and the structural safety of the tunnel.
- Documentation of the problems encountered during the excavation and measures applied, e.g. strengthening the ground or changing the projected type of support, based on monitoring results.

The above sequence of these basic documents also provides the general outline of the design procedure [1].

b) Elements of the Structural Design Model for Tunnels

In planning, designing, analyzing and detailing a structure, engineers promise that the structure will neither suffer structurally nor collapse during its projected lifetime. Thus, models of the reality are necessary for analysis in order to predict the behaviour of a tunnel during the excavation and during its lifetime. Models are also needed for bidding on projects [1].
The following main elements involved in the design procedure are presented as a flow chart in Figure 2.1[1]:

1. Geology and site investigations must confirm the line, orientation, depth, etc., of the opening, e.g. a cavern.
2. Ground probing and soil or rock mechanics must be applied to determine the ground characteristics, e.g. primary stresses, soil or rock strength, faults, water conditions.
3. Experience and preliminary estimates or calculations are used to determine the cross-section required and the choice of the excavation method or the tunnel driving machine to be used, as well as the methods of dewatering the ground and the selection of the supporting structural elements.
4. After steps 1 – 3 are completed, the tunnelling engineer must derive, or even invent, a structural model. By applying equilibrium and compatibility conditions to the model, the engineer has to arrive at those criteria that are factors in deciding whether or not the design is safe. Different models may be used for each excavation phase, for the preliminary and the final tunnel lining, or for different ground behaviour, e.g. in discontinuous rock or homogeneous soft soil. Modelling of the geometric features may vary greatly, depending on the desired intensity of the analysis.
5. A safety concept drawn from failure hypotheses may be based on criteria such as strains, stresses, deformation, or failure modes [1].

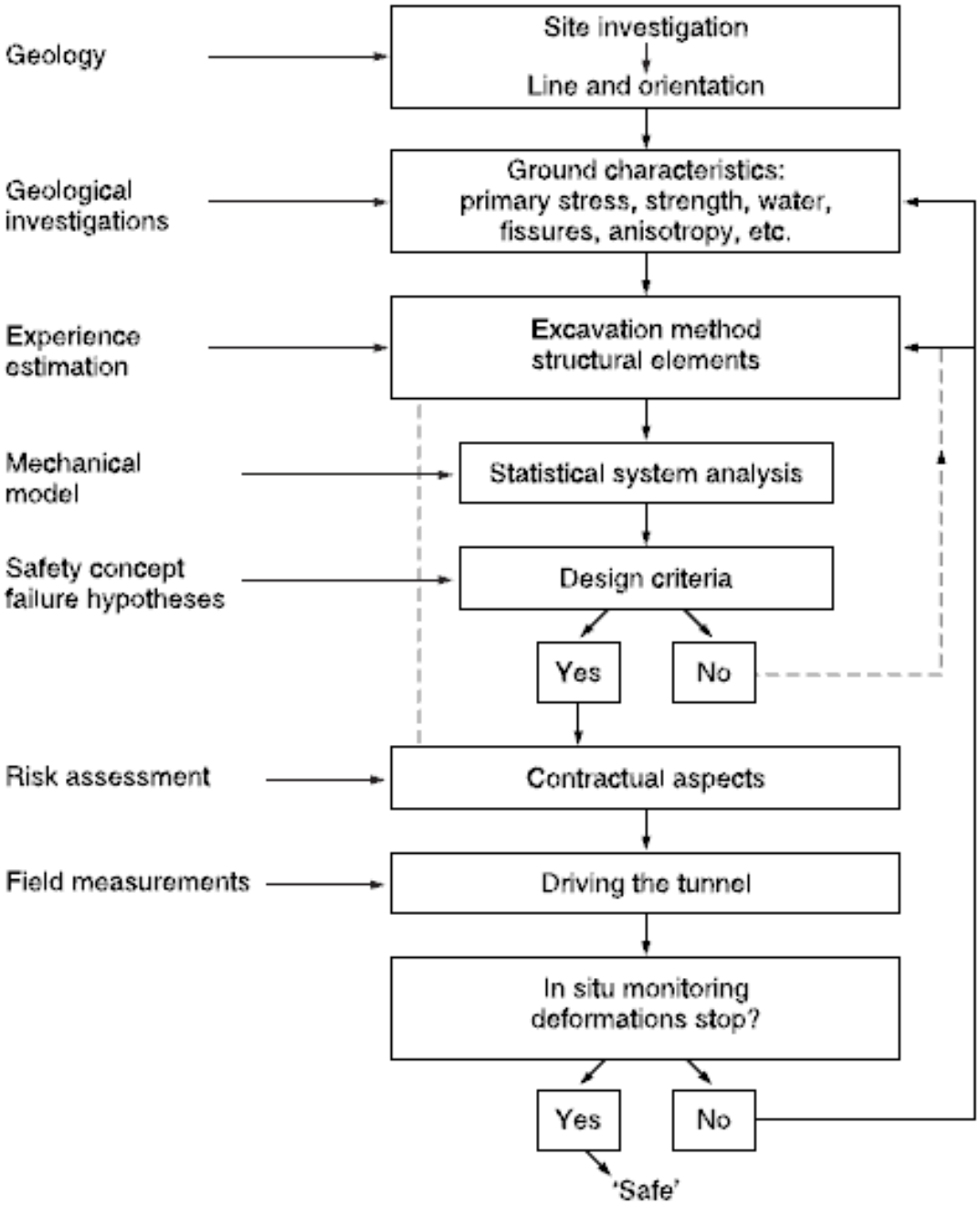


Figure 2.1 Design approach for rock tunnels [1, 4]

c) Design Criteria and Evaluation of Structural Safety

The structural design model should yield criteria related to failure cases, against which the tunnel should be designed safely. These criteria may be [1, 2, 4]:

- Deformations and strains.
- Stresses and utilization of plasticity.

- Cross-sectional lining failure.
- Failure of ground or rock strength.
- Limit-analysis failure modes.

In principle, the safety margins may be chosen differently for each of the failure cases listed above. However, in reality the evaluation of the actual safety margins is most complex and very much affected by the scattering of the involved properties of the ground and the structure. Therefore, the results of any calculation should be subject to critical reflection on their relevance to the actual conditions [1, 2, 3, 4].

d) Tunnels in rock/soil

The ground should be divided in geotechnical units for which the design characteristics may be considered uniform. However, relevant characteristics may display considerable variations within a geotechnical unit. The following aspects should be considered for the geological description of each zone [1, 2]:

- Name of the geological formation in accordance with a genetic classification.
- Geologic structure and fracturing of the rock mass with strike and dip orientations.
- Colour, texture and mineral composition.
- Degree of weathering.

Parameters of the rock mass (for example, in five rock classes of intervals, including) [1, 2, 3]:

- Thickness of the layers.
- Fracture intercept.
- Rock classification.
- Core recovery.
- Uniaxial compressive strength of the rock (derived from laboratory tests).
- Angle of friction of the fractures (derived from laboratory direct shear tests).
- Strength of the ground in on-site situations.
- Deformation properties (modulus).
- Effect of water on the rock quality.
- Seismic velocity

e) Primary stress field of the ground

For larger tunnel projects, tests evaluating the natural stresses in the rock mass may be recommended. For usual tunnel projects one should estimate the stress ratio σ_h/σ_v at tunnel level, where σ_h is the lateral ground pressure and σ_v is the major principal stress (usually in the vertical direction), for which the weight of the overlying rock generally may be taken. Tectonic (relative movement of earth's lithosphere) stresses should be indicated [1,2, 3].Two types of information about water conditions are required:

- Permeability coefficient k (m/s) (from field tests) and Lugeon unit (from tests in boreholes)
- Water pressure at the tunnel level (hydraulic head), at piezometric level in boreholes.

Deformability of the rock mass- in-situ tests are required to derive the two different deformation moduli, which can be determined either from static methods (dilatometer tests in boreholes, plate tests in adits, or radial jacking tests in chambers) or from dynamic methods (wave velocity by seismic-refraction or by geophysical logging in boreholes). Engineering judgment should be exercised in choosing the value of the modulus most appropriate for the design-for instance, by relevant tangent of the pressure-deformation curve at the primary stress level in the static method [1,2, 3]. The simplified design chart is shown in Figure 2.2.

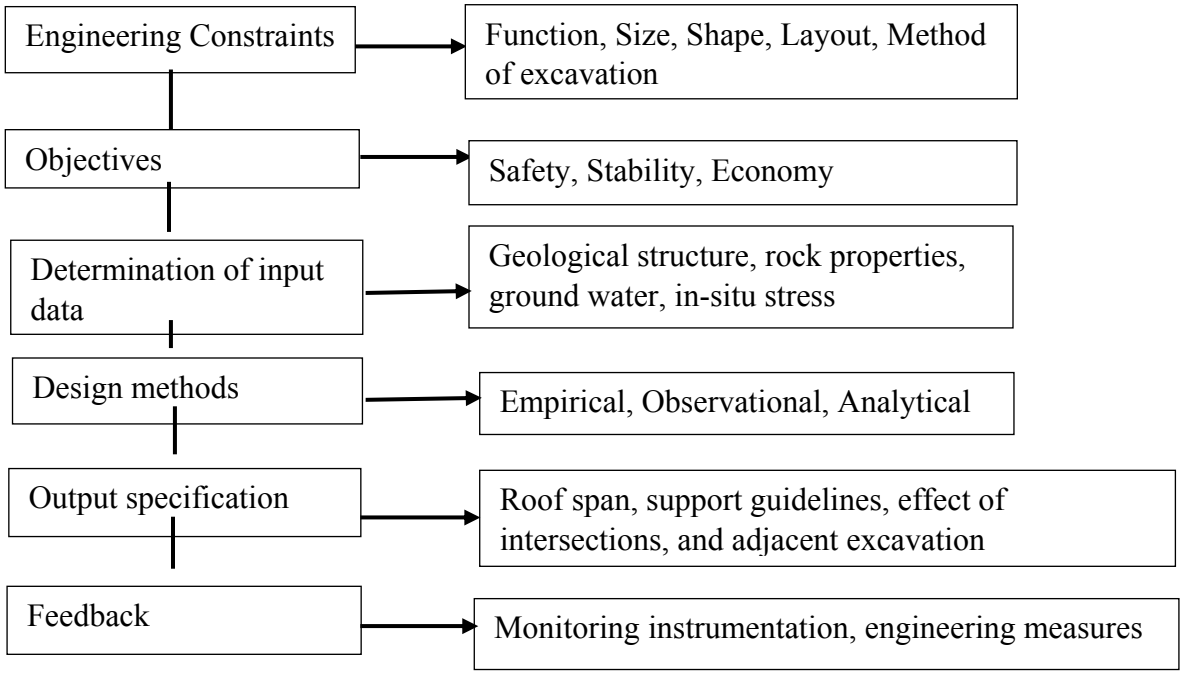


Figure 2.2. Simplified design charts for tunnelling [7]

2.1.2. Design Methods

The design methods/tools available are [4, 7, 8]:

1. Empirical methods
2. Observational methods
3. Analytical methods

2.1.2.1. Empirical methods

Rock classifications are the best known empirical approach for assessing the stability of underground openings in rock, including rock mass rating (RMR), rock mass quality (Q) system, etc. [7].

The empirical approach, based on rock mass classifications, is the most popular probably because of its basic purpose of simplicity and ability to managing uncertainties. The geological and geotechnical uncertainties can be tackled effectively using proper rock classifications.
Moreover, the designers can take on-the-spot decisions on supporting measures etc., if there is sudden change in the geology [7].

Empirical methods are based on assessments of precedent practice and generally have a successful track record in rock tunnels. Ideally the support recommendations have been 'calibrated' against actual performance for a wide range of tunnelling conditions and tunnel sizes. Some soft ground empirical rules have also been established but these tend to be based on local experience rather than being universally applicable [4, 24, 8, 23].

The most frequently used empirical design methods are the geomechanics rock classification (RMR) and rock mass quality (Q) (Table 2.1 and Table 2.2, respectively). These employ a combination of parameters such as the strength of the rock, its quality by using RQD values, joints, and number of sets, frequency, spacing and condition, and groundwater conditions to produce a rock mass classification. Using the product of these parameters, the support measures required can be determined from design charts or tables. A measure of their success has been the number of publications amplifying specific aspects and developments of the original format. Accordingly it is recommended that direct selection systems, such as RMR and Q, be used only when the strength of the rock mass adequately exceeds the ground stresses. Otherwise the rock mass should be treated as a continuum [4, 23].

a) Geomechanics rock classification (RMR) method

Six parameters are used in estimating RMR (equation 2.1)[7, 8]:

$$RMR = R_1+R_2+R_3+R_4+R_5+R_6 \quad (2.1)$$

Where R_1 = uniaxial compressive strength of intact rock material
R_2 = rock quality designation (RQD)
R_3 = Joint or discontinuity spacing
R_4 = joint condition or condition of discontinuity
R_5 = ground water condition
R_6 = joint orientation

Tables for the RMR calculation are presented in Appendix A.

Experience suggests that [7]:

- 10 points should be added to RMR to get RMR for undisturbed rock masses in situations where TBMs or roadheaders are used for tunnel excavation;
- 3 to 5 points may be added to RMR depending upon the quality of the controlled blasting.

b) Rock mass quality (Q) Method

The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1000 and is defined by Eq. 2.2 [5].

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF} \quad (2.2)$$

Where RQD = Rock Quality Designation ($0 \leq RQD \leq 100$)
J_n = joint set number ($0.5 \leq Jn \leq 20$)
Jr = joint roughness number ($1 \leq Jr \leq 4$)
Ja = joint alteration number ($0.75 \leq Ja \leq 20$)
Jw = joint water reduction factor ($0.05 \leq Jw \leq 1$)
SRF = stress reduction factor ($0.5 \leq SRF \leq 400$)

Table 2.1 General Procedures for applying rock mass classification systems [4]

No.	Design approach for rock tunnels
1	Classify rock (using RMR or Q-system classifications – ideally both)
2	Use RMR or Q-system design charts to make preliminary estimate of support
3	Consider overall stability and possible stress instability i. Consider in situ stress ii. Consider excavation geometry iii. Consider wedge stability
4	Consider excavation sequence and timing of support
5	Check rock-bolt design against other empirical rules
6	Draw up support and tunnel to scale in order to visualize the support and possible failure mechanisms (Note: this is difficult to do when it is a three-dimensional problem)

Table 2.2. Limitations of RMR and Q-systems [4]

RMR	Q-system
Applicable to tunnels and mines	Applicable to tunnels only
Considers orientation of joint sets	No explicit consideration of joint orientation
Provides information on stand-up time	Does not provide stand-up time
Applicable for permanent support only	For temporary and permanent support
Provides some information on lining loads	Provides some information on lining loads
Does not consider stress/strength ratio	Stress/strength ratio considered
Support chart is not up-to-date	Support chart is up-to-date
Does not consider time-dependent ground behavior or squeezing rock behaviour	Does not consider time-dependent ground behavior or squeezing rock behaviour

Bieniawski, Barton and others have suggested various correlations between RMR and other parameters. For the purpose of this study, the most applicable correlation between Q and RMR is given by Eq. 2.3 [3]:

$$Q = 10^{\left(\frac{RMR - 50}{15}\right)} \tag{2.3}$$

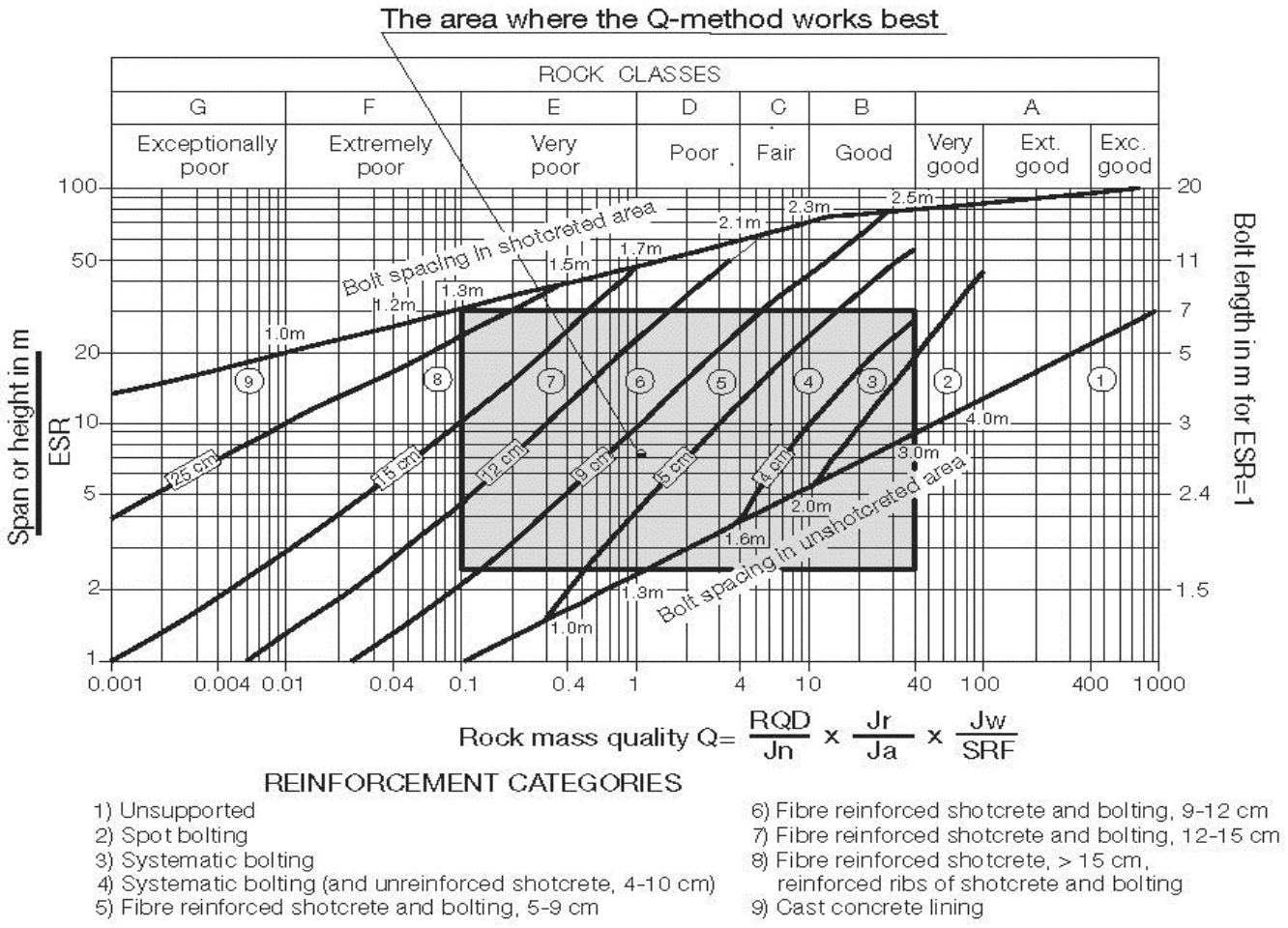


Figure 2.3. Up-to-date Q-Chart for rock support Design [5].

In the shaded area Q-method works best, i.e., the tunnel will be stable, safe and economical. Outside the shaded area supplementary methods/evaluations/calculations should be applied [5]

The RMR and Q systems have been adapted to derive a correlation between rock mass quality and rock mass modulus. While the classification is useful, care has to be taken at the low end of the RMR and Q ratings where the predicted modulus values are sensitive to small variations in the ratings. Unfortunately, these are also the ranges that are most important when evaluating the

performance of key excavations such as portals and optimizing support requirements in poor ground [4].
One of the benefits of the RMR system is that it is relatively easy to use. The result produced by the RMR classification, however, is rather conservative. This can lead to an overestimation of the support measures [5].
The strength of empirical methods generally lies in their simplicity, as well as speed and economy of use. Since the approaches are based on practical experience, they are particularly suitable for feasibility studies at the concept design stage. Also, because of the difficulty of investigating and modelling ground behaviour in complex rock masses, the averaging of rock mass properties and specifying rock support classes at the detailed design stage provides a useful and necessary basis for managing support selection during construction. During construction, the support can be chosen from the pre-designed support classes on the basis of the rock mass classification of the exposed face [2, 3, 4].

Experience over many years has shown that the methods are generally successful when implemented by experienced tunnel engineers or engineering geologists [4].

2.1.2.2. Observational Methods

Observational design methods rely on actual monitoring of ground movement during excavation to detect measurable instability and on the analysis of ground-support interaction. Although considered separate methods, these observational approaches are the only way to check the results and predictions of the other methods [8].

2.1.2.3. Analytical Methods

Analytical design methods utilize the analyses of stresses and deformations around openings. They include such techniques as closed-form solutions, numerical methods (finite elements, finite difference, boundary elements, etc.), analog simulations (electrical and photoelastic), and physical modeling [8].
Analytical design tools can be used for designing adjacent tunnels using the principle of superposition but this assumption may significantly underestimate the interaction of the tunnels, especially if the distance between the two tunnels is less than two clear diameters. Also, these models do not make any allowance for construction loads (e.g. from rams in a TBM) or the timing of support placement, with the exception of the Convergence–Confinement Method [1, 4].
In terms of the range of analytical solutions available, various methods exist for determining the stresses around a hole in an elastic or elasto-plastic homogeneous half-space. Various pressure distributions have been proposed to derive the stresses in the lining. These are of limited use in determining the loads on linings because they consider the ground alone [1, 4].

2.1.3. Tunnel Construction Methods

Tunnels can be constructed by using a number of different techniques. A useful analysis was carried out to estimate the performance of different tunnel construction techniques (TBM, roadheader and drill and blast) [5].
With most tunnelling methods the ground is mined in ‘advances’ i.e. a tunnel section is created as part of the total tunnel void in a single cycle. The section length, which is approximately 0.8m

to 4.0m, depends, amongst other things, on the stand-up time of the ground. The stand-up time being the time the void remains stable without any support. Soft ground only has a very short, or no, stand-up time. It is therefore necessary to support the void immediately. During the time that is needed for the installation of any support method, or until it takes effect, the construction method has to ensure the stability of the created void [5].

2.1.3.1. Tunnel Excavation methods

2.1.3.1.1. Drill and Blast

Drill and blast for tunnel construction can be used in geology ranging from hard rock with low strength (e.g. marl, loam, clay, gypsum, chalk) to the hardest rocks (such as granite, gneiss, basalt or quartz). Due to this large range of possible usage, drill and blast can be advantageous for very changeable ground conditions. The drill and blast work and the extent of the tunnel support can be adjusted with every heading advance if required [5].

In addition, tunnelling by using drill and blast is often preferable to TBM or road header tunnelling if, for example, the tunnel is relatively short (≤ 2km [9]) so that the high investment costs needed for a tunnelling machine are not economic, or when the ground hardness is very high so that a high wear of the cutter tools leads to an uneconomic application of the machine. Other reasons for using drill and blast for tunnelling can be when a cross sectional profile is required that differs from a circle, or when a very large tunnel profile is required which does not allow the application of a tunnel machine for technical or economic reasons. As with all tunnelling types, tunnelling using drill and blast is principally most economic when it is continuous and similar work processes are used. Therefore when planning a tunnel it is beneficial to split the tunnel into sections where the same advancing schema and the same tunnel support intensity can be used [5].

Most underground excavation in rock is still performed using drill and blasting techniques [4]. Computerized and automated hydraulic drilling jumbo are widely used. Drilling plans are stored in computer, precise positioning and navigation are by laser. Excavated profiles can be scanned for guiding next drilling and blasting (Figure 2.4 and Figure 2.5) [10].

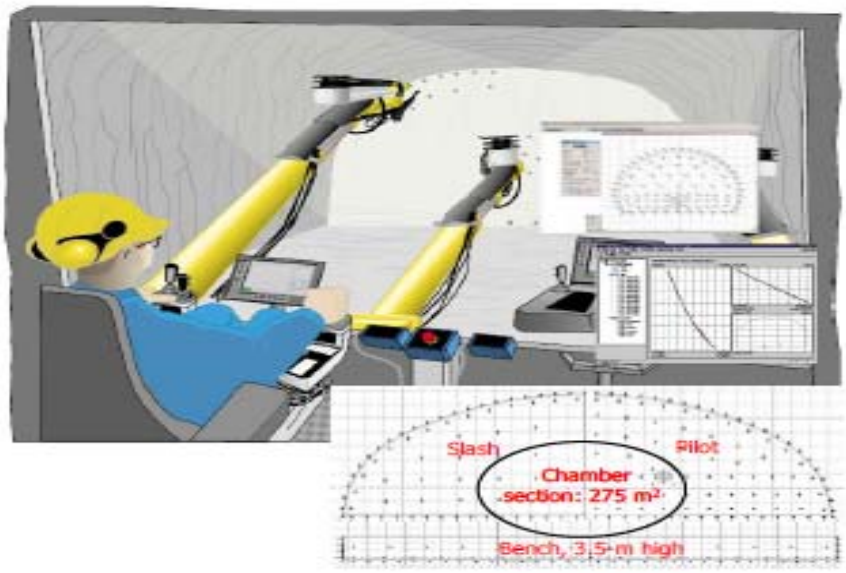
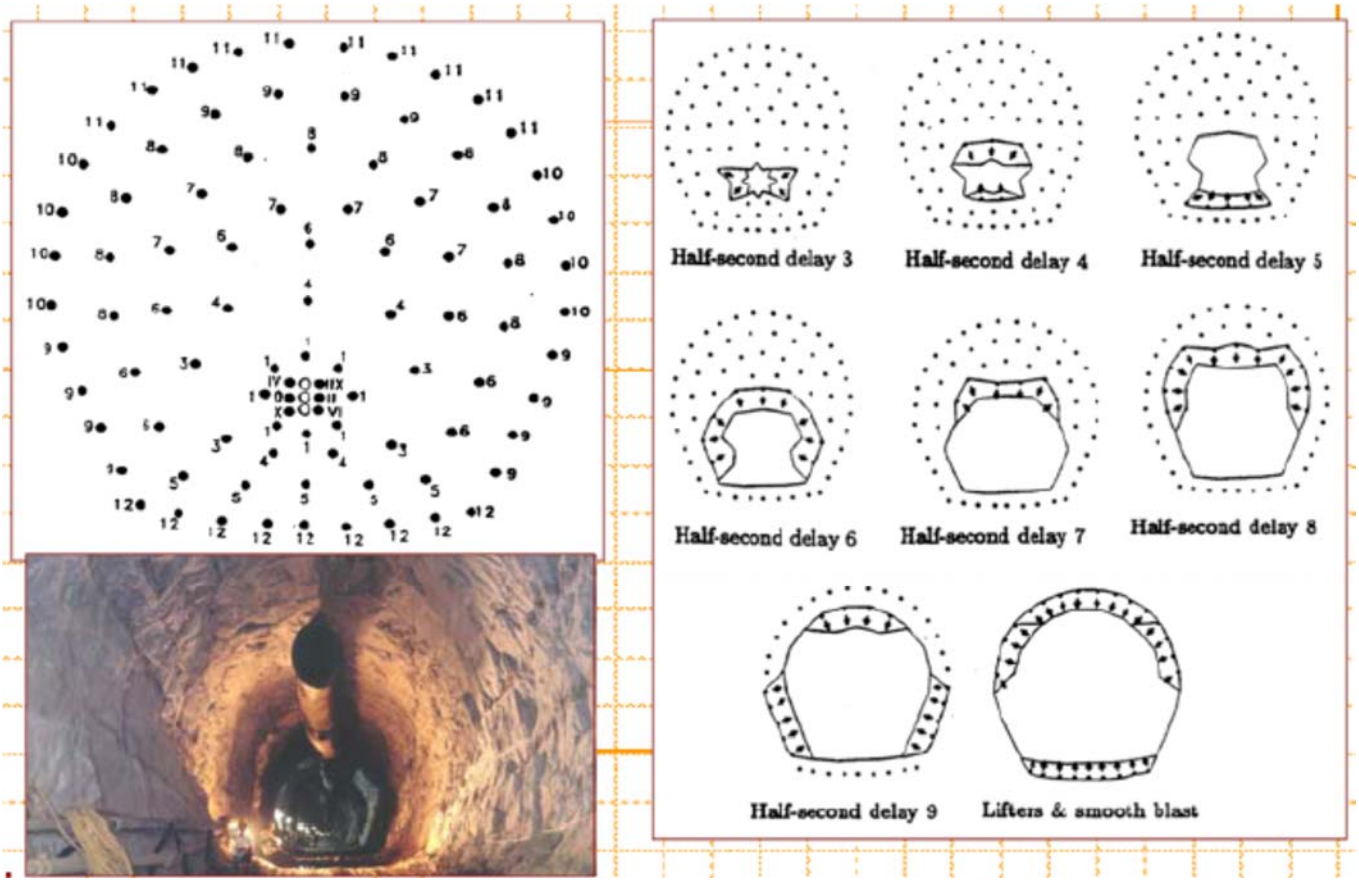
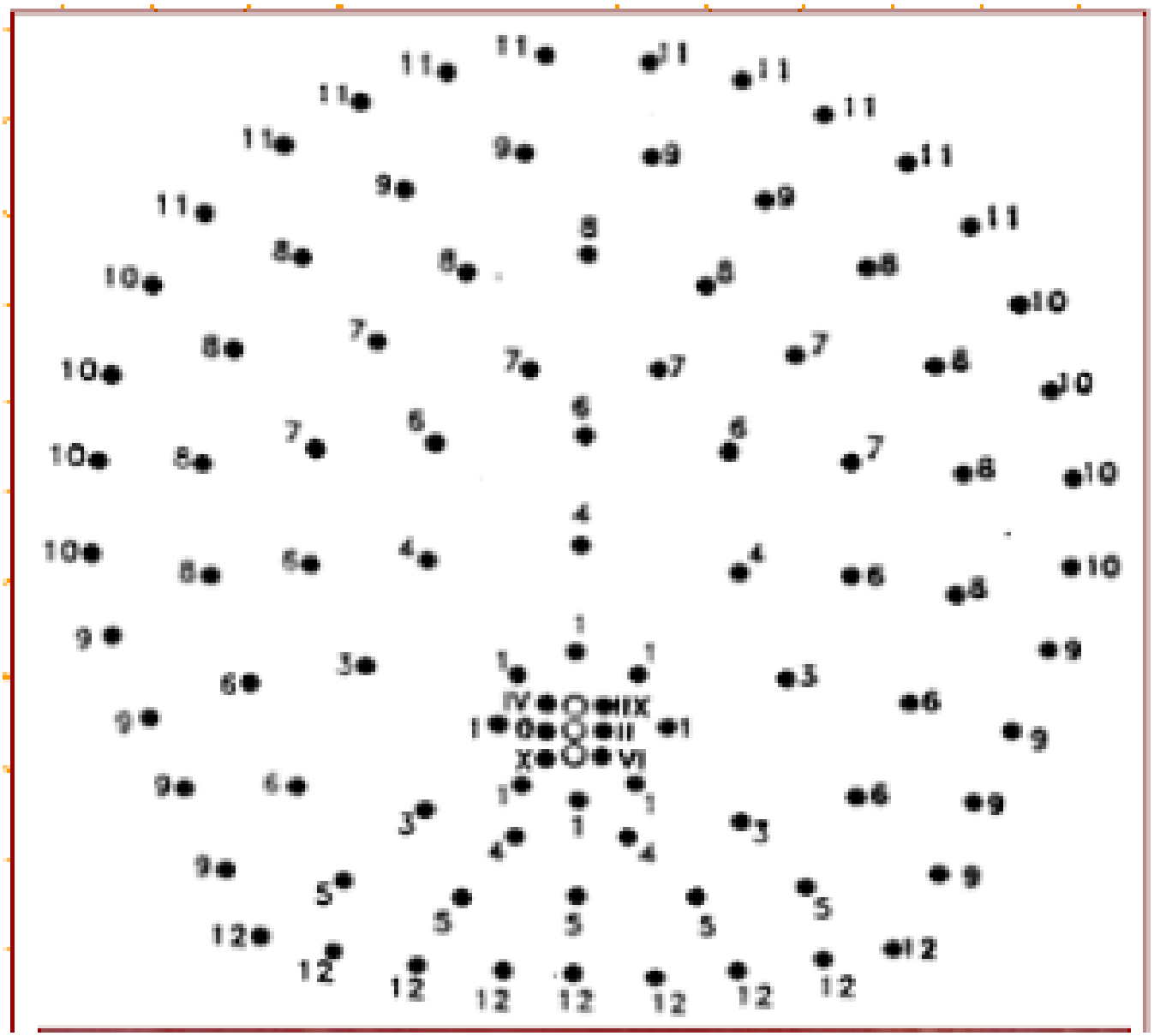


Figure 2.4 Computerized and automated hydraulic drilling jumbo [10]



(a)



(b)

Figure 2.5 Drill and Blast Pattern Design [9]

a) The excavation cycle

The typical cycle of excavation by blasting is performed in the following steps (Figure 2.6) [3, 4, 11]:

1. Drilling blast holes and loading them with explosives.
2. Detonating the blast, followed by ventilation to remove blast fumes.
3. Removal of the blasted rock (mucking).
4. Scaling crown and walls to remove loosened pieces of rock.
5. Installing initial ground support.
6. Advancing rail, ventilation, and utilities.

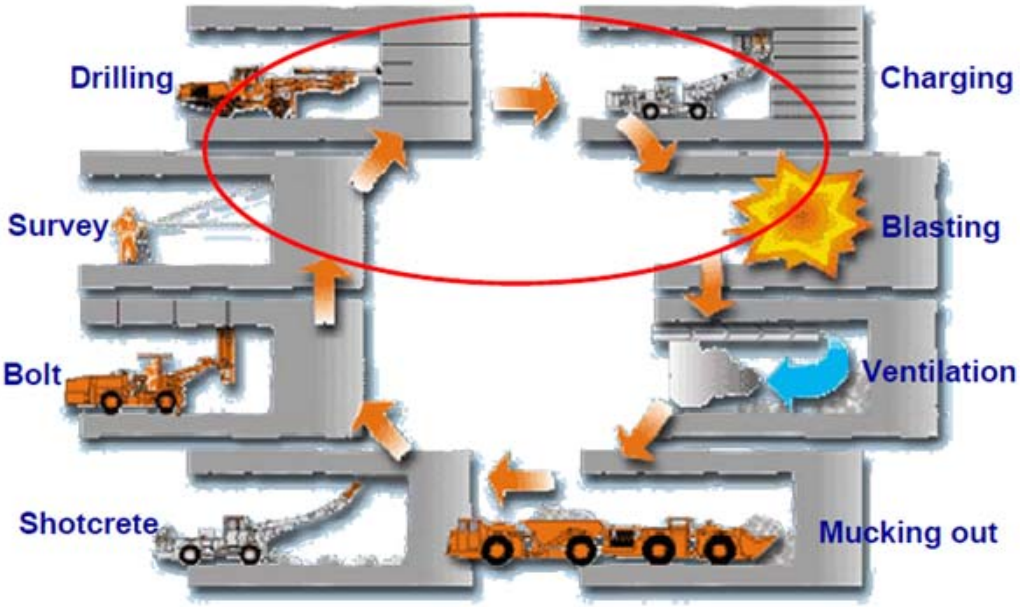


Figure 2.6 Typical cycle of excavation by blasting [10]

b) Full- and partial-face advance.

Most tunnels are advanced using full-face excavation. The entire tunnel face is drilled and blasted in one round. Blastholes are usually drilled to a depth somewhat shorter than the dimension of the opening, and the blast "pulls" a round a little shorter (about 90 percent with good blasting practice) than the length of the blastholes. The depth pulled by typical rounds are 2 to 4 m depth. Partial-face blasting is sometimes more practical or may be required by ground conditions or equipment limitations. The most common method of partial-face blasting is the heading-and-bench method, where the top part of the tunnel is blasted first, at full width, followed by blasting of the remaining bench. The bench can be excavated using horizontal holes or using vertical holes similar to quarry blasting. There are many other variations of partial-face blasting, such as a center crown drift, followed by two crown side drifts, then by the bench in one, two, or three stages.

Reasons for choosing partial as opposed to full-face blasting include the following [1, 2, 3, 4]:

- The cross section is too large for one drill jumbo for example: Underground openings of the sizes usually required for powerhouses, valve chambers, and two- or three-lane highway tunnels are usually excavated using partial-face blasting excavation.
- The size of blast in terms of weight of explosives must be limited for vibration control.
- The ground is so poor that the full width of excavation may not be stable long enough to permit installation of initial ground support.

c) Design of a blasting round

The individual blasting rounds are usually designed by a blasting specialist in the contractor's employ. The design is reviewed by the engineer for compliance with specifications [1, 2, 3, 4]. Drill and blast summarized in Figure 2.7 below.

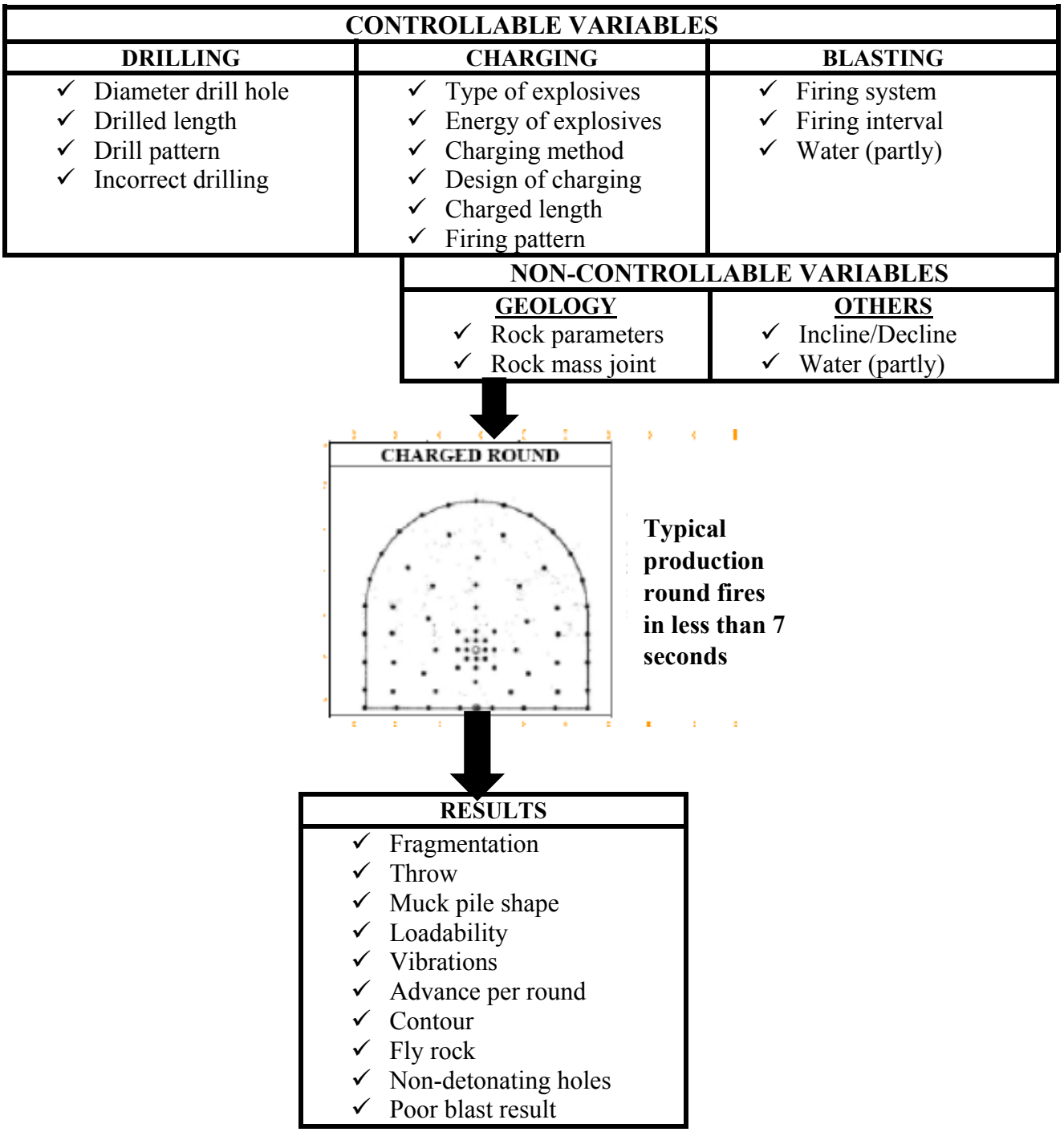


Figure 2.7. Drilling and Blasting Summary [9]

2.1.3.1.2. TBM and Roadheader Excavation

Much underground excavation today is performed by mechanical means. Tools for excavation range from excavators equipped with ripper teeth, hydraulic rams, and off roadheaders to TBMs of various designs. By far, TBMs are the most popular method of excavation. Roadheaders are versatile machines, useful in many instances where a TBM is not cost-effective. This section describes roadheader and TBM excavation methods and the factors that affect the selection of mechanical excavation methods [1, 2, 3, 4].

a) Roadheader excavation

Roadheaders come in many sizes and shapes, equipped for a variety of different purposes. They are used to excavate tunnels by the full face or the partial-face method, and for excavation of small and large underground chambers. They may also be used for TBM starter tunnels, ancillary adits, shafts, and other underground openings of virtually any shape and size, depending on rock hardness limitations. Most roadheaders include the following components [1, 2, 3, 4]:

- Rotary cutterhead equipped with picks.
- Hydraulically operated boom that can place the cutterhead at a range of vertical locations.
- Turret permitting a range of horizontal motion of the cutterhead.
- Loading device, usually an apron equipped with gathering arms.
- Chain or belt conveyor to carry muck from the loading device to the rear of the machine for offloading onto a muck car or other device.
- Base frame, sometimes with outriggers or jacks for stabilization, furnished with electric and hydraulic controls of the devices and an operator's cab.
- Propelling device, usually a crawler track assembly.

Most roadheaders can cut rock with an unconfined compressive strength of 60 to 100 MPa. The most powerful can cut rock with a strength of 150 MPa to 200 MPa for a limited duration. Generally, roadheaders cut most effectively into rocks of a strength less than 30 MPa, unless the rock mass is fractured and bedded [1, 2, 3, 4].

b) TBM Excavation

A TBM (Tunnel Boring Machine) (Figure 2.8) is a complex set of equipment assembled to excavate a tunnel. The TBM includes the cutterhead, with cutting tools and muck buckets; systems to supply power, cutterhead rotation, and thrust; a bracing system for the TBM during mining; equipment for ground support installation; shielding to protect workers; and a steering system. Back-up equipment systems provide muck transport, personnel and material conveyance, ventilation, and utilities [1, 2, 3, 4].

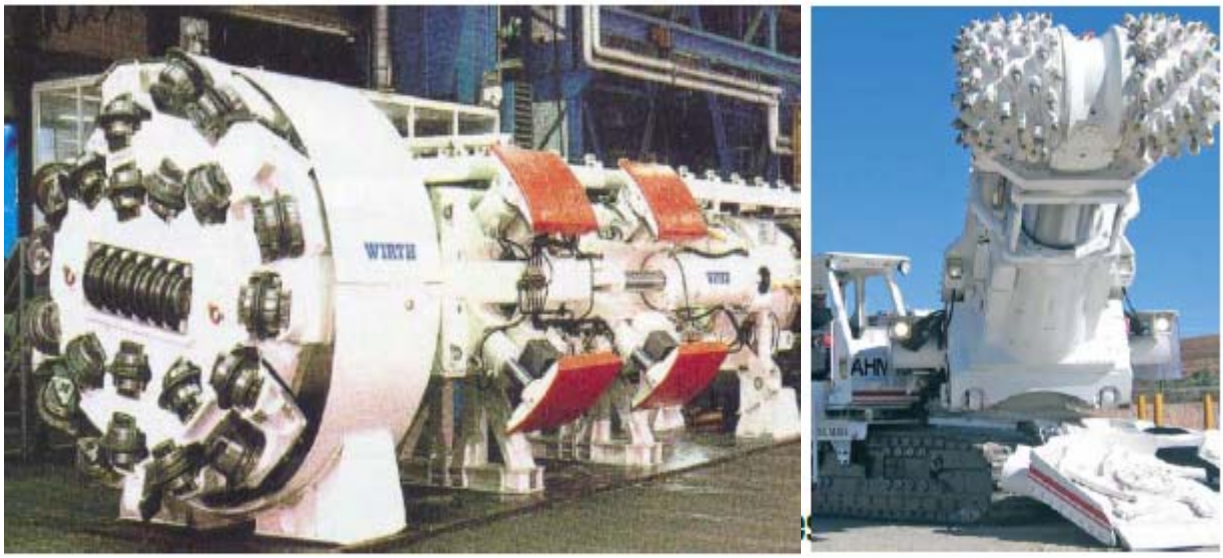
The advantages of using a TBM include the following:

- Higher advance rates.
- Continuous operations.
- Less rock damage.

- Less support requirements.
- Uniform muck characteristics.
- Greater worker safety.
- Potential for remote, automated operation.

Disadvantages of a TMB are the following:

- The fixed circular geometry.
- Limited flexibility in response to extremes of geologic conditions.
- Longer mobilization time, and
- Higher capital costs.



(a) **(b)**

Figure 2.8 Ground excavation machines (a) TBM, (b) Roadheader [10]

2.1.3.2. Temporary and permanent Linings

Most tunnels in rock, and all tunnels in softer ground, require a solid lining. The thickness of the permanent concrete lining is determined by the size of the tunnels, loading conditions, and the minimum required to embed the steel ribs of the primary lining. Loads on lining include its own weight and internal loads, weight of soil above the tunnel (submerged soil for tunnels below water level), reaction due to vertical loads, uniform horizontal pressure due to soil and water above the crown, and triangular horizontal pressure due to soil and water below the crown. Magnitude of loads on tunnel liners depends on types of soil, depth below surface, loads from adjacent foundations, and surface loads. These will require careful analysis, in which observations made on previous tunnels in similar materials will be most helpful [32].

In sufficiently strong rock permanent lining (inner lining) is not provided. Usually, however, conventionally driven tunnels obtain a permanent lining of cast in-place-reinforced concrete in

addition to the temporary lining of shotcrete, rock bolt, grouting, etc.. Benefits from inner lining are:

- Inner lining increases safety.
- A sealing membrane (if necessary) can be mounted between the outer and inner linings.
- A smooth surface of the tunnel wall is advantageous from the point of view of aerodynamics (ventilation) and illumination.

In the case of segmental linings (as used in shield driven tunnels), there is usually no inner lining of cast concrete [2].Permanent linings are required in many tunnels for two purposes [4]:

Structural: to support the exposed ground thus providing and maintaining the required operational cross-section and, if required, to provide a barrier to the passage of liquids.
Operational: to provide an internal surface and environment appropriate to the functions of the tunnel.

The chosen lining must be capable of safe and economic construction and in most cases be adaptable to varying conditions encountered during the works [4].

In order to begin to design any tunnel lining it is important to know and understand the functional requirements that the lining needs to achieve (Table 2.3). There can be a wide variety of requirements, which are influenced by many factors. A tunnel lining is fundamental to most underground construction projects, usually to enable the underground space to be used as required. Of paramount importance to this is the long-term integrity of the tunnel structure, which is totally dependent on the serviceability of the lining. The major requirements of the lining may be summarised as follows [4].

- Operational-usually determined by the owner/operator and dependent on the purpose of the tunnel and how it is to be operated.
- Serviceability-includes the anticipated design life and the owner/ operator's view on initial capital cost versus both longer-term maintenance and shorter-term issues such as fire resistance.
- Environmental -Including external influences from the surrounding environment, such as leakage, chemical and temperature effects, as well as the effects of the constructed tunnel on the surrounding environment, such as those from noise, vibration, changes in the groundwater regime, settlement and appearance.

Risk factors will also influence the determination of the form and detail of a tunnel lining. These factors will all play a part in defining the project requirements under which the tunnel lining will be designed, and will influence many of the technical decisions that have to be made throughout the design process [4, 8].

Table 2.3 sets out some of the significant spatial and loading constraints that need to be considered for linings for rail, road and utility tunnels [4].The principal functions for which tunnels are required fall into the following categories [4]:

1. Mining,

2. Military,
3. Transportation: road, rail, pedestrian, canal,
4. Utilities: water supply, sewerage, irrigation cables and piped services, hydroelectric power, cooling water,
5. Storage and plant: power stations, liquid storage (water, oil), gas storage, waste storage (e.g. radioactive).
6. Protection: civil defence shelters.

Table 2.3. Major spatial and loading considerations for tunnel lining design for rail, road and utility tunnels [4], M = Machinery & E = Equipment

	RAIL	ROAD	UTILITIES
Space	❖ Structure gauge/kinetic envelope ❖ Track and track-bed ❖ Access/escape walkway ❖ Ventilation equipment, services and M & E ❖ Electrical clearances ❖ Aerodynamics (cross-sectional area and roughness)	❖ Swept envelope ❖ Roadway ❖ Escape route ❖ Ventilation services and M & E ❖ Aerodynamics	❖ Capacity (volume, section, grade) ❖ Hydraulic shape ❖ Access (benching/ walkway) ❖ Roughness
Loading	❖ Internal fixing for: • Rails • Walkway • M & E • Signals • Telecommunications • Overhead Catenary System ❖ External: • Ground • water ❖ Adjacent tunnels ❖ Shape tolerances ❖ Temporary construction loads ❖ External influences from property development (surcharge loading, piles and foundations) ❖ Reduction from dredging of water crossings ❖ Long-term creep	❖ Internal fixings: • Roadway • Walkway • M & E • Traffic controls • Secondary cladding and ducting Signage ❖ External: • ground • water ❖ Adjacent tunnels ❖ Shape tolerances ❖ Temporary construction loads ❖ External influences from property development (surcharge loading, piles and foundations) ❖ Reduction from dredging of water crossings ❖ Long-term creep	❖ Internal pressures (surcharge) ❖ External • ground • water ❖ Adjacent tunnels ❖ Shape tolerances ❖ Temporary construction loads ❖ External influences from property development (surcharge loading, piles and foundations) ❖ Reduction from dredging of water crossings ❖ Long-term creep

In tunnels where saline intrusion is present or in railway tunnels where earth current leakage or induced currents can set up electrical cells within the reinforcement, corrosion can be particularly severe. In those circumstances it may be necessary to increase concrete cover to reinforcement or consider alternatives such as higher specification linings without steel reinforcement, coated reinforcement, and fibre reinforcement or, in the extreme, cathodic protection [4].

2.1.3.2.1. Lining design requirements

Because of the uncertainties surrounding the properties of the ground and the loads on the lining, it is important to understand that tunnel lining design is not a straightforward deterministic process. A design analysis should therefore be regarded as an attempt to provide an indication of the behaviour of the tunnel lining system, rather than the definitive solution. The general design procedures for achieving this are summarised in the ITA design process model in Figure 2.1 [4].

The ground will usually have the greatest influence on the loads. Very often geotechnical interpretative reports (GIR) provide a good basis on which to judge ground behaviour. Information from other tunnelling in similar ground is a valuable addition to the results of site investigations. Furthermore, the precision of the available analytical tools is much greater than the reliability and accuracy of the data obtained from site investigations [1, 4].

In the context of tunnel lining design, instrumentation and monitoring (I & M) is used to assess the pre-construction ambient state, the geotechnical parameters required for design, for performance monitoring during construction, for design verification, and for long-term post-construction monitoring [4].

The design of a permanent tunnel lining solution is influenced by the full range of project-specific operational requirements, i.e. not only the ground loading and water control. These include the electrical and mechanical operation and maintenance and safety aspects such as resistance under fire [1, 2, 3].

Design issues that need to be considered are [1, 2, 3]:

- Functionality, for example lining and material type and lining thickness, material properties, constructability;
- Durability, for example corrosion, adverse chemical reactions, fire;
- Appearance, for example the effect of water, cracking, deformation, surface texture.

External loading: Loadings under which a tunnel lining will be required to operate will depend largely on the tunnel use. Primary external ground and groundwater loads such as surcharge from buildings, foundations, piles and adjacent tunnels may need to be considered as well as possible accidental load cases from possible explosions or earthquakes and other seismic disturbances. Even reduction of loading in the long term from dredging operations or the like may need to be considered [4].

Internal loading: internal loads will also need careful consideration and these can be either permanent or transient. Some of these are likely to be relatively small by comparison with

external loading but may need early consideration. Accidental load cases may also need to be considered such as explosions as well as temporary construction loads from possible internal compressed air. Means of fixing, for example, need to be considered, as many tunnel owners do not allow post drilling of tunnel linings. Internal pressures in water storage and transfer tunnels need to be particularly assessed as they are likely to have a major influence on the variance of loading in the lining as well as influencing the detailing of watertightness both internally and externally [4].

2.1.3.2.2. Tunnel Excavation support options

The purpose of an initial support (sometimes called temporary lining, or temporary support of excavation) in rock tunnelling is to keep the opening open, stable and safe until the final lining is installed and construction is complete. As a consequence the initial support system in a rock tunnel can be one or a combination of a number of options [3].

From a tunnel design point of view, a rock mass is considered to be weak when its in-situ uniaxial compressive strength is significantly lower than the natural and excavation induced stresses acting upon the rock mass surrounding a tunnel. As a very approximate and simple estimation, Figure 2.9 can be directly used to predict squeezing potential by comparing rock mass strength (σ_{cm}) and in-situ stress (p_o). If finite element (FE) analysis results are available, one can simply predict the squeezing potential based on the calculated strains from the FE analysis. It should be noted that strains in Figure 2.9 are based on tunnels with no support installed [3].

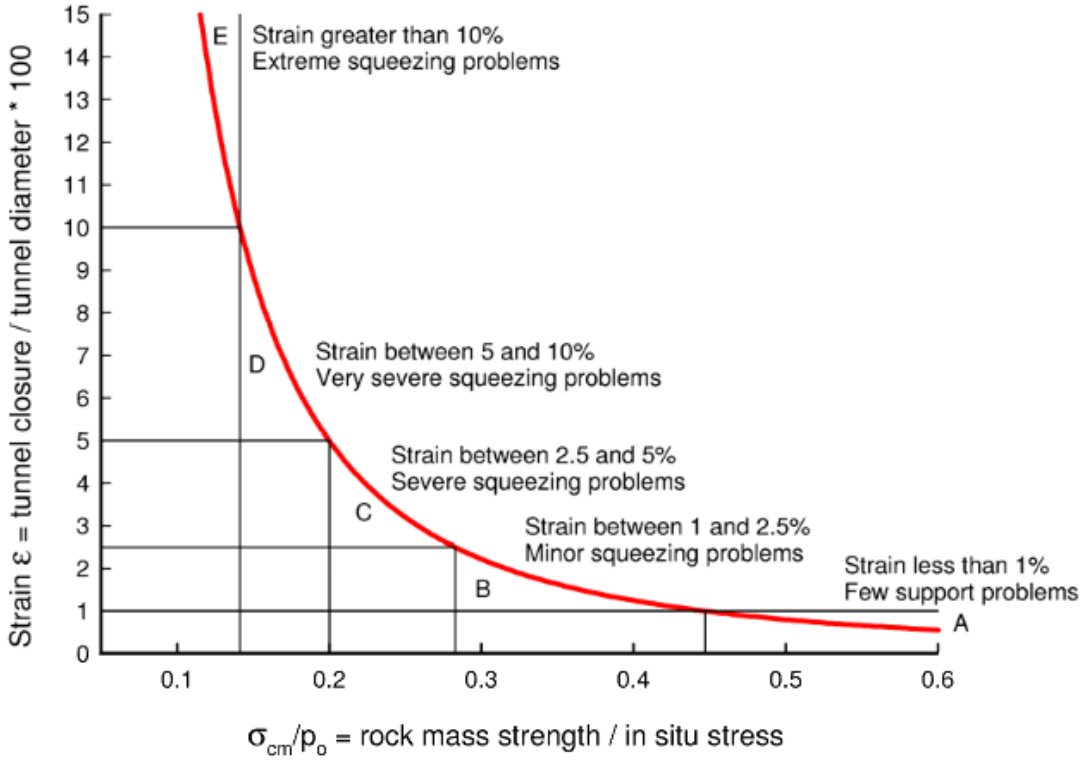


Figure 2.9. A Relationship between strain and Squeezing Potential of Rock Mass [3]

The principal objective in the design of an underground excavation support design is to assist the rock mass to support itself. The basic philosophy in tunnelling and rock engineering is that the extent of rock supports should reflect the actual rock condition. During excavation, at the working face, temporary supports are installed to ensure safe working conditions. Generally rock bolts and shotcrete are used for it. The permanent supports are generally installed after the excavation, behind the working face [12].

Ground support in a rock tunnel may be supplied by ribs and lagging or by shotcrete in combination with rock bolts or dowels, steel sets, lattice girders, wire mesh or various types of reinforcement mats. For the most part modern rock tunnels are supported by shotcrete and either rock bolts or lattice girders. Either system provides a flexible support that takes advantage of the inherent rock strength but that can be stiffened simply and quickly by adding bolts, lattice girders and/or shotcrete. In addition, lattice girders provide a simple template by which to judge the thickness of shotcrete. For other situations wire mesh or reinforcement mats have proven to successfully arrest and hold local raveling until sufficient shotcrete can be applied to knot/bond the whole system together and hold it until the shotcrete attains its strength[3].

a) Rock bolts

Bolt and anchor lengths depend on the dimensions of the excavations. Lengths used in the roof arch are usually related to the span, while lengths used in the walls are usually related to the height of the excavations. The ratio of bolt length to span tends to reduce as the span increases. Accordingly, the following recommendations are given as a simple rule-of-thumb, to be modified as in situ conditions demand [24].

Roof:	Bolts	$L=2+\frac{0.15B}{ESR}$	(2.4)
	Anchors	$L=\frac{0.40B}{ESR}$	(2.5)
Walls:	Bolts	$L=2+\frac{0.15H}{ESR}$	(2.6)
	Anchors	$L=\frac{0.35H}{ESR}$	(2.7)

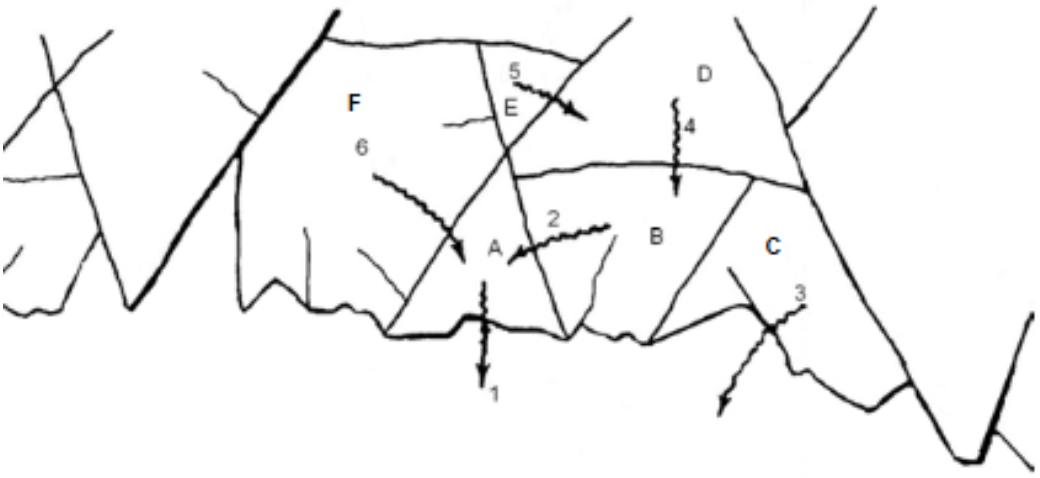
Where L = length in meters
B = span in meters
H = excavation height in meters
ESR = excavation support ratio

There are two types of rockbolt applications in tunnelling (Figure 2.10 and Figure 2.11) [2]:

- Spot bolting: individual rockbolts are placed to stabilise isolated blocks,

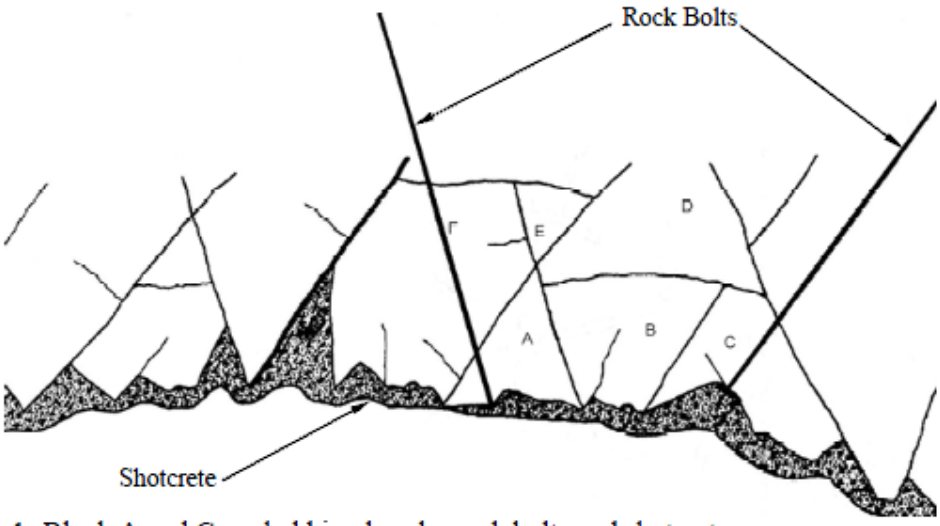
- Pattern bolting: systematic installation of a more or less regular array rockbolts. Despite some rational approaches, the design of pattern bolting is empirical.

In unstable rock, bolts should be installed immediately after drilling of the hole. In so doing, the long established safe practice of working forward from solid or secured ground towards unsupported or suspect ground must be adhered to [2].



Step 1- Block A drops down
Step 2- Block B rotates counterclockwise and drops out
Step 3- Block C rotates counterclockwise and drops out
Step 4- Block D drops out followed by block E
Step 5- Block E drops out followed by block F
Step 6- Block F rotates clockwise and drops out

Figure 2.10. Progressive Failure in Unsupported Blocky Rock [3]



Step 1- Block A and C are held in place by rock bolts and shotcrete
Step 2- Block B is held in place by Blocks A and C
Step 3- Block D is held in place by Blocks A, B, and C
Step 4- Blocks E and F are held in place by Blocks A, B, and D assisted by rock bolts and shotcrete

Figure 2.11. Prevention of Progressive Failure in Supported Blocky Rock[3]

b) Shotcrete

Shotcrete is simply concrete sprayed in to place through a nozzle. It contains additives to gain strength quicker and to keep it workable until it is sprayed. Shotcrete can be made with or without the addition of reinforcing fibres and can be sprayed around and through reinforcing bars or lattice girders. Both the quality and properties of shotcrete can be equal to those of cast in place concrete but only if proper care and control of the total placement procedure is maintained throughout (Figure 2.11) [3].

What distinguishes shotcrete (or 'sprayed concrete') from cast concrete is not the strength of the final product but the process of its placement. In tunnelling, shotcrete is applied (i) to seal freshly uncovered surfaces (in thicknesses of 3 to 5 cm) and (ii) for the support of cavities [1, 2, 3, 11]. The characteristics of sprayed concrete (shotcrete) are almost the same as those of usual concrete [2].

Steel Ribs

Support systems based on steel ribs have been used for many decades. This technique involves rolled steel sections being placed around the circumference of the excavated tunnel profile at specified intervals. It is inevitable that there will be gaps between the steel ribs and the ground and it is important that these gaps are suitably wedged to prevent excessive deformations. The importance of ensuring that the loads carried by the steel supports are evenly distributed around the tunnel profile is well recognized. Point loading of the steel supports significantly reduces their ultimate load bearing capacity.

It is common these days to combine steel ribs with sprayed concrete (Figure 2.12). If a layer of sprayed concrete is applied prior to erecting the steel ribs, this helps to overcome some of the problems with wedging. A subsequent layer of sprayed concrete is then applied to integrate the steel ribs into the lining and provide additional stability.

Figure 2.12. Steel ribs with wire meshes and shotcrete [13]

The legs of the steel arches are often set into concrete blocks to help distribute the loads into the ground and prevent settlement. Lattice girders, rather than rolled steel sections, combined with sprayed concrete are also commonly used these days. Figure 2.13 shows the mesh being installed ready for shotcreting, in combination with lattice girders [1, 2, 3].

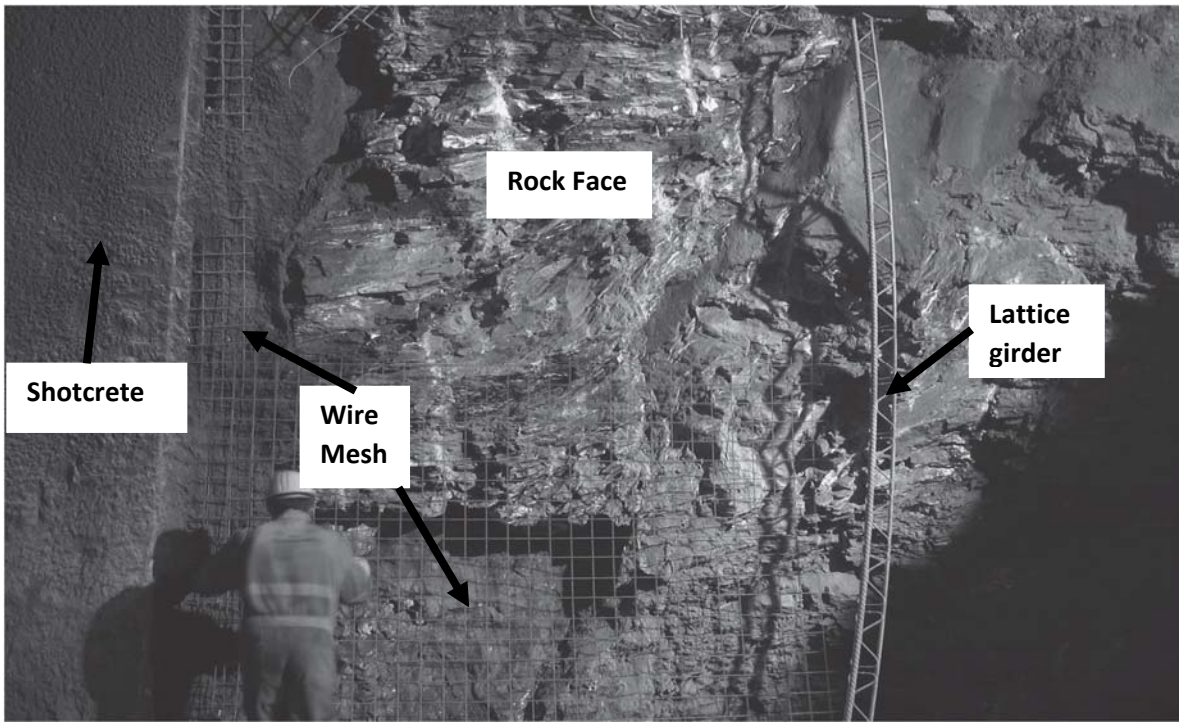


Figure 2.13. Mesh installation and a lattice girder arrangement [5]

c) Cast-in-place reinforced concrete linings

In situ concrete linings can either be used in self-supporting ground (for example rock) as the main lining, or as a second-stage lining where a temporary support system has already been placed during the excavation process (for example steel arches, sprayed concrete or rock bolting). In both cases the lining is cast in situ using a formwork system, which provides a gap between the ground, or initial support system, and the formwork into which wet concrete is placed (Figure 2.14). The concrete lining can either be plain or reinforced. Once the concrete has reached a suitable early stage strength the formwork is 'struck', i.e. removed. In 'wet' ground, either a waterproof membrane is used between the initial support system and the cast in situ lining (Figure 2.14), or a watertight cast in situ lining can be used [5].

These linings are often used with 'system formwork' where travelling steel or wood forms are advanced, often as separate 'invert' and 'arch' forms, in tunnels with a suitable length of regular cross section and where the operation can be developed around a 24 hour cycle. Although expensive, 'system formwork' can become economically viable with extended use (Winter 2006). One benefit of using formwork is that it can be built as required to any shape and it is therefore highly adaptable. Thus it can be used in tunnels where there are junctions or tunnels of varying cross section [5].

Figure 2.14. Cast-in-place concrete Construction of the inner lining [5]

Disadvantages of a cast-in-place concrete lining are as follows [3]:

- Concrete placement, especially around reinforcement can be difficult. The nature of the construction of the lining restricts the ability to vibrate the concrete. This can result in incomplete consolidation of the concrete around the reinforcing steel.
- Reinforcement when used is subject to corrosion and resulting deformation of the concrete. This is a problem common to all concrete structures, however underground structures can be also be subject to corrosive chemicals in the groundwater that could potentially accelerate the deterioration of reinforcing steel.
- Cracking that allows water infiltration can reduce the life of the lining.
- Chemical attack in certain soils can reduce lining life.
- Construction requires a second operation after excavation to compete the lining.

d) Grouting:

Grouting is required to ensure that the in situ lining is in full contact with the initial ground support and ground. Bleed pipes will be required to ensure that air does not become trapped during grouting or concreting. The grouting pressure should be limited so that it does not damage the new lining, and the grouting arrangement should be compatible with the waterproofing design [1, 3, 4].

The purpose of grouting is to fill the annulus around the tunnel lining and achieve the following:

- maintain the tunnel ring shape
- distribute ground pressures evenly onto the lining while maintaining stability
- reduce seepage/ground water inflow
- Limit surface settlement.

It is necessary to consider the proposed annulus grouting system and method during the detailed design of the tunnel lining.

e) Steel meshes

Steel meshes (Figure 2.13) (mesh size ≥ 10cm*10cm, Ø< 10 mm, concrete cover ≥ 2 cm) are manually mounted and should, therefore, be not too heavy. A usual weight is 5 kg/m^2. Mesh installation is labour intensive and relatively hazardous, as the personnel are exposed to small rock falls. For drill & blast heading the mesh adjacent to the face (proximity < 1 m) can be damaged by the subsequent blast [2, 5].

CHAPTER 3. HYDROPOWER TUNNELS IN ETHIOPIA

3.1 Introduction

Tunnelling is new in Ethiopia and has been recently used in hydropower projects. Till now, tunnels have been built in six hydropower projects such as Fincha hydropower, Beles hydropower, Tekeze hydropower, and the three Gilgel Gibe hydropower projects. In this chapter, the Gilgel Gibe I, II, III and Beles hydropower tunnels will be reviewed briefly. The construction method used in the GG I and GG III is drill and blast whereas TBM excavation method used in the excavation of GG II and Beles hydropower tunnels.

3.2 Gilgel Gibe I Hydropower Tunnels

Gibe I is located at about 260 km south-west of Addis Ababa on the road to Jimma town. Gibe I headrace tunnel was constructed by Conventional drilling and blasting method and its length is about 9300m. The Powerhouse was constructed underground, in a 68,000m^3 Cavern, 98m long, 38m high and 18m wide, excavated inside the massive limestone rock on the right bank of the river [17].

The construction methods, techniques and procedures of the GG I hydropower tunnels are the same to that of the GG III hydropower tunnels. Therefore, the details are discussed in section 3.5 and in chapter 4.

3.3 Gilgel Gibe II Hydropower Tunnels

a) Location

The Gibe II is located at 240 km south-west of Addis Ababa, coordinates **(**7°45′25″N, 37°33′44″E). The project channels the water discharged from the Gilgel Gibe I dam through a 26km long tunnel and a steep drop directly to the valley of the Omo River [18].

b) Design

The Gilgel Gibe II consists of a power station on the Omo River that is fed with water from a headrace tunnel and sluice gate on the Gilgel Gibe River. The headrace tunnel runs 26 km under the Fofa Mountain and its internal diameter is 6.98m, and at its end, it converts into a penstock with a 500 m hydraulic head/drop. The design discharge was 100m^3/s and the design internal pressure was minimum 2 bar and maximum 7 bar.

As shown in Figure 3.1, the tunnel has a diameter of 6.98 meters and extends for 26 kilometers and is designed to withstand a maximum pressure of 7 bar (0.7 MPa). The tunnel is buried deep in the ridge formed by Fofa Mountain at depths varying from 300m to 1.3km at its deepest section [18].

The ridge through which the tunnel is dug is made up of a non-uniform rock formation with several types of rocks with the dominant ones being basalt on the west side followed by trachyte and rhyolite on the east side of the tunnel. As the figure shows, the ridge is intersected by numerous faults generally inclined in the east direction [17].

i. Main summary of ground Investigations

- The rock mass rating (RMR) was between 17 and 19.
- The rock class V - very poor.
- No ground water was encountered but the rock was only damp.
- The temperature was varying from 42ºc to 53ºc.

Five rock formations along the tunnel alignment. Mainly tertiary volcanic rocks: rhyolite, trachyte, basalt:

- Uniaxial compressive strength of intact rock (σ_c) = 120 MPa
- Number of joints per meter = 6
- Orientation of discontinuities with respect to tunnel axis = Oblique
- Stand up time >48 hrs
- Ground water inflow at tunnel face = 0 liters /sec [17].

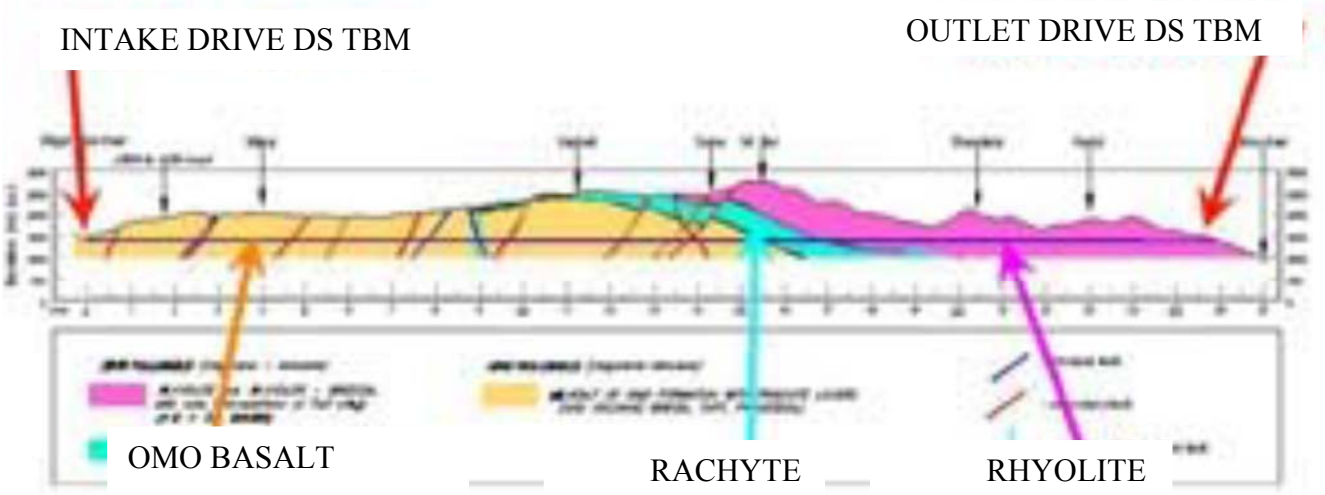


Figure 3.1. Gilgel Gibe II geological profile [30]

ii. Geometrical Characteristics of Lining [17]

- Segment Type …….Hexagonal 4 pieces
- Segment Thickness ……..……..25 cm
- Segmental Lining Outside Diameter ……..……....6,800 mm
- Segmental Lining final internal diameter……6,300 mm
- Segment Width ……..……..1,600 mm.

c) Challenges and Remedial Measures of the collapsed section

The first failure happened, during the construction of the tunnel that was started in June 2005. In October 2006 the tunneling crew hit a pocket of wet earth along a major fault line only 4 km heading. Wet and humid mud under a pressure of as much as 40 Bars and with temperatures reaching 40ºC gushed out. This immense pressure which is almost 6 times the maximum pressure of 7 Bars that the tunnel was designed for not only damaged the tunnel itself and its linings but also the 255 ton TBM. A summary of the failures as reported by the tunneling engineering consultant read as follows: **'**Event at the chainage 4+196 from intake heading. At the end of October 2006, the TBM was pushed back as consequence of the sudden extrusion and collapse of the tunnel face against the cutter head and the front shield. The tunnel face moved towards the TBM 40-60 mm/hour. The TBM has been pushed back more than 60 cm and displaced laterally more than 40 cm. As consequence, severe damages occurred to the shields, the cylinders and the last 7 segment rings installed behind the TBM.'' [19]

Figure 3.2. Image of the damaged section of Gibe II tunnel that failed in October 2006 under pressures that exceeded the design pressure by six times [19]

The second failure occurred in June 2007 resulting in the collapse of the front face of an exploratory adit (access tunnel) which eventually filled 80m of the main tunnel with mud. Further, in August 2007, unexpected fault line was crossed during excavation; but with no reported damage at that time.

i. Remedial Measures:

The solutions to these series of unforeseen events involved [19]:

1. Building a new 230meters of bypass tunnel,
2. Changing the original layout (direction) of the tunnel,
3. Dredging as much as 40,000 m^3 of mud continuously for 2 years (between October 2006 – August 2008),

4. Drilling of 1,600m of drainage/exploratory holes [17], and
5. Filling out the failed section of the tunnel with concrete.

On December 2007, monitoring measurements have shown lowering of the rock stresses, while the right exploratory adit was crossing the fault zone [17]

ii. Further Remedial Measures

- Completion of the back chamber through a concave shot-concrete wall, reinforced with horizontal steel ribs HEB200;
- Construction of a new assembly chamber and TBM launching chamber;
- Segmental lining dismantling and casting of a concrete plug in the power tunnel;
- Resuming of the intake drive excavation along a new alignment at Chainage 3+805.

To facilitate TBM steering at the excavation resuming and to reinforce the pillar between the two tunnels, in the transition zone the old tunnel has been filled with concrete. The TBM has been refurbished and re-assembled in the tunnel [17].

The excavation diameter was enlarged from 6980mm to 7074mm peripheral cutter housings (from n°37 to n°44) were re-positioned.

These remedial measures, constituted significant departure from the original design. [19]

The success of the intervention was possible only after the releasing of the pressure and the stresses acting in the area and surrounding the TBM. Large mud/water inflows, unstable faces of ravelling/running and blocky ground, high rapid convergences, high ground loads, and very hot water and gas inflows were conditions faced by the crews of two double shield tunnel boring machines (DS TBM) working towards each other from the portals. After starting in October 2006 the machines met in June 2009 and the tunnel was watered-up in September 2009 [20].

iii. Tunnel Collapse after Inauguration and repair

In January 2010, about 8,500m^3 of material, ranging from silt and mud to blocks of rock up to 2-3m wide, crashed through a 15m long section of the 8.10m diameter TBM driven segmentally lined tunnel, bringing operation of the facility to a standstill just three months after inauguration [20]. About 1.3 km of the tunnel was inaccessible because of the rocks that blocked passage and trapped the water. It was also stressed that a large quantity of rock and mud fall into the tunnel [19].

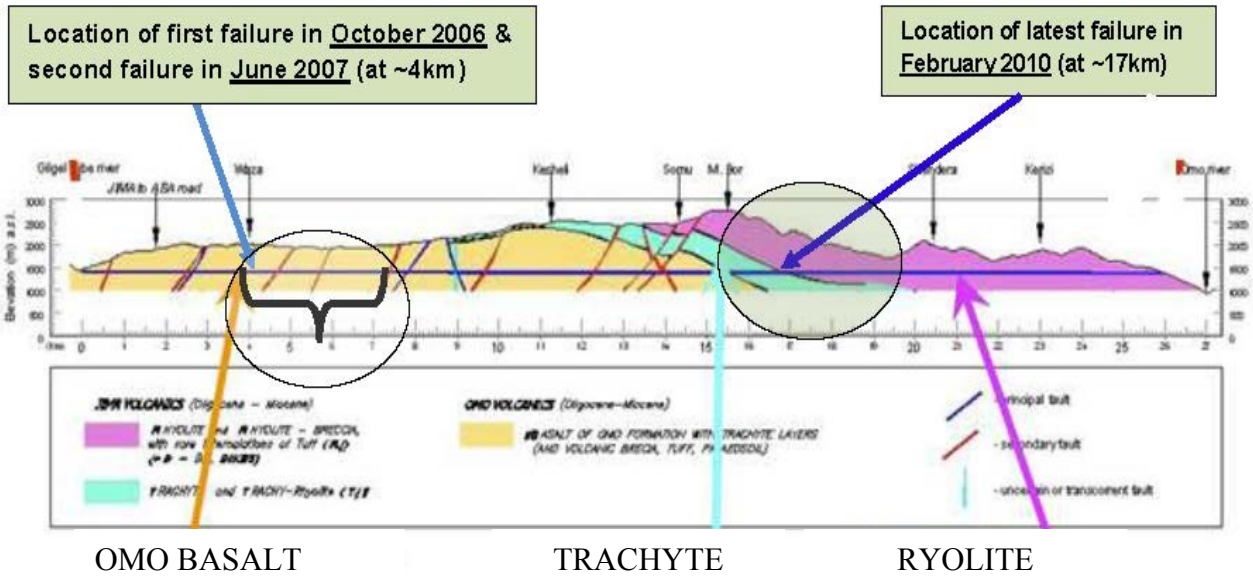


Figure 3.3. Cross-section of the tunnel showing multitudes of fault lines. Later failures showed that there were still more fault lines unforeseen at the beginning of the project [19].

The latest failure occurred three and half-years after the first series of failures. In the words of the tunneling consultant engineer, "the cause was extremely high rock/mud pressure of a major fault associated with loose cyclopic dolerite blocks and round boulders. One possibility is that the faulty ground (high pressure mud mix with round boulders) in this area was separated from the tunnel by a relatively thin diaphragm of a fractured dolerite dike. When mud pressure was able to break the diaphragm, a sudden concentrated and dynamic load of mud and blocks acted on the tunnel section causing the damage." There is no possibility, and need according to engineers, to drive a lateral access [19].

Recovery operations were divided into three phases - drilling of geological investigation boreholes; clearing of the collapsed material from the tunnel; and reconstruction of the waterway headrace tunnel [20].

The 26km long TBM driven through complex geological formations of the Great Rift Valley region of Africa is recognized as one of the most difficult TBM tunneling projects on record [20].
According to SELI engineers (sub-contractor for tunnel excavation), the poor rock conditions that broke into the tunnel subsequent to inauguration were not crossed or detected during excavation. When the TBM passed through, the rock was "hard but fissured" and only blocks falling from the face and damaging some of the cutters interrupted good average performance. Man-entry inspection of the tunnel after dewatering confirmed that the inrush of debris created a collapse hole that extends about 30m-40m up into the rock mass overhead. A 20m length of collapsed tunnel plus a further 10m of damaged lining on the downstream outlet side was in need of repair [20].

"The reason for the collapse is the suspected presence of a major fault, perhaps an old river bed, hidden behind a thin diaphragm of hard but fissured rock. The pressure of this fault - and remember we measured 40 bars during the first geological event under much less cover - must have been so big as to break the rock diaphragm and apply a high concentrated load on a few segments" causing them to crush [20].

After a major tunnel inundation event during excavation in 2006, the recent collapse event occurred at chainage 8.980 from the outlet portal and three months into operation of the tunnel.

3.4 Beles Hydropower Tunnels

a) Location

The Beles hydropower plant is located in the Amhara region, about 150km from the town of Bahir Dar on the south-western bank of Lake Tana.

b) Design

The Beles Hydroelectric Power Plant receives water from Lake Tana where it is transferred to a power station through 12km headrace tunnel and then discharged through 7.2km tailrace tunnel and finally into the Beles River. This is first accomplished by an inlet on Lake Tana, where the power station can utilize $9.12x10^9m^3$ water of the lake's volume for power production. The inlet channel is 43 m wide, 11.5 m high and its flow into the headrace tunnel is controlled by five floodgates. The headrace tunnel transfers the water to the southwest along its 12 km length within its diameter of 8.1 m. At the end of the headrace tunnel, it converts into a 6.5 m diameter and 270 m long penstock before reaching the power station. At the power station, water is delivered to four Francis Turbines powering four 115 MW generators. The power station is an underground cavern type and is 82 m long, 17.6 m high and 38.5 m wide. It has a 91.2 m deep and 8 m diameter surge/pressure shaft as well. Once the water is used in hydroelectric production it is then discharged from the power station to the Beles River via a 7.2 km tailrace tunnel with the same diameter as the headrace. Normal water level at the inlet is 1,800 m above sea level and the power station resides at 1,450 m, allowing for 350 m of hydraulic head [21].

c) Construction

The double shield earth pressure balanced (EPB) dual-mode tunnel boring machine (TBM) on the Beles headrace was able to cope with different geological formations and conditions by adapting its configuration and operational mode to the ground characteristics and behaviour.
The double shielded TBM broke through on November 8, 2009. The finishing works completed by January 2010.

Figure 3.4. Beles Headrace Breakthrough, Nov. 8/2009[21]

The tunnel ran through a variety of ground formations with about 10km passing through hard granite rock of up to 350MPa in compressive strength and intersected by local faults, and another 1.8km of loose soils, basically lake deposits, through which the machine advanced with a daily output of about 15-20m. The tunnel is lined with 30cm-thick precast concrete segments in 6+1 configuration and the annulus backfilled with pea-gravel and mortar.
Technical data of the Beles headrace tunnel TBM:

- Boring diameter: 8.10m
- Cutters: 132x43cm (52 x 17in)
- Max. Cutterhead thrust: 11.440kN
- Cutterhead speed: 6 rpm
- Cutterhead drive power: 2.100kW

The 7.2km long tailrace tunnel was excavated using a second 8.07m diameter double shielded TBM. It too was lined with an open type parallel ring rhomboidal segmental system with a pea gravel and mortar annular fill. Excavation of the tailrace began in June 2007 and was completed in May 2008, after less than 12 months of TBM progress and to achieve a final breakthrough 46 days ahead of the contractual deadline.

d) Challenges and Remedial Measures

Configuration change from double shield mode to earth pressure balanced (EPB) mode took about three weeks to accomplish. At about half way through the headrace tunnel a massive failure of the face blocked the TBM cutterhead and brought progress to a standstill. Among the

many options available to deal with the difficulty, SELI (sub-contractor) chose to employ polymer resins and a three-stage process to secure the heading and resume TBM advance.
First, the void ahead and above the cutterhead was filled with high expansion polymer resins that support the void to prevent further material falling. TIS-foam DRY K (a high expansion ratio organo-silicate resin) was injected through a set of appropriately positioned pipes through the TBM cutterhead and at sufficient volume to completely fill the cavity [21].

Secondly, the crown and face area was consolidated using TIS-sill STAB (a low expansion ratio organo-mineral resin). TIS-sill STAB has exceptional mechanical resistance and adhesion and was injected again through cutterhead and via pipes positioned appropriately to consolidate the heap of loose rubble in the face and around the cavity.

Following that a hand-mined heading was advanced out above the shield to remove rocks and free up the cutterhead, being assured at all times that all ahead is stabilized and made safe [21].

3.5 Gilgel Gibe III Hydropower Tunnels

This section is a review of different documents of the GG III hydropower tunnels. The documents mainly include ground investigation reports or geological reports, hydrogeological reports, geotechnical interpretative reports, design reports, method statements, photographs, drawings, challenges and remedial works report, excavation face mappings, etc.

3.5.1 Location

The Gibe III Hydroelectric project is located within the Omo Gibe River Basin in the middle reach of the Omo River around 450km South of Addis Ababa. The approximate geographic coordinates of the location of the dam axis is between 312,044E and 757,343N and 312,542E and 757,107N. Figure 3.5 shows the location of the project area [22].

Figure 3.5. Location Map of the Project Area [22]

3.5.2 Tunnel Geology

During the design processes, the design inputs were carried out in the site investigations and those data were updated during excavation of the tunnels.

Based on the geological, geotechnical and hydrogeological reports, the power tunnels and the Omo-River diversion tunnels of Gilgel Gibe III were designed to pass through mainly three types of Trachytic geological formations. The tunnels geological profile show mainly (See geological profiles in Appendix B) [14, 15]:

- Slightly weathered trachyte (SW-T),
- Unweathered trachyte (U-T), and
- Brecciated trachyte (Br-T).

From the site investigation reports and the geotechnical interpretative report (GIR), the design UCS value of 98 MPa adopted for the unweathered trachyte, corresponds to an R4/R5 rock class (ITA rock classification); the UCS value of 65 MPa adopted for the slightly weathered trachyte, corresponds to an R4 rock class, while the UCS value of 50 Mpa adopted for brecciated trachyte, corresponds to an R3/R4 rock [14].

The design modulus of elasticity of the intact rock has been selected as E = 25GPa for the unweathered trachyte, E = 22GPa for the slightly weathered trachyte and E = 20GPa for the brecciated trachyte, and poisson's ratio $\nu = 0.3$ (Table 3.1) [14, 15].

Table 3.1. Summary of Design geotechnical parameters for intact rocks of the GG III HPTs [14]

ROCK TYPE	Unit weight	UCS		Young's Modulus of Intact Rock, E	m_i		Seismic Velocity
	kN/m^3	Lab Tests [(1)] (MPa)	Design (MPa)	Design (GPa)	Refer [(2)]	Design	Lab Tests [(3)] (m/s)
U-T	**24.5**	109.5	**98**	**25**	20/25	**14**	3915
SW-T	**23.9**	67.5	**65**	**22**		**20**	3376
Trachytic Breccia	**24.5**	50 [(4)]	**50**	**20**	19±5	**19**	

[(1)] Average value from 7 (light grey Trachyte), 7 (dark grey Trachyte), 26 (slightly weathered Trachyte) UCS tests
[(2)] Average value from D.C. Wyllie'03 "Rock Slope Engineering"
[(3)] Average value from 16 (light grey Trachyte), 16 (dark grey Trachyte), 45 (slightly weathered Trachyte) seismic velocities laboratory tests
[(4)] Estimated from point load tests, considering the intact rock samples only.

3.5.3 Design of Ground Excavation Supports

The designer of the GG III hydropower tunnels used geological strength index (GSI) and Q-method for the design of ground excavation initial supports (Table 3.2).

Table 3.2. Ground Excavation Initial Supports Comparison between GSI- and empirical design Q-method [14]

Tunnel geology		U/S WETHERED TRACHYTE				TRACHYTIC BRECCIA	
GSI values		71-95	56-70	36-55	23-35	33-46	23-32
Support type		A	B	C	D	D	E
Design support	Bolts (@, L) (m)	Spot	2.5, 6	1.8, 6	1.5, 6	1.5, 6	-
	Shotcrete ≥	-	100mm	150mm	250mm	250mm	300mm
	Steel sets	-	-	-	-	-	HEB180 @1m
Q values		8.25-120	1.4-8.25	0.15-1.4	0.04-0.15	0.11-0.51	0.04-0.11
Design support	Bolts (@, L)(m)	2.3, 2.4	1.8, 2.4	1.3, 2.4	1.2, 2.4	1.3, 2.4	1.2, 2.4
	Shotcrete (mm)	-	50-90	90-120	150	120-150	150
	Steel sets	-	-	-	-	-	-

3.5.4 Construction Method and Techniques

The construction method used in Gilgel Gibe III hydropower tunnels was drill and blast. The tunnels were excavated in two phases, i.e., top heading and then bench excavation techniques.

3.5.5 Final Tunnel Lining

The inner/final linings include cast-in-place reinforced concrete, steel lining and grouting. The thickness of the cast-in-place reinforced concrete was designed to be of 65cm and 70cmand the finished final tunnel radius is to be 5.5m. The concreting of the permanent lining of the left and right power tunnels was realized all along the power tunnels except at the portions covered by the chambers and steel linings [15, 16].

3.5.6 Challenges and remedial measures

In GG III hydropower tunnels encountered challenges during construction of tunnels. Especially, a major collapse occurred in the central river diversion tunnel. Causes of collapse and remedial measures are explained in chapter 4.

CHAPTER 4. ANALYSIS AND EVALUATION OF GILGEL GIBE IIIHYDROPOWER TUNNELS

In this chapter the design parameters and methods, construction methods and techniques, challenges encountered, etc. will be analysed and evaluated. At the end of this chapter, the author has performed proper design for ground excavation initial supports using the up-to-date Q-design chart.

4.1. Evaluation of design parameters and design methods

The design parameters include rock mass classification, average stand-up time, cohesion of the rock mass and angle of internal friction of the rock mass. According to the geomechanics rock classification method (RMR), the design parameters and engineering properties of the rock mass are shown in Table 4.1 [7, 8].

Table 4.1. Design Parameters and Engineering Properties of Rock Mass [7, 8]

S. No.	Design Parameters/ properties of Rock Mass	Rock Mass Rating, RMR (Rock Class)				
		81-100 (I)	61-80 (II)	41-60 (III)	21-40 (IV)	≤ 20 (V)
1	Classification of rock mass	Very good	Good	Fair	Poor	Very poor
2	Average stand-up time	10 years for 15m span	6 months for 8m span	1 week for 5m span	10 hours for 2.5m span	30 min. for 1m span
3	Cohesion of rock mass (Mpa)	>0.4	0.3-0.4	0.2-0.3	0.1-0.2	<0.1
4	Angle of internal friction of rock mass	>45º	35º-45º	25º-35º	15º-25º	15º

4.1.1. Rock Classifications and design of ground excavation supports

The most widely used rock classification methods or empirical tunnel support design methods are geomechanics rock classification (or rock mass rating, RMR) and rock mass quality (Q).RMR and Q are indeed in the same category of assessing rock mass quality for tunnelling purposes and providing input for engineering design and construction. One of the benefits of the RMR system is that it is relatively easy to use. The result produced by the RMR classification, however, is rather conservative [2, 3]. The geological strength index (GSI) is different from these two (RMR and Q) empirical methods.

The rock classifications (Figure 4.1 and 4.2) of the GG III hydropower tunnels executed according to the geomechanics rock classification (RMR) method. The rock qualities of the GG III HPTs were classified in to three classes. The designer used RMR method to carry out the rock classifications but the design of ground excavation primary supports performed by the GSI values which are calculated from the correlation Eq. 4.1.

$$GSI = RMR - 5 \quad (4.1)$$

According to Richard Z. T. Bieniawski Preinl, GSI is an index of rock mass characterization, i.e., it is not meant for replacing a classification system of the type of RMR or Q method. In addition, the GSI values (Eq. 4.1) might be valid when the RMR values are less than fifteen (RMR<15) [6]. “GSI has no end use other than to provide input for the Hoek-Brown criterion as a rock mass characterization index”.

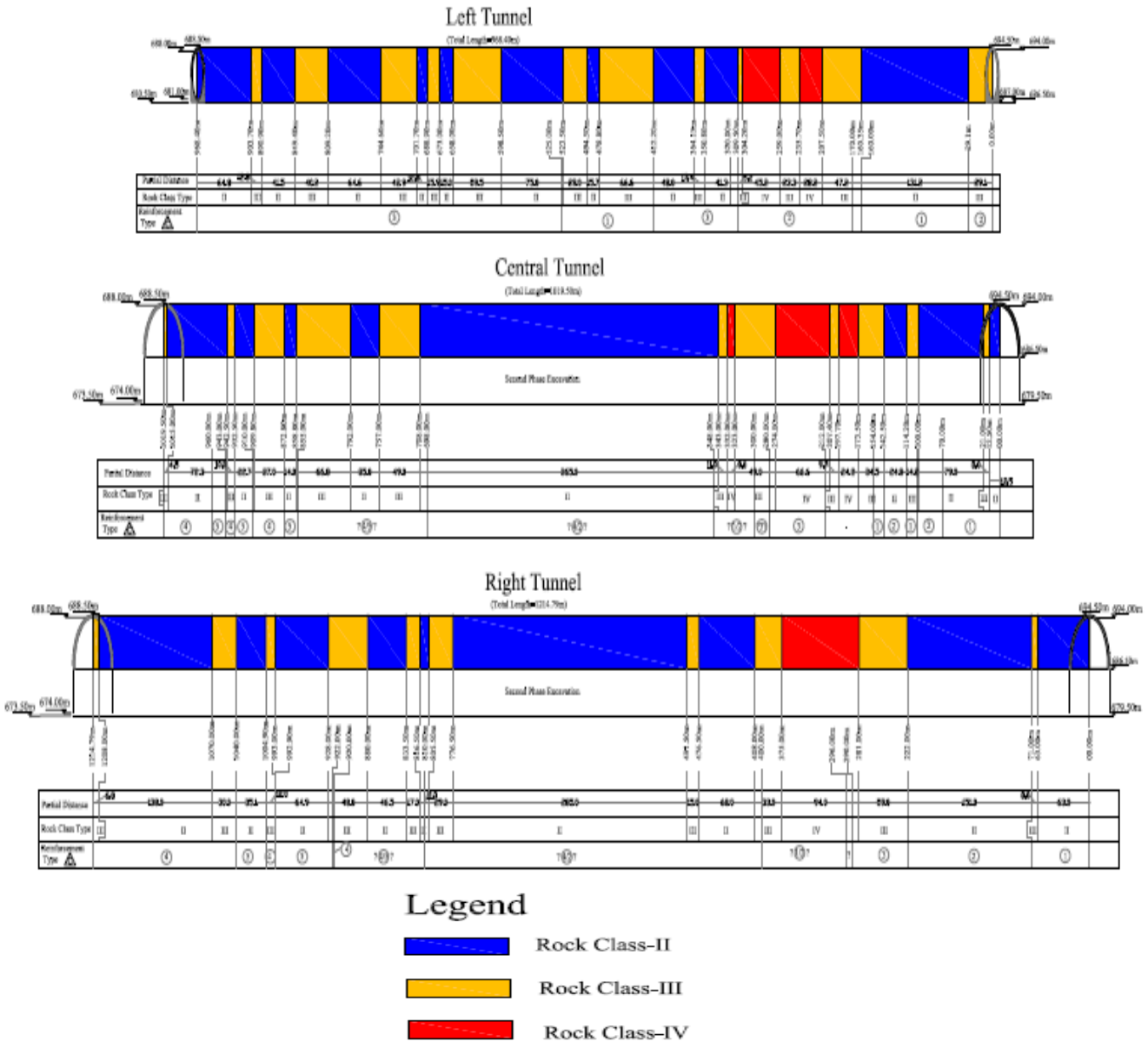


Figure 4.1. Rock Classifications (class II, III and IV) of the River Diversion Tunnels (it is more visible in Appendix C)

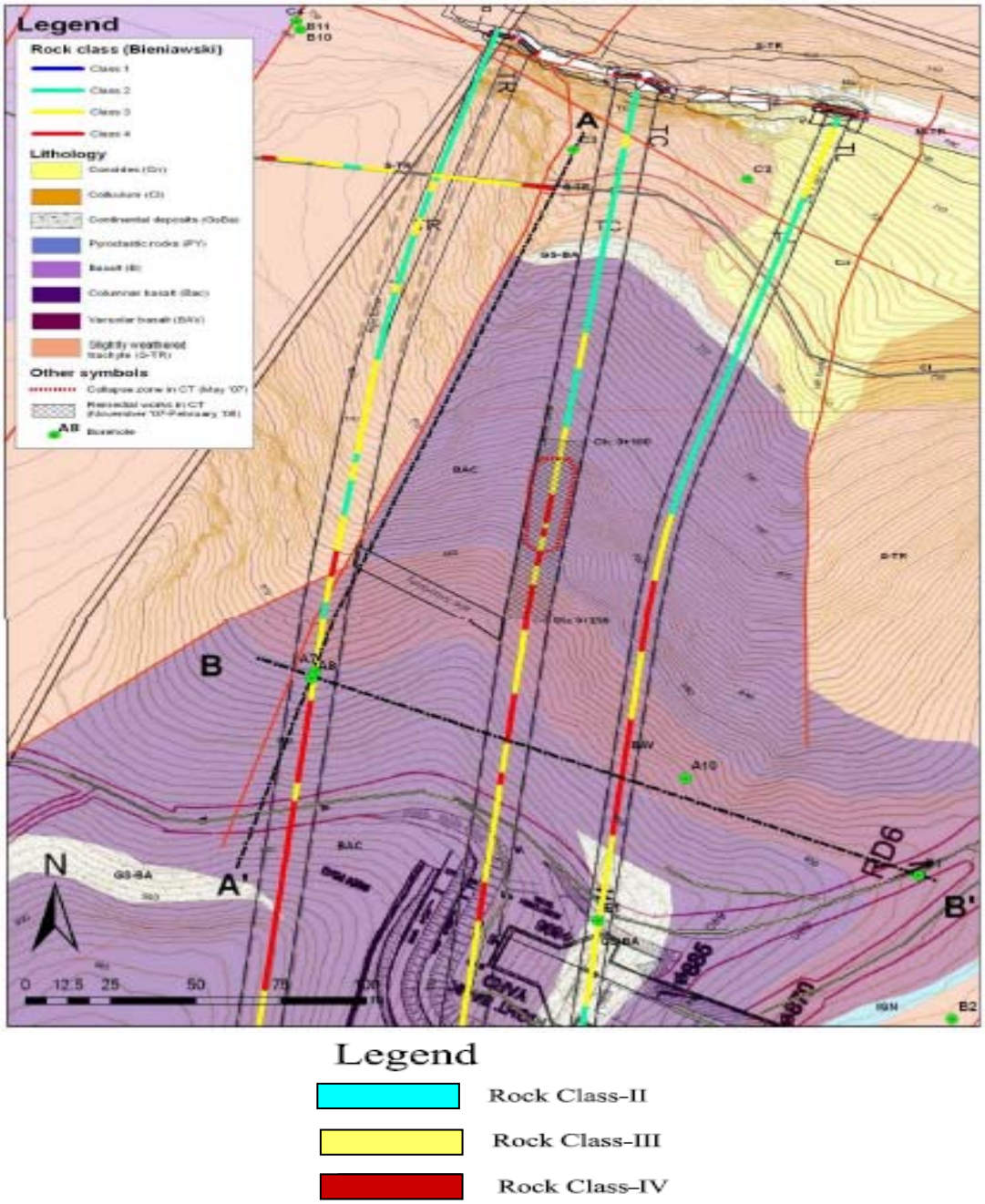


Figure 4.2 Rock Classifications (class II, III and IV) of the Power Tunnels (it is more visible in Appendix C)

Since the RMR values of the GG III hydropower tunnels (HPTs) are between 21 and 80, using the GSI values for design are inappropriate. As explained above, the RMR values are conservative and then the GSI values (Eq. 4.1) are more conservative than the RMR values. Therefore, for the case of the GG III HPTs, the geological strength index (GSI) is not appropriate support design method and the supports that were designed using the GSI values are uneconomical.

The designer considered the rock mass quality (Q) empirical primary support design method but they used wrong formula in the calculation of the rock bolt length. The formula they used to calculate the rock bolt length was Eq. 4.2 [14, 15]:

$$L = \frac{2 + 0.15B}{ESR} \tag{4.2}$$

Where:

L = rock bolt length in meter
B = width or height of tunnel profile, in meter
ESR = excavation support ratio

L = (2+0.15*12.6)/1.6 = 2.4m (shown in Table 3.2, Q-method)

The corrected formula to calculate the rock bolt length is Eq. 4.2 [2, 13]:

$$L = 2 + \frac{0.15B}{ESR} \tag{4.3}$$

Where:

L = rock bolt length in meter
B = width or height of tunnel profile, in meter
ESR = excavation support ratio

L = 2+0.15*12.6/1.6 = 3.2m

The designer compared the rock bolt lengths that were calculated from the rock mass quality Q-method (2.4m) and from the GSI values (6m) (Table 3.2). The designer selected 6m rock bolt length, to the safe side, to be installed in the construction of the GG III HPTs. However, the 6m rock bolt length is uneconomical as explained above.

Therefore, the proper design procedures and design of economical ground excavation supports will be performed in this chapter in section 4.4.

4.1.2. Average Stand-Up Time

Data are needed to predict the stand-up time for the size and orientation of the tunnel and the conditions for blasting during construction, i.e. strength, stratigraphy, description and classification of the ground, water, gas, quartz content and abrasivity (this is obviously essential for tunnelling techniques, except for possibly immersed tube tunnels) [3, 5]. Almost all ground properties are time dependent and strain controlled, both in the short and long term. In the short term, the concept of stand-up time is used as a practical means of indicating the sensitivity of the ground to imposed stress changes. This influences both the support requirements and geometry of tunnel linings. Consolidation, swelling and squeezing may be significant in the longer term [2, 3, 5].

According to the geomechanics rock classification (RMR) method, the average Stand-up time is one of the design parameters and the standard stand-up time for each rock class is explained in Table 4.1.

The values of all design parameters (Table 4.1) should be considered in the design and construction processes in order to have a stable, safe and economical project accomplishment. The values of the average stand-up time are not evaluated in the design and construction of the GG III HPTs. Due to this inconsideration of stand-up time, the construction processes in GG III HPTs were very difficult, i.e., more construction challenges occurred. The challenges include:

1. Rock wedge failures at chainage 0+843 during shotcreting lately in the right manifold of the power tunnel in which a worker lost due to the rock failure.
2. Rock and shotcrete wedges failure at chainage 0+175, 0+178 and 0+185 and major collapse of tunnel section occurred at chainage 0+175 in the central river diversion tunnel during the construction. This is discussed in detail in section 4.3.

4.2. Evaluation of Construction Methods and Excavation Techniques

Tunnels can be constructed by using a number of construction methods and techniques (TBM, roadheader and drill and blast). Although there is a difference between soft ground and hard ground (rock), tunnelling techniques are being used in a wider range of ground conditions and this boundary is becoming increasingly blurred. It should be noted that from a practical point of view, if the ground is stable, the construction of the tunnel can focus on economics and be driven by the limits of the tunnelling equipment. For soft ground, which needs immediate support, construction is driven by the need to support the ground immediately after creation of the void [5].

4.2.1. Excavation Method

The GG III hydropower tunnels consist of about twelve tunnels, namely, power tunnels, river diversion tunnels, drain and access tunnels (see Appendix D).The ground of the GG III hydropower tunnels is stable and its rock properties are classified into three classes, class II, III and IV. The GG III hydropower tunnels are relatively short and are not straight. The length of each tunnel is less than 2km. The diameters of the GG III hydropower tunnels are variable due to the rock conditions (see Appendix F). The shape of these tunnels, except the left river diversion tunnel, is horse shoe (Table 4.2 and Figure 4.3). For these conditions, therefore, drill and blast (D &B) excavation method, used in GG III HPTs, is more economical rather than TBM or road header excavation methods.

The main geometric parameters of the tunnels are summarized in Table 4.2 and Figure 4.3in which DT is river diversion tunnel and PT is power tunnel.

Table 4.2. Summary of parameters of Power Tunnels (PT) and Diversion Tunnels (DT) [14]

Tunnel Type	Length (m)	Diameter/Height (m)	Average x-section Area (m^2)	Shape of Tunnel	Intake Elevation (m)	Outlet Elevation (m)
Right DT	1222	14/15	175	Horse Shoe	680	674
Central DT	1026	14/15	175	Horse Shoe	680	674
Left DT	969	8	57	Circular	687	681
Left PT	1050	12.2/12.3/12.6	126	Horse Shoe	780	775
Right PT	975		126	Horse Shoe	780	775

Figure 4.3 Schematic representation of Gilgel Gibe III hydroelectric project

4.2.2. Excavation Techniques

Large cavities are less stable than small ones. Therefore, in many cases the tunnel cross section is not excavated at once, but in parts called partial face excavation. According to NATM, the most

widespread methods of partial face excavation are (i) top heading and bench, and (ii) sidewall drift. Top heading and bench is a type of excavation techniques in which the crown is excavated first and then the bench is excavated (Figure 4.4). A soon construction of a temporary invert of the crown section or, better, the soon excavation and support of the bench and invert helps avoiding large settlements of the abutments of the crown arch. This means that the length $a = a_1 + a_2$ (Figure 4.5) should be kept as small as possible. On the other hand, a_1 should be sufficiently large (1 to 1.5 times the tunnel diameter, depend on the rock classes/properties) to enable efficient excavation and support works in the crown (Figure 4.6)[2, 53].The top heading and bench excavation technique is implemented improperly in the construction of the GG III HPTs.

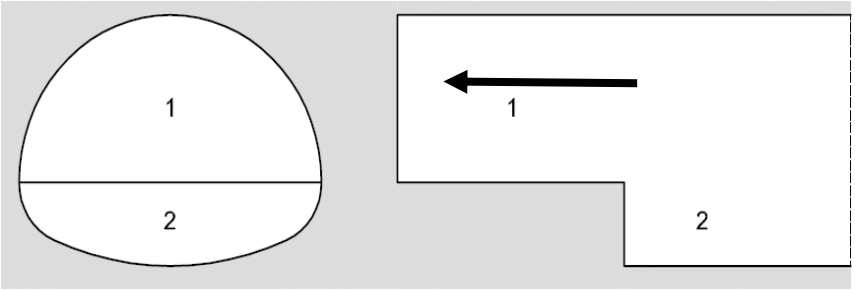


Figure 4.4. Top heading, cross and longitudinal sections. 1: calotte, 2: bench [2]

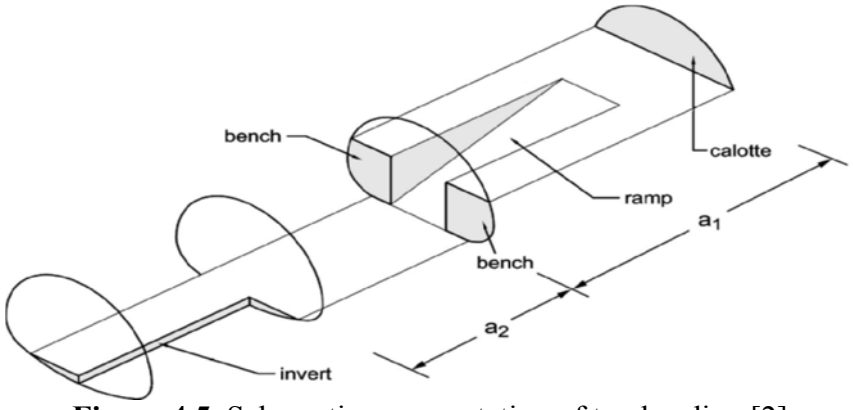


Figure 4.5. Schematic representation of top heading [2]

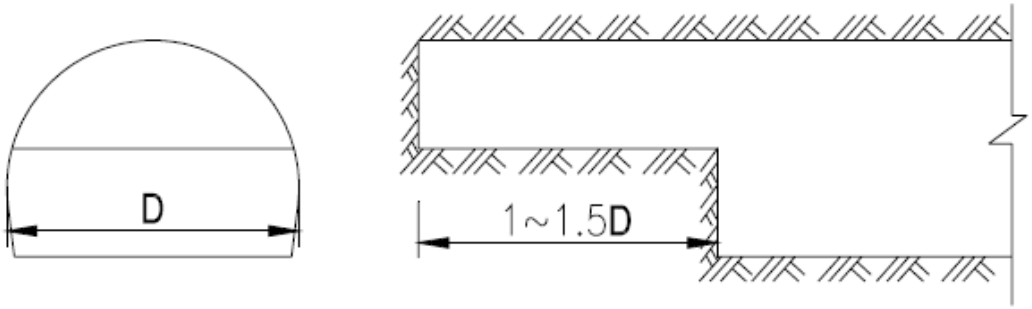


Figure 4.6. Sketch of two-step (top heading and bench) excavation technique [31]

In GG III hydropower tunnels, Portals of all tunnels excavated in full face excavation technique whereas the full length of the tunnels excavated in top heading and bench excavation technique.

But the top heading excavation technique (Figure 4.5 and Figure 4.6) was not implemented in the proper way, i.e. The bench excavation has been started after completion of top heading excavation of the full length (Figure 4.7) (see phase excavation in Appendix E). The control of convergence and settlement of the abutments and crown of the top heading is principally achieved by providing early invert closure [5]. In the construction of the GG III HPTs, no provision of early invert closure was made (Figure 4.7).

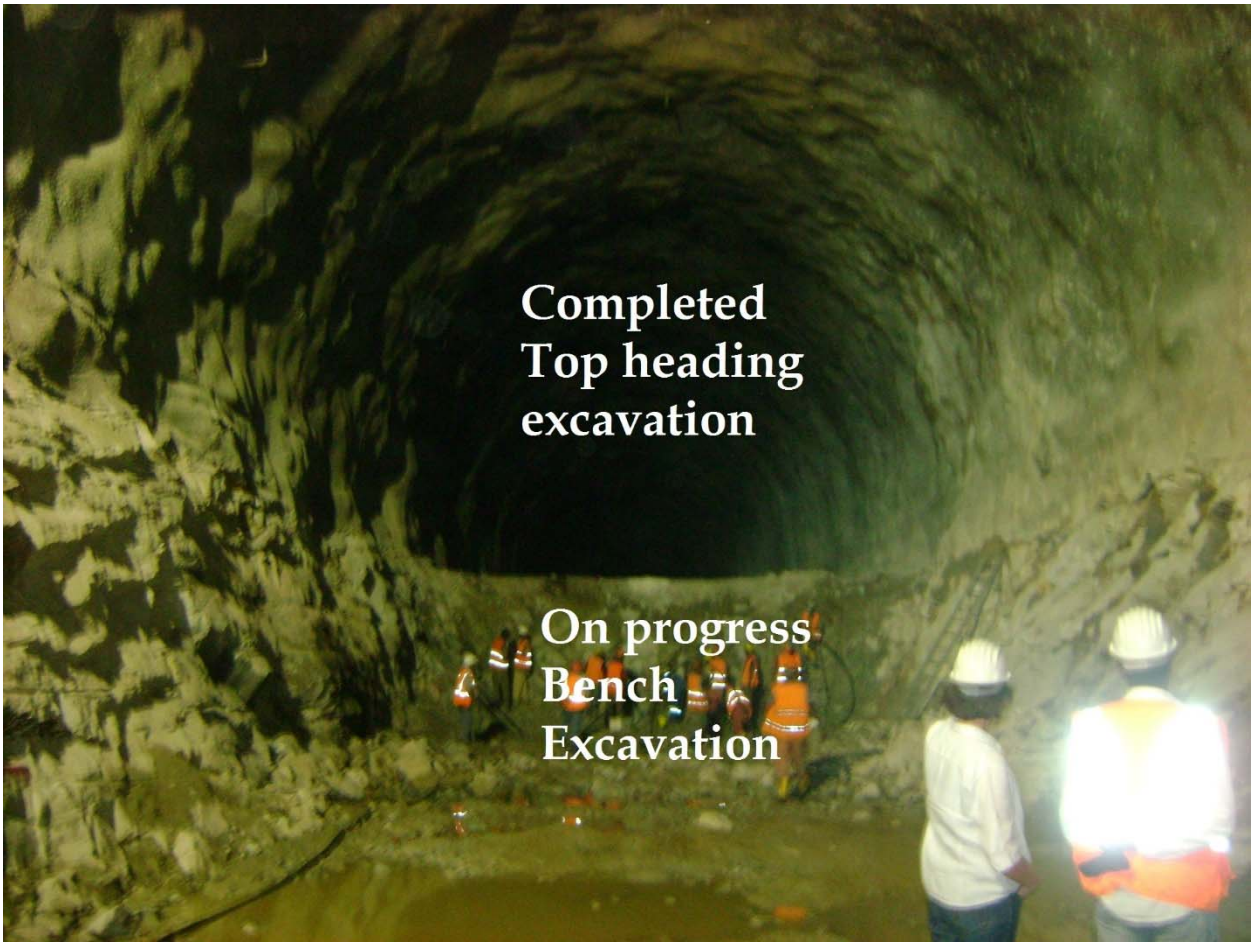


Figure 4.7. The progress of bench excavation after completion of top heading excavation [25]

The control of convergence and settlement is also principally achieved by limiting the length of advance per stage [5]. But in GG III HPTs, the length of advance in the bench excavation was 15 m to 20m per stage. This is a huge blast in a tunnel construction and its disturbing factor is very high in the crown arch which has been excavated in top heading.
The excavation technique used in the GG III HPTs will lead to large settlements and convergences of the abutments and crown. This settlements and convergences also lead to wedge (rock and shotcrete) failures and finally tunnel section collapse. Challenges have been faced in the construction of the GG HPTs due to this excavation technique and the support requirements.

The author has observed overbreak tunnel cross-sections in the power tunnels (Figure 4.8 and Figure 4.9). These overbreak tunnel x-sections required to be filled with concrete mortar. The

cast-in-place reinforced concrete permanent tunnel lining was designed to be 65cm and 70cm in thickness (Figure 4.10 and Figure 4.11) however, due to these overbreak voids, around the permanent tunnel linings, the quantity of concrete required to fill the voids is absolutely very large.

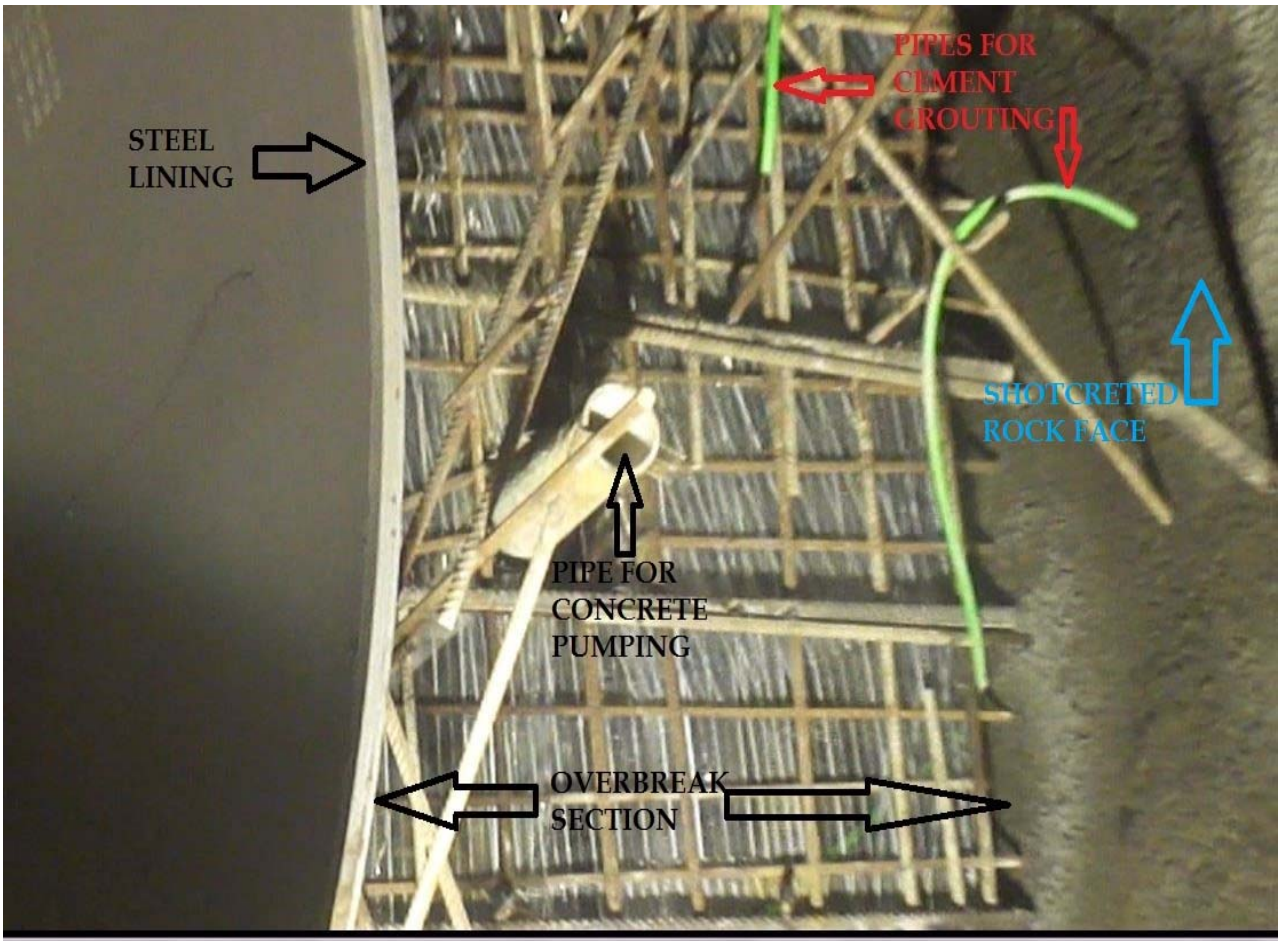


Figure 4.8. Typical overbreak sections and steel lining in the power tunnels

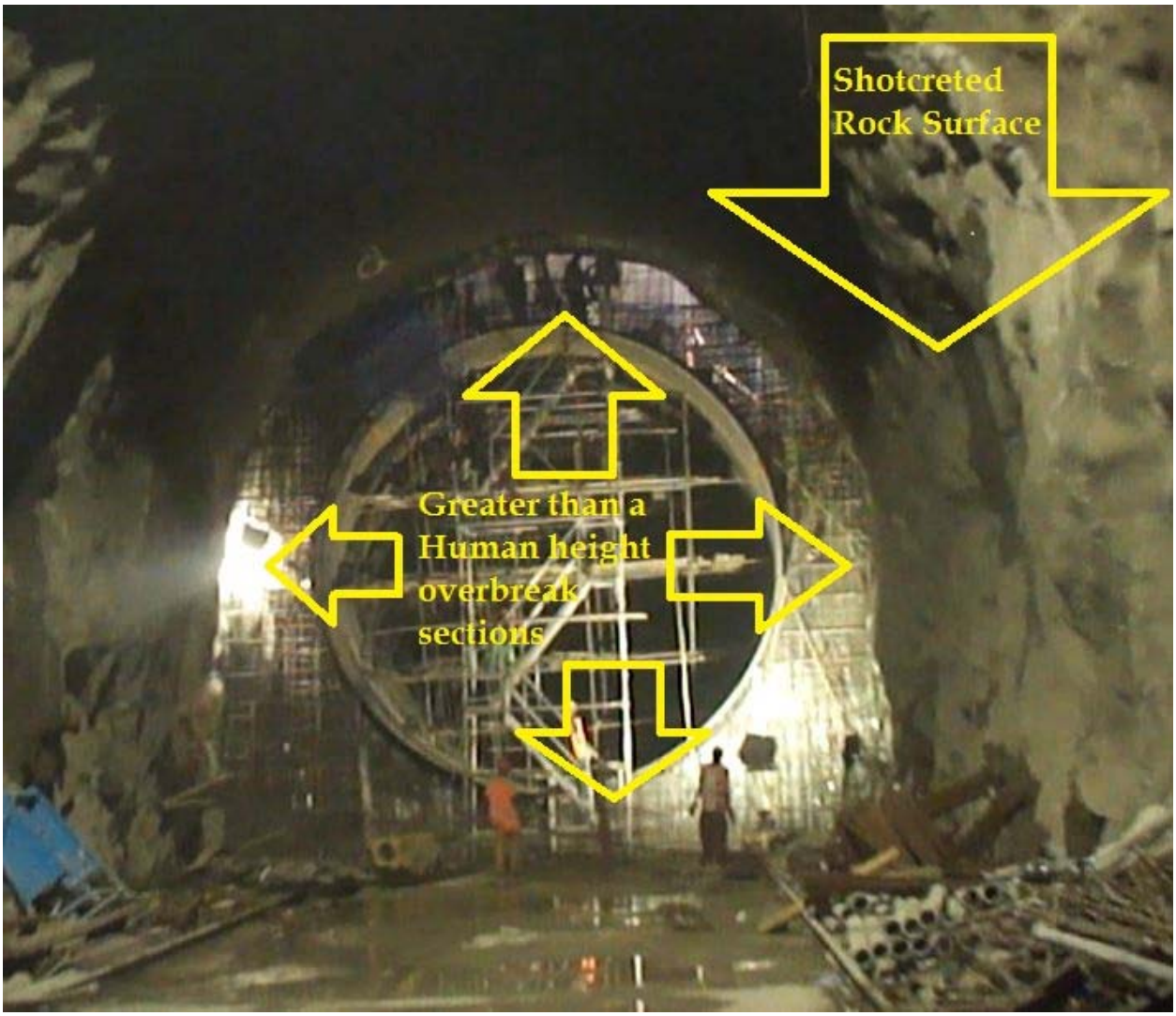


Figure 4.9. Typical overbreak sections and steel lining in the power tunnels

This is a lack of controlled blasting knowledge and it, therefore, leads to uncontrolled cost overrun.

4.2.3. Lining/Support

Sprayed concrete (shotcrete) lining uses an incremental excavation sequence and sprayed concrete used as a primary (initial) support with, or without, weldmesh, fibres, lattice arches (girders), dowels, anchors and bolts. The primary support details are to be determined in advance of the construction by the designer and are to be validated during construction by instrumentation and monitoring devices [2, 3, 5].

Sprayed concrete lining (SCL) or shotcrete, welded wire mesh, rock bolting and cement grouting are used in the GG III hydropower tunnels as an initial support systems whereas cast-in-place reinforced concrete and steel linings used as a final lining together with cement grouting. These lining systems are suitable for the drill and blast construction method. As discussed above, the excavations executed with overbreak tunnel cross-sections.

The designer accomplished clearly the length of drilled holes and the length of blasted rock portions together with charging and blasting patterns for the two phase excavation techniques (see Appendix E). The cast-in-place reinforced concrete finial lining was designed to be the thickness of 70cm and 65cm (Figure 4.10 and Figure 4.11, respectively, Appendix H). However, due to lack of controlled blasting problems, the final lining thicknesses obviously increased. Thus, the volume of the required concrete and grouting also increased incredibly because the overbreak voids are to be filled with concrete and then cement grouting.

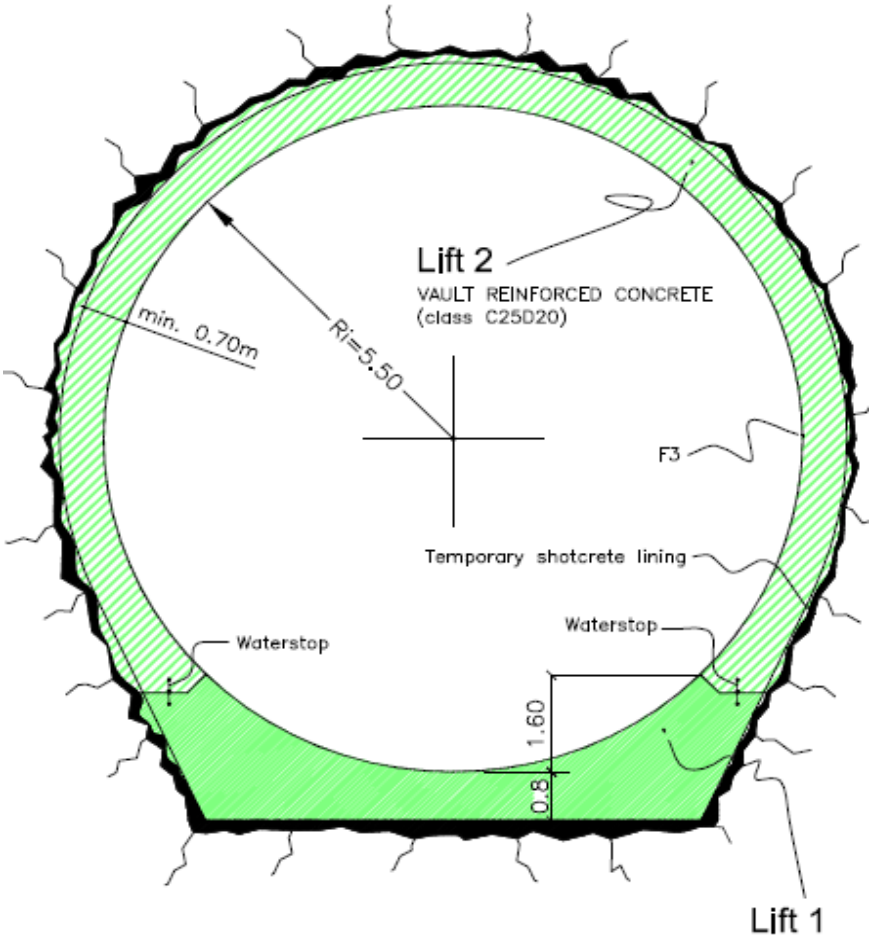


Figure 4.10. Type I Cast-in-place reinforced concrete final lining [as per design documents]

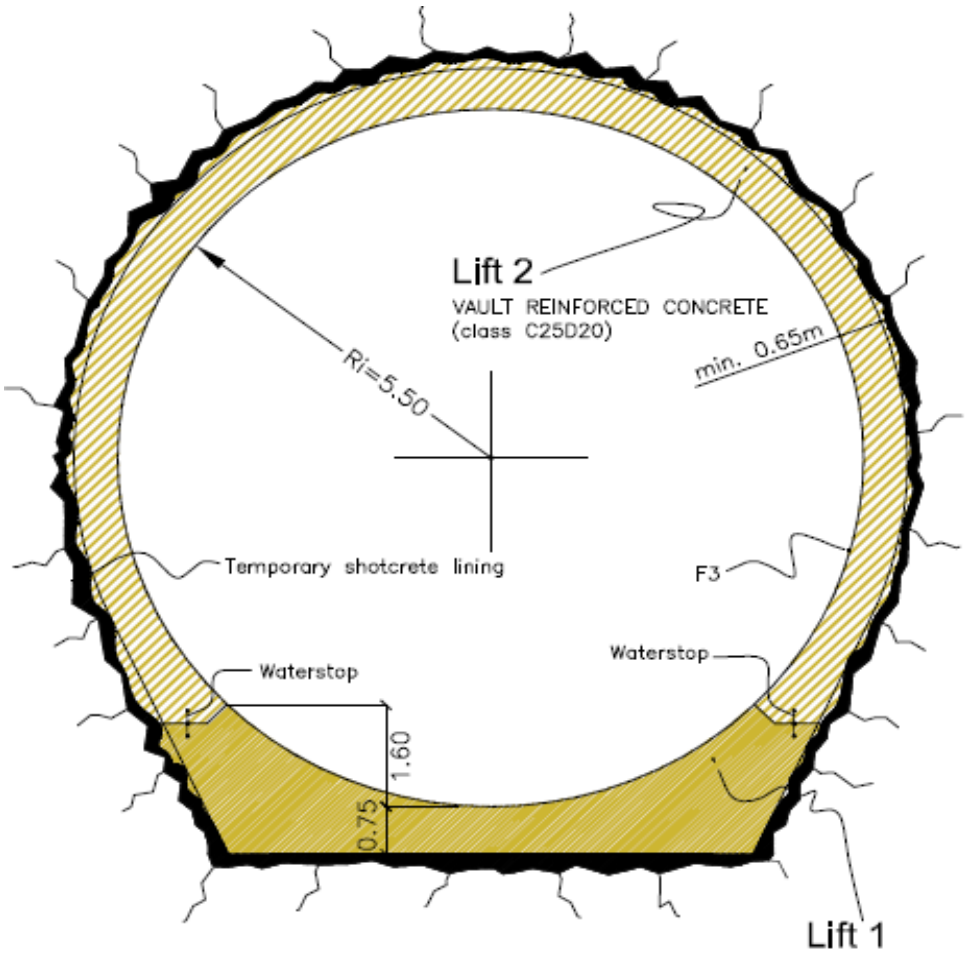


Figure4.11. Type II Cast-in-place reinforced concrete final lining [as per design documents]

Consolidation cement grouting is applied all along the tunnels to fill the voids between the initial ground excavation supports and the final linings as well as to fill the joints of the surrounding rocks (see Appendix H).

4.3. Challenges Encountered in Gilgel Gibe III

Certain challenges occurred in the GG III hydropower tunnels during its construction. Especially, a major collapse occurred in the central river diversion tunnel, and rock and shotcrete wedges failure in the excavation of the central diversion and right power tunnels as discussed below.

4.3.1. Challenges with regard to geotechnical parameters

Challenges were not recorded with regard to the geotechnical parameters, i.e., no changes in geological structures, rock properties, ground water conditions, etc.

4.3.2. Challenges with regard to construction Procedures and support requirements

Rock wedge failures occurred in power and diversion tunnels during tunnel construction at different locations. A major rock wedge failure occurred in the right power tunnel at manifold section at chainage 0+843. A man died due to this rock wedge failure when a shotcreting of the manifold section is on progress (Figure 4.12 and Figure 4.13). The rock class is II, RMR = 69. This type of rock might have remained stable for about six months for only 8m of unsupported excavated span (Table 4.1), according to the RMR method. In the manifold section, the power tunnel branched in to five tunnels each of which will take water in to five turbines. Therefore, the excavated width, span and height, in the manifold section, are more than 8m (Table 4.1).

Figure 4.12. Rock Failure on the Shotcreting Machine [25]

Figure 4.13. Rock Failure Section at chainage 0+843 [25]

The rock wedges failure encountered in the central river diversion tunnel at stations 0+175, 0+178 and 0+185 during excavation. A major collapse occurred after 74 days of excavation at chainage 0+175 and this collapse extended into both directions (backward to station 0+160 and forward to station 0+250). The causes of this collapse and remedial measures that are used to repair the collapsed section will be discussed below.

The investigations of causes were carried out by Pietrangeli (Designer, from Italy) and Lombardi Engineering (from Switzerland). They used the recorded geological reports (for example bore holes of A10 and E1) (Figures 4.14), recorded actual face mappings (Table 4.3), visiting the collapsed section, etc.

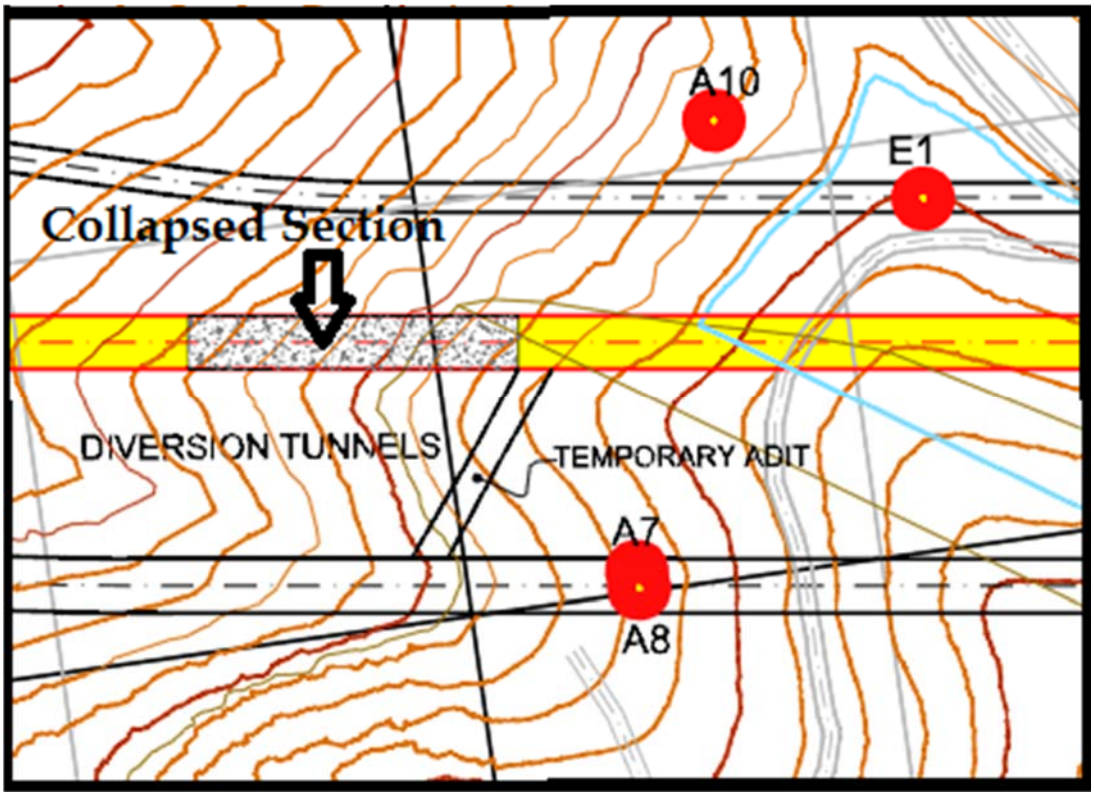


Figure 4.14. Bore Holes A10 and E1 with collapsed section [Appendix B]

The recorded in-place face mappings and rock classifications carried out during excavations of the collapsed area are listed in Table 4.3. The rock wedge failure occurred at station 0+178 during excavation but the section collapsed at station 0+175 after 74 days and 67m advance of the construction. This collapse extended into both directions, therefore, these data are used for cause investigations.

Table 4.3 As built rock classifications of the collapsed area [26]

Chainage	Mapping date	RMR	Rock class	Remark
0+148	18-Feb-2007	57	III	Fair rock
0+164	21-Feb-2007	59	III	Fair rock
0+169	23-Feb-2007	55	III	Fair rock
0+178	01-Mar-2007	39	IV	Poor rock*
0+204	18-Mar-2007	57	III	Fair rock
0+218.5	27-Mar-2007	57	III	Fair rock
0+228	14-Apr-2007	57	III	Fair rock
0+242	28-Apr-2007	57	III	Fair rock

* Rock failure occurred during excavation, the section collapsed on 13th May 2007.

4.3.3. Causes of collapse

Causes of collapse [26], in brief, include:

1. Excessive delays in the installation of the required ground excavation supports.
2. Installation of improper ground excavation supports.
3. Convergence monitoring was not installed.
4. Drainage holes were not installed.

1. Excessive delays in the installation of the ground excavation initial supports. Installation of the support delayed about seventy four (74) days from 01-Mar-2007 to 13th May 2007. The tunnel advanced sixty seven (67) meters, from station 0+175 to station 0+242, when the section collapsed (Table 4.3). The collapsed section is in rock class IV (RMR = 39, Table 4.3)(Figure 4.15). These rocks might have stabled for about10hours for 2.5m of unsupported excavated span (Table 4.1).

Figure 4.15. Collapse of Central Diversion Tunnel at chainage 0+ 175[26]

2. Installation of inappropriate ground excavation initial supports. The Lombardi Engineering Limited remedial works report [26] said "all along the tunnel, the rock surface was covered

with skin shotcrete and some pitons (swellex)" (Figures 4.18 and 4.19). As explained above, the collapsed section is rock class IV (RMR = 39). The initial supports that were designed for rock class IV include:

- Shotcrete Thickness ≥25cm,
- Welded Wire Mesh: ϕ5mm, 10cmx10cm,
- Pattern Rock Bolt: 23ϕ25mm, L= 6m@1.5m with cement grouting.

These supports were not installed in place. As said above, the installed supports were skin (very thin layer) shotcrete and some (spot) swellex. This indicates that the quality control body has not carried out his obligation.

Figure 4.16 Collapsed section supported with Skin shotcrete and pitons [26]

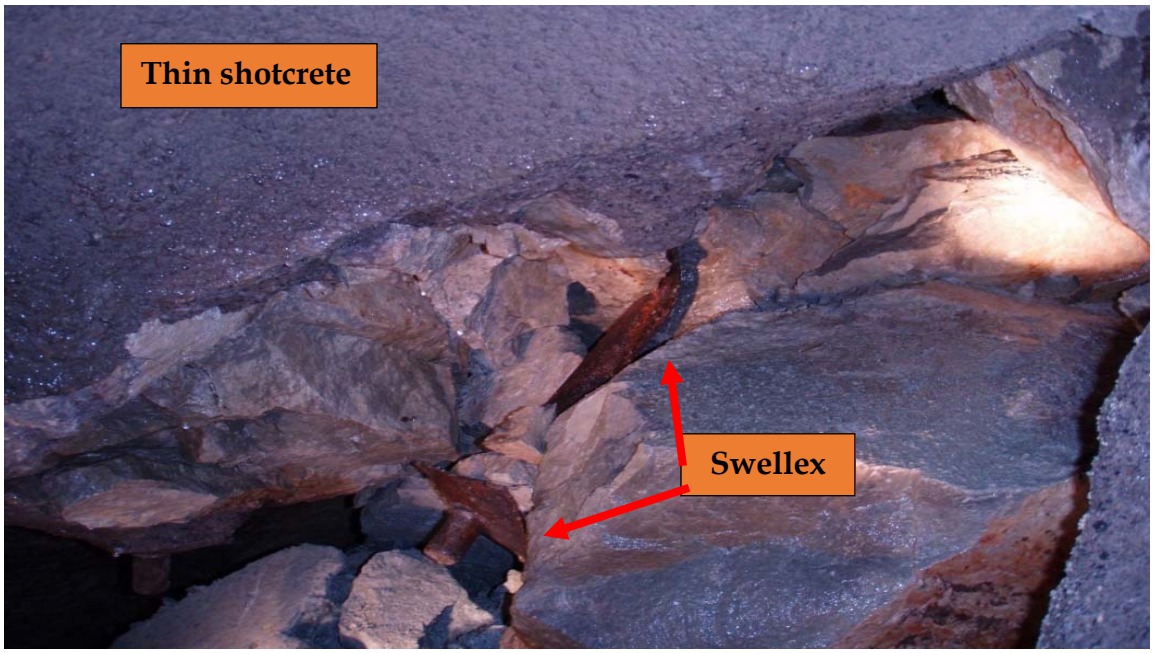


Figure 4.17 Collapsed section supported with some (spot) swellex and skin (thin)shotcrete [26]

3. Convergence monitoring were not installed. The Lombardi Engineering Limited remedial works report [26] said "convergence measures and monitoring sections were not installed". Convergence monitoring within the tunnel can help to provide data to review parameter uncertainty. A combination of roof settlement and an inward movement of the footings is called convergence (the distance shortens) (Figure 4.18) [5].

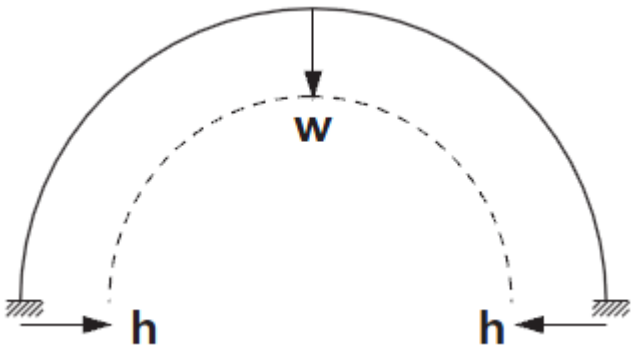


Figure 4.18. Convergence in combination with roof settlement [5]

Convergence measurement and flat jack test were installed in the construction of the power tunnels (Figure 4.19).

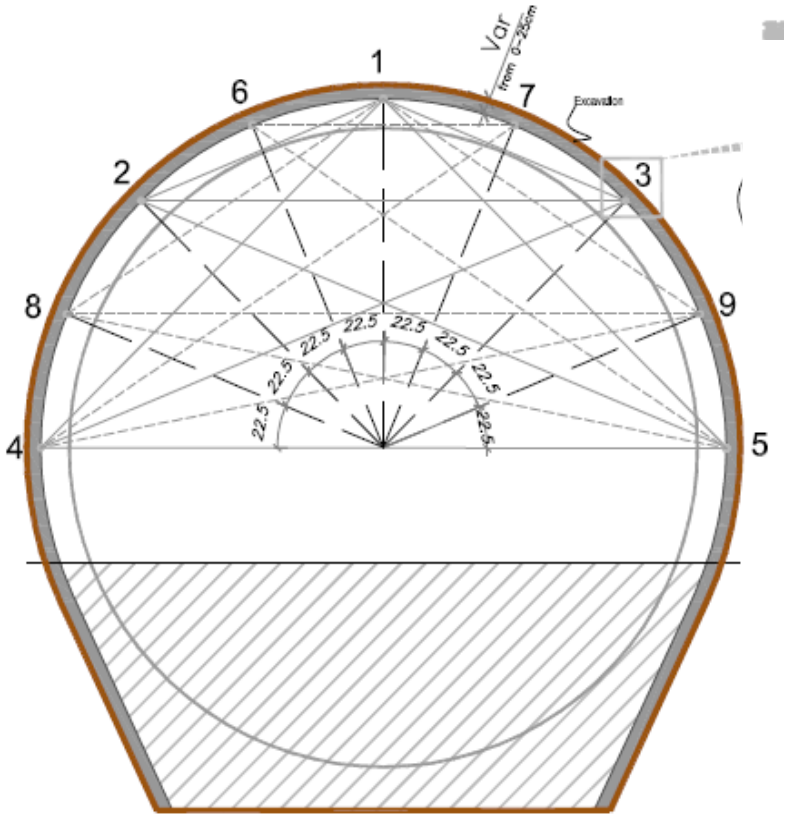


Figure 4.19. Convergence measurements targets, see details in Appendix F

4. Drainage holes were not installed. The Lombardi Engineering Limited remedial works report [26] said "during the excavation no water was reported, however some dripping and wet shotcrete observed around the collapsed area at the moment of the inspection. It is assumed that the water reduces the cohesion of the filling of the discontinuities increasing the loads on the supports. The installed supports were not able to resist the new loads and some blocks collapsed and then other blocks lost their lateral support, finally, the collapsed sections enlarged rapidly into both directions", from chainage 0+175 backward to 0+160 and forward to 0+250[26].

4.3.4. Remedial Works

The remedial works started from the intake side, where free access to the collapsed area is available, and progressed towards the core of the collapsed section. This remedial works started at ten meters (10m) before the beginning of the collapsed section (Figure 4.20).

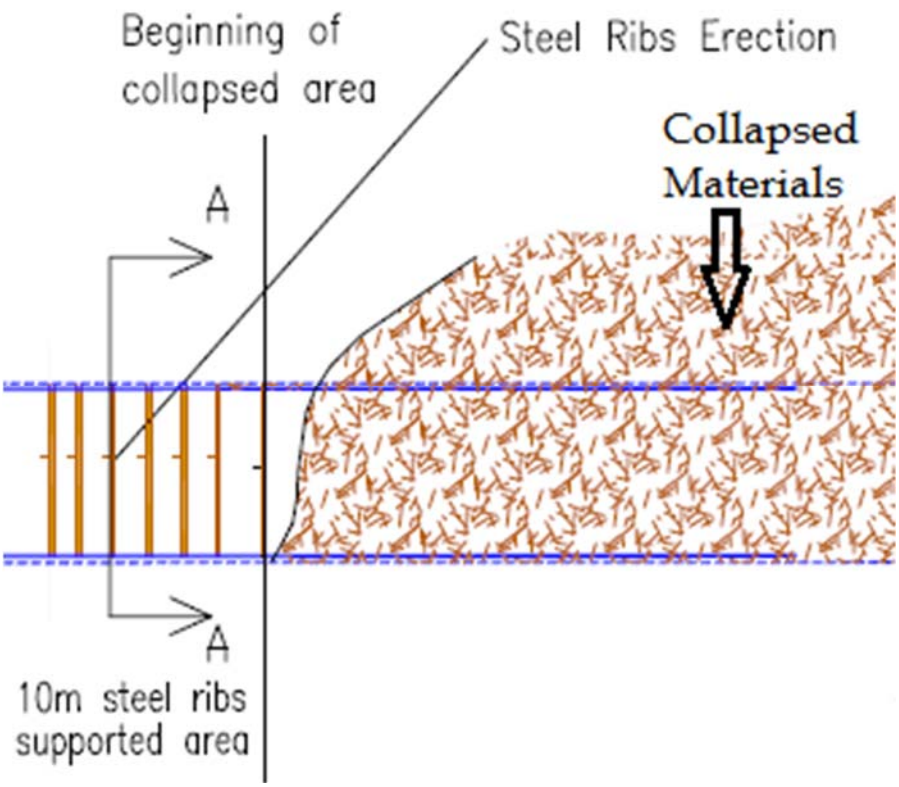


Figure 4.20. Remedial works details (see Appendix G)

The remedial work procedures are summarized briefly below [27] (see Appendix G):

1. The first step is installing steel ribs in 10m span before the beginning of the collapsed area (Figure 4.20). The Steel ribs are erected by 1m spacing and fixed to the rock walls by anchor bolts and steel plates. Wire meshes are also fixed between the steel ribs and then shotcrete is applied (Figure 4.21).

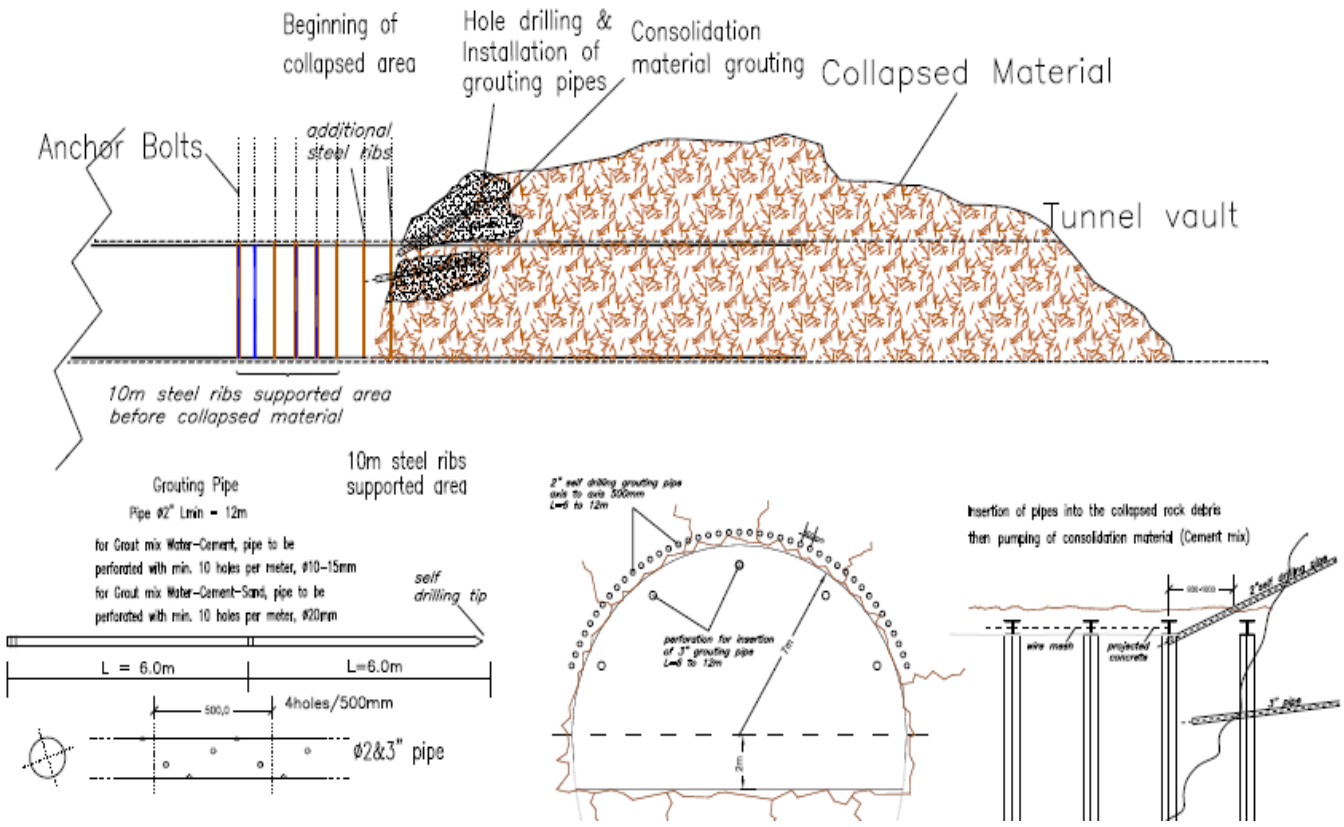


Figure 4.21. Pipes installation and consolidation grouting into the collapsed debris (see details in Appendix G)

2. Small diameter perforated pipes, 12m long, inserted into the vault and debris of the collapsed area in order to apply consolidation grouting of cement mortar and polyurethane foam to consolidate the debris (Figure 4.21). Cement mortar and polyurethane foam are injected through the inserted pipes in order to consolidate the collapsed debris and then slow excavation followed by using proper machines to muck out the debris of one meter (1m) length (Figure 4.22).

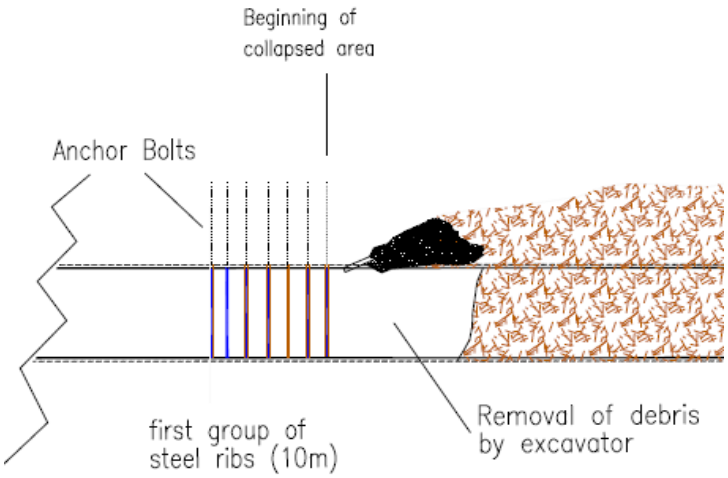


Figure 4.22. Removal of 1m span consolidated debris (Appendix G)

3. After the removal of one meter span consolidated debris, the above two steps have been repeatedly used until the collapsed area is successfully restored (Figure 4.23).

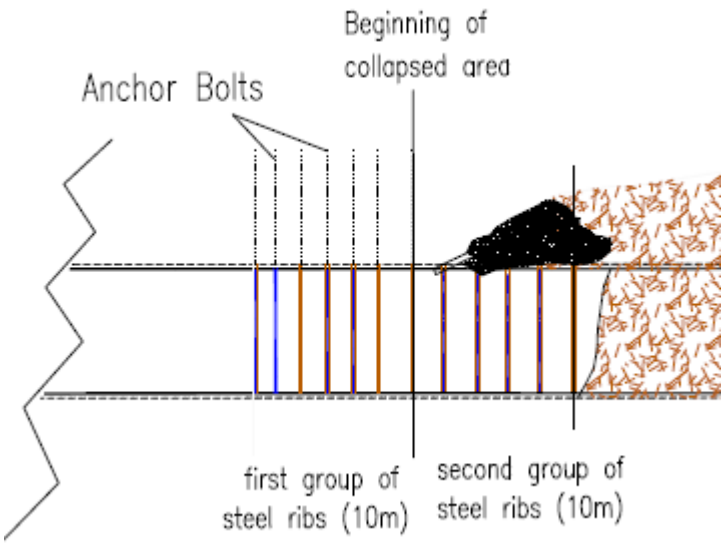


Figure 4.23. Progress of remedial works step by step (Appendix G)

In general, various techniques and unexpected amount and type of supports such as steel ribs, steel plates, wire mesh, shotcrete, cement mortar, anchor bolts, fixing anchors, consolidation grouting of cement mortar and polyurethane foam, etc. used in the remedial works. In addition, due to this collapse, unexpected 63.25m long access tunnel (adit) (Figures 4.26 and 4.27) constructed from right river diversion tunnel to central river diversion tunnel in order to proceed the excavation of the central diversion tunnel. Almost nine months have been lost in the restoration of the collapsed tunnel section, from May 2007 to February 2008. As a result of this collapsed tunnel section, the cost and construction time of the project have been increased.

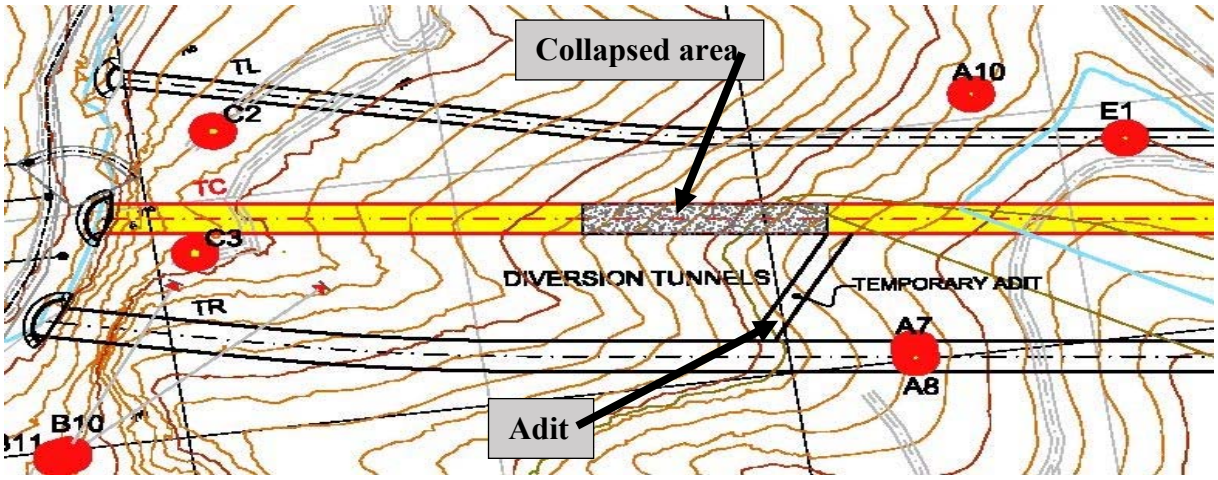


Figure 4.24 Collapsed central tunnel section and temporary adit [28]

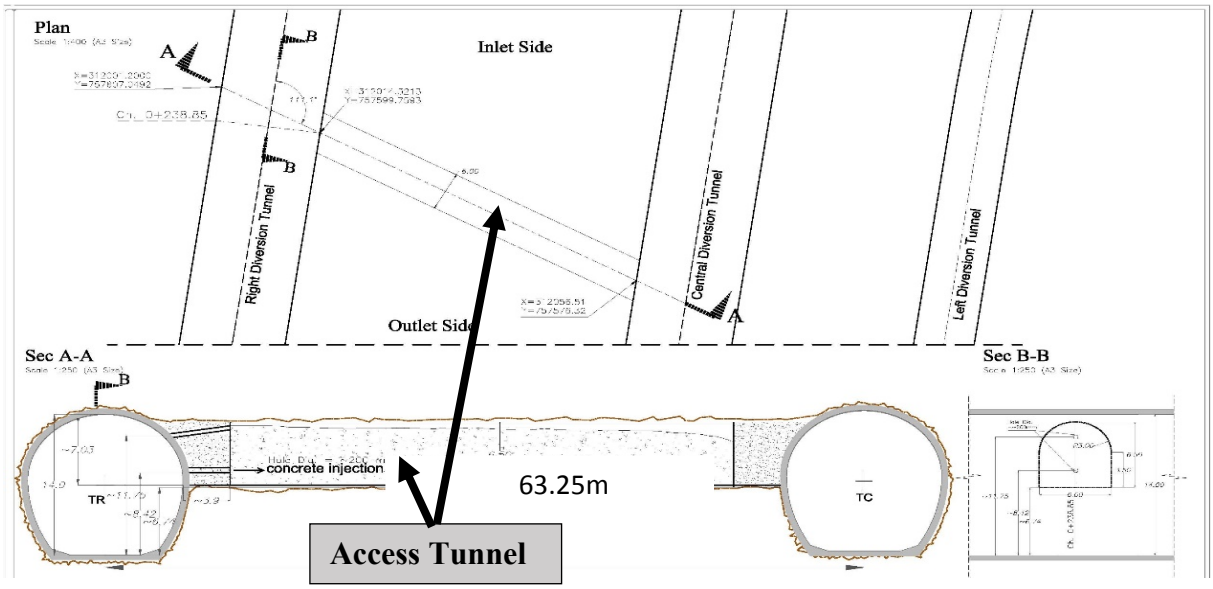


Figure 4.25 Temporary adit/access tunnel [29]

4.4. Design of Ground Excavation Initial Supports

As discussed above in this chapter, ground excavation initial supports were not designed properly and economically. Therefore, the proper design procedures and the economical design of ground excavation initial supports are carried out in this section.

(a) Conversion of RMR to Q system

The two most widely used rock mass classification methods are geomechanics rock classification (RMR) and rock mass quality (Q). The rock properties of the GG III hydropower tunnels are classified based on the geomechanics rock classification (RMR) method but the support design chart of the RMR method is not up-to-date whereas that of the Q method is up-to-date. Therefore, the conversion of RMR to Q system is required. RMR and Q incorporate geotechnical, geometrical, and engineering parameters. Using rock mass classifications and equivalent dimension of the tunnel, which is defined as ratio of dimension of tunnel and excavation support ratio (ESR), Barton proposed a number of support categories and the up-to-date Q-support design chart (Figure 2.3) [3].The correlation values of RMR and Q systems are shown in Table 4.4 (Eq. 2.3).

Table 4.4 Correlationvalues between RMR and Q [14]

Tunnel Geology	Unweathered/slightly weathered Trachyte				Trachytic Breccia (Moderately weathered)	
Support Type	A	B	C	D	D	E
RMR	76-100	61-75	41-60	28-40	38-51	28-37
Q	8.25-120	1.4-8.25	0.15-1.4	0.04-0.15	0.11-0.51	0.04-0.11

The value of ESR is related to the intended use of the excavation and to the degree of safety, which has an influence on the support system to be installed in order to maintain the stability of the excavation (Table 4.5) [5].

Table 4.5 Excavation Support Ratio (ESR) values for various underground structures [3, 23, 24]

Category	Description of Excavation Category	Suggested ESR value
A	Temporary mine openings.	3 – 5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Barton provide additional information on rockbolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations.

(b) Determination of Equivalent Dimension

In order to determine the stability of the excavation and support requirements, the equivalent dimension, De, of the excavation is determined in (Eq. 4.6) [5].

$$D_e = \frac{B(m)}{ESR} \tag{4.3}$$

Where:

D_e = equivalent dimension in meter
B = excavation span or width or height (Table 4.2) = 12.6m, for the power tunnels
ESR = excavation support ratio =1.0 to use up-to-date Q-support design chart

Therefore, D_e is estimated as:

$$D_e = \frac{12.6}{1.0} = 12.6\text{m} \qquad \text{Use De = 13m}$$

(c) Determination of Rock Bolt Length

The length L of rockbolts can be estimated from the excavation width (B) and the excavation support ratio (ESR) [2, 13] as (Eq. 4.2):

$$L = 2 + \frac{0.15B}{ESR}$$

Where:

B = excavation span or width or height (Table 4.2) = 12.6m, for the power tunnels
ESR = excavation support ratio =1.0 to use up-to-date Q-support design chart

$$L = 2 + \frac{0.15*B}{ESR} = 2 + \frac{0.15*12.6m}{1.0} = 3.89m$$ Use L = 4m

(d) Determination of Maximum Unsupported Span

The maximum unsupported span can be estimated from Equation 4.7 as [23]:

Maximum span (unsupported) = $2ESR\ Q^{0.4} = 2*1.6*Q^{0.4}$ (4.4)

Where:

ESR = excavation ratio = 1.6 for hydropower tunnel
Q = rock mass quality

The calculations of the maximum unsupported span are performed in Table 4.6for each support category and Q values.

(e) Determination of Rock Bolt Spacings and Shotcrete Thicknesses

Actually every empirical method has its own limitations. The Q-method is most applicable within a certain range of parameters as shown by the shaded area in Figure 2.3. If the excavation supports are within the shaded area, the up-to-date Q-support design chart works best, i.e., the tunnel will be Stable, Safe and Economical [5].
Select the rock bolts spacings and thicknesses of the shotcrete (sprayed concrete) for each support types by using up-to-date Q-support design chart (Figure 2.3) and the calculated values of:

- Rock bolt length, L = 4m,
- Equivalent dimension, D_e = 13m,
- Rock classes, and
- Q-values (to be conservative, the minimum Q value is considered in each range).

The design of ground excavation initial supports are performed in excel sheet and summarized in Table 4.6.

Table 4.6 Design of Initial supports using up-to-date Q-support design chart.

TUNNEL GEOLOGY		U/SW TRACHYTE				TRACHYTIC BRECCIA	
Q-VALUESS		8.25 - 120	1.4 - 8.25	0.15 - 1.4	0.04 - 0.15	0.11 - 0.51	0.04 - 0.11
SUPPORT TYPE		A	B	C	D	D	E
DESIGN SUPPORT	ROCK BOLTS (L, @)*	4m, 2.3m	4m, 2.1m	4m, 1.5m	4m, 1.2m	4m, 1.5m	4m, 1.2m
	SHOTCRETE	R4	R5	R7	R8	R7	R8
Unsupported span		7 - 21m	3 - 7m	1.5 - 3m	0.80 -1.5m	1- 2m	0.80 -1m

* L = Rock bolt length, @ = Rock bolt spacing

KEY:

- R4: Systematic Bolting and Unreinforced shotcrete of thickness 40-100mm.
- R5: Fibre reinforced shotcrete of thickness 50-90mm.
- R7: Fibre reinforced shotcrete of thickness 120-150mm.
- R8: Fibre reinforced shotcrete of thickness greater than 150mm, reinforced ribs of shotcrete and bolting.

All but R8 (region 8 in Figure 2.3) supports are in the shaded region of Figure 2.3.For the rock class region type R8: supplementary methods or evaluations or calculations should be applied during tunnel excavations. According to the up-to-date Q-support design chart, these supports will make the tunnel stable, safe and economical because these supports are within the shaded region in which the Q-support design chart works best.

There are two types of reinforced shotcrete: steel fibre reinforced shotcrete (Q-method) and traditional wire mesh reinforced plain shotcrete (RMR- method). The designer used welded wire meshes (Φ5mm @10cmx10cm) with shotcrete based on the RMR method whereas the Author proposes fibres with shotcrete based on the Q-method. Mesh installation is labour intensive and relatively unsafe, as the personnel are exposed to rock wedge falls whereas fibre reinforced shotcreting is simple, fast and safe. The designer used 6m rock bolts whereas the Author proposes 4m rock bolts. The rock bolt spacings and the shotcrete thicknesses are explained in Table 4.7.

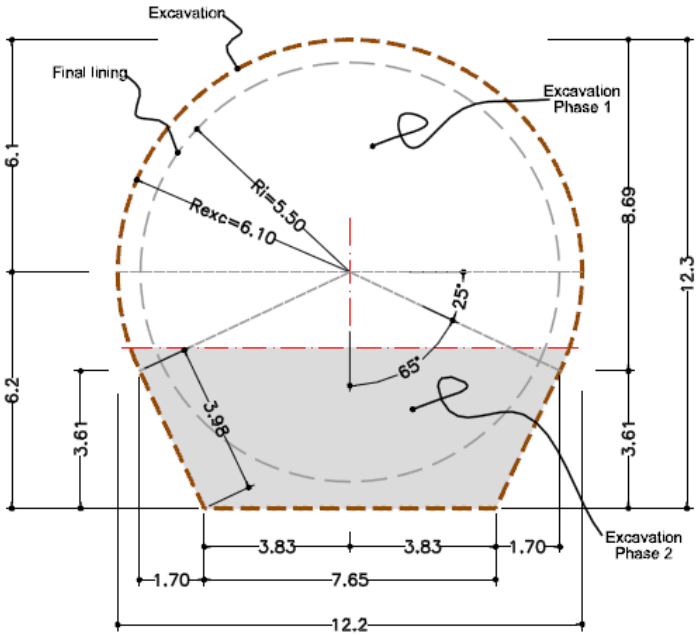
Table 4.7 Ground excavation supports design comparison (Table 3.2 and Table 4.6)

Tunnel Geology			Unweathered/slightly weathered Trachyte				Brecciated Trachyte	
Support Type			A	B	C	D	D	E
RMR -Values			76-100	61-75	41-60	28-40	38-51	28-37
Q -Values			8.25-120	1.4-8.25	0.15-1.4	0.04-0.15	0.11-0.51	0.04-0.11
Designer	GSI	Bolt spacing (m)	spot	2.5	1.8	1.5	1.5	-
		Shotcrete (mm) ≥	-	100	150	250	250	300
		Steel sets	-	-	-	-	-	HEB180 @1m
	Q	Bolt spacing (m)	2.3	1.8	1.3	1.2	1.3	1.2
		Shotcrete (mm)	-	50-90	90-120	150	120-150	150
Author	Q	Bolt spacing (m)	2.3	2.1	1.5	1.2	1.5	1.2
		Shotcrete (mm)	40-100	50-90	120-150	>150	120-150	>150
		Reinforced ribs of shotcrete	-	-	-	Yes	-	Yes

(f) Rock bolt quantity calculations

Number of rock bolts can be calculated for each rock bolt pattern and for each type of supports (A, B, C, D or E) using Eq. 4.5and Figure 4.26 is used to calculate the supportable length (the crown part plus the side walls) (Table 4.8).

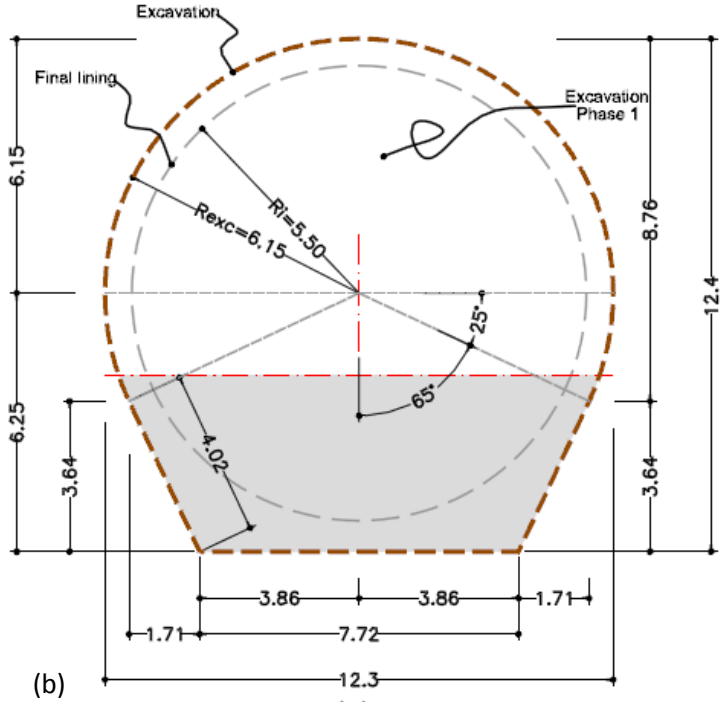
TYPE A, B



(a)

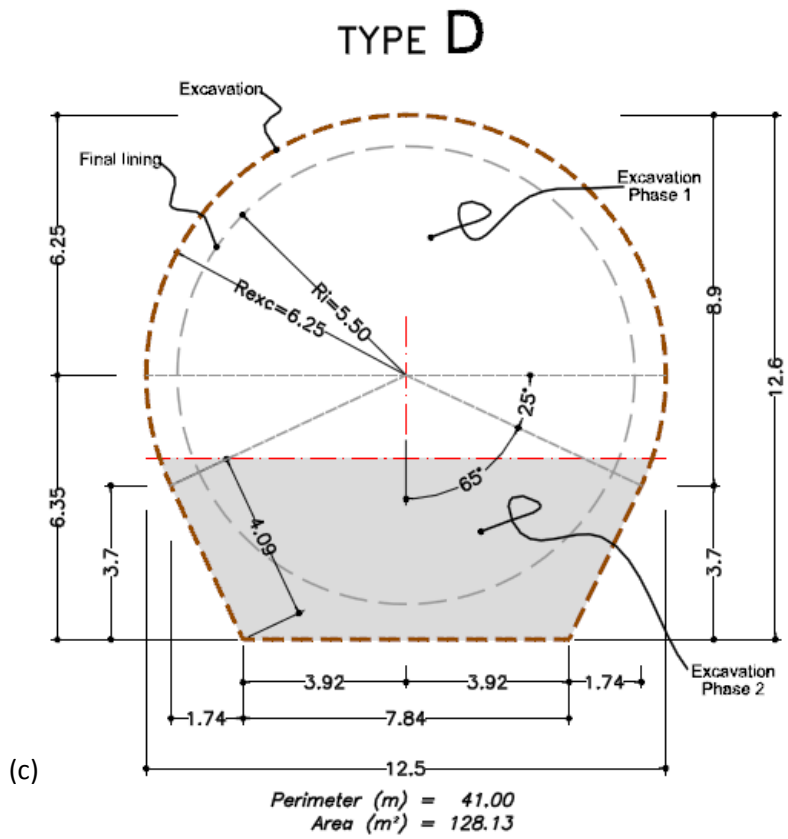
Perimeter (m) = 40.02
Area (m²) = 122.05

TYPE C



(b)

Perimeter (m) = 40.34
Area (m²) = 124.06



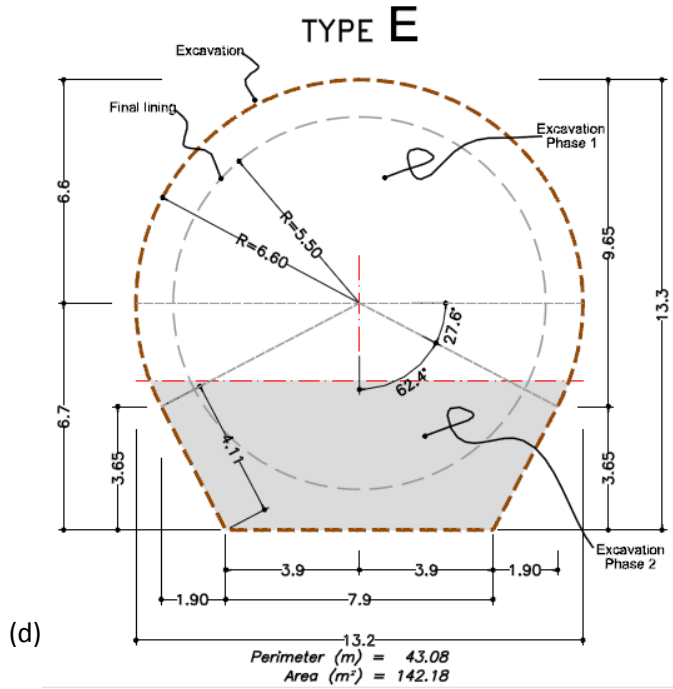


Figure 4.26. Typical Tunnel Cross-sections [from (a) to (d)] for two phase excavations for each Type of Support [15, 16], See Appendix F

Table 4.8 Spacing, length and numbers of the rock bolts for each type of support [as per the Author]

Tunnel Geology	Unweathered/Slightly weathered Trachyte				Brecciated Trachyte	
Support type	A	B	C	D	D	E
Rock Bolt L = 4m @ spacing (m):	2.3m	2.1m	1.5m	1.2m	1.5m	1.2m
Crown + Sidewalls (m)	32.36m	32.36m	32.62m	33.16m	33.16m	35.28m
No. of bolts per pattern	15	17	23	27	23	31

$$\text{Number of rockbolts per pattern} = \frac{(\text{Crown} + \text{side walls})\ (\text{m})}{\text{Rock bolt spacing}\ (\text{m})} + 1 \qquad (4.5)$$

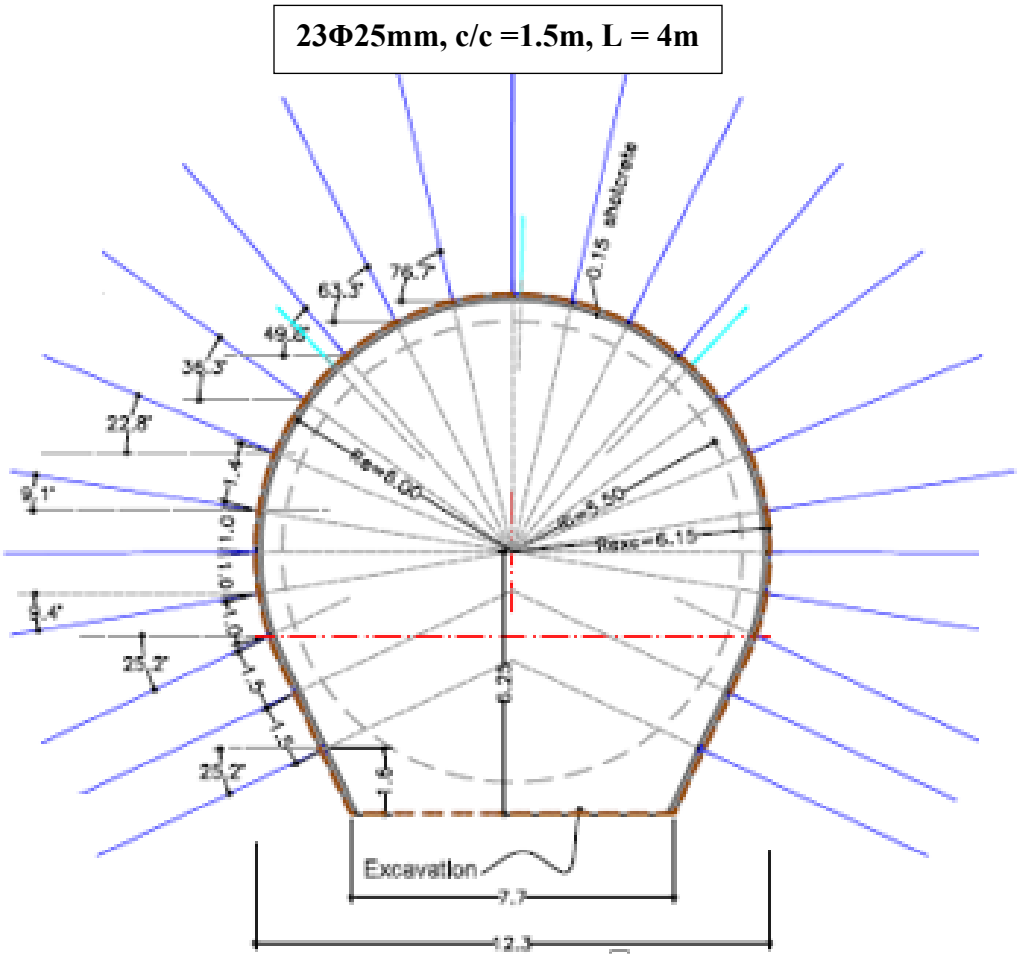


Figure 4.27. Pattern bolting for support type C and D, See Appendix F

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

The objectives of this research were to evaluate the design and construction approaches, to assess challenges encountered in the design and construction of the GG III hydropower tunnels, and to recommend future tunnel design and construction considerations based on the lessons learned from the GG III hydropower tunnels. Conclusions and recommendations are presented below as well as suggestions for future study also have been proposed.

5.1. Conclusions

In this research, the author has found design and construction problems of the Gilgel Gibe III hydropower tunnels. The lessons learned from the GG III hydropower tunnels include the following.

The design problems include:

- The most globally used empirical design methods include geomechanics rock classification (RMR) and the rock mass quality (Q). The designer used RMR method for rock classifications, and geological strength index (GSI) values and Q method for primary support design. The RMR values calculated from the rock properties of the GG III HPTs are between 20 and 80 (rock class II, III & IV). The correlation values of GSI and RMR might be valid when the RMR values are less than fifteen (15) but not. Thus, the GSI values are inappropriate, however, they have calculated 6m rock bolt length from the GSI values and that have been used as a primary supports in the construction of the GG III HPTs. Therefore, the ground excavation primary supports are improper and uneconomical.
- In the RMR empirical design method, average stand-up time is one of the design parameters. The excavated ground must be supported before it loses its self-supporting capability. But the time of self-supporting capability is not considered in the design and construction processes of the GG III HPTs. In the review of the GG II headrace tunnel design data (the same contractor), the stand-up time and the rock strength have been used with wrong interpretations. The data obtained from the GG II headrace tunnel includes: The rock mass rating (RMR) values range from 17 to 19; rock class V (very poor rock); UCS = 120Mpa; stand up time greater than 48 hours. According to the geomechanics rock classification (RMR) method, for this rock class V (very poor rock, RMR between 17 and 19), the uniaxial compressive strength (UCS) is between 2 and 10Mpa (not 120MPa) (Table A-1 in Appendix A) and the stand-up time is 30 minutes (not more than 48hours) for 1m unsupported excavated span (Table 4.1). Therefore, due to the unknown time of the self-supporting capability of the excavated rocks/soils the construction activities will be in very high danger.

The construction problems include:

- They have not followed the correct procedure of NATM. According to NATM, the most widespread techniques of partial face excavation are (i) top heading and bench excavation, and (ii) sidewall drift excavation. Top heading and bench excavation

technique is used in the construction of the GG III HPTs inappropriately, i.e., the bench excavation has been started after 100% excavation of the full length of the crown part. The bench must be excavated, with the bench length being 1 to 1.5 times the tunnel diameter, before the ground loses its self-supporting capability so that the reinforcement structure can be timely closed. In addition, the bench excavation has been advanced by blasting 15 meters to 20 meters per stage which is out of the excavation design procedures of the project.

- They are not implemented the proper design supports in the construction. The initial supports designed for the rock class IV, for example, were: (a) Shotcrete Thickness ≥25cm, (b) Welded Wire Mesh: ϕ5mm, 10cmx10cm, (c) Pattern Rock Bolt: 23ϕ25mm, L= 6m at spacing of 1.5m with cement grouting. However, these supports are not implemented in the actual construction site.
- Convergence and settlement measures as well as drainage holes were not installed in the construction of river diversion tunnels.

The consequences of the above problems are:

- Rock and shotcrete wedges failure occurred at stations 0+175, 0+178 and 0+185 in the central diversion tunnel and rock wedge failure happened at station 0+843 in the right power tunnel.
- The rock wedge failure at the beginning of manifold (station 0+843) has led to a los of a worker while shotcreting the excavated rock face.
- After seventy four (74) days and sixty seven (67) meters advance of the construction of central river diversion tunnel, the tunnel section collapsed at station 0+175 which was extended to both directions (onward to 0+250 and rearward to 0+160). Due to this collapse:
    - The unexpected quantity and various types of rock supports, ground improvement materials, various construction techniques, a construction of additional 63.25m access tunnel (adit), etc. are used in the remedial works. The rock supports and ground improvement materials used for the remedial works include steel ribs, steel plates, wire mesh, shotcrete, cement mortar, anchor bolts, fixing anchors, consolidation grouting of cement mortar and polyurethane foam, etc.
    - The remedial works took almost nine months, from May 2007 to February 2008.
    - As a result of this collapse, the construction cost and the construction time of the project absolutely increased.

The design and construction approaches of the GG III hydropower tunnels are used in this research. The ground excavation primary supports can be designed by one of the three design methods (Empirical methods, or Observational methods or analytical methods) whereas the permanent supports (linings) are purely a structural design. The design procedures of the primary supports are the same for all function of tunnels, i.e., the design depends on geotechnical interpretative reports. The design of permanent supports depends on the function of tunnels. Therefore, the lessons learned from the GG III hydropower tunnels are very important and that can be applied in any future tunnels in Ethiopia.

5.2. Recommendations for future Tunnels in Ethiopia

The author recommends the following based on the problems encountered in the design and construction of the GG III hydropower tunnels.

1. The most globally used empirical design methods such as the geomechanics rock classification (RMR) and the rock mass quality (Q) shall be used in the design and construction of tunnels.
2. In the empirical design methods, all the design parameters shall be evaluated/used during design and construction processes.
3. In the GG III HPTs, Rocscience software products, e.g. phase 2, unwedge, etc… are used for the design analysis. But these softwares mainly used the GSI values as input. Since the correlation values of the GSI with RMR are inappropriate for the RMR values greater than fifteen (15), it's better to use discrete element methods (DEM) or finite element methods (FEM).
4. Whatever the contract type, the consulting company shall carry out his obligation properly during design and construction processes.
5. There must be installation of controlling instrumentations to control convergence and settlement in a tunnel construction.
6. Tunnel construction is very challenging and full of risks. Thus, technically skilled and experienced persons shall participate in the tunnel design and construction processes because the geological and geotechnical uncertainties can be tackled effectively using proper rock classifications on site.

Finally, the author suggests for any researcher, he/she can do his/her investigations on the following:

1. Verification analysis for the relationship of GSI with RMR using different modelling software.
2. The reasons of Gigel Gibe II headrace tunnel collapse and the remedial measures used/applied.

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APPENDICES

Appendix A: Tables for the calculation of the RMR values.

Appendix B: Profiles of Power Tunnels and water Diversion Tunnels.

Appendix C: RMR rock classification Procedure and rock Classes.

Appendix D: Plan of Diversion, Power, Access, and Drain Tunnels.

Appendix E: Phase Excavations-Drilling/Charging/Blasting Patterns.

Appendix F: Convergence Measurement Section, Rock Bolt Grouting Mixes and Typical Sections of Tunnels.

Appendix G: Attachments of Drawings for Remedial Works.

Appendix H: Attachments of Drawings for permanent lining, Consolidation Grouting, etc.

Appendix A: Tables for the calculation of the RMR values.

Table A-1. Strength of Intact Rock Material [47, 24]

Qualitative description	UCS (Mpa)	Point load strength index (Mpa)	Rating, R_1
Exceptionally strong	>250	8	15
Very strong	100-250	4-8	12
Strong	50-100	2-4	7
Average	25-50	1-2	4
Weak	10-25	Use of UCS is preferred	2
Very weak	2-10	Use of UCS is preferred	1
Extremely weak	1-2	Use of UCS is preferred	0

Note: At compressive strength less than 0.6 Mpa, many rock material would be regarded as soil.

Table A-2. Rock quality designation RQD [47, 24]

Qualitative description	RQD	Rating, R_2
Excellent	**90-100**	**20**
Good	**75-90**	**17**
Fair	**50-75**	**13**
Poor	**25-50**	**8**
Very poor	**<25**	**3**

Table A-3. Spacing of discontinuities [47, 24]

Description	Spacing (m)	Rating, R_3
Very wide	**>2**	**20**
Wide	**0.6-2**	**15**
Moderate	**0.2-0.6**	**10**
Close	**0.06-0.2**	**8**
Very close	**<0.06**	**5**

Note: If more than one discontinuity sets are present and the spacing of discontinuities of each set varies, consider the set with lowest rating [47, 24].

Table A-4. Condition of discontinuity [47, 24]

Description	Rating, R_4
Very rough and unweathered, wall rock tight and discontinuous, no separation	**30**
Rough and slightly weathered, wall rock surface separation <1mm	**25**
Slightly rough and moderately to slightly weathered, wall rock surface separation <1mm	**20**
Slickensided wall rock surface or 1-5mm thick gouge or 1-5mm wide continuous discontinuity	**10**
5mm thick soft gouge, 5mm wide continuous discontinuity	**0**

Table A-5. Ground water condition [47, 24]

Inflow per 10m tunnel length (Liter/minute)	**None**	**<10**	**10-25**	**25-125**	**>125**
Joint water pressure/major principal stress	**0**	**0-0.1**	**0.1-0.2**	**0.2-0.5**	**>0.5**
General description	**Completely dry**	**damp**	**wet**	**dripping**	**Flowing**
Rating, R_5	**15**	**10**	**7**	**4**	**0**

Table 2.6. Adjustment for joint orientation [47, 24]

Joint orientation Assessment for Tunnels	**Very Favourable**	**Favourable**	**Fair**	**Unfavourable**	**Very unfavourable**
Rating, R_6	**0**	**-2**	**-5**	**-10**	**-12**

Appendix B: Profiles of Power Tunnels and water Diversion Tunnels.

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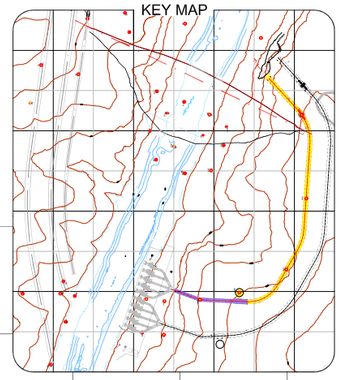

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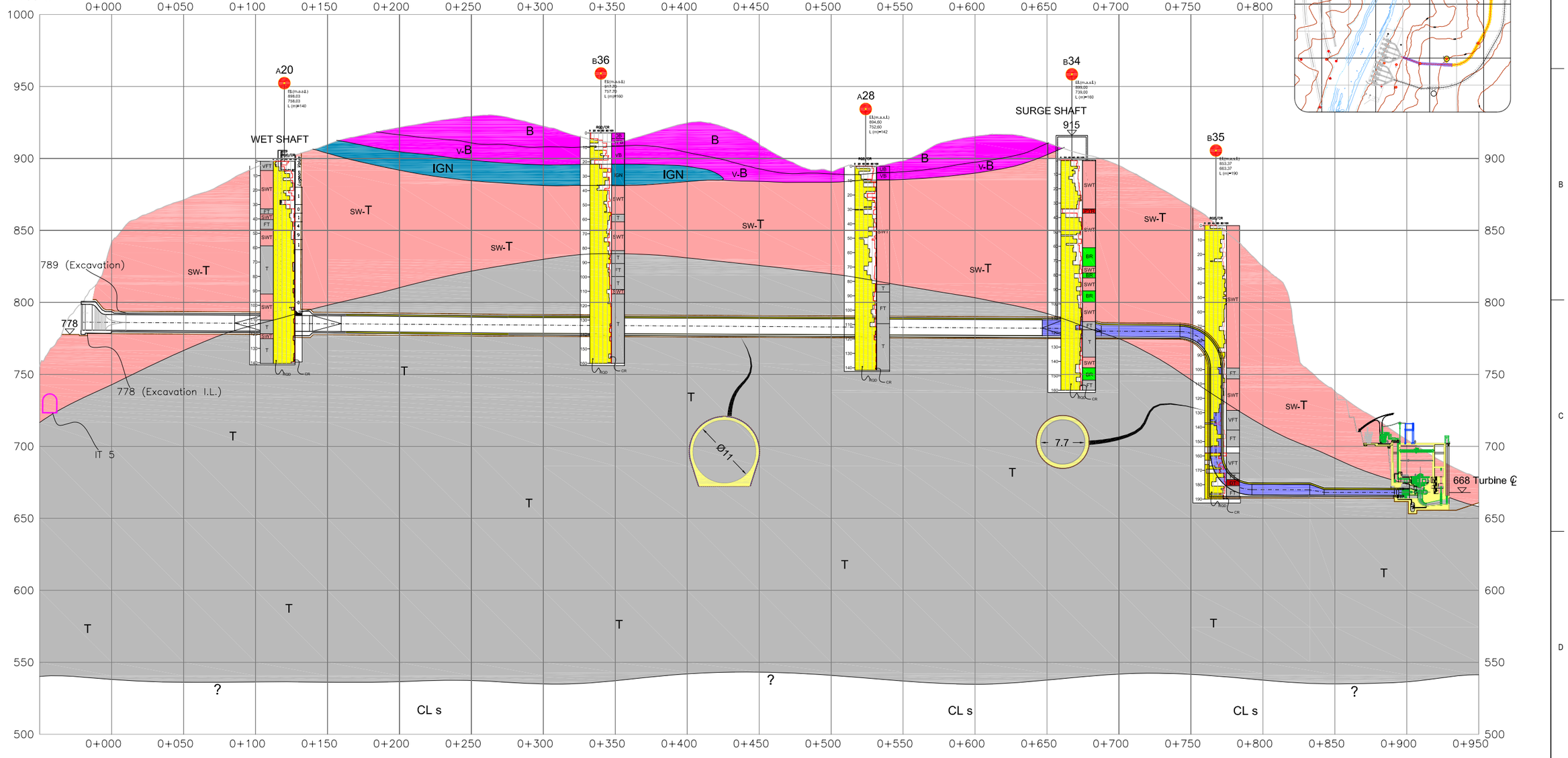

LEGEND

BOREHOLES
- COMPLETED
- PLANNED

SEISMIC PROFILE (SL202, SL203)
PROJECTED
4.0 km/s SEISMIC PROFILE VELOCITY
1600 SEISMIC ISOPAQUES (Estimated)

TUNNELS
IT

GEOMECHANICAL STATION
R6 | RMR | MAIN JOINT FAMILIES | DIP/DIP DIRECTION

- MAIN FAULT
- INFERRED FAULT
- FRACTURE LINEATIONS

ROCKS / SOILS DESCRIPTION

- T = TRACHYTE
- DT = DECOMPOSED TRACHYTE
- WT / SWT = W / SW TRACHYTE
- VT = VESICULAR TRACHYTE
- B = BASALT
- CB = COLUMNAR BASALT
- VB = VACUOLAR BASALT
- SOIL (USCS)
- BK = BACKFILL
- BR = BRECCIA
- COL = COLLUVIUM
- ALL = ALLUVIUM
- PY = PYROCLASTIC ROCK
- I = IGNIMBRITE

D = DECOMPOSED
F = FRACTURED
VF = VERY FRACTURED
SW = SLIGHTLY WEATHERED

0 10 20 30 40 50 100m
scale 1:2000 (A3 size)

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 505 = INVESTIGATION
GSC = GEOLOGICAL SECTIONS
RIGHT POWER TUNNEL Profile

issue	day	month	year	drawn	checked	approved
A	06	11	2008	EDS	TER	TER

modifications

section	kind works	type	ROMA office	serial n°	rev.	date
505	GSC	DSP	302	A	06	06 Nov 2008

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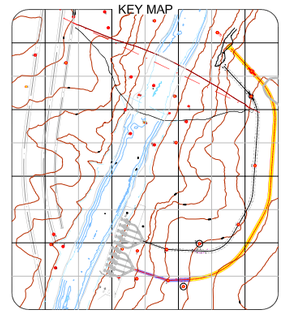

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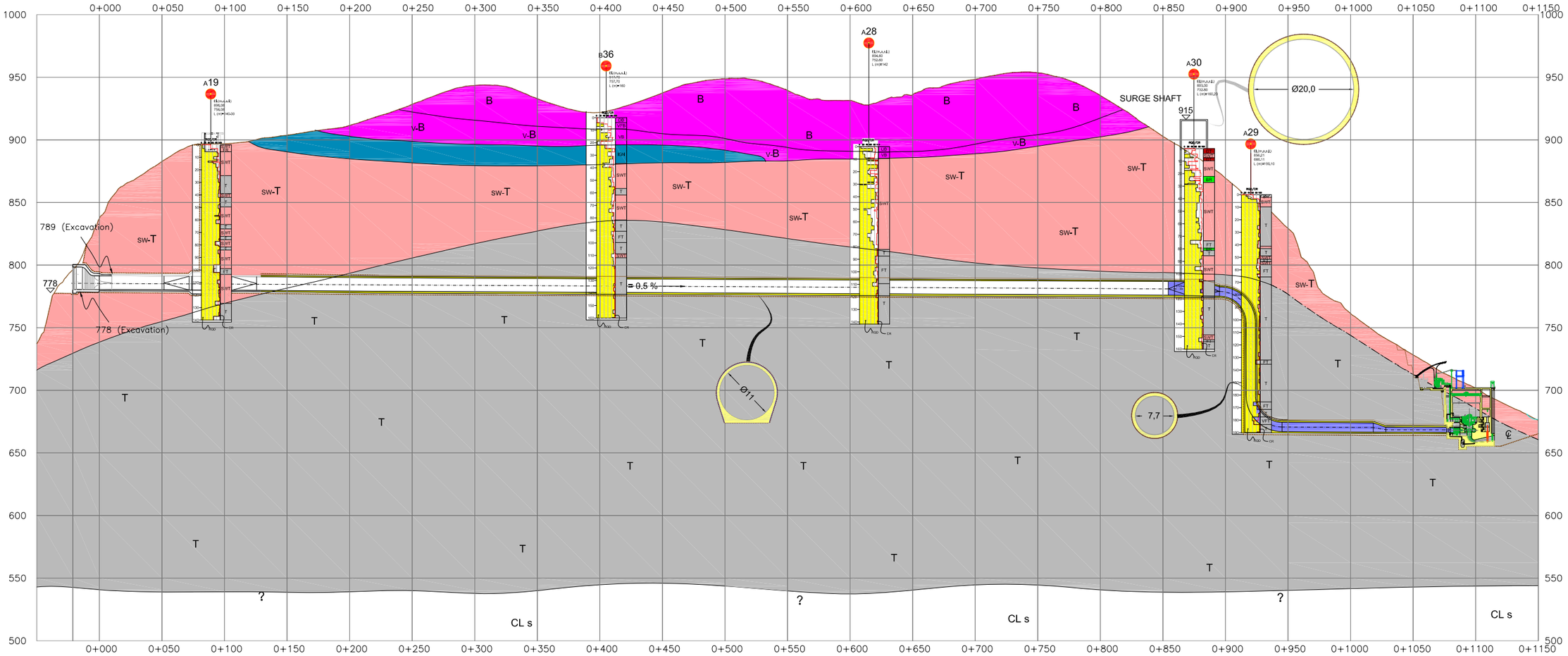

LEGEND

BOREHOLES
- COMPLETED
- PLANNED

SEISMIC PROFILE
PROJECTED
4.0 km/s SEISMIC PROFILE VELOCITY
1600 SEISMIC ISOPAQUES (Estimated)

TUNNELS
IT

GEOMECHANICAL STATION
R6 | RMR | MAIN JOINT FAMILIES | DIP/DIP DIRECTION

- MAIN FAULT
- INFERRED FAULT
- FRACTURE LINEATIONS

ROCKS / SOILS DESCRIPTION

- T = TRACHYTE
- DT = DECOMPOSED TRACHYTE
- WT / SWT = W / SW TRACHYTE
- VT = VESICULAR TRACHYTE
- B = BASALT
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- VB = VACUOLAR BASALT
- SOIL (USCS)
- BK = BACKFILL
- BR = BRECCIA
- COL = COLLUVIUM
- ALL = ALLUVIUM
- PY = PYROCLASTIC ROCK
- I = IGNIMBRITE

D = DECOMPOSED
F = FRACTURED
VF = VERY FRACTURED
SW = SLIGHTLY WEATHERED

0 10 20 30 40 50 100m
scale 1:2000 (A3 size)

issue	day	month	year	drawn	checked	approved
A	06	11	2008	EDS	TER	TER

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 550 = POWER TUNNEL
GEO = GEOLOGY
LEFT POWER TUNNEL Profile

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505	GSC	DSP	301	A	06	06 Nov 2008

\\SERVER02\SP_progetti\GIBE III\500 - LEVEL 2 DESIGN\505 - INVESTIGATIONS\02 GSC - Geological Sections\505 GSC D SP 301 A- PTL GEOL PROFILE - 06nov08.dwg eds, sp - Roma 10/NOV/2008 17:41:02

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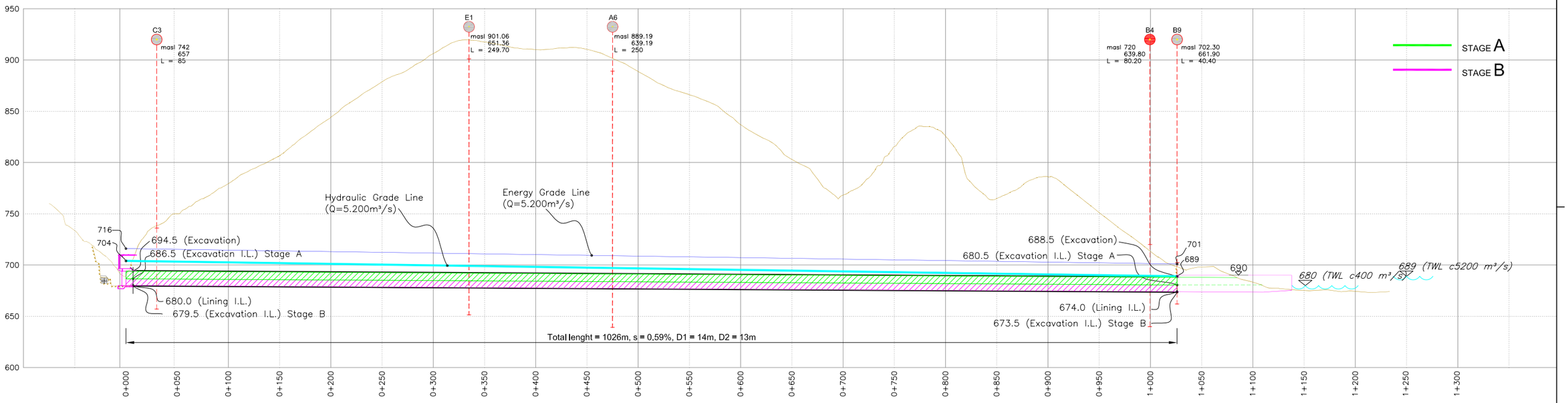
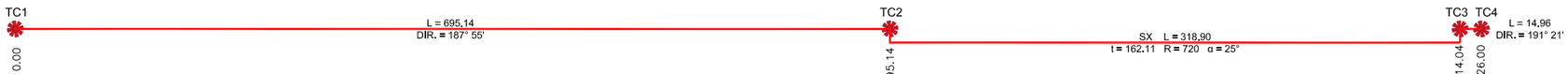
Z:\GIBE III\500 - LEVEL 2 DESIGN\510 - RIVER DIVERSION\TUN - TUNNELS\D - DRAWINGS\510-TUN-D-SP-002-A, TC Profile, rev.04_jan_2008.dwg dal, sp - Roma 07/JAN/2008 19:05:41

TUNNEL CENTRAL

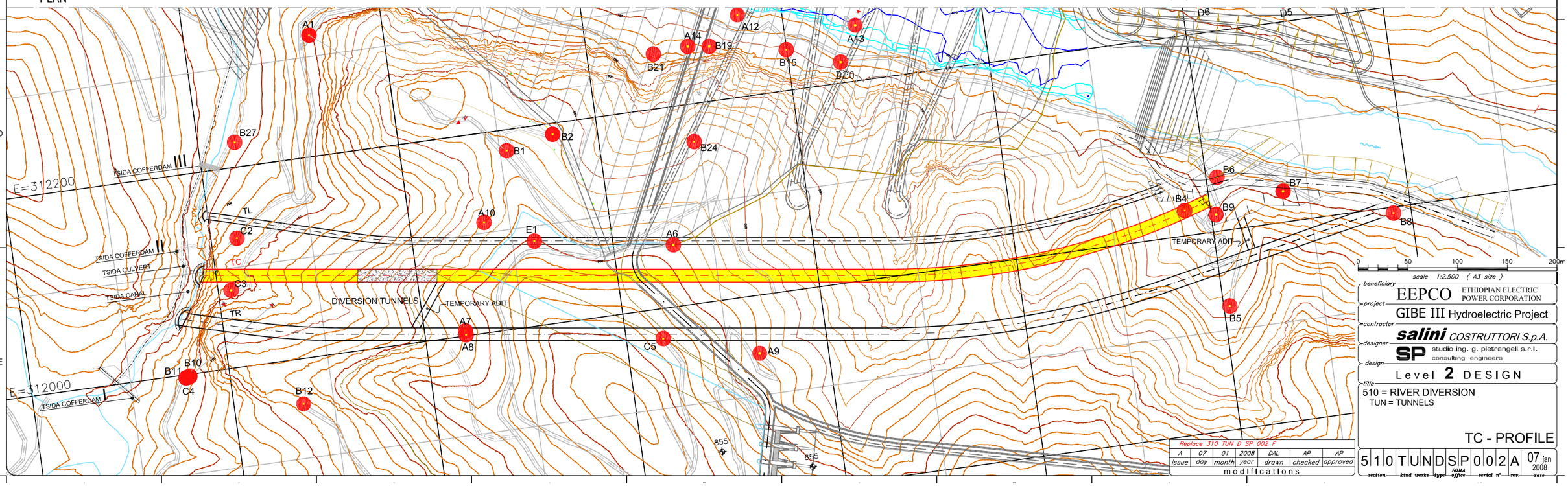


WF T / BR

Rock Type	U/SW F T								WF T – WF BR	U/SW F T						U/SW F T												
GSI (design)	55–71	35–55	55–71	35–55	55–71	35–55	55–71	32–46		32–46	35–55	35–55	35–55	32–46	35–55	35–55	55–71	35–55	55–71	55–71	35–55	55–71	35–55	55–71	35–55	35–55	35–55
Temporary Support Type	B	C	B	C	B	C	B	D	REMEDIAL WORKS	E	C	D	C	D	C	C	B	C	B	B	C	B	C	B	C	D	E
Design MAX Internal Pressure (average)	170								155							140		110										
Final Lining Type	1	1	3	1	3	1	3	1		1	2	2	2	1	2	3	4	3	5	5	3	5	3	5	3	1	1
ch.	0.00	12	21	40	70	100	114	142	165	245	278	295	310	320	335	348	402	708	757	792	872	910	932	943	960	1000	1017	1026



PLAN



Replace 310 TUN D SP 002 F

issue	day	month	year	drawn	checked	approved
A	07	01	2008	DAL	AP	AP

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION

project: GIBE III Hydroelectric Project

contractor / designer: salini COSTRUTTORI S.p.A.

design: SP studio ing. g. pietrangeli s.r.l. consulting engineers

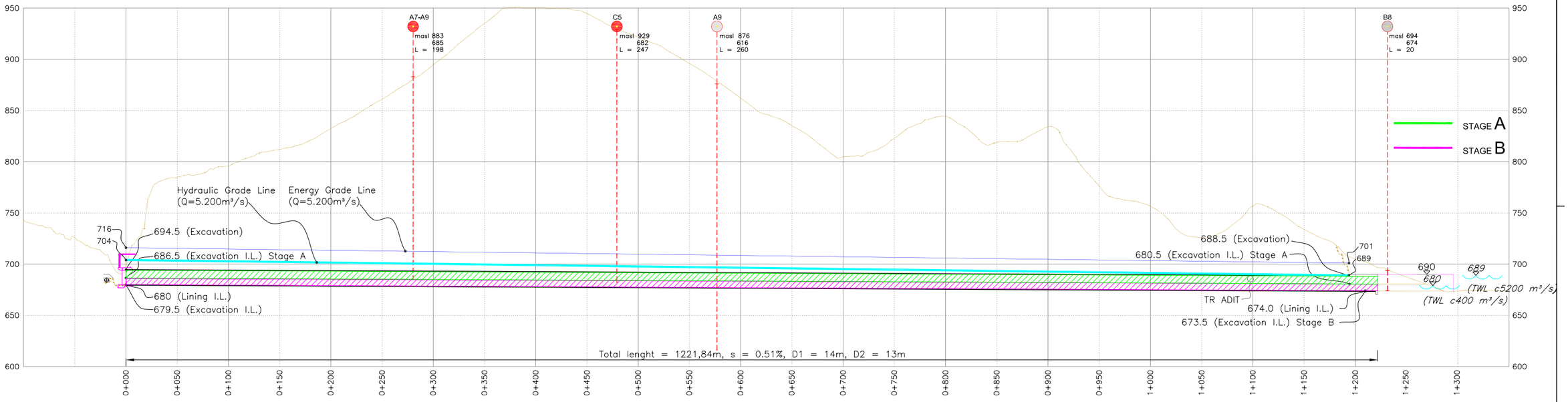
Level 2 DESIGN

title: 510 = RIVER DIVERSION
TUN = TUNNELS

TC - PROFILE

section	kind works	type	ROMA office	serial n°	rev.	date
510	TUN	D	SP	002	A	07 jan 2008

TUNNEL RIGHT



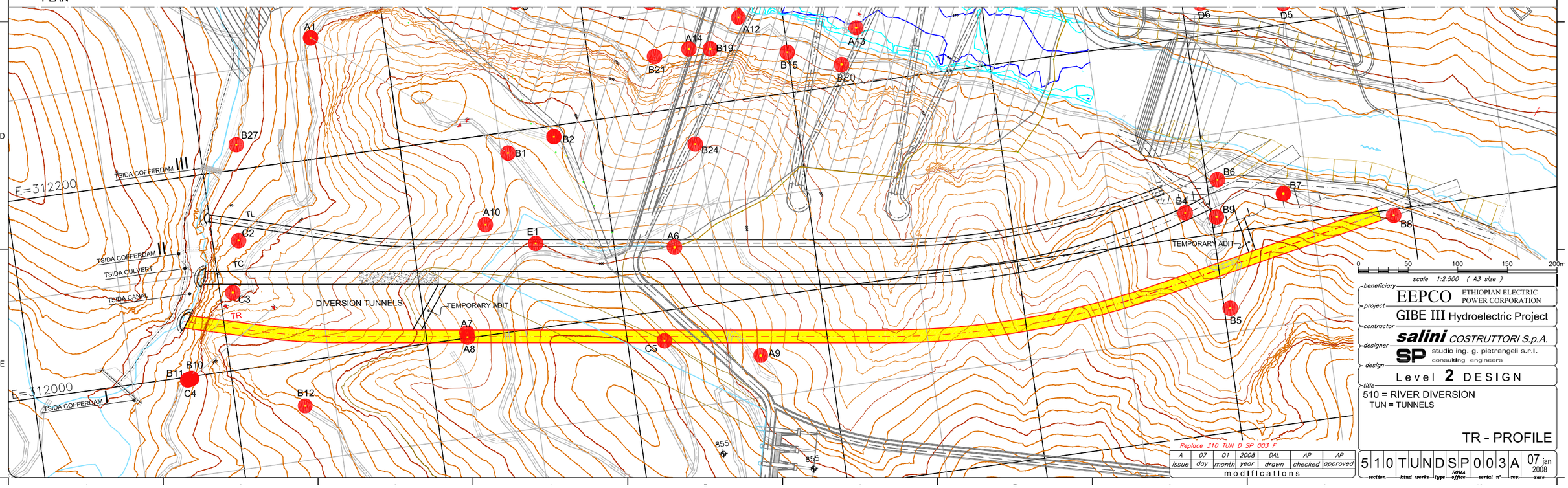
Rock Type	U/SW F T						WF T–WF BR			U/SW F T																							
GSI (design)	35–55	35–55	55–71	35–55	55–71	55–71	32–46	23–32	32–46	55–71	55–71	35–55	55–71	55–71	55–71	35–55	55–71	35–55	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71	55–71
Temporary Support Type	E	C	B	C	B	C	D	E	D	B	B	C	B	C	B	C	B	C	B	C	B	C	B	C	B	C	B	C	B	C	B	C	E
Design MAX Internal Pressure (average)	170						155			140						110																	
Final Lining Type	1	1	3	1	3	2	2	1	2	4	5	3	5	5	5	3	5	2	5	5	5	4	1										

ch. 0.00 6 20 63 71 170 222 281 320 408 648 771 805 817 833 993 1005 1045 1080 1095 1115 1160 1206 1221.84

TR1 L = 58.76 DIR. = 195° 35' | TR2 SX L = 95.70 t = 47.86 R = 730 α = 8° | TR3 L = 95.70 DIR. = 187° 55' | TR4 L = 283.76 t = 143 R = 844 α = 19° | TR5 L = 238.84 DIR. = 191° 21'

0.00 58.76 154.34 699.10 982.87 1221.84

PLAN



Replace 310 TUN D SP 003 F

issue	day	month	year	drawn	checked	approved
A	07	01	2008	DAL	AP	AP

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 510 = RIVER DIVERSION
TUN = TUNNELS

TR - PROFILE

section	kind works	type	ROMA office	serial n°	rev.	date
510	TUN	DSP	003	A	07	jan 2008

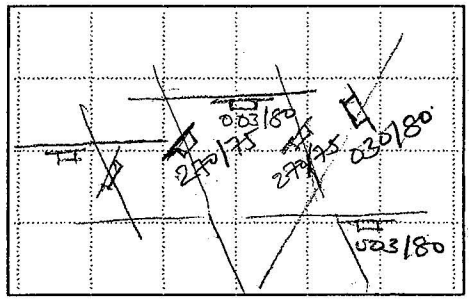
Z:\GIBE III\500 – LEVEL 2 DESIGN\510 – RIVER DIVERSION\TUN – TUNNELS\D – DRAWINGS\510–TUN–D–SP–003–A,TR Profile, rev.04_jan_2008.dwg dal, sp – Roma 07/JAN/2008 19:03:20

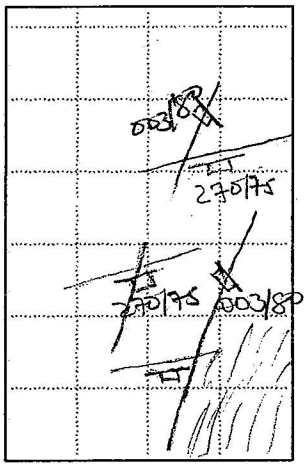
Appendix C: RMR rock classification Procedure and rock Classes.

Salini COSTRUTTORI S.P.A GEOTECHNICAL SECTION	GEOLOGICAL & GEOTECHNICAL RECORDS OF TUNNEL FACE MAP	GIBE III HYDROELECTRIC POWER PROJECT ETHIOPIAN ELECTRIC POWER CORPORATION
	LOCATION: Left - Diversion Tunnel	Inlet

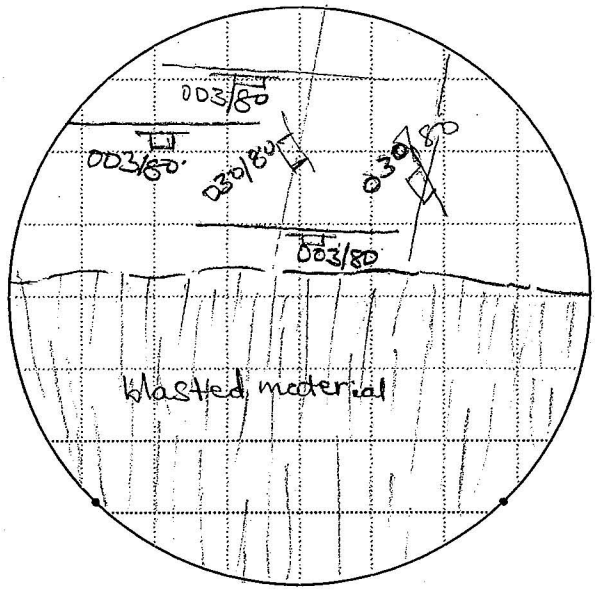
TUNNEL HEAD:	INLET	SEQ. NO:	020	DATE:	Jan 24, 2007	TIME:	9:10 A.M	SHIFT:	day
TUNNEL AXIS:	175°	CHAINAGE (m):	~0+230.0						

ROCK MASS DESCRIPTION: Grey to Greenish grey, fine grained, slightly to moderately weathered, Strong Trachyte with medium persistent, closely Spaced joints, smooth to Slickensided & planar joint Surfaces, partly open joints along with altered and fractured fault affected rocks.
Dry ground water conditions.

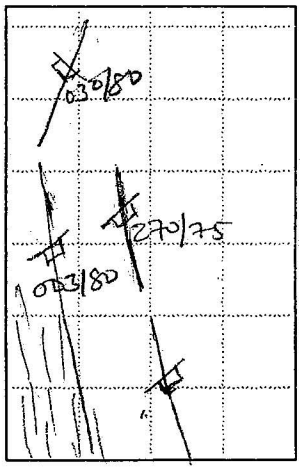




L.S.W



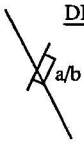
FACE



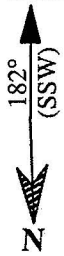
R.S.W

LEGEND:

DISCONTINUITY TRACES:



a: dip direction
b: dip amount



SCALE:

1) Joint spacing:

Sav	>2m	0.6m - 2m	20cm - 60cm	6cm - 20cm	<6cm
index	20	15	10	8	5

INDEX FOR RMR EVALUATION: 8

MAPPED BY: Zinabu

Rock Mass Rating system (After Bieniawski 1989)

TUNNEL HEAD:	INLET	SEQ. NO:		DATE:		TIME:		SHIFT:	
TUNNEL AXIS:		CHAINAGE (m) :							

2) RQD (Rock Quality Designation):

RQD = 115-3.3Jv	0 - 5.3	5.3 - 9.6	9.6 - 16.8	16.8 - 26.9	>26.9
	90% - 100%	75% - 90%	50% - 75%	25% - 50%	<25%
index	20	17	13	8	3

3) Compressive strength of intact rock: **INDEX FOR RMR EVALUATION:** 8

σ_c (MPa)	>250	100 - 250	50 - 100	25 - 50	5 - 25	1 - 5	<1
index	15	12	7	4	2	1	0

INDEX FOR RMR EVALUATION: 7

4) Joint condition:

LENGTH	<1m	1 -3m	3 - 10m	10 - 20m	>20m	
index	6	4	2	1	0	**INDEX:** 2
OPENING	0	< 0.1mm	0.1 - 1mm	1 - 5mm	>5mm	
index	6	5	4	1	0	**INDEX:** 3
ROUGHNESS	very rough	rough	slightly rough	smooth	slickensided	
index	6	5	3	1	0	**INDEX:** 1
FILLING	none	hard & <5mm	hard & >5mm	soft &<5mm	soft &>5mm	
index	6	4	2	2	0	**INDEX:** 1
WEATHERING	unweathered	slightly weathered	moderately weathered	highly weathered	decomposed	
index	6	5	3	1	0	**INDEX:** 3

5) Ground water condition:

Condition	dry	damp	wet	dripping	flowing
index	15	10	7	4	0

INDEX FOR RMR EVALUATION: 15

6) Rating Adjustment for Discontinuity orientations:

very favourable	favourable	fair	unfavourable	very unfavourable	fair	fair
0	-2	-5	-10	-12	-5	-5
strike perpendicular to tunnel axis				strike parallel to tunnel axis		irrespective of strike
drive with dip		drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 0°-20°

INDEX FOR RMR EVALUATION: -10

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	poor rock	Very poor rock

TOTAL RMR (ROCK MASS RATING):	38
ROCK CLASS:	IV (poor rock)

For the CONTRACTOR	For the EMPLOYERS REPRESENTATIVE
Name: Zinabu	Name: Yared D.
Signature:	Signature:

Rock Mass Rating system (After Bieniawski 1989)

salini **COSTRUTTORI S.p.A** **GEOTECHNICAL LABORATORY**	GEOLOGICAL MAPS	**GIBE III HYDROELECTRIC PROJECT** **ETHIOPIAN ELECTRIC POWER CORPORATION** REV.0

ROCK MASS CLASSIFICATION PARAMETERS

LOCATION	LEFT POWER TUNNEL FLOOR								
ELEVATION	FROM	-	TO	-	CHAINAGE	FROM	0+100	TO	0+070

WIDTH(m)	FROM	0	TO	10	DATE	4-Jun-12

1) Joints spacing

sav	> 2 m	0,6 m - 2 m	20 cm - 60 cm	6 cm - 20 cm	< 6 cm
index	20	15	10	8	5

INDEX FOR ROCK MASS RATING EVALUATION **10**

2) RQD (Rock Quality Designation)

RQD	90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
index	20	17	13	8	3

INDEX FOR ROCK MASS RATING EVALUATION **17**

3) Compressive strength of intact rock

σ_c	> 250	100 - 250	50 - 100	25 - 50	5 - 25	1 - 5	< 1
index	15	12	7	4	2	1	0

INDEX FOR ROCK MASS RATING EVALUATION **12**

4) Joints condition

LENGTH	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m	length =	
index	6	4	2	1	0	INDEX =	**1**
OPENING	0	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm	opening =	
index	6	5	4	1	0	INDEX =	**5**
ROUGHNESS	very rough [VR]	rough [R]	slightly rough [SR]	smooth [S]	slickensided [SL]	roughness =	
index	6	5	3	1	0	INDEX =	**4**
FILLING	none [N]	hard & < 5 mm [HE]	hard & > 5 mm [H]	soft & < 5 mm [SE]	soft & > 5 mm [S]	filling =	
index	6	4	2	2	0	INDEX =	**4**
WEATHERING	unweathered [UW]	slightly weathered [SW]	moderately weath. [MW]	highly weathered [HW]	decomposed [D]	weathering =	
index	6	5	3	1	0	INDEX =	**5**

INDEX FOR ROCK MASS RATING EVALUATION **19**

5) Ground water

CONDITION	dry [D]	damp [DA]	wet [W]	dripping [DR]	flowing [F]	ground water =	
index	15	10	7	4	0	INDEX =	15

INDEX FOR ROCK MASS RATING EVALUATION **15**

ROCK MASS RATING	**73**

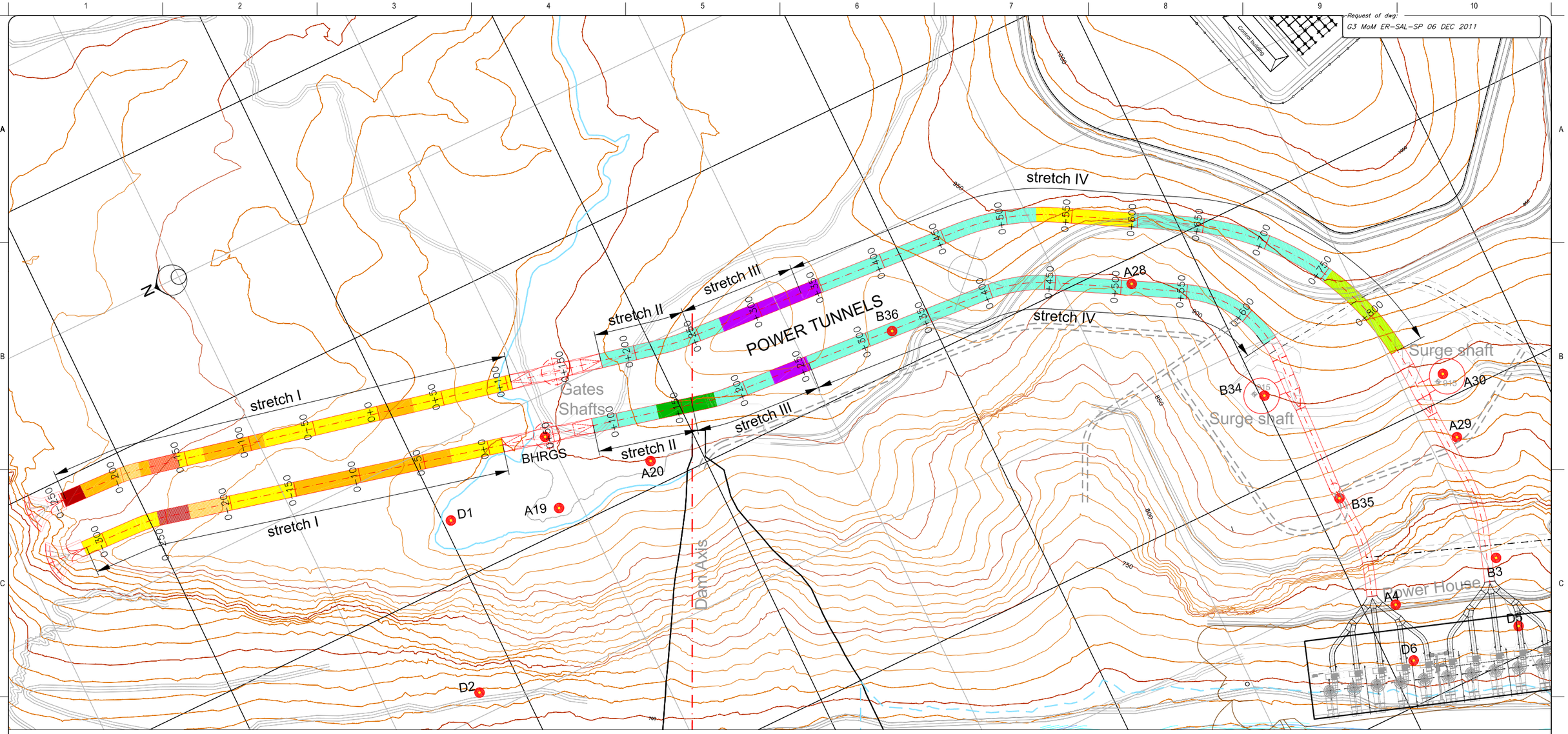
RMR	100 - 81	80 - 61	60 - 41	40 - 21	<20
ROCK CLASS	I - very good	II - good	III - fair	IV - poor	V - very poor

GSI CLASSIFICATION

GSI PARAMETERS	STRUCTURE	SURFACE CONDITION	GSI RANGE VALUE
	BLOCKY	GOOD	66-70

For the CONTRACTOR
Name : Nebil

Signature :



ROCK TYPE	GSI	Type A	Type B	Type C	Type D	Length of Tunnels [m] LEFT	RIGHT	TOT
Trachyte slightly weathered unweathered	70					80	30	110
	65					-	-	-
	60					426	470	896
	55					-	45	45
	50					78	-	78
	45					260	152	412
	40					112	122	234
	35					-	-	-
	30					17	-	17
Brecciated - Trachyte / Breccia (weathered)	40					16	30	46
	35					22	-	22
	30					-	24	24

NOTES

(1) The rock GSI is deducted from the geological mappingmade on site every 8 m of tunnels excavation. (ref. 003 & 004)

reference drawings

ref.	title code
4	550 GEO D SAS 004
3	550 GEO D SAS 003
2	550 GEN D SP 004
1	550 GEN D SP 003

modifications

issue	day	month	year	drawn	checked	approved
First issue						
A	08	08	2011	EDS	ROS	ACG
B	17	12	2011	ROS	ROS	ACG

GSI updated (PTL from ch. 0-232m to ch. 0-246m, PTR from ch. 0-256m to ch. 0-317m)

0 5 10 15 20 25m

scale 1:500 (A3 size)

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION

project: GIBE III Hydroelectric Project

contractor / designer: salini COSTRUTTORI S.p.A. / SP studio ing. g. pietrangeli s.r.l. consulting engineers

design: Level 2 DESIGN

title: 550 = POWER TUNNELS
RMK = ROCK MECHANICS

PT EXCAVATION GEOTECHICAL ASSESSMENT

section	kind works	type	ROMA office	serial n°	rev.	date
550	RMK	DSP	200	B	17	17 Dec 2011

Z:\GIBE III\500 - LEVEL 2 DESIGN\550 - POWER TUNNELS\03 RMK\DWGs\550 RMK D SP 200 B - PT excavation geotechnical assessment - DRAFT 17dec2011.dwg eds, sp - Roma 19/DEC/2011 08:58:47

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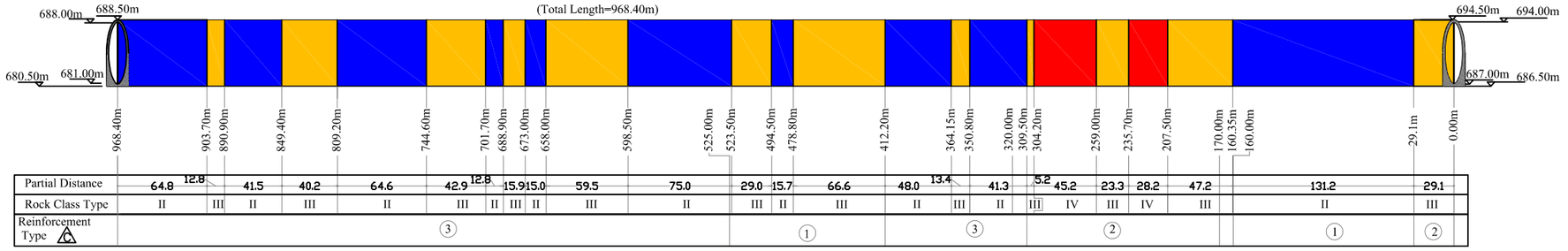
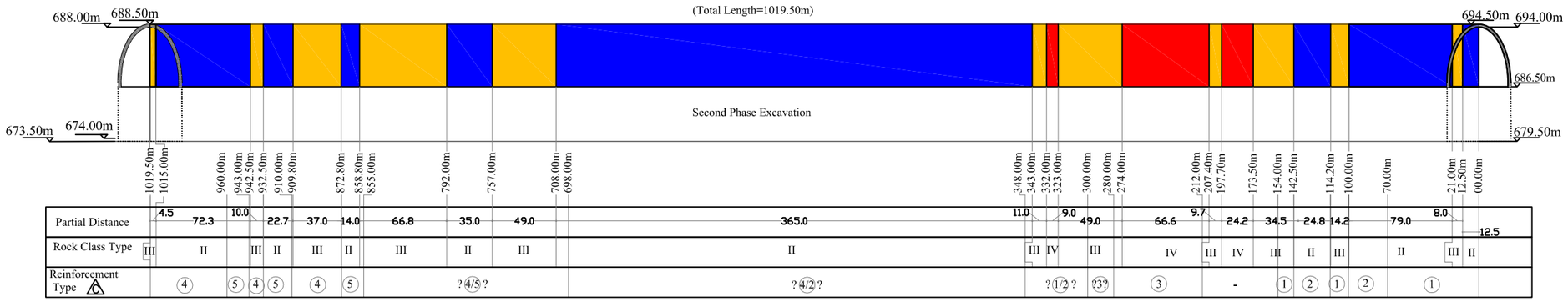
Left Tunnel

(Total Length=968.40m)



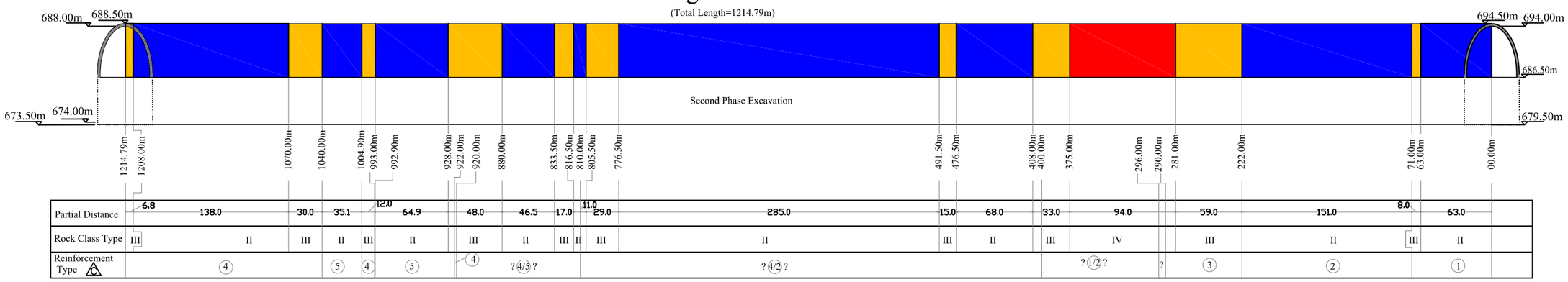
Central Tunnel

(Total Length=1019.50m)

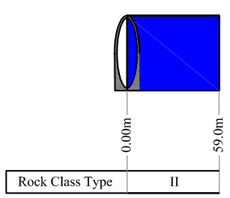


Right Tunnel

(Total Length=1214.79m)



Right Tunnel Adit



Scale : Vertical= 6 X Horizontal

Legend

- Rock Class-II
- Rock Class-III
- Rock Class-IV

Date	Rev	Descriptions
27/12/07	C	TC–TR & TL Reinforcement types
09/12/07	B	Tunnel Total Rock Class
22/05/07	A	Rock Class until 22May2007

Revisons

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION

project: GIBE III HYDROELECTRIC PROJECT

contractor: salini COSTRUTTORI S.p.A.

designer:

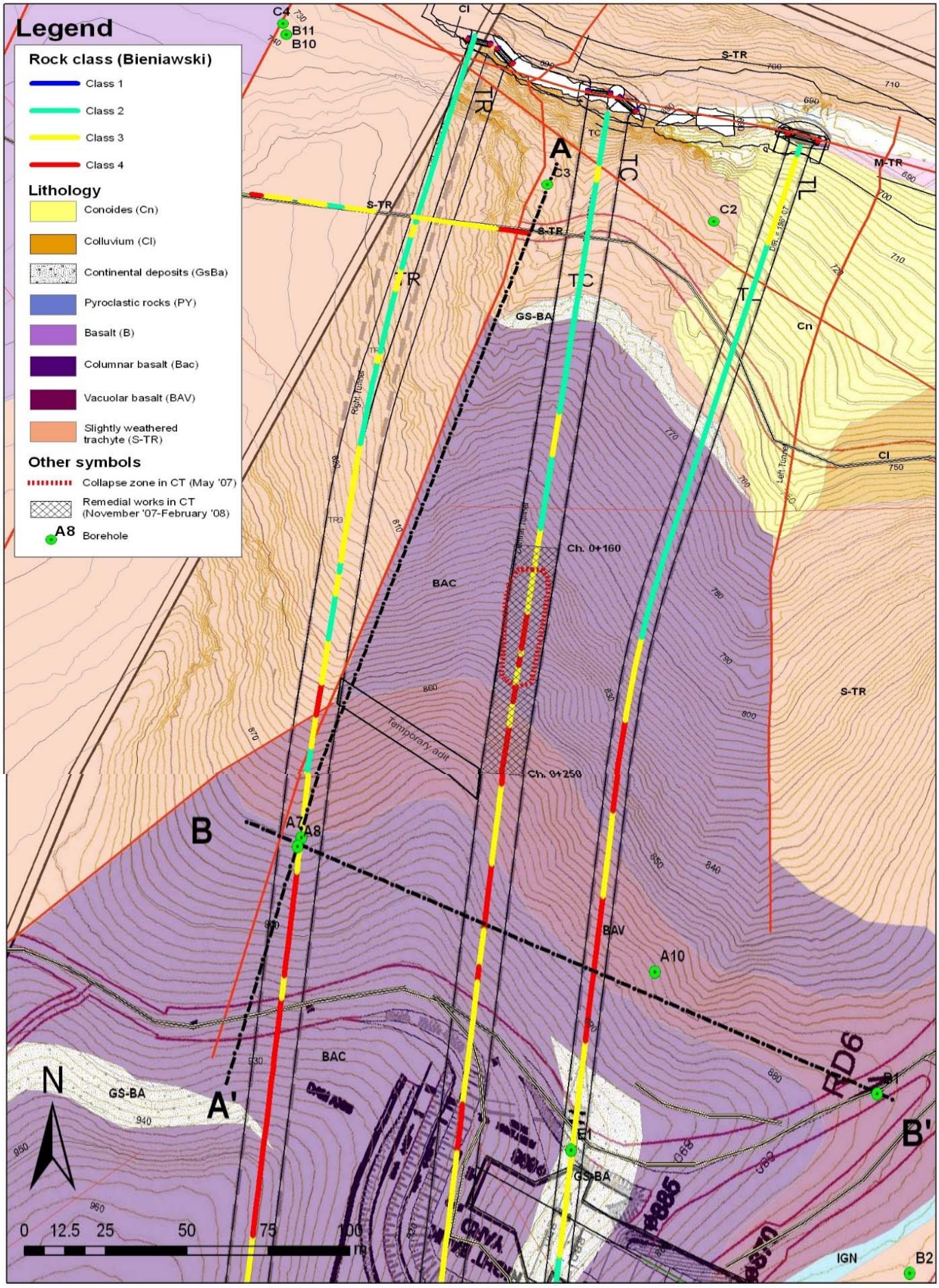
subject: DIVERSION TUNNELS

title: TUNNELS ROCK CLASSES SUMMARY

Size	Scale
A3	1:4000
A1	1:2000

section	s.sect.	work	office	drawing nr.	rev.	date
310	TUN	D	SAS	024	C	09 Dec. 2007

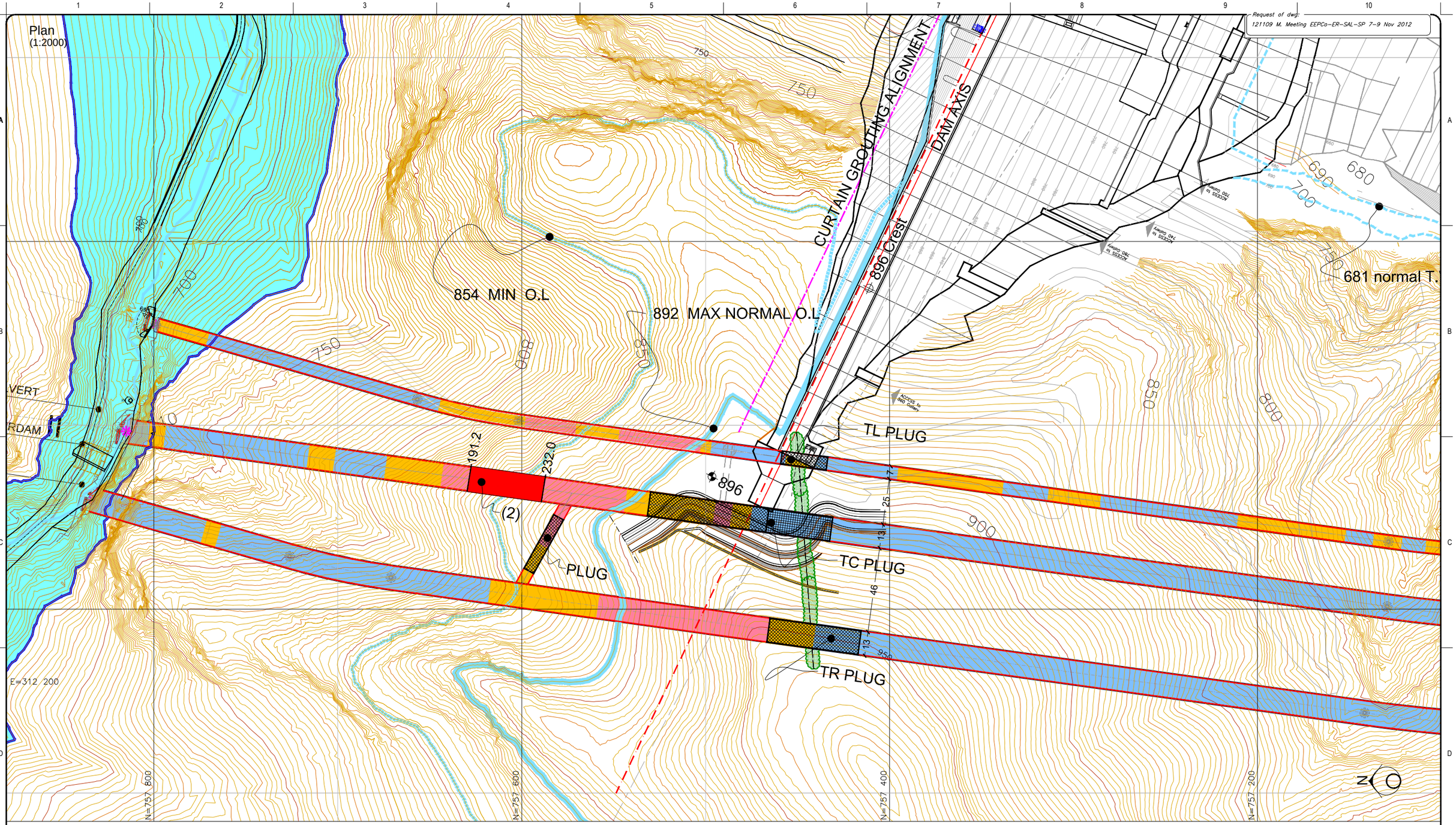
310–TUN–D–SAS–024–C–27Dec2007–Diversion Tunnel's Rock Rock CalssSummary.dwg\GG\


Legend
Rock class (Bieniawski)
Class 1
Class 2
Class 3
Class 4
Lithology
Conoides (Cn)
Colluvium (Cl)
Continental deposits (GsBa)
Pyroclastic rocks (PY)
Basalt (B)
Columnar basalt (Bac)
Vacuolar basalt (BAV)
Slightly weathered trachyte (S-TR)
Other symbols
Collapse zone in CT (May '07)
Remedial works in CT (November '07-February '08)
A8 Borehole
C4
B11
B10
Cl
S-TR
M-TR
TR
TC
TL
A
C3
C2
GS-BA
Cn
Right Tunnel
Left Tunnel
Ch. 0+160
Ch. 0+250
BAC
BAV
Temporary adit
A7
A8
A10
B1
E1
B
B'
A'
RD6
IGN
B2
N
0 12.5 25 50 75 100 m

Appendix D: Plan of Diversion, Power, Access, and Drain Tunnels.

Request of dwg:
121109 M. Meeting EEPCo-ER-SAL-SP 7-9 Nov 2012

Plan
(1:2000)



ROCK LEGEND

- Rock class-II
- Rock class-III
- Rock class-IV

NOTES:

1) GEOLOGICAL MAPPING according to dwg 310 TUN D SAS 050
2) COLLAPSED AREA

0 10 20 30 40 50 100 m
scale 1:2000 (A3 size)

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 515 = RIVER DIVERSION CLOUSURE
GEN = GENERAL

TC Plugs
Location

reference documents		
a	515 GEN R SP 001	
ref.	title code	

reference drawings	
5	310 TUN D SAS 001
4	310 TUN D SAS 050
3	510 TUN D SP 004
2	520 GRT D SP 002
1	515 GEN D SP 002x012
ref.	title code

modifications						
issue	day	month	year	drawn	checked	approved
A	10	10	2012	GIB	ACG	PAP

section	kind works	type	office	serial n°	rev.	date
515	GEN	D	SP	001	A	10 Oct 2012

Z:\GIBE III\500 - LEVEL 2 DESIGN\515 - RIVER CLOSURE\01 - GEN\D - DRAWINGS\515 GEN D SP 001-011 A - Dry & Wet Season - 07nov2012.dwg gib, sp - Roma 20/NOV/2012 10:12:25

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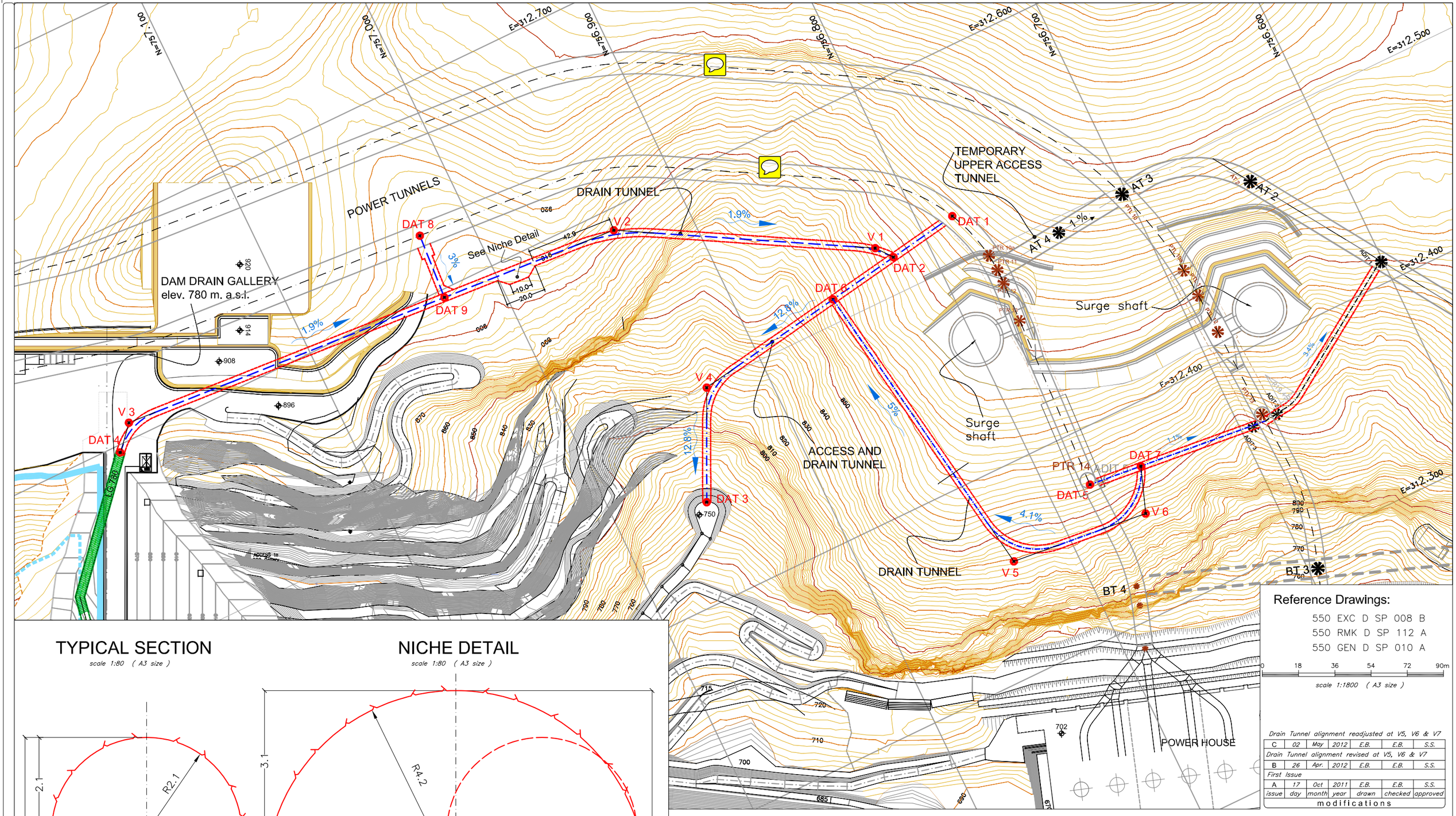
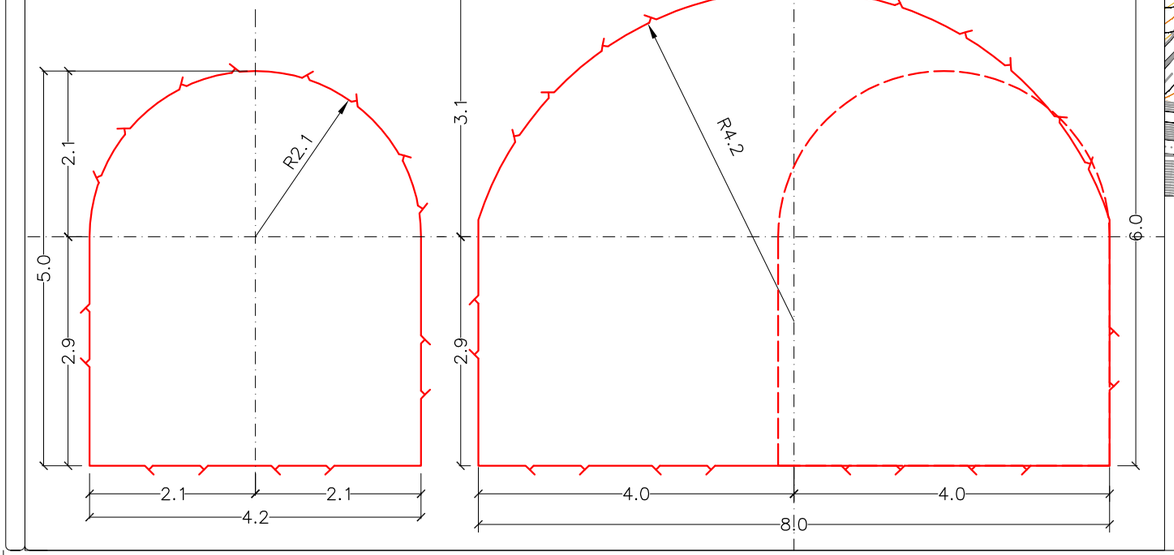


TYPICAL SECTION

scale 1:80 (A3 size)



NICHE DETAIL

scale 1:80 (A3 size)

SETTING OUT POINTS

Point	Easting	Northing	Elevation
DAT1	312523.1358	756783.1681	776.00
DAT2	312517.4093	756818.3297	772.05
DAT3	312447.9261	756954.6729	750.00
DAT4	312596.4540	757206.4694	780.00
DAT5	312373.3146	756779.3374	778.18
DAT6	312511.5191	756854.4962	767.22
DAT7	312370.6399	756752.6145	777.96
DAT8	312628.9500	757025.6900	777.65
DAT9	312596.0297	757028.0061	776.54

Point	Easting	Northing	Radius	Curve Length
V1	312525.3673	756824.5466	51.00	19.94
V2	312589.6794	756937.7455	87.00	38.84
V3	312607.8622	757196.1897	28.10	25.82
V4	312499.2170	756930.0328	30.00	28.84
V5	312355.3360	756829.9032	30.00	39.88
V6	312348.4045	756760.6488	30.00	39.72

Reference Drawings:

550 EXC D SP 008 B
550 RMK D SP 112 A
550 GEN D SP 010 A

0 18 36 54 72 90m

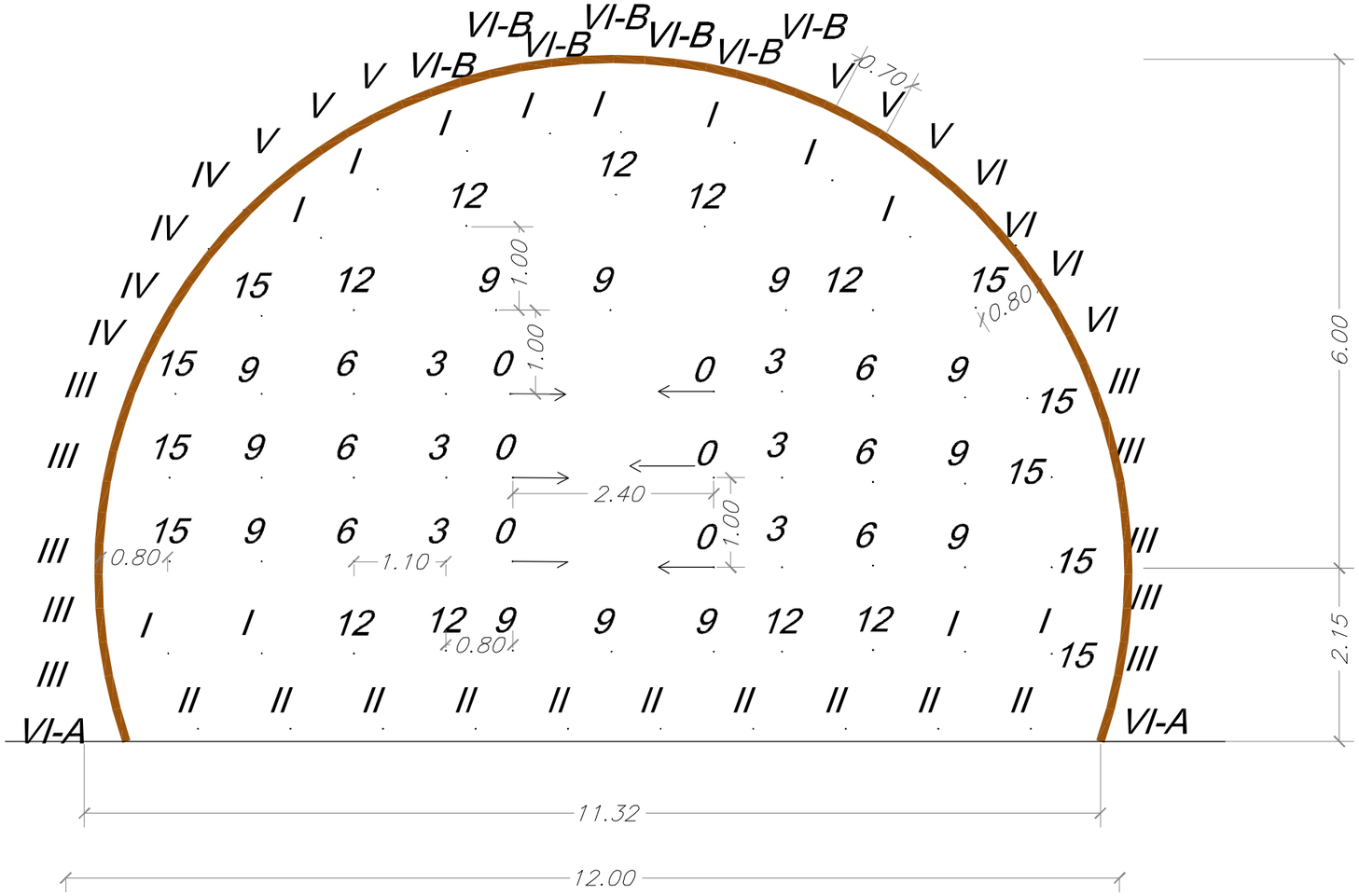
scale 1:1800 (A3 size)

Drain Tunnel alignment readjusted at V5, V6 & V7						
C	02	May	2012	E.B.	E.B.	S.S.
Drain Tunnel alignment revised at V5, V6 & V7						
B	26	Apr.	2012	E.B.	E.B.	S.S.
First Issue						
A	17	Oct	2011	E.B.	E.B.	S.S.
issue	*day*	*month*	*year*	*drawn*	*checked*	*approved*
modifications						

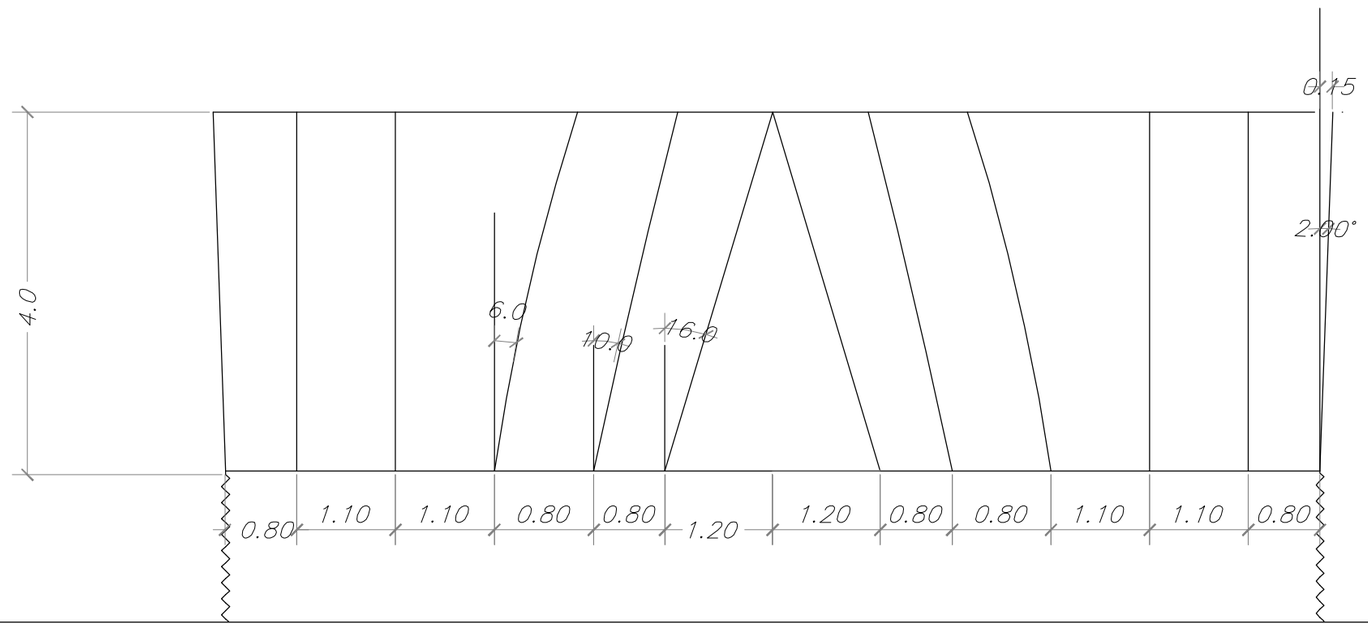
beneficiary EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project GIBE III Hydroelectric Project
contractor salini COSTRUTTORI S.p.A.
designer
design
Level 2 DESIGN
title
POWER TUNNELS
EXCAVATION
Drain and Access Tunnels
Plan and Setting Out Points

section	s.sect.	work	office	drawing nr.	rev	date
550	EXC	D	SAS	0 4 8	C	02 May 2012

Appendix E: Phase Excavations- Drilling/ Charging/ Blasting Patterns.

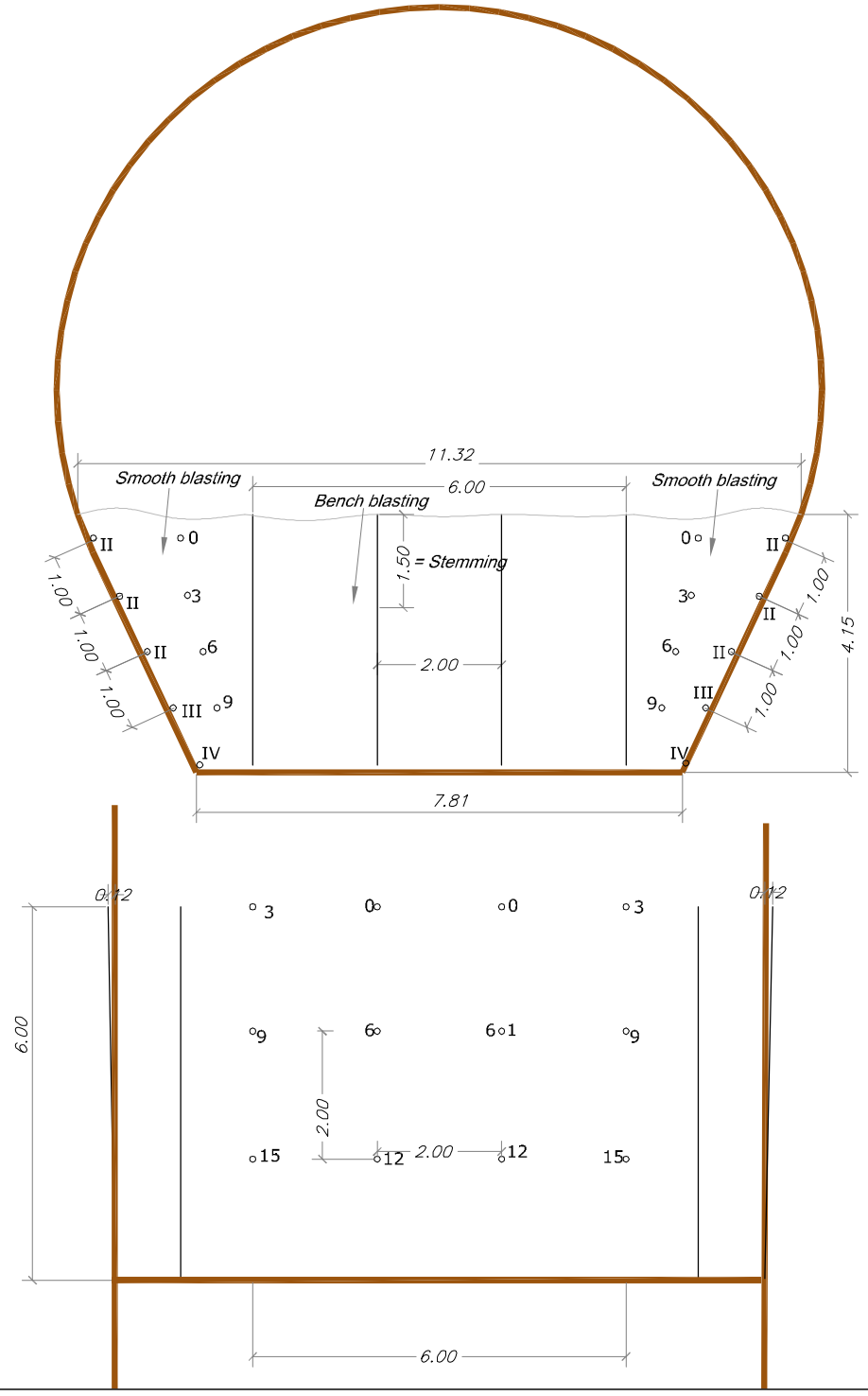


Holes Ø 51 Nr.59



scale 1:100 (A4 size)

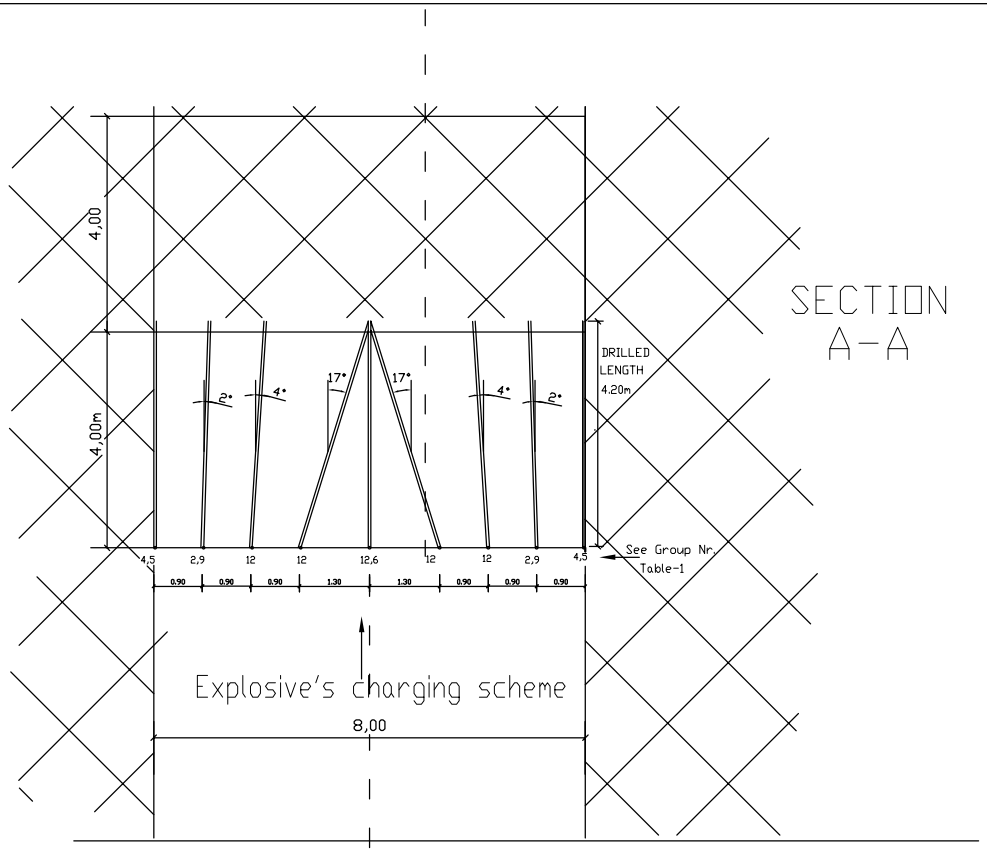
1st Power tunnels Phase Excavation V-Cut Blast Drilling Pattern



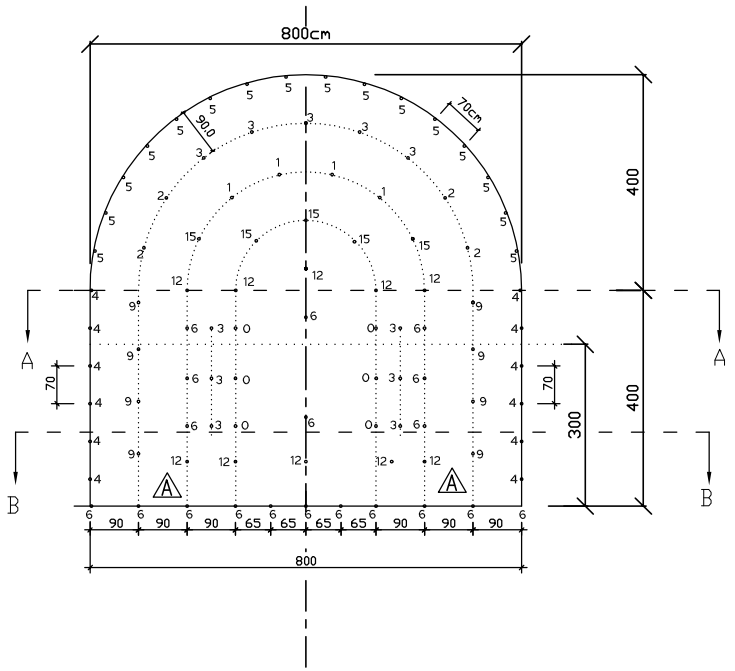
scale 1:100 (A4 size)

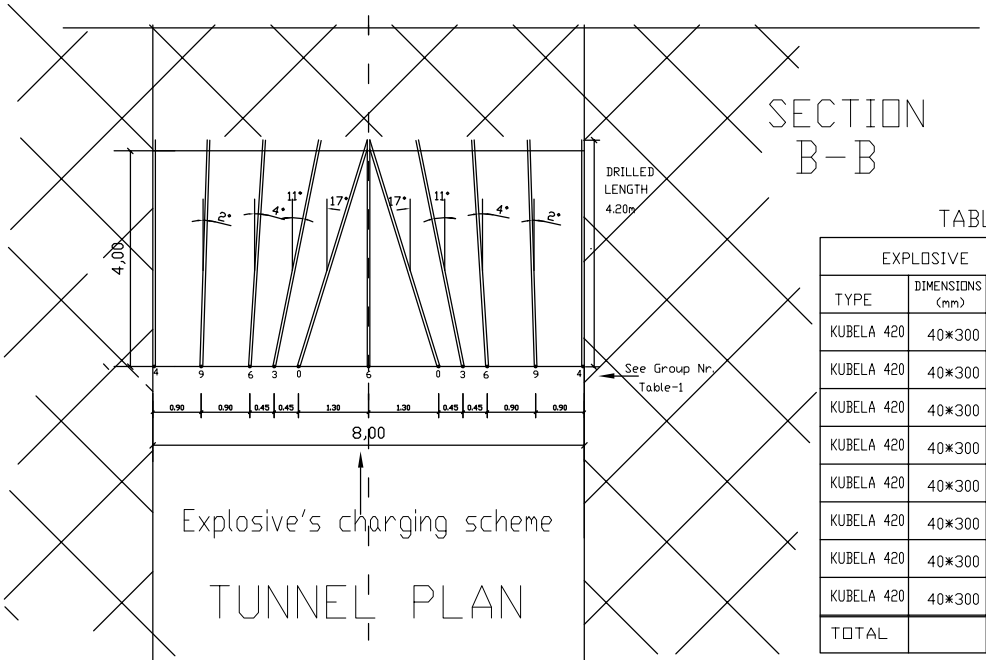
Power tunnels
2nd Phase Excavation
Drilling Pattern

Blasting scheme with electric retarders



SECTION A–A





SECTION B–B

TABLE-1

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	6	6*12 =	72	28.08	0	25 Ms	0.025	6
KUBELA 420	40*300	0.39	6	6*10 =	60	23.4	3	25 Ms	0.050	6
KUBELA 420	40*300	0.39	8	8*9 =	72	280.8	6	25 Ms	0.075	8
KUBELA 420	40*300	0.39	8	8*8 =	64	24.96	9	25 Ms	0.100	8
KUBELA 420	40*300	0.39	15	15*8 =	120	46.8	12, 15	25 Ms	0.125	15
KUBELA 420	40*300	0.39	13	13*7 =	91	35.49	1, 2, 3	500 Ms	0.625	13
KUBELA 420	40*300	0.39	28	28*8 =	224	87.36	4/5	500 Ms	1.125	28
KUBELA 420	40*300	0.39	11	11*11 =	121	47.19	6	500 Ms	1.625	11
TOTAL			95		824	321.36				95

Drilling holes diameter = 51mm
Length of drillied holes = 4.20m
Length of blasted rock portion = 4.00m
Blasted face area = 57.13sq.m.
Volume of blasted rock = 228.50cub.m.
Average volume of loose rock = 319.94 cub.m.
Explosive per cubic meter, ratio = 1.40 Kg/cub.m.

Gilgel Gibe III Hydroelectric Power Project

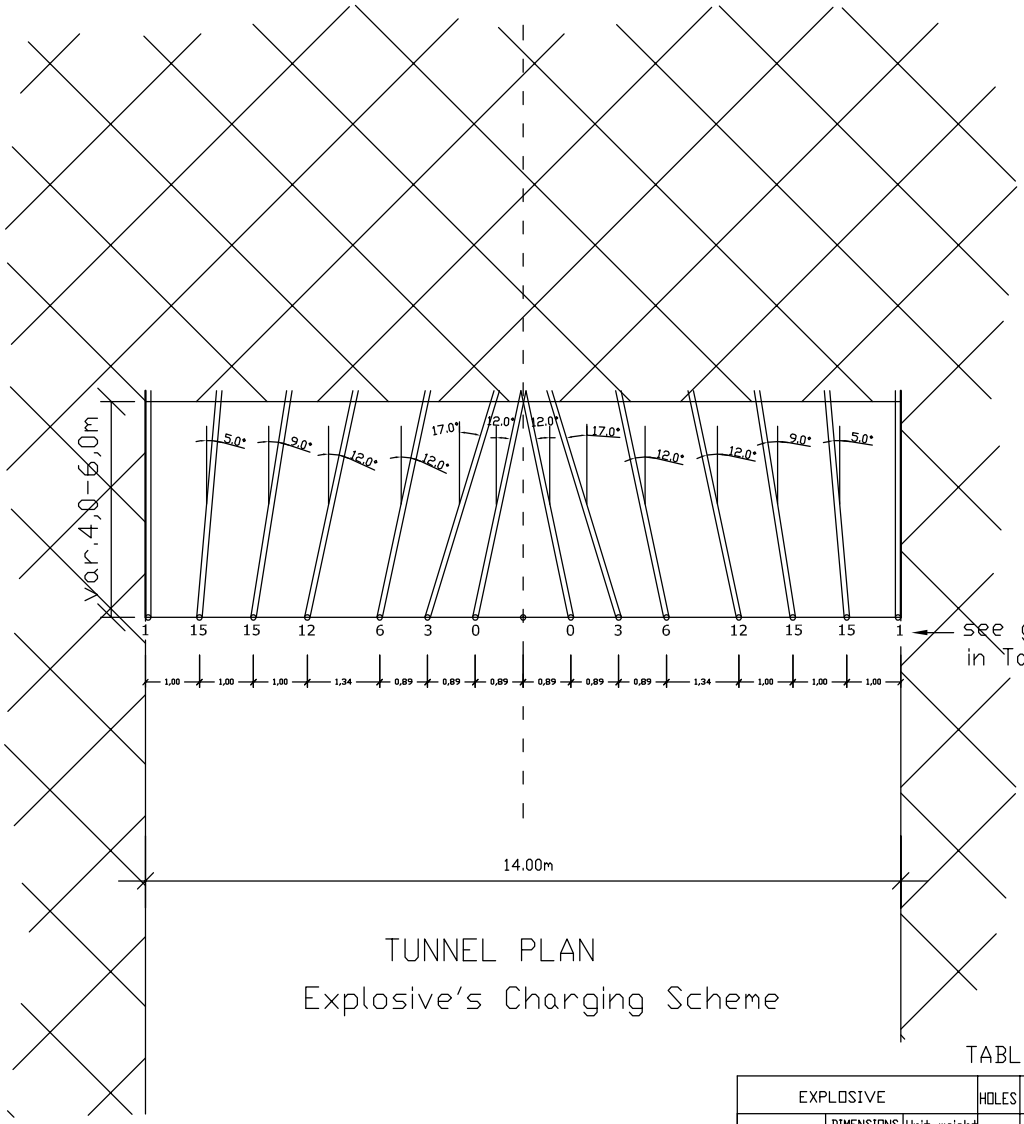
DIVERSION TUNNEL'S DRILLING & BLASTING

BLASTING SCHEME | Size A3 | Scale

LEFT TUNNEL | Rev.

METHOD STATEMENT n.01 | Attachment 05a | 0

310	TUN	M	SAS	105	Rev. 0	20–Feb–2007



TUNNEL PLAN

Explosive's Charging Scheme

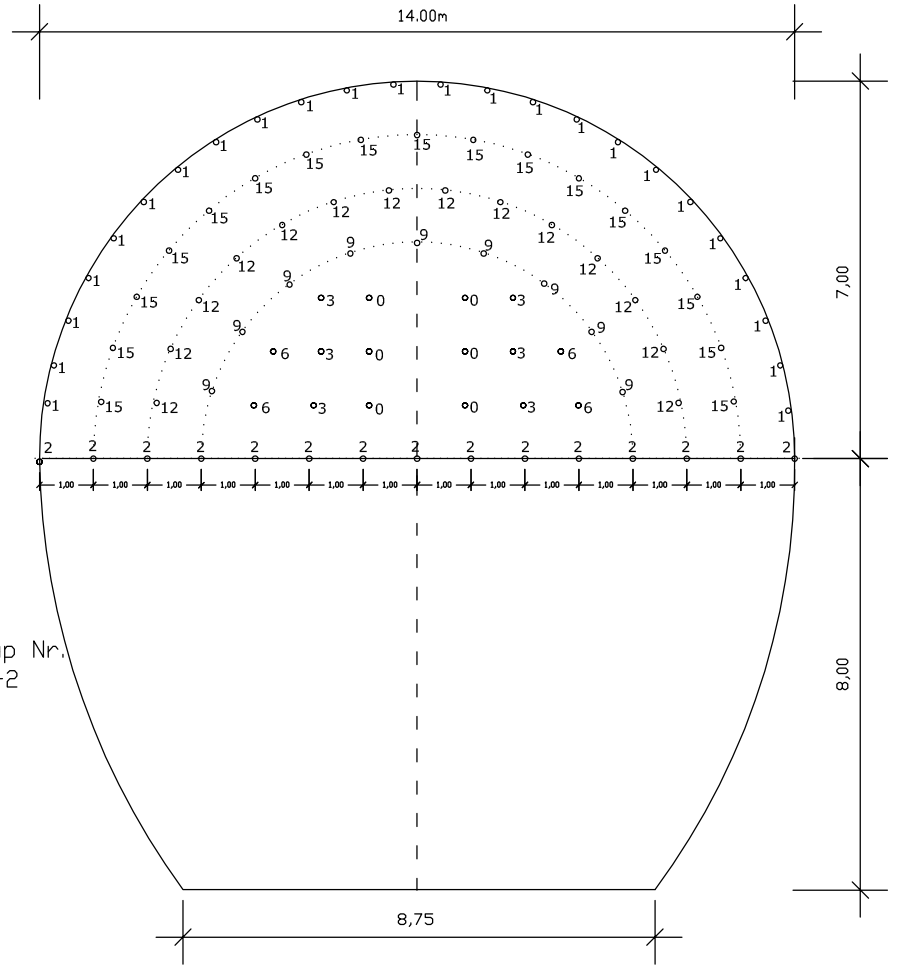


TABLE-2

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	6	6*12 =	72	28.08	0	25 Ms	0.025	6
KUBELA 420	40*300	0.39	6	6*12 =	72	28.08	3	25 Ms	0.050	6
KUBELA 420	40*300	0.39	4	4*11 =	44	17.16	6	25 Ms	0.075	4
KUBELA 420	40*300	0.39	14	14*11 =	154	60.06	9	25 Ms	0.100	14
KUBELA 420	40*300	0.39	16	16*11 =	176	68.64	12	25 Ms	0.125	16
KUBELA 420	40*300	0.39	17	17*10 =	170	66.30	15	25 Ms	0.150	17
KUBELA 420	40*300	0.39	24	24*9 =	216	84.24	1	500 Ms	0.650	24
KUBELA 420	40*300	0.39	15	15*12 =	180	70.20	2	500 Ms	1.150	15
TOTAL			102		1084	422.76				102

TYPE ROCK CLASS 1,2,3,4,5

Drilling holes diameter = 51mm
Length of drillied holes = 4.20m / 7.00m
Length of blasted rock portion = 4.00m
Blasted face area = 76.98sq.m.
Volume of blasted rock = 323.30cub.m.
Average volume of loose rock = 484.50 cub.m.
Explosive per cubic meter, ratio = 1.310Kg/cub.m.

Gilgel Gibe III Hydroelectric Power Project

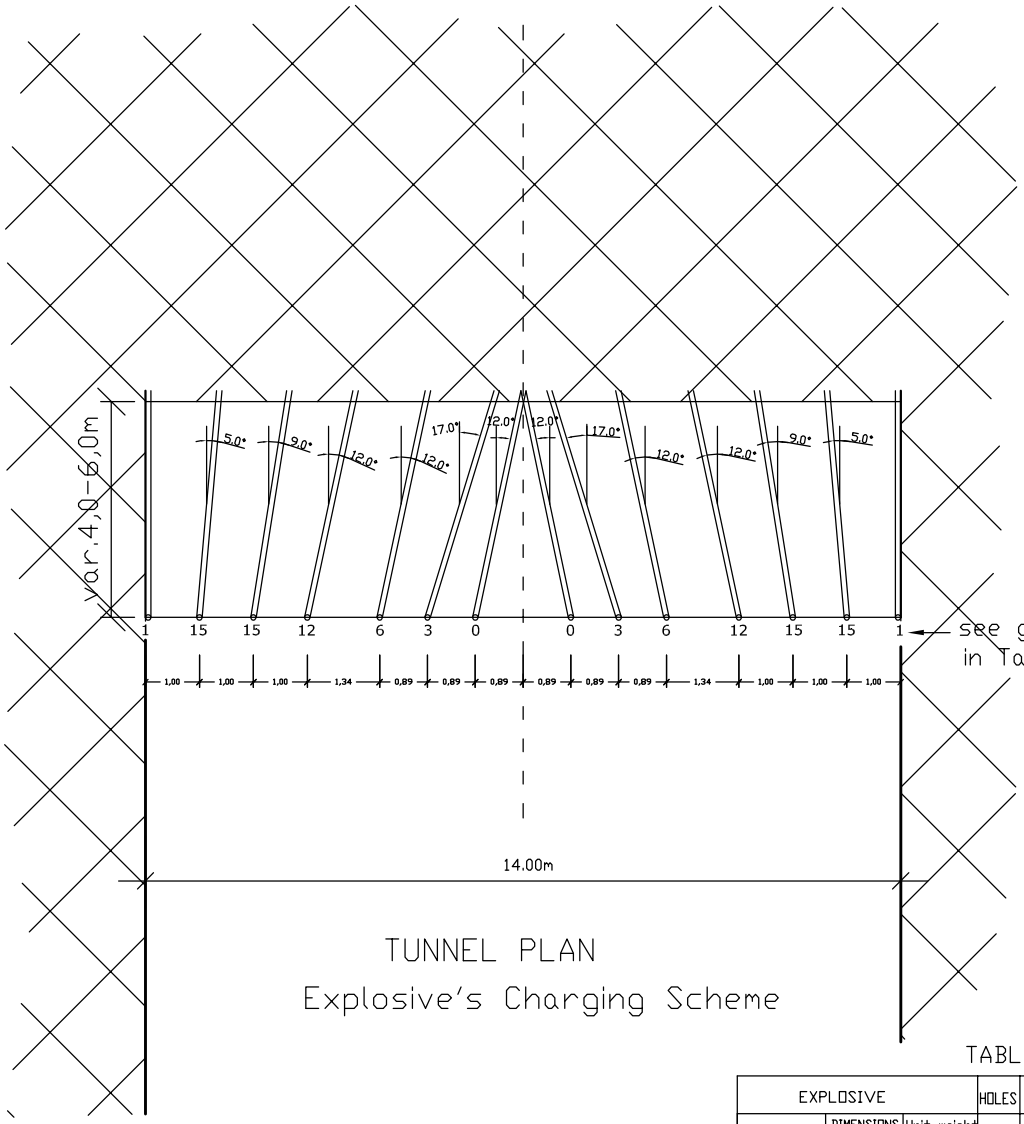
DIVERSION TUNNEL'S DRILLING & BLASTING

BLASTING SCHEME | Size A4 | Scale

RIGHT & CENTRAL TUNNEL 1st Phase | Rev.

METHOD STATEMENT n.01 | Attachment 05b | 0

310 | TUN | M | SAS | 105 | Rev. 2 | 01-Dec-2007



TUNNEL PLAN

Explosive's Charging Scheme

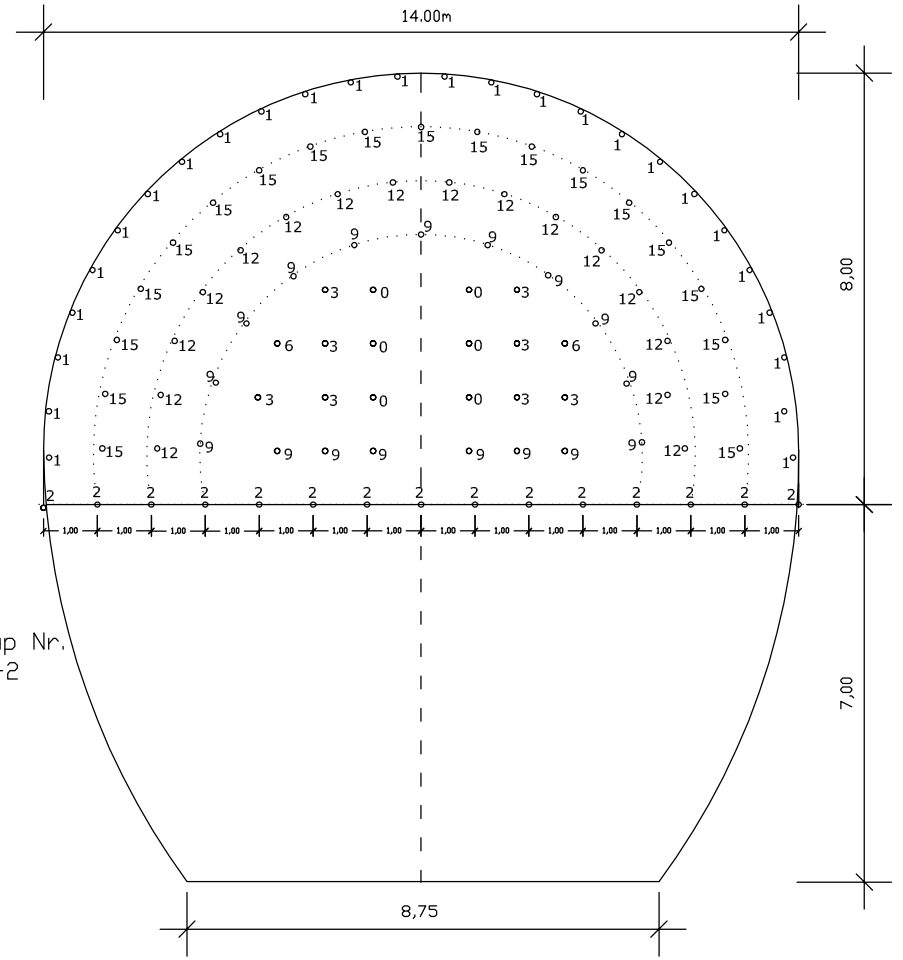


TABLE-2

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	6	6*12 =	72	28.08	0	25 Ms	0.025	6
KUBELA 420	40*300	0.39	6	6*12 =	72	28.08	3	25 Ms	0.050	6
KUBELA 420	40*300	0.39	6	6*11 =	66	25.74	6	25 Ms	0.075	6
KUBELA 420	40*300	0.39	17	17*11 =	187	72.93	9	25 Ms	0.100	17
KUBELA 420	40*300	0.39	16	16*11 =	176	68.64	12	25 Ms	0.125	16
KUBELA 420	40*300	0.39	19	19*10 =	190	74.10	15	25 Ms	0.150	19
KUBELA 420	40*300	0.39	26	26*9 =	234	91.26	1	500 Ms	0.650	26
KUBELA 420	40*300	0.39	15	15*12 =	180	70.20	2	500 Ms	1.150	15
TOTAL			111		1177	459.03				111

TYPE ROCK CLASS 1,2,3,4,5

Drilling holes diameter = 51mm
Length of drillied holes = 4.20m / 7.00m
Length of blasted rock portion = 4.00m
Blasted face area = 90.96sq.m.
Volume of blasted rock = 378.00cub.m.
Average volume of loose rock = 567.00 cub.m.
Explosive per cubic meter, ratio = 1.214Kg/cub.m.

Gilgel Gibe III Hydroelectric Power Project

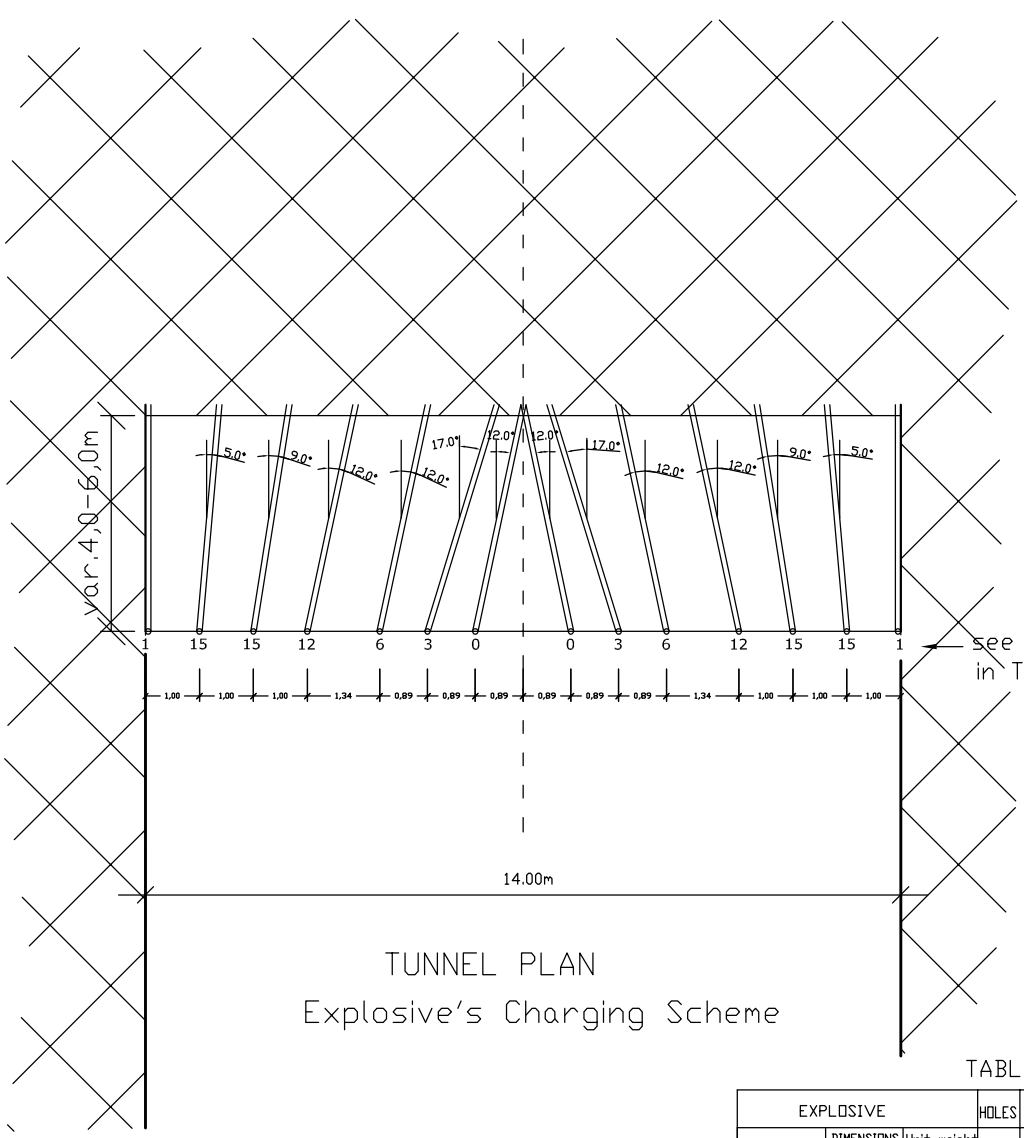
DIVERSION TUNNEL'S DRILLING & BLASTING

BLASTING SCHEME | Size A4 | Scale

RIGHT & CENTRAL TUNNEL 1st Phase | Rev.

METHOD STATEMENT n.01 | Attachment 05c | 0

310 | TUN | M | SAS | 105 | Rev. 2 | 01-Dec-2007



TUNNEL PLAN

Explosive's Charging Scheme

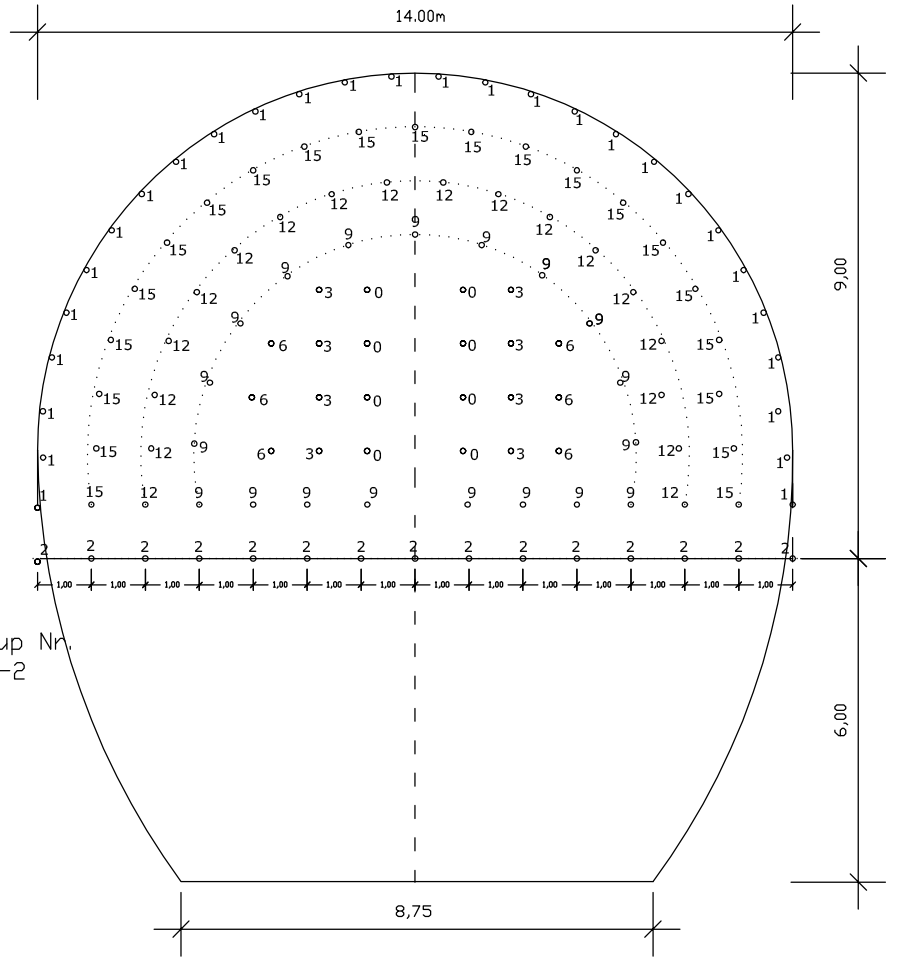


TABLE-2

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	8	8*12 =	96	37.44	0	25 Ms	0.025	8
KUBELA 420	40*300	0.39	8	8*12 =	96	37.44	3	25 Ms	0.050	8
KUBELA 420	40*300	0.39	6	6*11 =	66	25.74	6	25 Ms	0.075	6
KUBELA 420	40*300	0.39	13	13*11 =	143	55.77	9	25 Ms	0.100	13
KUBELA 420	40*300	0.39	17	17*11 =	187	72.93	12	25 Ms	0.125	17
KUBELA 420	40*300	0.39	21	21*10 =	210	81.90	15	25 Ms	0.150	21
KUBELA 420	40*300	0.39	28	28*9 =	252	98.28	1	500 Ms	0.650	28
KUBELA 420	40*300	0.39	15	15*12 =	180	70.20	2	500 Ms	1.150	15
TOTAL			116		1230	479.7				116

TYPE ROCK CLASS 1,2,3,4,5

Drilling holes diameter = 51mm
Length of drillied holes = 4.20m / 7.00m
Length of blasted rock portion = 4.00m
Blasted face area = 104.81sq.m.
Volume of blasted rock = 441.00cub.m.
Average volume of loose rock = 661.5.00 cub.m.
Explosive per cubic meter, ratio = 1.087g/cub.m.

Gilgel Gibe III Hydroelectric Power Project

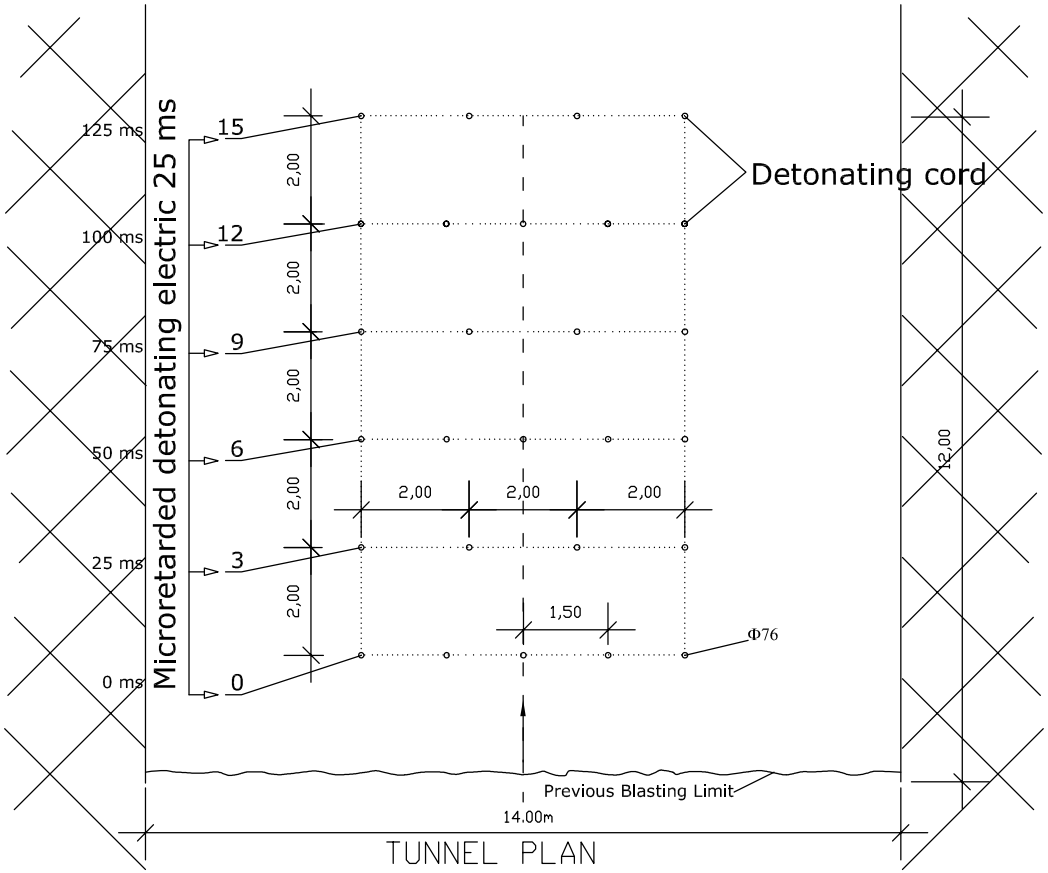
DIVERSION TUNNEL'S DRILLING & BLASTING

BLASTING SCHEME | Size A4 | Scale

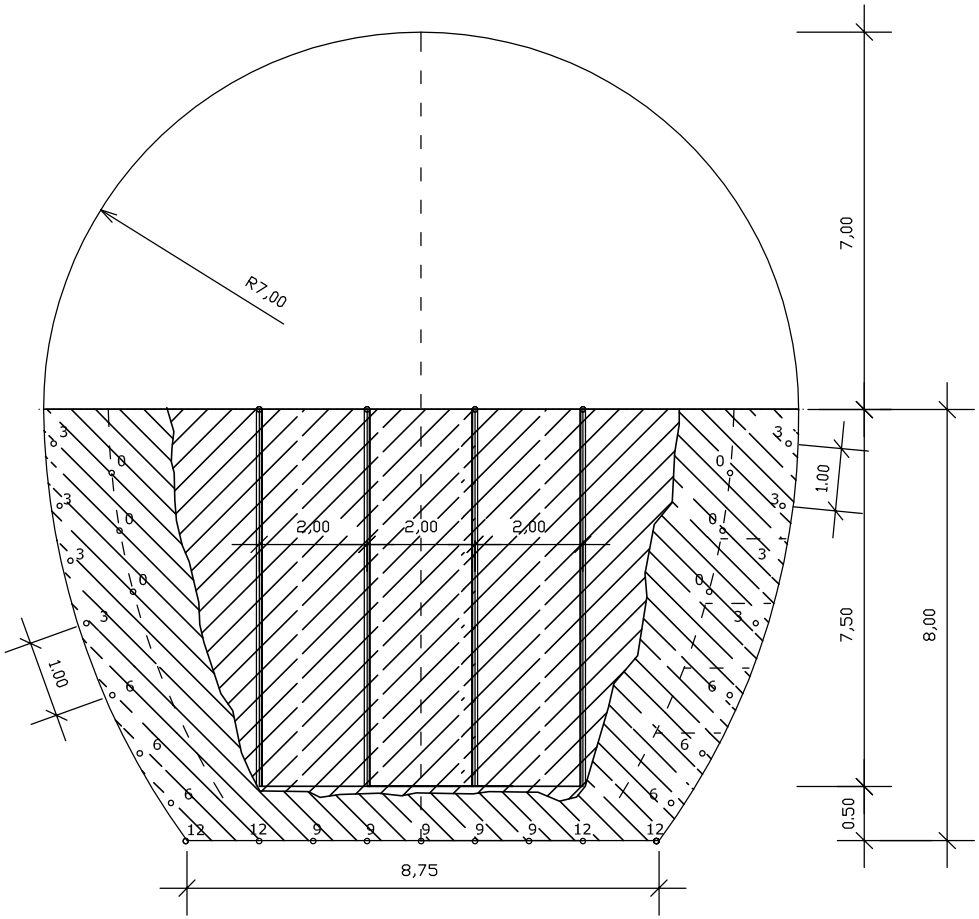
RIGHT & CENTRAL TUNNEL 1st Phase | Rev.

METHOD STATEMENT n.01 | Attachment 05d | 0

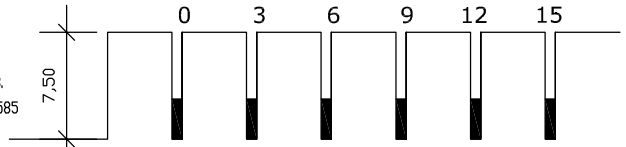
310 | TUN | M | SAS | 105 | Rev. 2 | 01-Dec-2007



TUNNEL PLAN



Explosive's Charging Scheme



Vertical Drilling holes diameter = 76mm
Length of drilled holes = 7.50m
Length of blasted rock portion = 12.00m
Volume of blasted rock = 540 cub.m.
Explosive type Kubela 50 x 585 Unit Weight Kg 2.08.
Total Explosive charge = 224,64 Kg KUBELA 50 x 585
Explosive per cubic meter, ratio = 0.416 Kg/cub.m.

1° Blasting

TABLE VERTICALS HOLES

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 520	50*585	2.08	5	5*4 =	20	41.60	0	25 Ms	0.000	5
KUBELA 520	50*585	2.08	4	4*4 =	16	33.28	3	25 Ms	0.025	4
KUBELA 520	50*585	2.08	5	5*4 =	20	41.60	6	25 Ms	0.050	5
KUBELA 520	50*585	2.08	4	4*4 =	16	33.28	9	25 Ms	0.075	4
KUBELA 520	50*585	2.08	5	5*4 =	20	41.60	12	25 Ms	0.100	5
KUBELA 520	50*585	2.08	4	4*4 =	16	33.28	15	25 Ms	0.125	4
TOTAL			27		108	224.64				

2° Blasting

TABLE HORIZONTALS HOLES

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	6	6*1 =	6	2.34	0	25 Ms	0.025	6
KUBELA 420	40*300	0.39	8	8*1 =	8	3.12	3	25 Ms	0.050	8
KUBELA 420	40*300	0.39	6	6*1 =	6	2.34	6	25 Ms	0.075	6
KUBELA 420	40*300	0.39	5	5*3 =	15	5.85	9	25 Ms	0.100	5
KUBELA 420	40*300	0.39	4	4*3 =	12	4.68	12	25 Ms	0.125	4
TOTAL			29		47	18.33				46

Horizontal Drilling holes diameter = 51mm
Length of drilled holes = 6.00m
Length of blasted rock portion = 12.00m
Volume of blasted rock = 115.20cub.m.
Explosive type Kubela 40 x 300 Unit Weight Kg 0.39.
Total Explosive charge = 18,33 Kg KUBELA 40 x 300
Explosive per cubic meter, ratio = 0.159 Kg/cub.m.

Gilgel Gibe III Hydroelectric Power Project

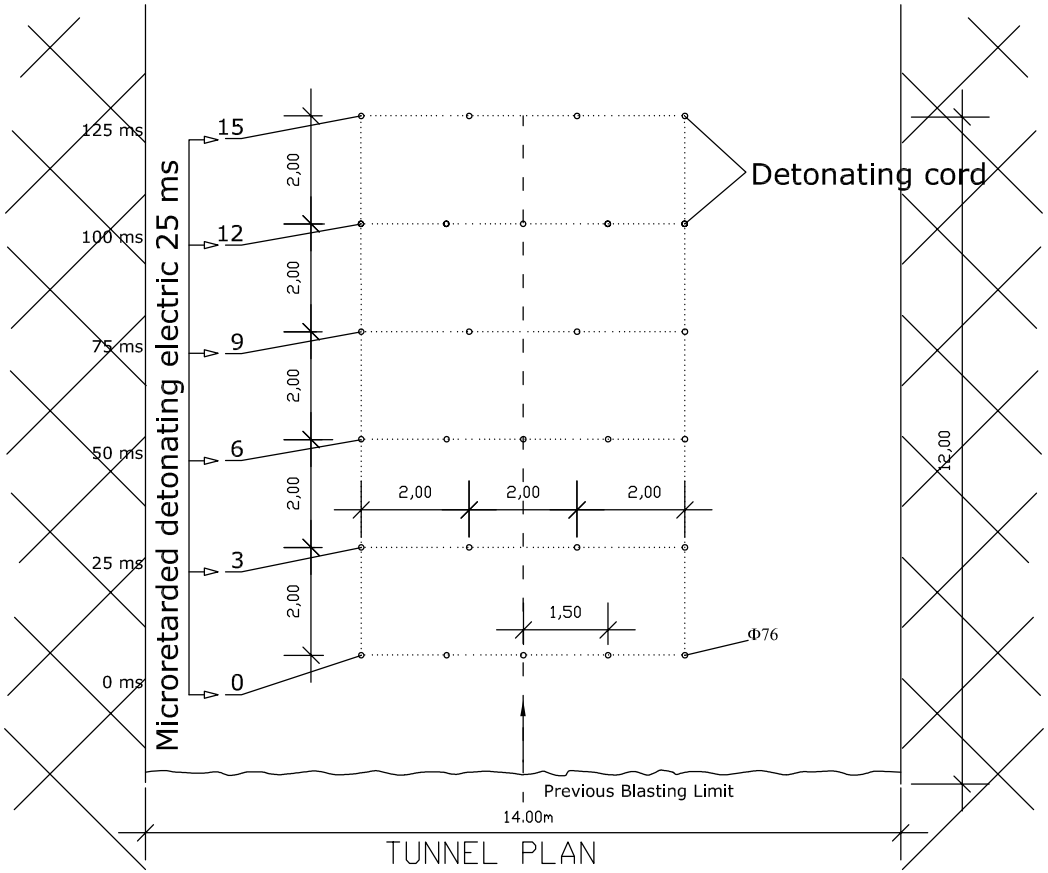
DIVERSION TUNNEL'S DRILLING & BLASTING

BLASTING SCHEME R.class II&III | Size A4 | Scale

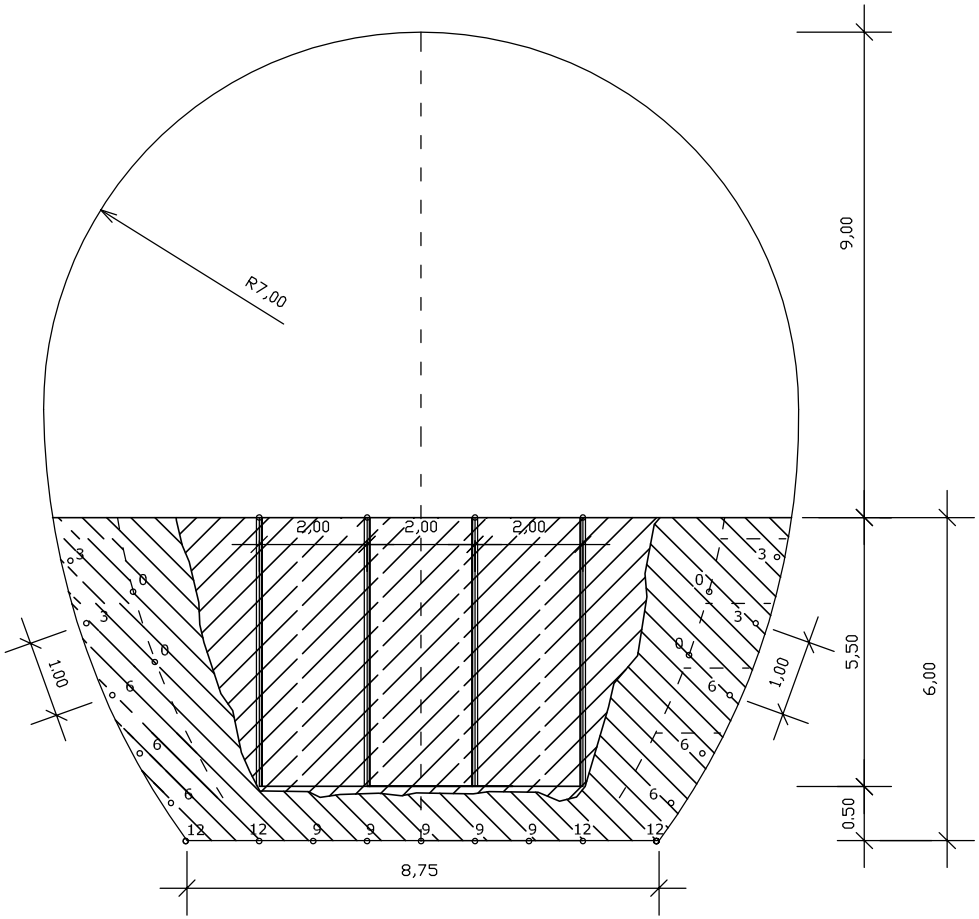
RIGHT & CENTRAL TUNNEL 2nd Phase | Rev.

METHOD STATEMENT n.01 | Attachment 05e | 3

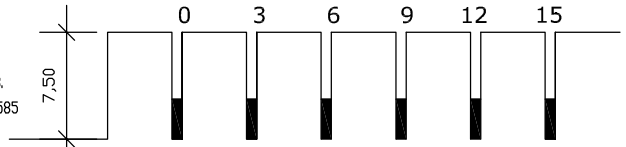
310	TUN	M	SAS	105	Rev. 3	16-Feb-2008



TUNNEL PLAN



Explosive's Charging Scheme



Vertical Drilling holes diameter = 76mm
Length of drilled holes = 5.50m
Length of blasted rock portion = 12.00m
Volume of blasted rock = 396 cub.m.
Explosive type Kubela 50 x 585 Unit Weight Kg 2.08.
Total Explosive charge = 224,64 Kg KUBELA 50 x 585
Explosive per cubic meter, ratio = 0.567 Kg/cub.m.

1° Blasting

TABLE VERTICALS HOLES

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 520	50*585	2.08	5	5*4 =	20	41.60	0	25 Ms	0.000	5
KUBELA 520	50*585	2.08	4	4*4 =	16	33.28	3	25 Ms	0.025	4
KUBELA 520	50*585	2.08	5	5*4 =	20	41.60	6	25 Ms	0.050	5
KUBELA 520	50*585	2.08	4	4*4 =	16	33.28	9	25 Ms	0.075	4
KUBELA 520	50*585	2.08	5	5*4 =	20	41.60	12	25 Ms	0.100	5
KUBELA 520	50*585	2.08	4	4*4 =	16	33.28	15	25 Ms	0.125	4
TOTAL			27		108	224.64				

2° Blasting

TABLE HORIZONTALS HOLES

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	4	4*1 =	4	1.56	0	25 Ms	0.025	4
KUBELA 420	40*300	0.39	4	4*1 =	4	1.56	3	25 Ms	0.050	4
KUBELA 420	40*300	0.39	6	6*1 =	6	2.34	6	25 Ms	0.075	6
KUBELA 420	40*300	0.39	5	5*3 =	15	5.85	9	25 Ms	0.100	5
KUBELA 420	40*300	0.39	4	4*3 =	12	4.68	12	25 Ms	0.125	4
TOTAL			23		39	15.99				21

Horizontal Drilling holes diameter = 51mm
Length of drillied holes = 6.00m
Length of blasted rock portion = 12.00m
Volume of blasted rock = 86.40cub.m.
Explosive type Kubela 40 x 300 Unit Weight Kg 0.39.
Total Explosive charge = 15,99 Kg KUBELA 40 x 300
Explosive per cubic meter, ratio = 0.185 Kg/cub.m.

Gilgel Gibe III Hydroelectric Power Project

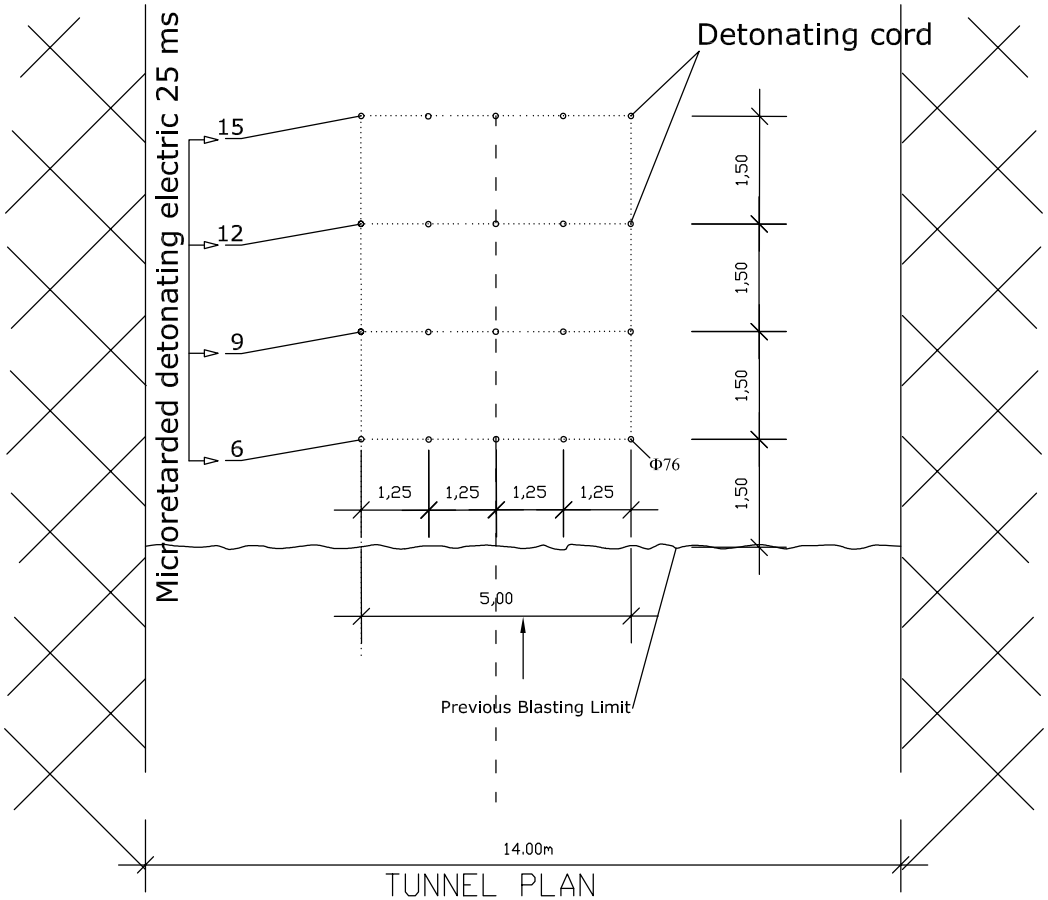
DIVERSION TUNNEL'S DRILLING & BLASTING

BLASTING SCHEME R.class II&III | Size A4 | Scale

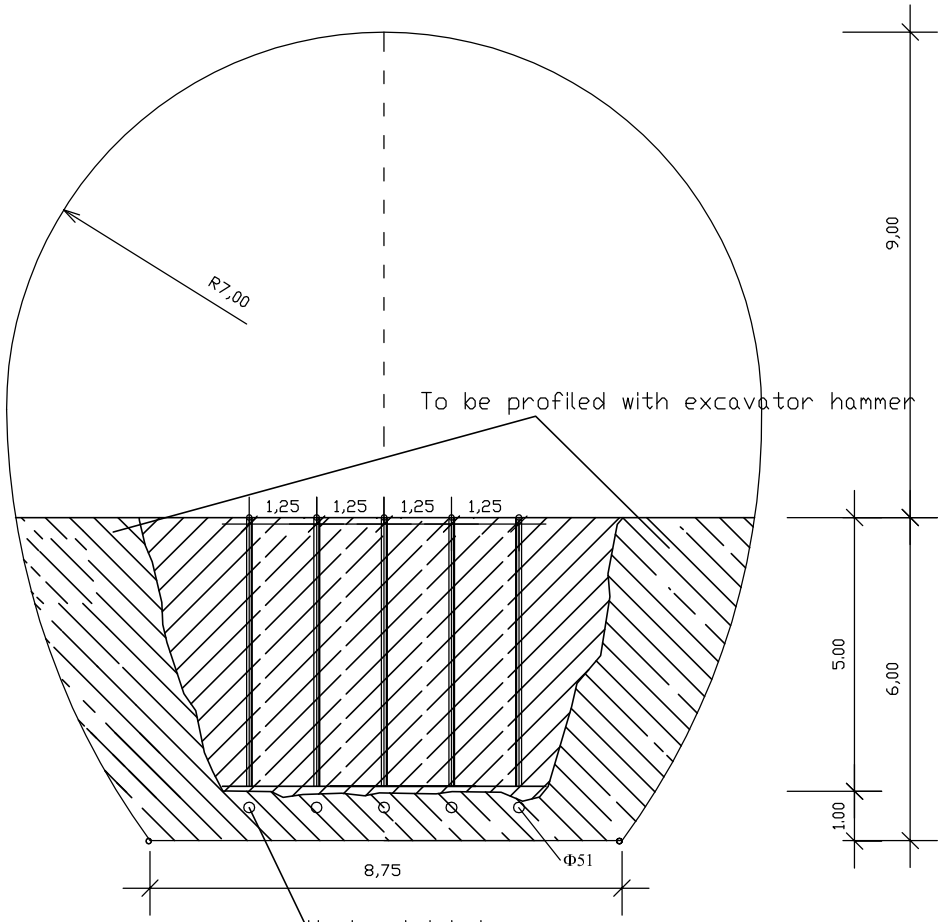
RIGHT & CENTRAL TUNNEL 2nd Phase | Rev.

METHOD STATEMENT n.01 | Attachment 05f | 3

310 | TUN | M | SAS | 105 | Rev. 3 | 16-Feb-2008



TUNNEL PLAN



Explosive's Charging Scheme

Vertical Drilling holes diameter = 76mm
Length of drillied holes = 5.00m
Volume of blasted rock = 150 cub.m.
Length of blasted rock portion = 6.00m
Explosive type Kubela 50 x 585 Unit Weight Kg 2.08.
Total Explosive charge = 124,80 Kg KUBELA 50 x 585
Explosive per cubic meter, ratio = 0832 Kg/cub.m.

1° Blasting

TABLE VERTICALS HOLES

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 520	50*585	2.08	5	5*3 =	15	31.20	6	25 Ms	0.050	5
KUBELA 520	50*585	2.08	5	5*3 =	15	31.20	9	25 Ms	0.075	5
KUBELA 520	50*585	2.08	5	5*3 =	15	31.20	12	25 Ms	0.100	5
KUBELA 520	50*585	2.08	5	5*3 =	15	31.20	15	25 Ms	0.125	5
TOTAL			20		60	124.80				

2° Blasting

TABLE HORIZONTALS HOLES

EXPLOSIVE			HOLES	No OF CHARGE	Q.ty	TOTAL CHARGE	BLASTING RETARDERS			
TYPE	DIMENSIONS (mm)	Unit weight Kg			Nr.	Kg	Group Nr.	partial retards	TIME (sec.)	QTY
KUBELA 420	40*300	0.39	5	5*12 =	60	23.40		25 Ms		5
TOTAL			5		60	23.40				

Horizontal holes

Horizontal Drilling holes diameter = 51mm
Length of drillied holes = 6.00m
Volume of blasted rock = 30.00 cub.m.
Length of blasted rock portion = 6.00m
Explosive type Kubela 40 x 300 Unit Weight Kg 0.39.
Total Explosive charge = 23,40 Kg
Explosive per cubic meter, ratio = 0.78 Kg/cub.m.

Gilgel Gibe III Hydroelectric Power Project

DIVERSION TUNNEL'S DRILLING & BLASTING

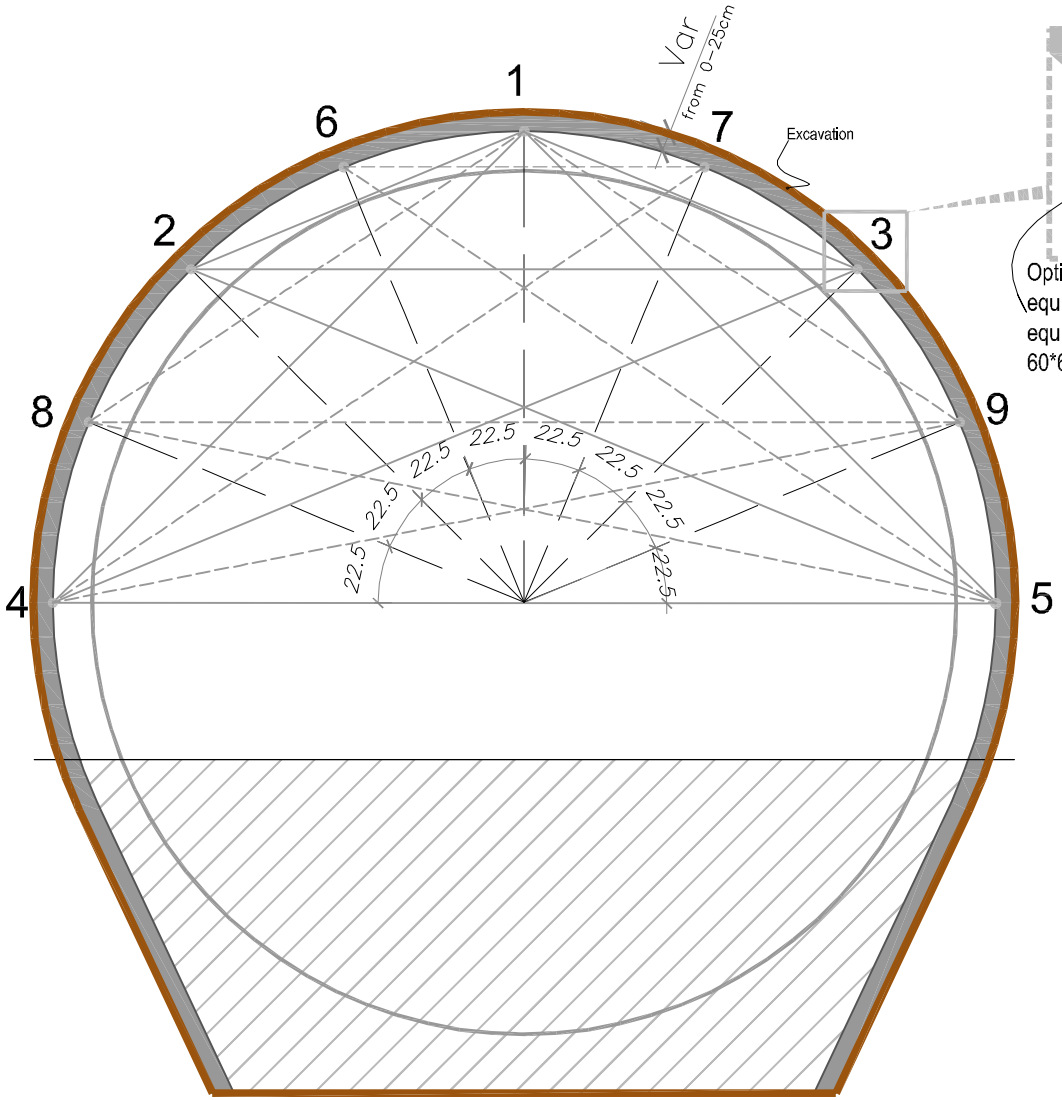
BLASTING SCHEME R.class IV&V | Size A4 | Scale

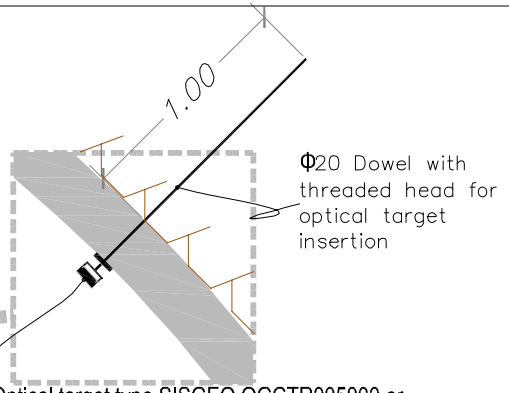
RIGHT & CENTRAL TUNNEL 2nd Phase | Rev.

METHOD STATEMENT n.01 | Attachment 05g | 3

310	TUN	M	SAS	105	Rev. 3	16-Feb-2008

Appendix F: Convergence Measurement Section, Rock Bolt Grouting Mixes and TypicalSections of Tunnels





Optical target type SISGEO OGCTR005000 or equivalent with adaptor type OGCTR38AD00 or equivalent or Leica Zielmarken DG3970 WEIB 60*60mm

POWER TUNNELS CONVERGENCE MEASUREMENT DATA RECORD

RIGHT/LEFT POWER TUNNEL

Chainage :

DATA DATE :

ID point	Coordinates not refers to Site general system		
(two readings)	X	Y	Elev.
1			
1			
2			
2			
3			
3			
4			
4			
5			
5			
6			
6			
7			
7			
8			
8			
9			
9			

ID point	(average reading)		
	X	Y	Elev.
1			
2			
3			
4			
5			
6			
7			
8			
9			

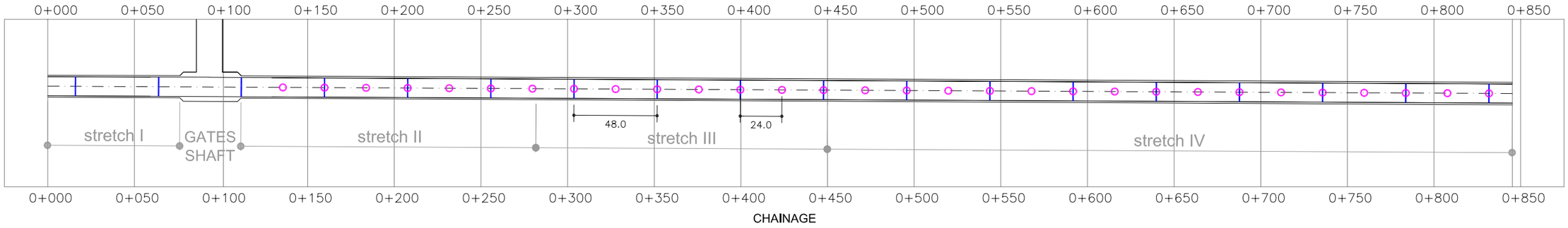
Segments (see sketch hereunder)	Initial date measure	Present data measure (m)	**variations (mm)**
4-1			
1-5			
5-4			
1-3			
3-4			
4-9			
9-1			
4-7			
6-5			
8-9			
8-1			
5-8			
2-1			
3-2			
5-2			
6-7			

The result will be plotted and compared to with previous readings in

Convergence measurments targets and data record table

Request of dwg:
GG3_L2_SAS SP WCM 25_140310

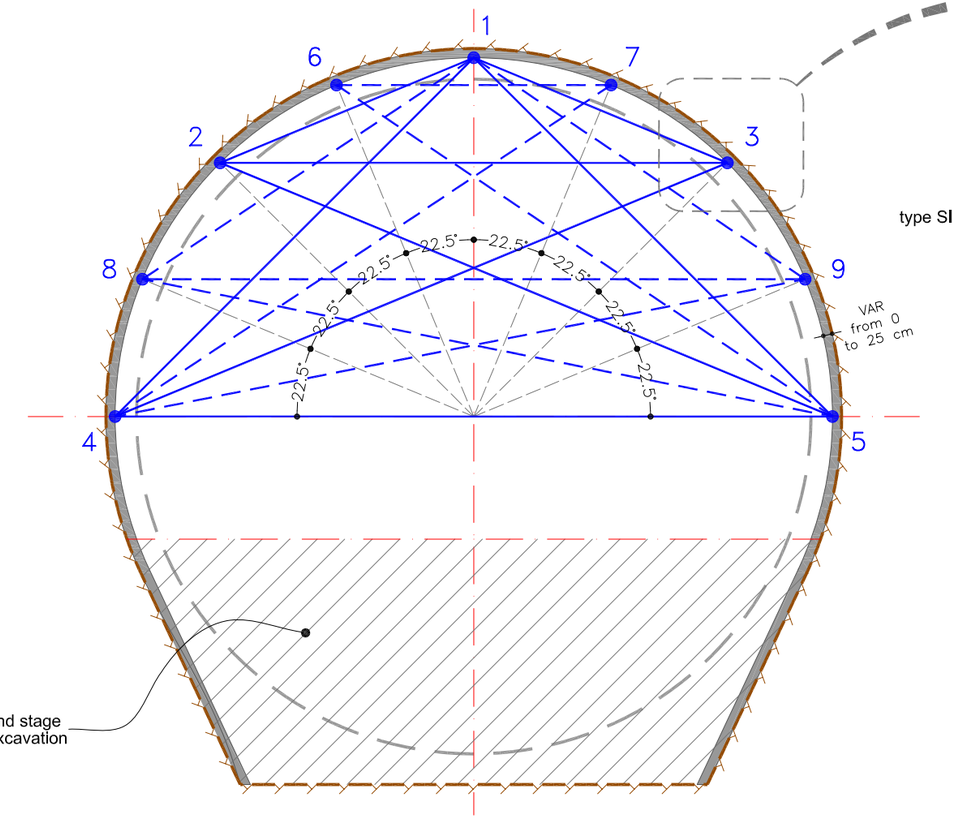
PTL PROFILE



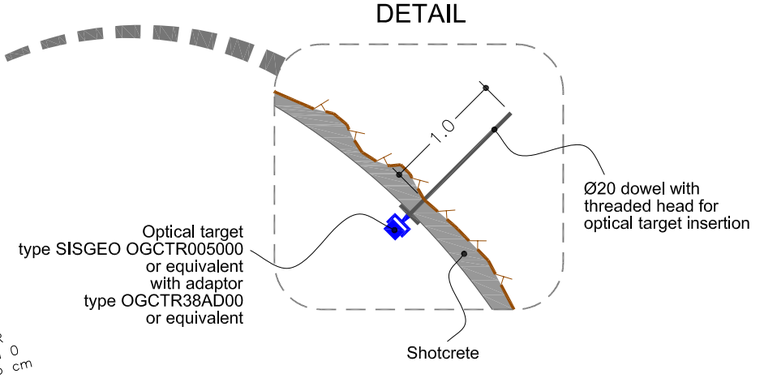
NOTES:
(1) Convergence measurement shall be carried out every 48 m and, in any case, at each GSI class variation recorded during excavation
(2) PTL: n° 18 sections instrumented for convergence measurement as minimum; n° 30 flat jack tests
(3) PTR: n° 13 sections instrumented for convergence measurement as minimum; n° 21 flat jack tests
(4) The number and/or the position of tests can be modified depending on convergence measurement and tests results, and geological/geotechnical conditions found during excavation

LEGEND
- Convergence measurement
- Flat Jack test

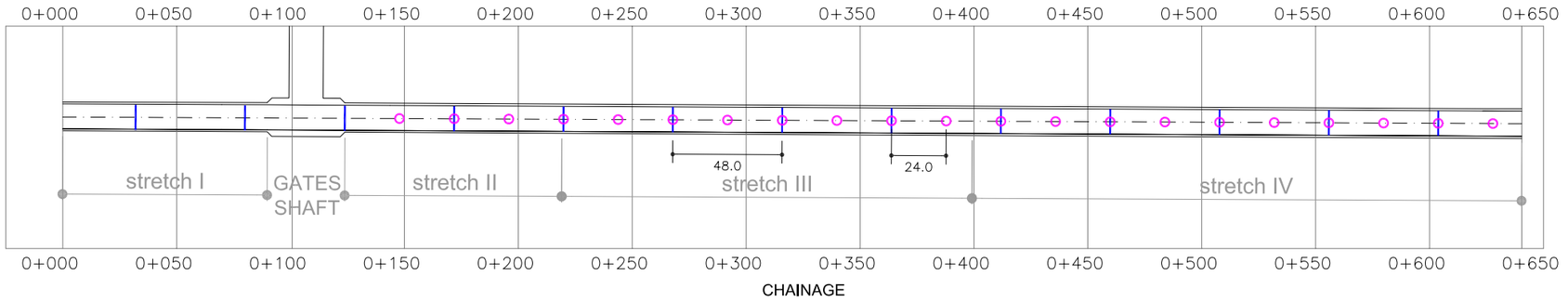
POWER TUNNEL - TYPICAL SECTION



DETAIL



PTR PROFILE



0 1 2 3 4 5m
(1:100 A3 size)

beneficiary	EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project	GIBE III Hydroelectric Project
contractor	salini COSTRUTTORI S.p.A.
designer	SP studio ing. g. pietrangeli s.r.l. consulting engineers
design	Level 2 DESIGN
title	550 = POWER TUNNEL RMK = Rock Mechanics CONVERGENCE MEASUREMENT AND FLAT JACK TEST

section	kind works	type	ROMA office	serial n°	rev.	date
5	5 0	RMK	D S	P	1 0 9	B	16 Mar 2010

reference drawings

ref.	title code
1	550 RMK D SP 110

modifications

issue	day	month	year	drawn	checked	approved
B	16	03	2010	MAG	ACG	PAP	Detail added Convergence measur. and flat jack test location
A	09	02	2010	MAG	ACG	PAP	First issue

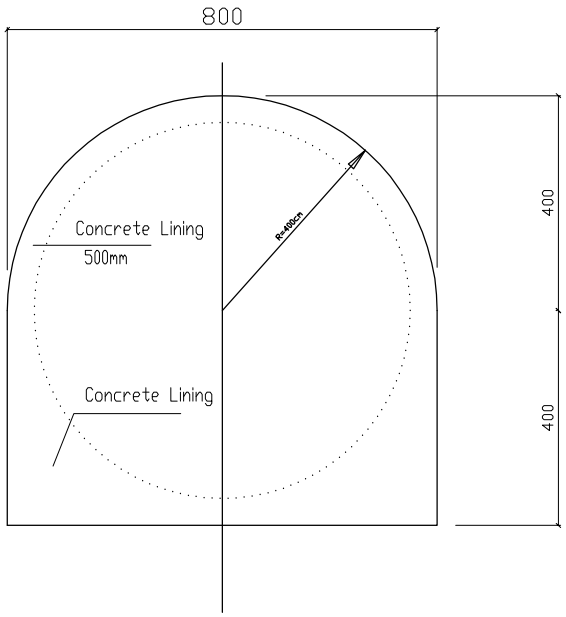
\\Server01\SP_Progetti\GIBE III\500 - LEVEL 2 DESIGN\550 - POWER TUNNELS\03 RMK - Rock Stabilization\DWGs\550 RMK D SP 109 B - PT Convergence Measurement and Flat Jack Test - 16mar2010.dwg msg, sp - Roma 16/MAR/2010 15:13:26

THIS DRAWING IS PROPERTY OF SP AND MAY NOT BE REPRODUCED OR RELEASED TO THIRD PARTIES WITHOUT WRITTEN AUTHORIZATION

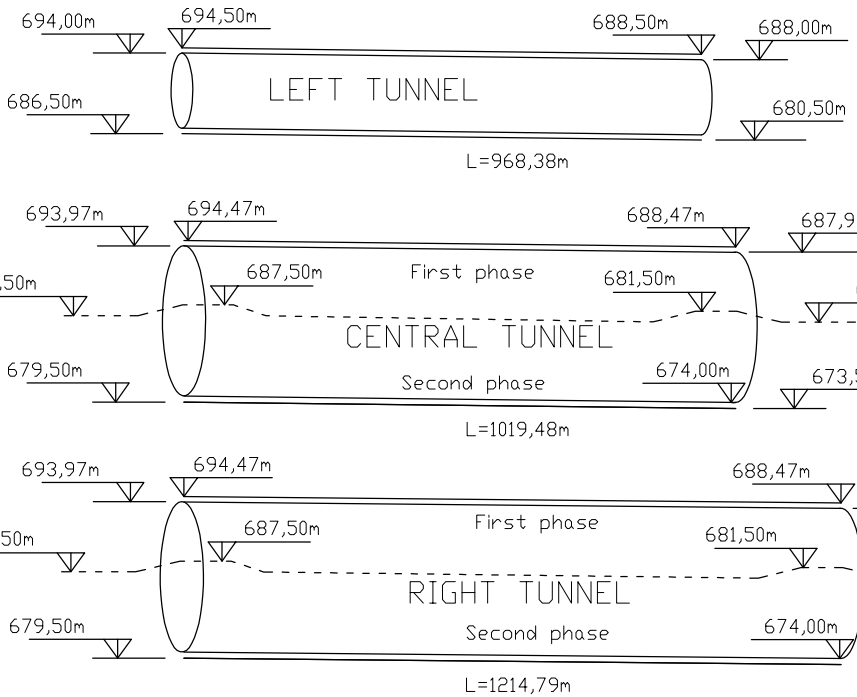
salini COSTRUTTORI S.p.A GEOTECHNICAL LABORATORY	**ROCK BOLT GROUTING MIXES**	GIBE III HYDROELECTRIC PROJECT ETHIOPIAN ELECTRIC POWER CORPORATION

Mix Type	W/C Ratio	Water lt/ton	Cement kg/ton	Sikament 340 %cem w.	Bentonite % cem w.	Theoretical Density gr/cmc	Cement Temp. °C	Mix Temp. °C	Viscosity sec	Real Density gr/cmc	Bleeding 2hr	Compressive Strength N/mm^2			
												2 days	3 days	7 days	28 days
A	**0.34**	**255**	**745**	**0.00**	**0.00**	**2.038**	**-**	**28**	**Non viscous**	**2.031**	**0.5%**	**-**	**-**	**-**	**-**
B	**0.35**	**260**	**740**	**0.00**	**0.00**	**2.022**	**-**	**27**	**Non viscous**	**2.021**	**0.5%**	**36.0**	**50.0**	**71.0**	**102.0**
C	**0.36**	**265**	**735**	**0.00**	**0.00**	**2.008**	**-**	**27**	**Non viscous**	**2.01**	**1.0%**	**-**	**-**	**-**	**-**

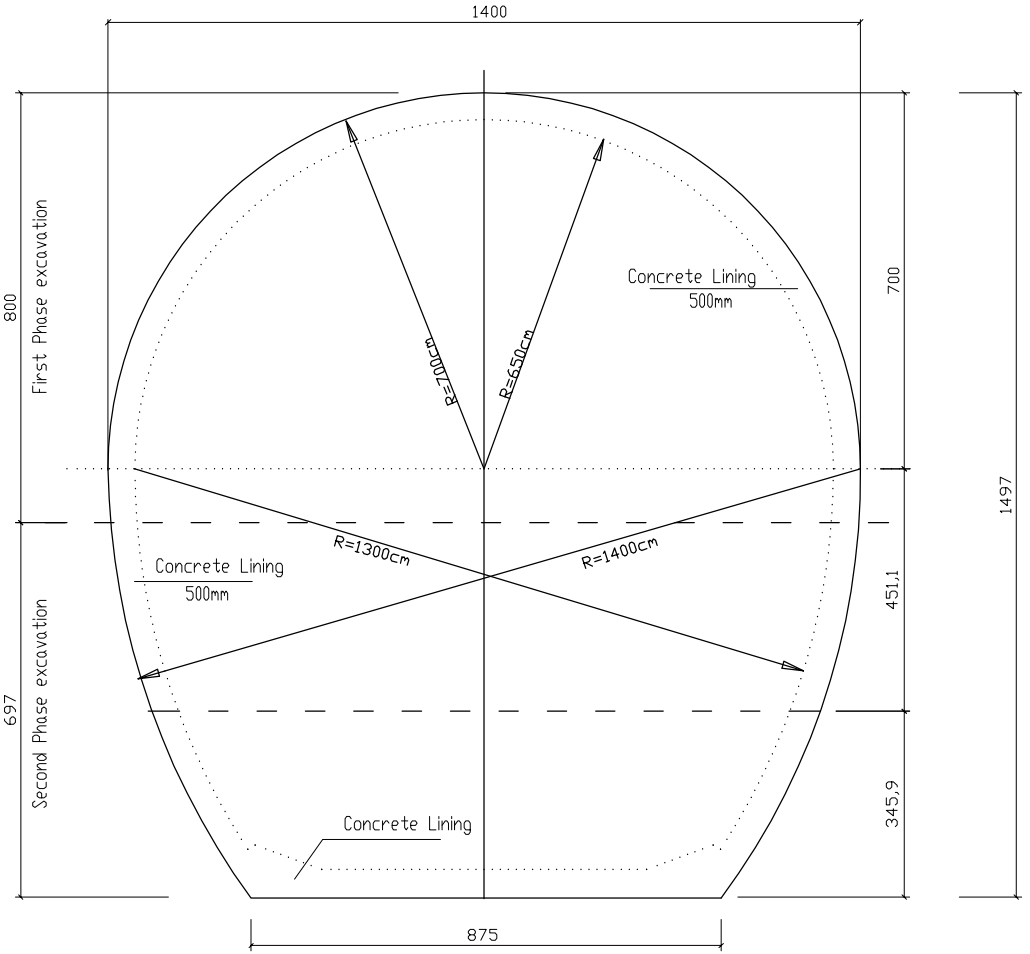
LEFT TUNNEL



EXCAVATION LEVELS

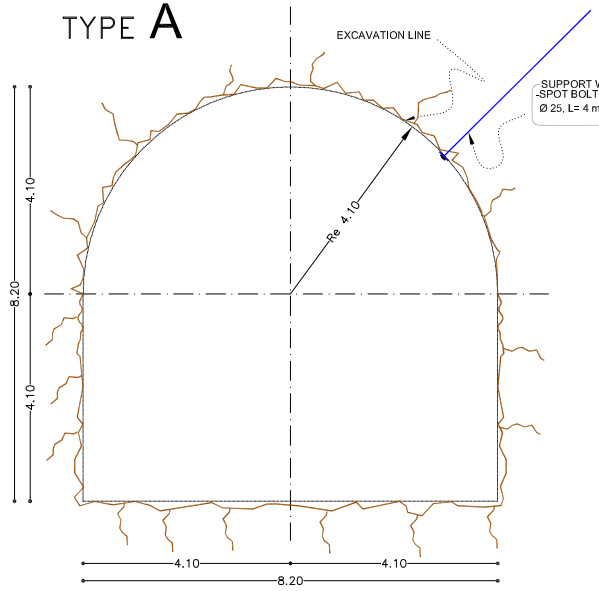


CENTRAL & RIGHT TUNNEL

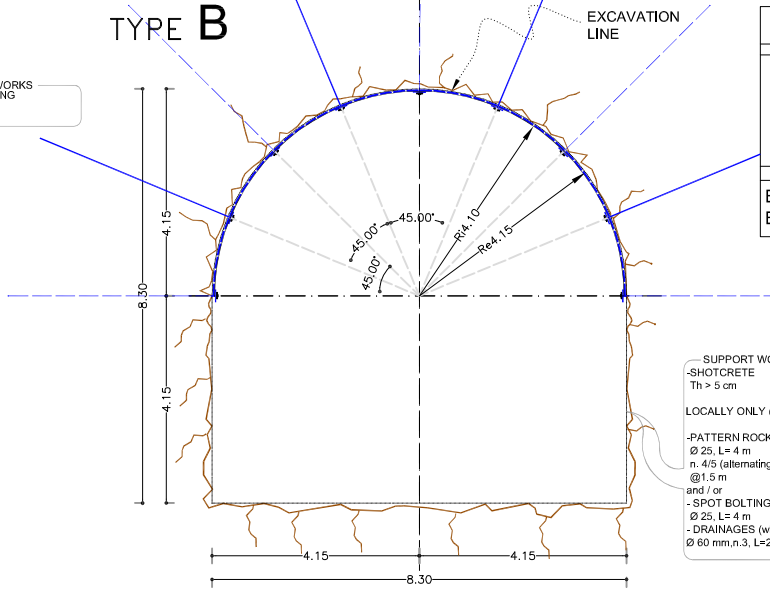


Gilgel Gibe III Hydroelectric Power Project							
DIVERSION TUNNEL'S DRILLING & BLASTING							
TYPICAL SECTIONS						Size A3	Scale
DIMENSIONS							Rev.
METHOD STATEMENT n.01					Attachment 02		2
310	TUN	M	SAS	102	Rev. 2	31-Jan-2008	

TYPE A



TYPE B



SUPPORT WORKS
-SHOTCRETE
Th > 5 cm

LOCALLY ONLY (If required)

-PATTERN ROCK BOLTS
Ø 25, L= 4 m
n. 4/5 (alternating)
@1.5 m
and / or
- SPOT BOLTING
Ø 25, L= 4 m
- DRAINAGES (when required)
Ø 60 mm,n.3, L=2m,@3m

ROCK TYPE	GSI MIN	GSI MAX	RMR 89' MIN	RMR 89' MAX	Type A	Type B	Type C	Type D	Type E
Trachyte slightly weathered unweathered	71	95	76	100	■				
	55	71	60	76		■			
	35	55	40	60			■		
	23	35	28	40				■	
Brecciated - Thachyte / Breccia (weathered)	32	46	37	51				■	
	23	32	28	37					■

Notes :

- SHOTCRETE SURFACE PREPARATION
Before applying shotcrete loose rock and dust must be removed with air-water jet leaving a damp surface

- SHOTCRETE THICKNESS = mInImum thIckness above surface. Average thickness some cms higher

- GSI < 23
Currently not expected for TL. If necessary specific support will be foreseen depending on local conditions.

- RMR 89' = RMR 89
Having: groundwater rating = 15 and Joint orientation = 0

- DRAINAGES
Where damp conditions or water dripping is observed

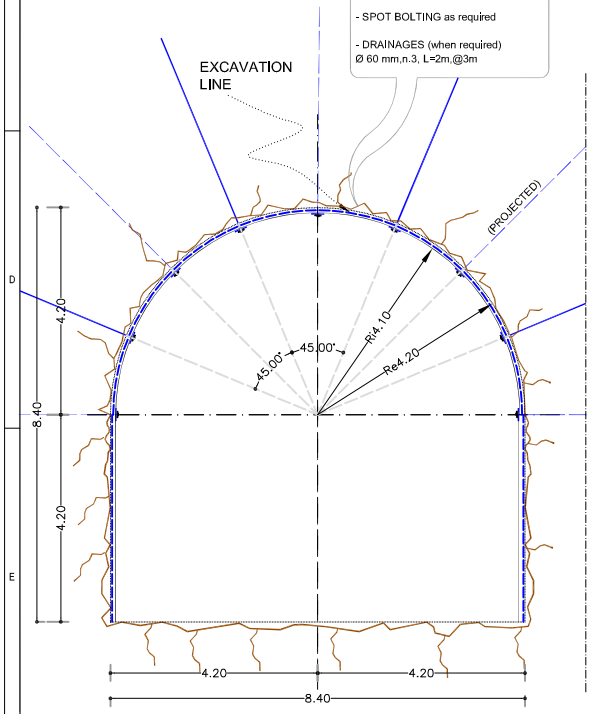
TYPE C

SUPPORT WORKS
-SHOTCRETE
Th > 10 cm
reInforced wIth WELDED MESH
Ø 5 10x10 or STEEL FIBER

-PATTERN ROCK DOWELS
Ø 25, L= 4 m
n. 4/5 (alternatIng)
@1.5 m

- SPOT BOLTING as required

- DRAINAGES (when required)
Ø 60 mm,n.3, L=2m,@3m



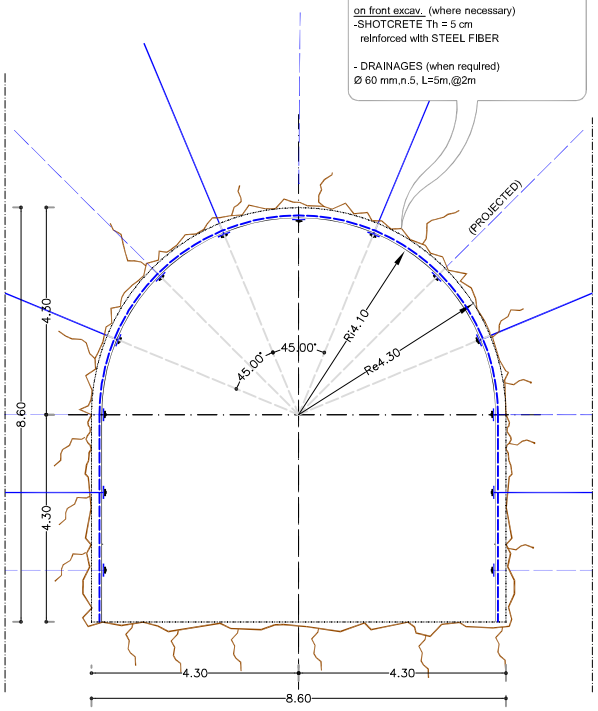
TYPE D

SUPPORT WORKS
-SHOTCRETE
Th > 20 cm
reinforced with WELDED MESH
Ø 5 10x10 or STEEL FIBER

-PATTERN ROCK DOWELS
Ø 25, L= 4 m
n. 6/7 (alternatIng)
@1.5 m

on front excav. (where necessary)
-SHOTCRETE Th = 5 cm
reInforced wIth STEEL FIBER

- DRAINAGES (when required)
Ø 60 mm,n.5, L=5m,@2m

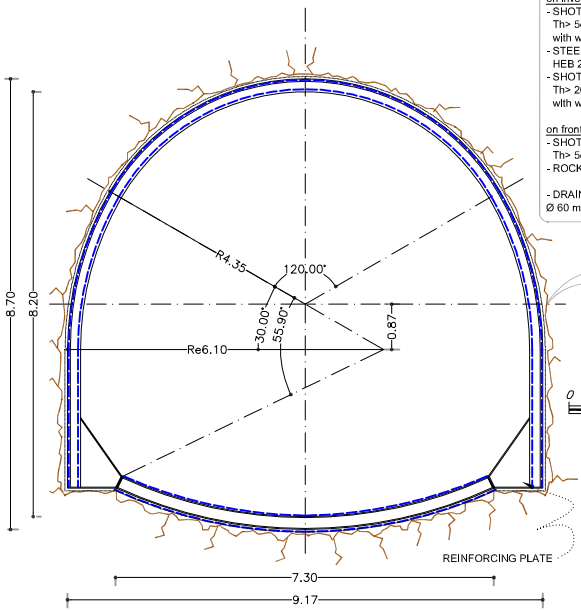


TYPE E

SUPPORT WORKS
on arch and side wall:
- SHOTCRETE
Th> 5cm
wIth welded mesh Ø5 10x10 or steel fIber
- STEEL SETS
HEB 200 @ 1.0m
where required spacIng can be varIed followIng
the local geotechnIcal condItIons
- SHOTCRETE
Th> 20cm
wIth welded mesh Ø5 10x10 or steel fIber
on Invert:
- SHOTCRETE
Th> 5cm
with welded mesh Ø5 10x10 or steel fiber
- STEEL SETS
HEB 200 @ 1.0m
- SHOTCRETE
Th> 20cm
wIth welded mesh Ø5 10x10 or steel fIber

on front excavation (where necessary):
- SHOTCRETE
Th> 5cm
- ROCK DOWELS Ø 25, 1@ 4m²

- DRAINAGES (when required)
Ø 60 mm,n.5, L=5m,@2m



0 0.5 1 1.5 2 2.5 5 m
(1:100 A3 size)

Modify dimension of cross section excavation						
9	16	11	2007	MT-ROD-TER		
Replace 310 TUN D SP 121 C						
A	29	08	2007	SER	AP	AP
issue	day	month	year	drawn	checked	approved

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 510 = RIVER DIVERSION
TUN = TUNNELS
TL TEMPORARY SUPPORT TYPICAL SECTIONS

section	kind works	type	ROMA office	serial n°	rev.	date
510	TUN	DSP	121	B	16	16 nov 2007

Z:\GIBE III\500 - LEVEL 2 DESIGN\510 - RIVER DIVERSION\TUN - TUNNELS\510-TUN-D-SP-121-A TL temporary support typical sections rev. 2007_08_29.dwg aser, sp - Roma 30/AUG/2007 10:33:32

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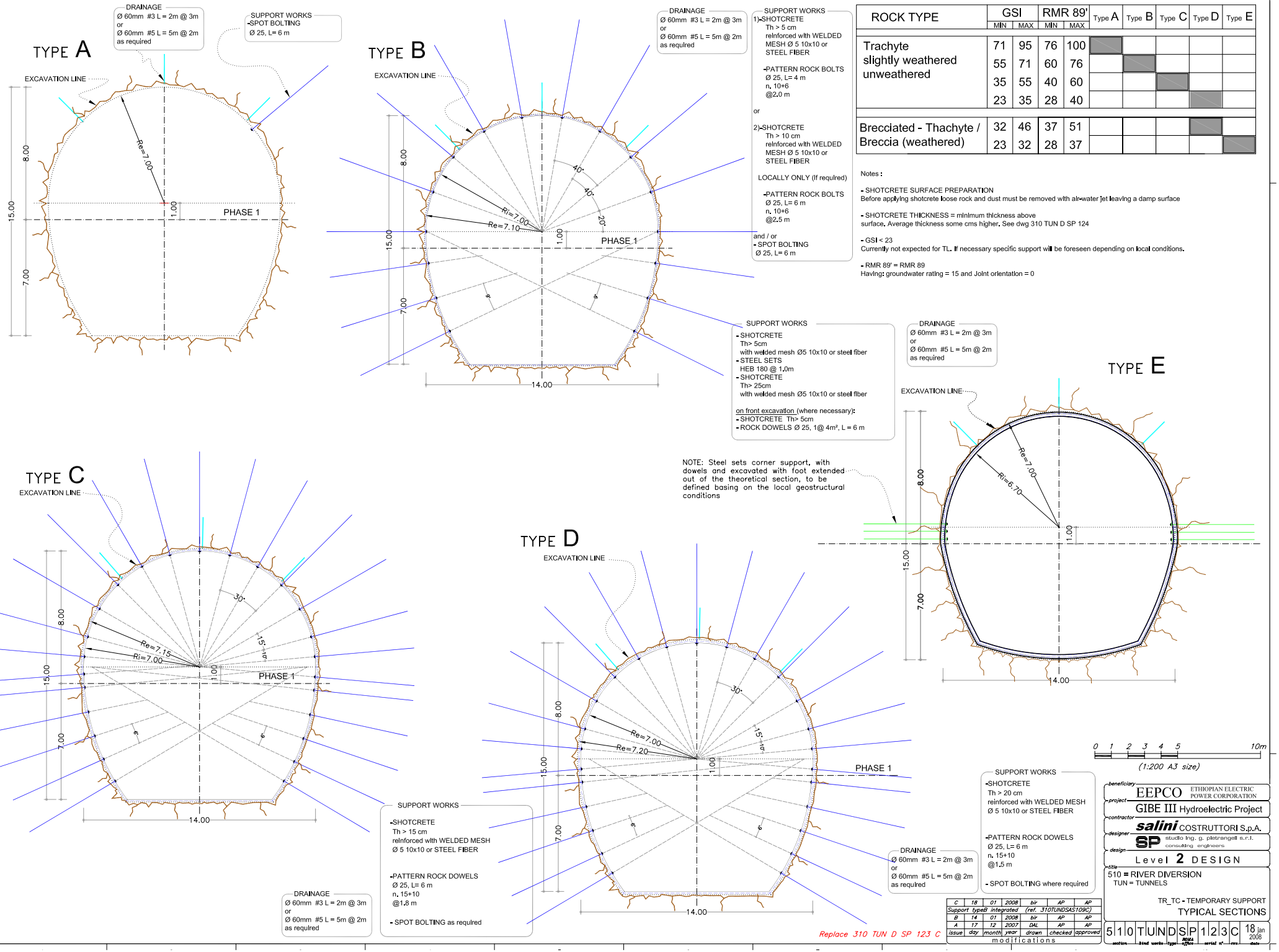


ROCK TYPE	GSI MIN	GSI MAX	RMR 89' MIN	RMR 89' MAX	Type A	Type B	Type C	Type D	Type E
Trachyte slightly weathered unweathered	71	95	76	100	■				
	55	71	60	76		■			
	35	55	40	60			■		
	23	35	28	40				■	
Brecciated - Thachyte / Breccia (weathered)	32	46	37	51				■	
	23	32	28	37					■

Notes :

- SHOTCRETE SURFACE PREPARATION
Before applying shotcrete loose rock and dust must be removed with air-water jet leaving a damp surface

- SHOTCRETE THICKNESS = minimum thickness above surface. Average thickness some cms higher. See dwg 310 TUN D SP 124

- GSI < 23
Currently not expected for TL. If necessary specific support will be foreseen depending on local conditions.

- RMR 89' = RMR 89
Having: groundwater rating = 15 and Joint orientation = 0

0 1 2 3 4 5 10m
(1:200 A3 size)

issue	day	month	year	drawn	checked	approved
C	18	01	2008	bir	AP	AP
Support typeB integrated (ref. 310TUNDSAS109C)						
B	14	01	2008	bir	AP	AP
A	17	12	2007	DAL	AP	AP

modifications

Replace 310 TUN D SP 123 C

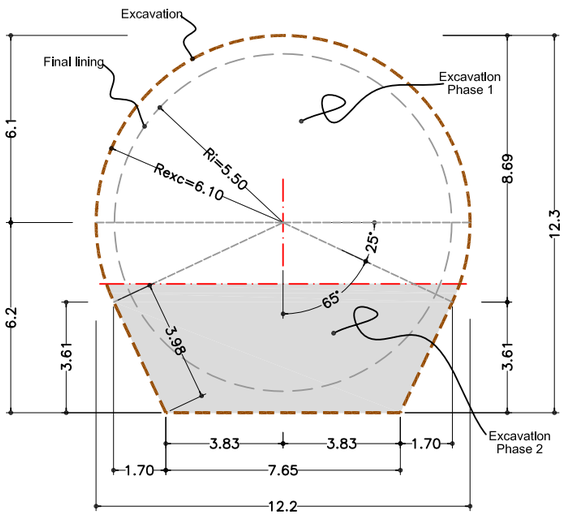
beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 510 = RIVER DIVERSION
TUN = TUNNELS
TR_TC - TEMPORARY SUPPORT
TYPICAL SECTIONS

section	kind works	type	serial n°	rev.	date
510	TUNDSP	123	C		18 jan 2008

Z:\GIBE III\500 - LEVEL 2 DESIGN\510 - RIVER DIVERSION\TUN - TUNNELS\D - DRAWINGS\510-TUN-D-SP-123 C TR_TC temporary support typical sections rev 18jan08.dwg bir, sp - Roma 18/JAN/2008 17:36:06

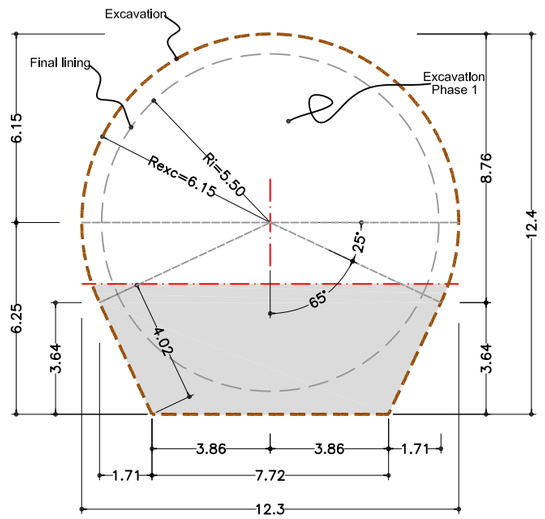
Request of dwg:
WCM 26 SAL – SP 20 Apr 2010

TYPE A, B



Perimeter (m) = 40.02
Area (m²) = 122.05

TYPE C



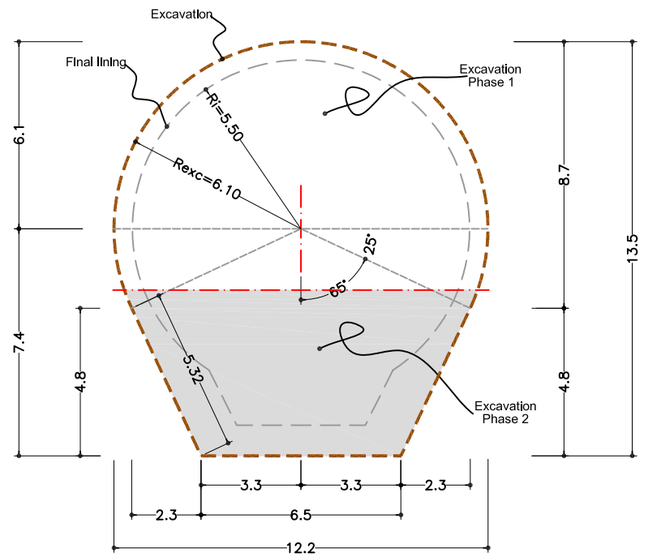
Perimeter (m) = 40.34
Area (m²) = 124.06

ROCK TYPE	GSI MIN	GSI MAX	RMR 89' MIN	RMR 89' MAX	Type A	Type B	Type C	Type D	Type E
Trachyte slightly weathered unweathered	71	95	76	100	■				
	56	70	61	75		■			
	36	55	41	60			■		
	23	35	28	40				■	
Brecciated - Thachyte / Breccia (weathered)	33	46	38	51				■	
	23	32	28	37					■

RMR 89' = RMR 89 Having: groundwater rating = 15 and Joint orientation = 0

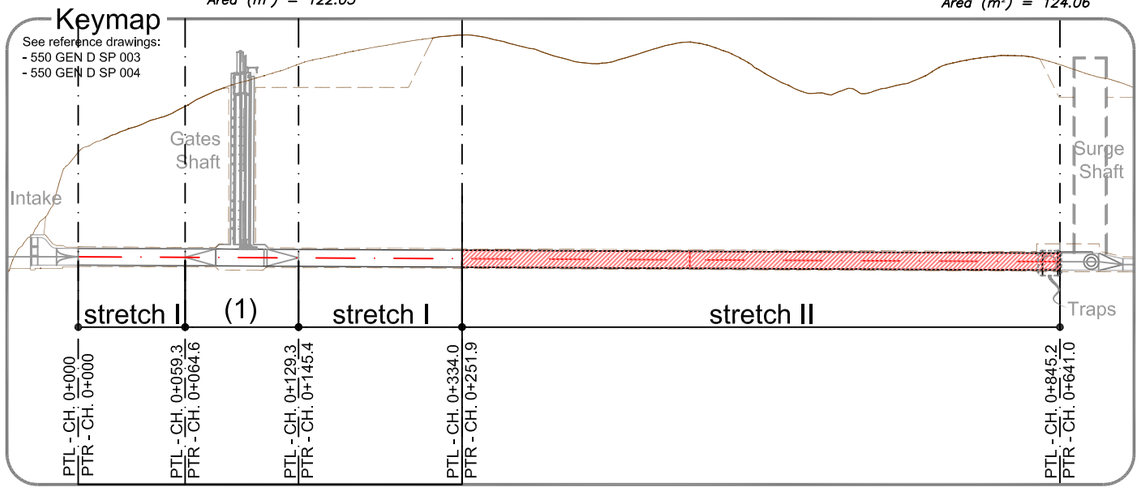
TYPCAL FOR TRAPS ONLY

PTR from ch. 0+625.0 to ch. 0+639.0
PTL from ch. 0+829.2 to ch. 0+843.2

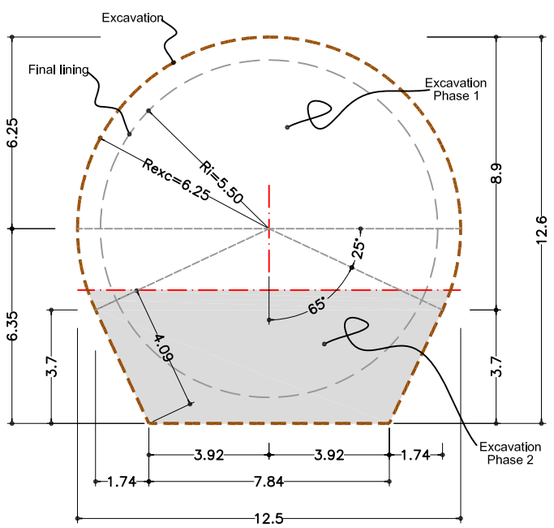


Keymap

See reference drawings:
- 550 GEN D SP 003
- 550 GEN D SP 004

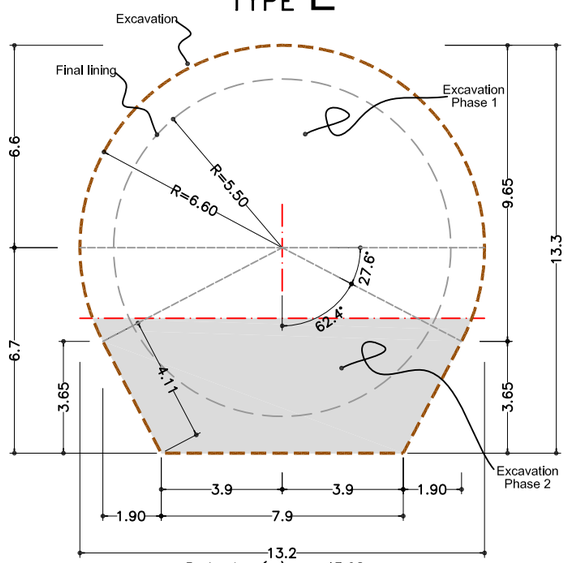


TYPE D



Perimeter (m) = 41.00
Area (m²) = 128.13

TYPE E



Perimeter (m) = 43.08
Area (m²) = 142.18

NOTES:
(1): GATES SHAFT STRETCH
Layout, excavation and lining of P.T. stretch in correspondence of gates shaft is described separately, in dedicated drawings series 552.

0 1 2 3 4 5 10m
(1:200 A3 size)

ref.	reference drawings: title code
6	550 CON D SP 004
5	550 EXC D SP 101
4	550 EXC D SP 102
3	550 EXC D SP 104
2	550 GEN D SP 004
1	550 GEN D SP 003

issue	day	month	year	modifications: drawn	checked	approved
Tunnel stretch chainages revised						
D	26	04	2010	SCOL	ACG	PAP
Excavation dimension variable according to rock type.						
C	30	09	2009	MAC	ACG	PAP
Excavations phases						
B	03	12	2008	GIOA	AP	AP
First issue						
A	06	11	2008	NEM	PAP	AP

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 550 = POWER TUNNEL
EXC = EXCAVATION
TEMPORARY SUPPORT
TYPICAL SECTIONS
STRETCH II

section	kind of works	type	ROMA office	serial n°	rev.	date
550	EXC	DSP	103	D		26 Apr 2010

\\Server01\sp_progetti\GIBE III\500 – LEVEL 2 DESIGN\550 – POWER TUNNELS\02 EXC – Excavation\DWG\550 EXC D SP 103 D – PT TEMPORARY SUPPORT – TYPICAL SECTIONS – Stretch II – 26apr2010.dwg scal, sp – Roma 26/APR/2010 18:22:07

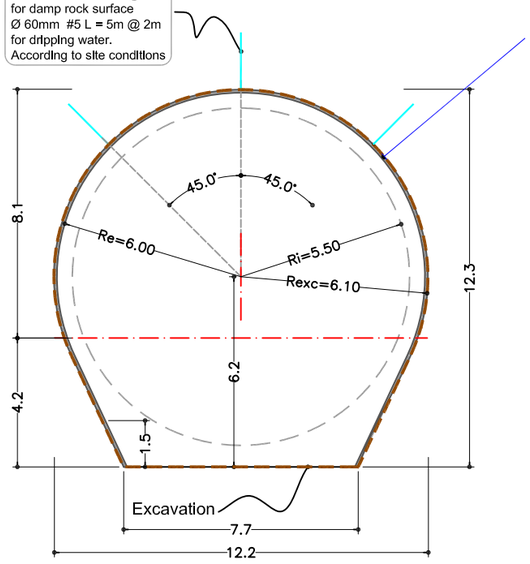
Request of dwg:
WCM 26 SAL – SP 20 Apr 2010

TYPE A

DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
SHOTCRETE
Th > 10 cm (1) (2) (3)
reinforced with WELDED
MESH Ø 5 10x10 cm

-SPOT BOLTING
where required
Ø 25, L= 6 m



TYPE B

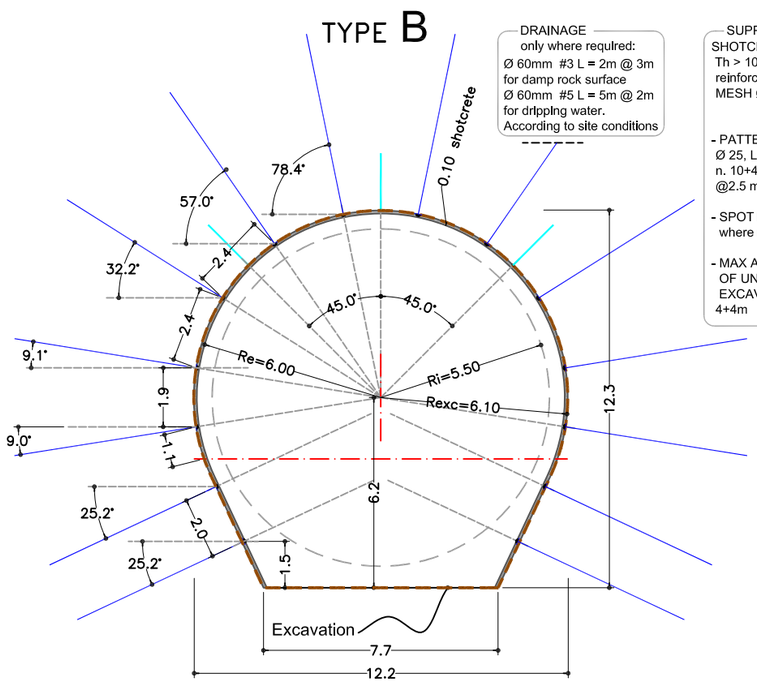
DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
SHOTCRETE
Th > 10 cm (1) (2) (3)
reinforced with WELDED
MESH Ø 5 10x10 cm

- PATTERN ROCK DOWELS
Ø 25, L= 6 m
n. 10+4
@2.5 m

- SPOT BOLTING
where required

- MAX ALLOWED LENGTH
OF UNSUPPORTED
EXCAVATION
4+4m

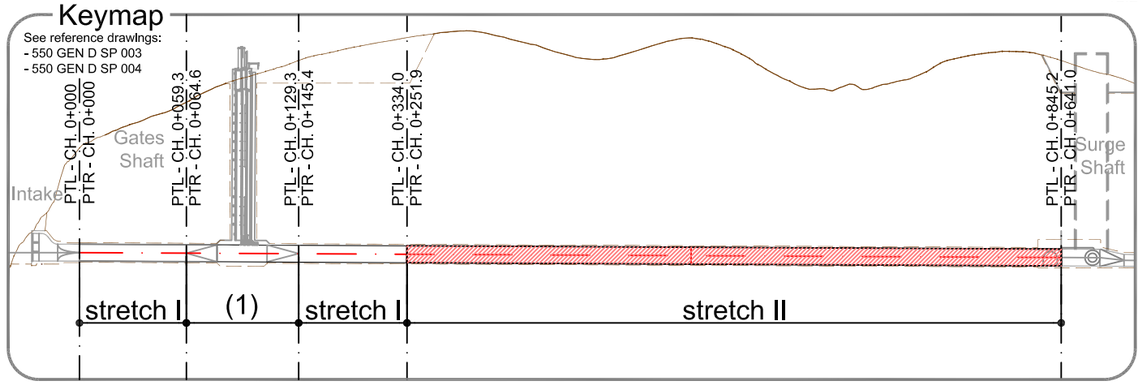


ROCK TYPE	GSI MIN	GSI MAX	RMR 89' MIN	RMR 89' MAX	Type A	Type B	Type C	Type D	Type E
Trachyte slightly weathered unweathered	71	95	76	100	■				
	56	70	61	75		■			
	36	55	41	60			■		
	23	35	28	40				■	
Brecciated - Thachyte / Breccia (weathered)	33	46	38	51				■	
	23	32	28	37					■

NOTES:
(1) Potential wedge failure by three suvertical joints cannot be excluded on first 200 m of tunnel .
Where these potential wedges are found during the excavation the shotcrete thickness shall be increased up to 20 cm or specific additional spot bolting applied.
(2) - SHOTCRETE SURFACE PREPARATION
Before applying shotcrete loose rock and dust must be removed with air-water jet leaving a damp surface.
(3) - SHOTCRETE THICKNESS = minimum thickness above surface. Average thickness some cms higher. Not included preparatory skin shotcrete and void filling shotcrete to be done as far as necessary before support works application. See dwg 500 RMK D SP 005
(4) - GSI < 23 Currently not expected for P.T. If necessary specific support will be foreseen depending on local conditions.
(5) - RMR 89' = RMR 89 Having: groundwater rating = 15 and Joint orientation = 0
(6) - Possible improvement of the support measures for section D and E (including possible revision of steel sets spacing) can be considered according to the observation at site of the behaviour of the excavation and support made.

Keymap

See reference drawings:
- 550 GEN D SP 003
- 550 GEN D SP 004



NOTE: Steel sets corner: support with dowels and excavated with foot extended out of the theoretical section, to be defined basing on the local geostructural conditions (steel plates, bifurcated steel ribs ends, bolts or other devices can be also used according to the need, to be defined at site)

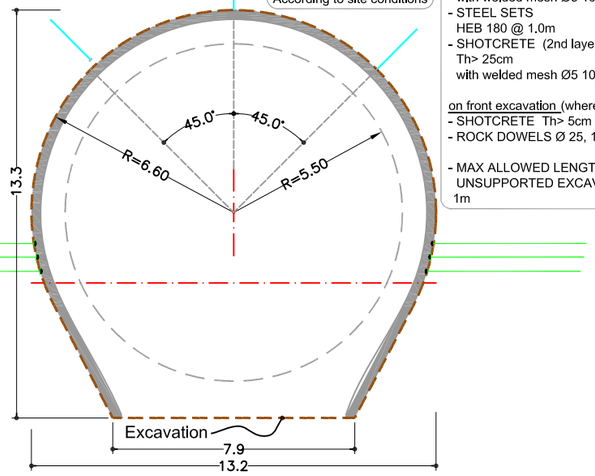
TYPE E

DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
- SHOTCRETE (1st layer)
Th> 5cm
with welded mesh Ø5 10x10 cm
- STEEL SETS
HEB 180 @ 1.0m
- SHOTCRETE (2nd layer)
Th> 25cm
with welded mesh Ø5 10x10 cm

on front excavation (where necessary):
- SHOTCRETE Th> 5cm
- ROCK DOWELS Ø 25, 1@ 4m², L = 6 m

- MAX ALLOWED LENGTH OF UNSUPPORTED EXCAVATION
1m



TYPE C

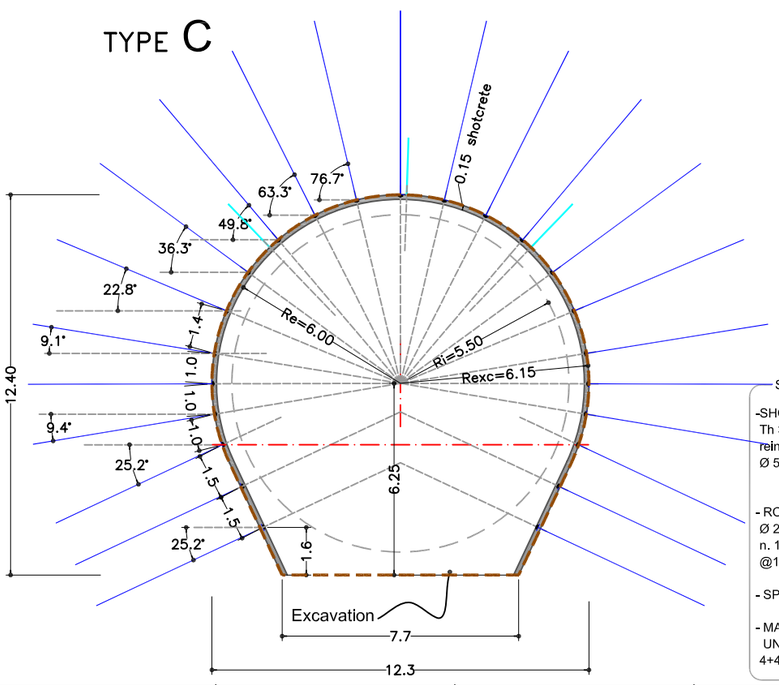
DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
-SHOTCRETE
Th > 15 cm (1) (2) (3)
reinforced with WELDED MESH
Ø 5 10x10 cm

- ROCK DOWELS PATTERN
Ø 25, L= 6 m
n. 17+6
@1.8 m

- SPOT BOLTING where required

- MAX ALLOWED LENGTH OF
UNSUPPORTED EXCAVATION
4+4m



TYPE D

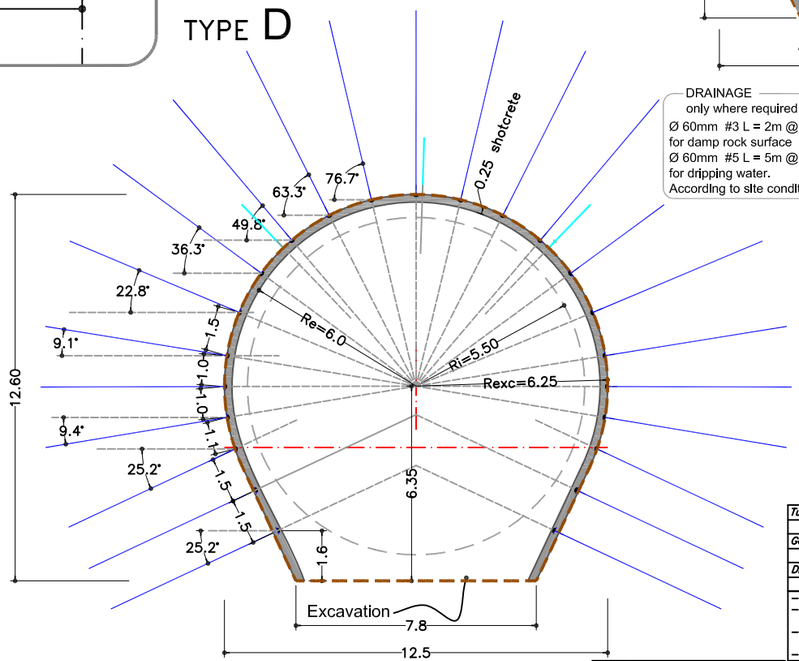
DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
-SHOTCRETE
Th > 25 cm (2) (3)
reinforced with WELDED MESH
Ø 5 10x10 cm

- ROCK DOWELS PATTERN
Ø 25, L= 6 m
n. 17+6
@1.5 m

- SPOT BOLTING where required

- MAX ALLOWED LENGTH OF
UNSUPPORTED EXCAVATION
4+4m for Trachyte slightly
weathered / unweatered
2+2m for Brecciated - Trachyte /
Breccia (weatered)



0 1 2 3 4 5 10m
(1:200 A3 size)

ref.	reference drawings
5	550 EXC D SP 101
4	550 EXC D SP 102
3	550 EXC D SP 103
2	550 GEN D SP 004
1	550 GEN D SP 003

issue	day	month	year	drawn	checked	approved
Tunnel stretch chainages revised						
F	26	04	2010	SCOL	ACG	PAP
Gate shaft transition chainage updated						
E	03	12	2009	MDG	ACG	PAP
Dimensions updated						
D	30	09	2009	MDG	ACG	PAP
– Updated type D shotcrete thickness – Excavations enlarged according to shotcrete thickness at each section type – Max allowable length of unsupported excavation added – Note 6 added						
C	02	04	2008	EDS	ACG	AP
Excavation phases and pattern dowels						
B	02	12	2008	EDS	AP	AP
First issue						
A	06	11	2008	NEM	AP	AP

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 550 = POWER TUNNEL
RMK = Rock Mechanics
TEMPORARY SUPPORT
TYPICAL SECTIONS
STRETCH II

section	kind works	type	RMK sub-type	serial n°	rev.	date
550	RMK	DSP	101	F		26 Apr 2010

Z:\GIBE III\500 – LEVEL 2 DESIGN\550 – POWER TUNNELS\03 RMK – Rock Stabilization\DWGs\550 RMK D SP 101 F – Temporary Support – Typical Sections Stretch II – 26apr2010.dwg scal, sp – Roma 26/APR/2010 18:36:23

\\Server01\sp_progetti\GIBE III\500 - LEVEL 2 DESIGN\550 - POWER TUNNELS\03 RMK - Rock Stabilization\DWGs\550 RMK D SP 102 C - Temporary Support - Typical Sections Stretch I - 21apr2010.dwg scal, sp - Roma 22/APR/2010 17:06:01

Request of dwg:
WCM 26 SAL – SP 20 Apr 2010

ROCK TYPE	GSI MIN	GSI MAX	RMR 89' MIN	RMR 89' MAX	Type A	Type B	Type C	Type D	Type E
Trachyte slightly weathered unweathered	71	95	76	100	■				
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	36	55	41	60			■		
	23	35	28	40				■	
Brecciated - Thachyte / Breccia (weathered)	33	46	38	51				■	
	23	32	28	37					■

NOTES:
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(6) - Possible improvement of the support measures for section D and E (including possible revision of steel sets spacing) can be considered according to the observation at site of the behaviour of the excavation and support made.

TYPE A

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only where requlred:
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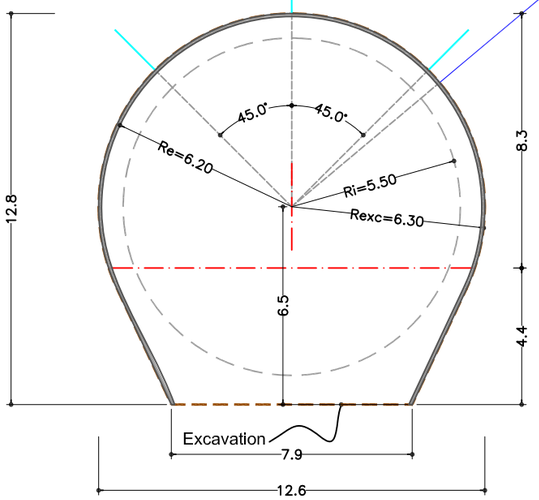
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
SHOTCRETE
Th > 10 cm (1) (2) (3)
reinforced with WELDED
MESH Ø 5 10x10 cm

-SPOT BOLTING
where required
Ø 25, L= 6 m



TYPE B

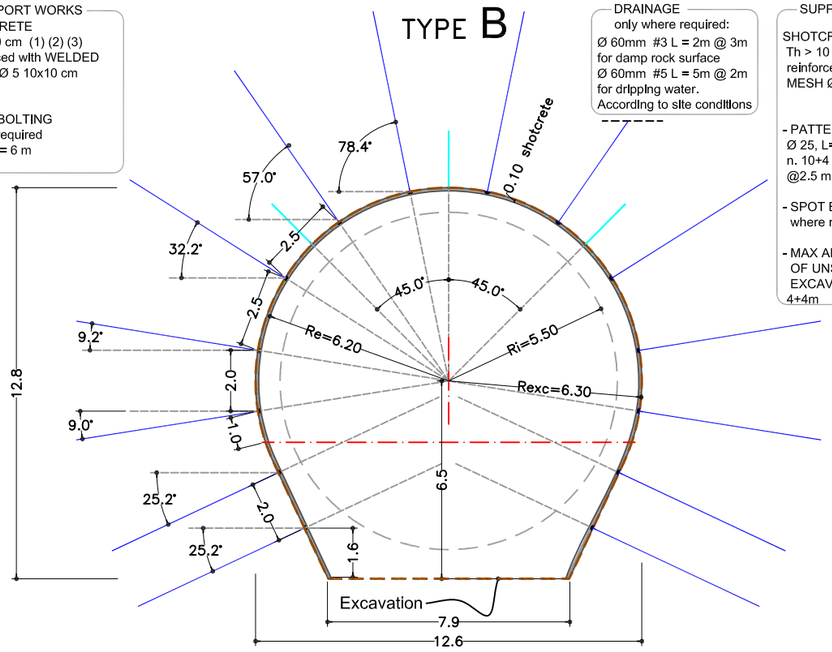
DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
SHOTCRETE
Th > 10 cm (1) (2) (3)
reinforced with WELDED
MESH Ø 5 10x10 cm

- PATTERN ROCK DOWELS
Ø 25, L= 6 m
n. 10+4
@2.5 m

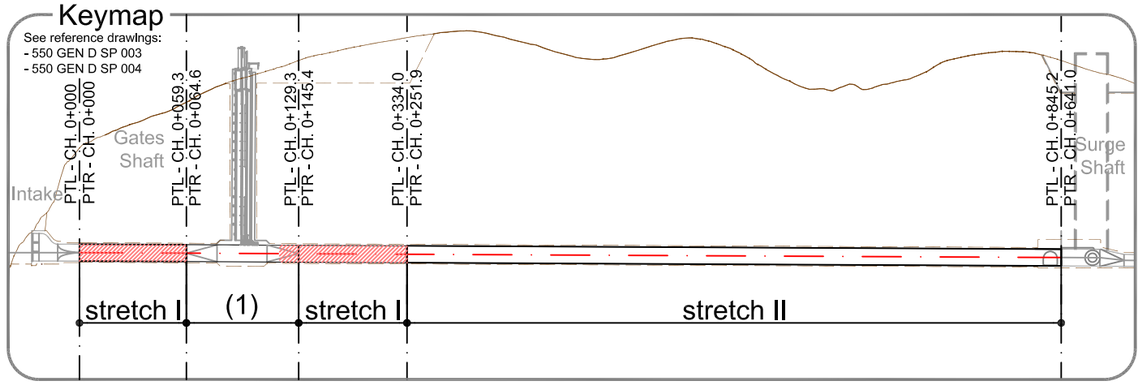
- SPOT BOLTING
where required

- MAX ALLOWED LENGTH
OF UNSUPPORTED
EXCAVATION
4+4m



Keymap

See reference drawings:
- 550 GEN D SP 003
- 550 GEN D SP 004



NOTE: Steel sets corner: support with dowels and excavated with foot extended out of the theoretical section, to be defined basing on the local geostructural conditions (steel plates, bifurcated steel ribs ends, bolts or other devices can be also used according to the need, to be defined at site)

TYPE C

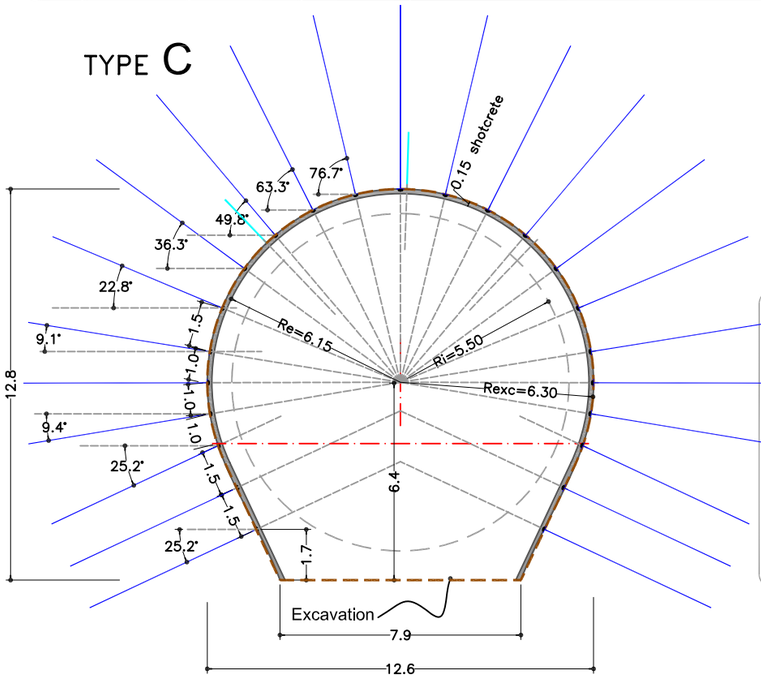
DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
-SHOTCRETE
Th > 15 cm (1) (2) (3)
reinforced with WELDED MESH
Ø 5 10x10 cm

- ROCK DOWELS PATTERN
Ø 25, L= 6 m
n. 17+6
@1.8 m

- SPOT BOLTING where required

- MAX ALLOWED LENGTH OF
UNSUPPORTED EXCAVATION
4+4m

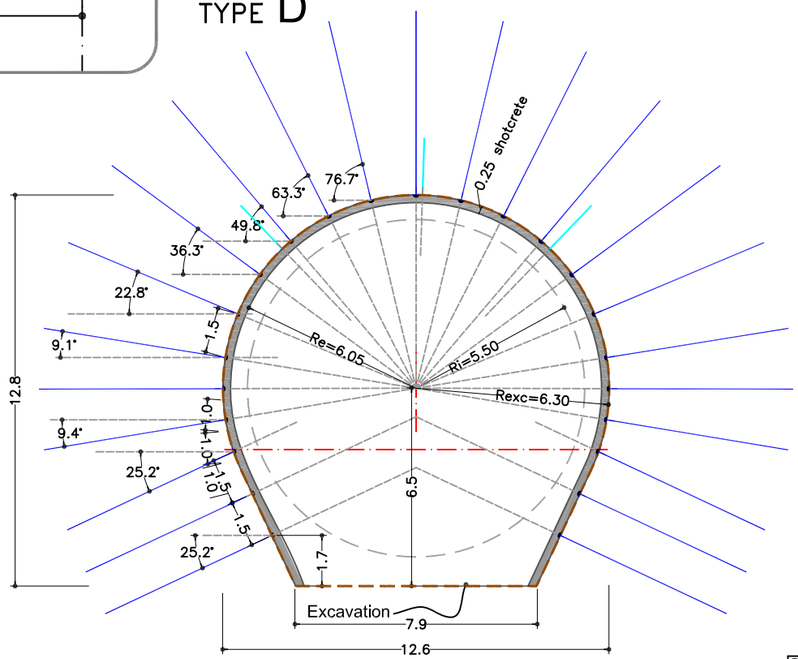


TYPE D

SUPPORT WORKS
- SHOTCRETE (1st layer)
Th> 5cm (2) (3)
with welded mesh Ø5 10x10 cm
- STEEL SETS
HEB 180 @ 1.0m
- SHOTCRETE (2nd layer)
Th> 25cm
with welded mesh Ø5 10x10 cm

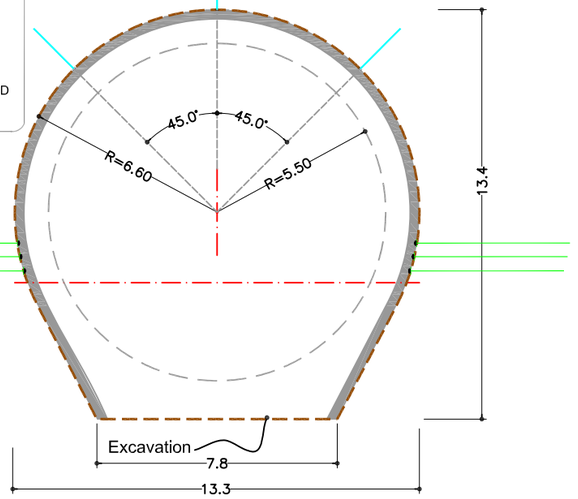
on front excavation (where necessary):
- SHOTCRETE Th> 5cm
- ROCK DOWELS Ø 25, 1@ 4m², L = 6 m

- MAX ALLOWED LENGTH OF UNSUPPORTED
EXCAVATION
1m



TYPE E

DRAINAGE
only where required:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions



DRAINAGE
only where requlred:
Ø 60mm #3 L = 2m @ 3m
for damp rock surface
Ø 60mm #5 L = 5m @ 2m
for dripping water.
According to site conditions

SUPPORT WORKS
-SHOTCRETE
Th > 25 cm (2) (3)
reinforced with WELDED MESH
Ø 5 10x10 cm

- ROCK DOWELS PATTERN
Ø 25, L= 6 m
n. 17+6
@1.5 m

- SPOT BOLTING where required

- MAX ALLOWED LENGTH OF
UNSUPPORTED EXCAVATION
4+4m for Trachyte slightly
weathered / unweatered
2+2m for Brecciated - Trachyte /
Breccia (weatered)

0 1 2 3 4 5 10m
(1:200 A3 size)

ref.	title code
5	550 EXC D SP 101
4	550 EXC D SP 102
3	550 EXC D SP 103
2	550 GEN D SP 004
1	550 GEN D SP 003

reference drawings

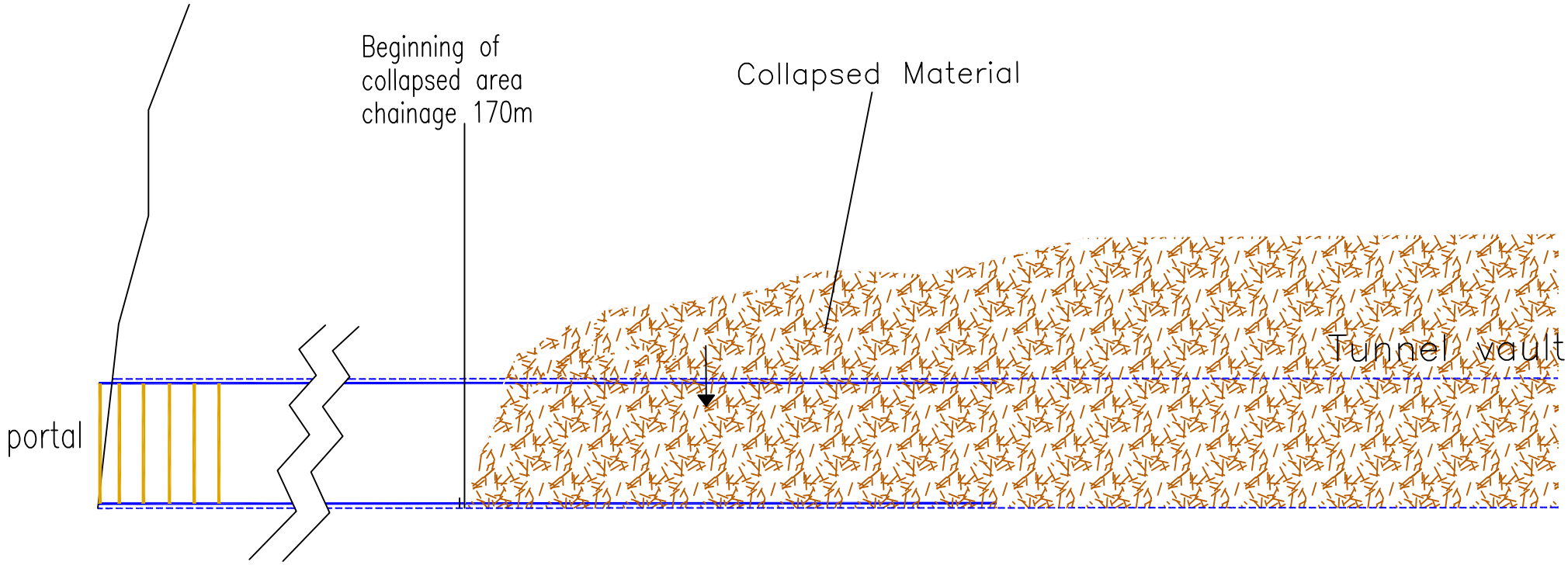
issue	day	month	year	drawn	checked	approved
Tunnel stretch chainages revised						
C	26	04	2010	SCOL	AP	AP
Gate shaft transition chainage updated						
B	03	12	2009	MDG	AP	AP
First issue						
A	30	09	2009	MDG	AP	AP

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor / designer: salini COSTRUTTORI S.p.A.
design: SP studio ing. g. pietrangeli s.r.l. consulting engineers
Level 2 DESIGN
title: 550 = POWER TUNNEL
RMK = Rock Mechanics
TEMPORARY SUPPORT
TYPICAL SECTIONS
STRETCH I

section	kind works	type	RMK/A office	serial n°	rev.	date
550	RMK	D SP	102	C		26 Apr 2010

Appendix G: Attachments of Drawings for Remedial Works.



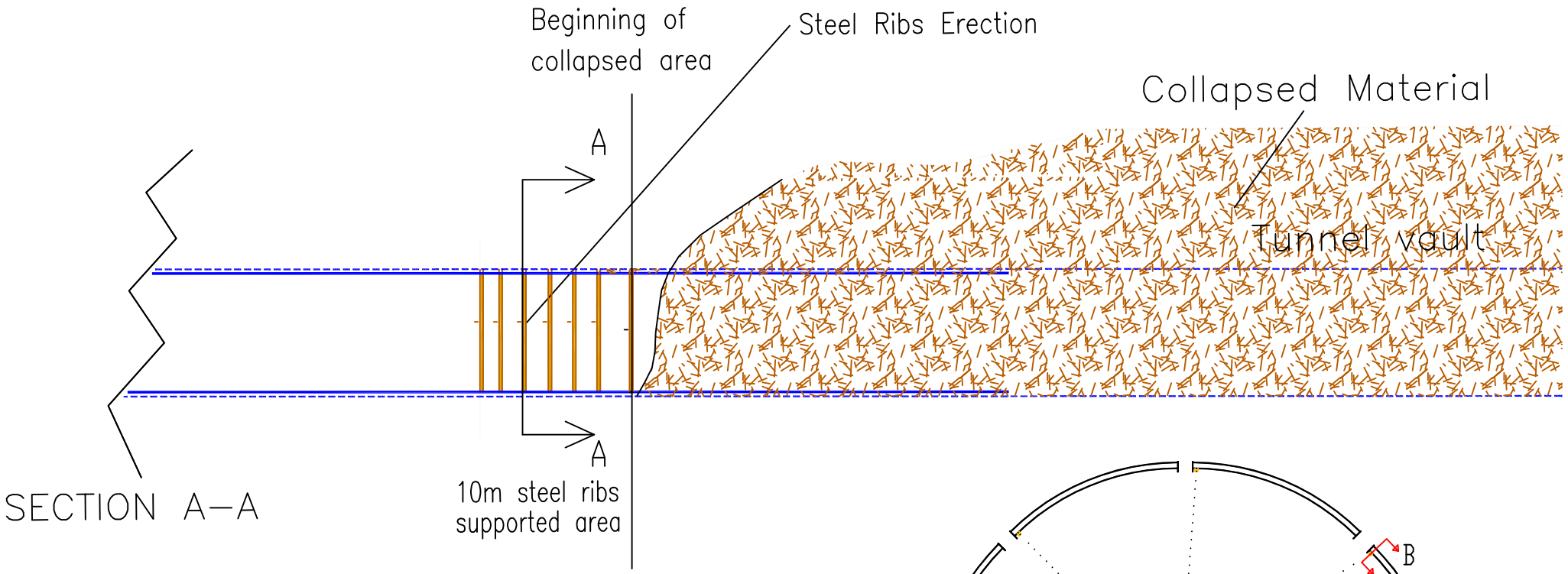
Gilgel Gibe III Hydroelectric Power Project

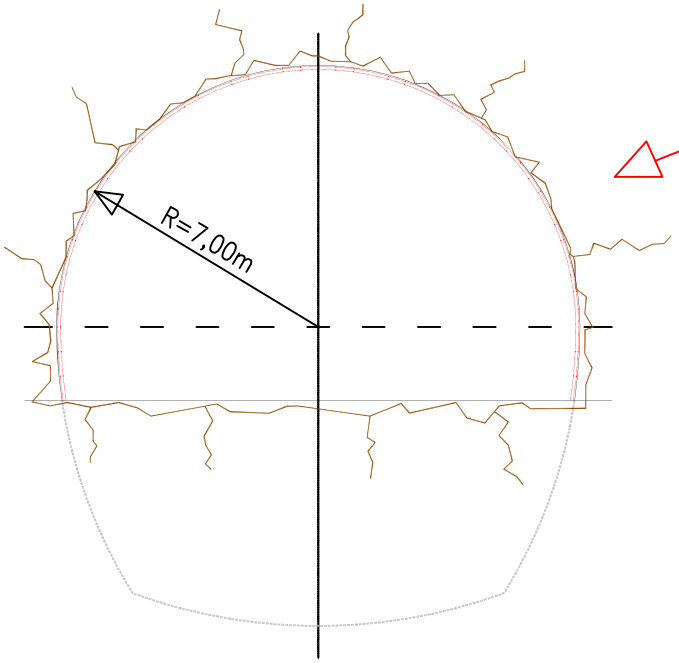
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS

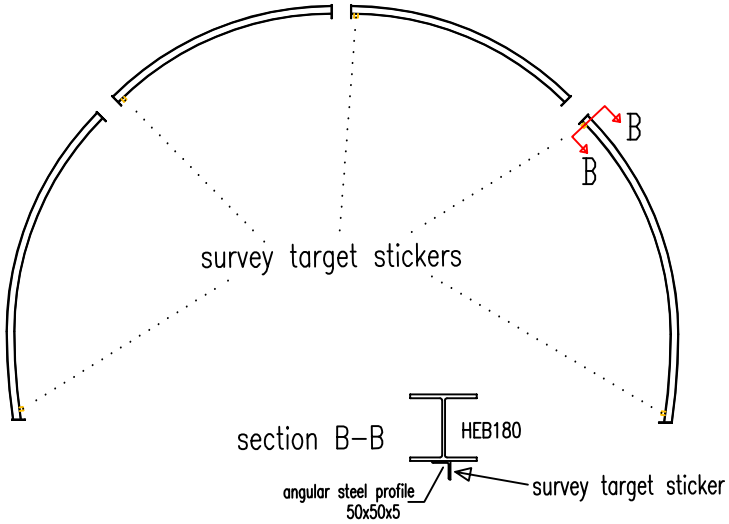
Profile of the tunnel with typical collapse	Size A4	Scale / Rev.

METHOD STATEMENT n.03	Attachment 1	0

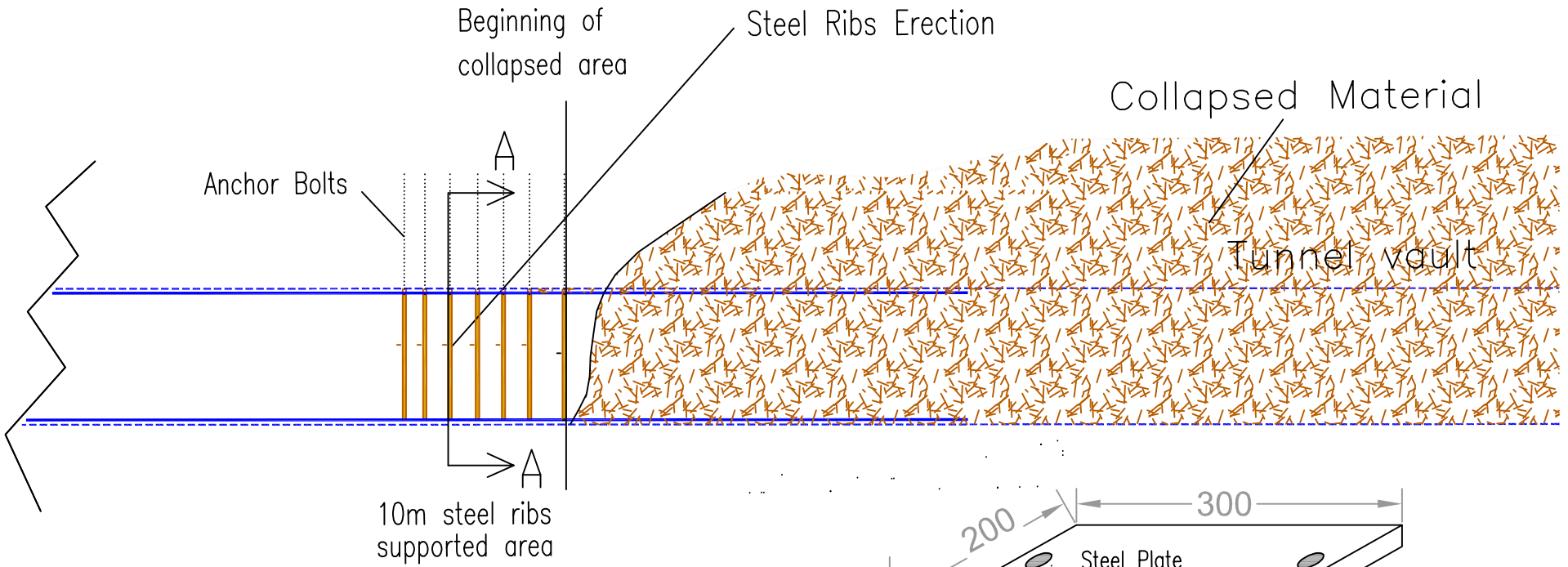
310	TUN	M	SAS	131	Rev. 0	15-May-2007

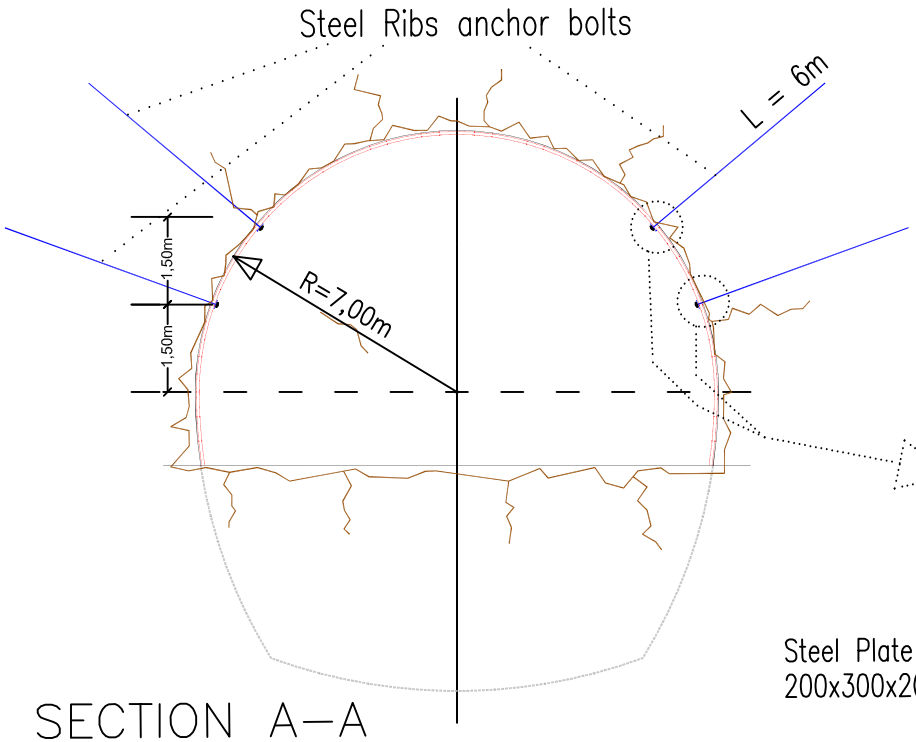




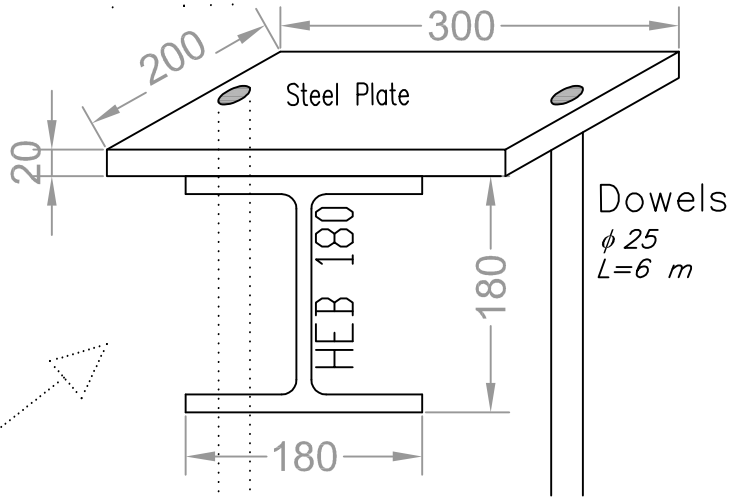


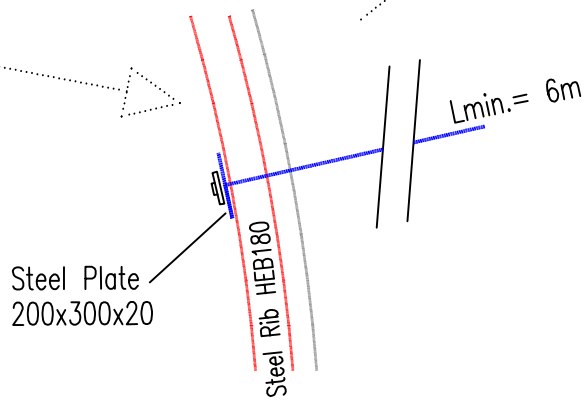
Gilgel Gibe III Hydroelectric Power Project							
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS							
Steel Ribs Erection						*Size* A4	*Scale*
Assembly and erection of steel ribs							*Rev.*
METHOD STATEMENT n.03	Attachment 2						0
310	*TUN*	*M*	*SAS*	*132*	*Rev. 0*	*15–May–2007*	



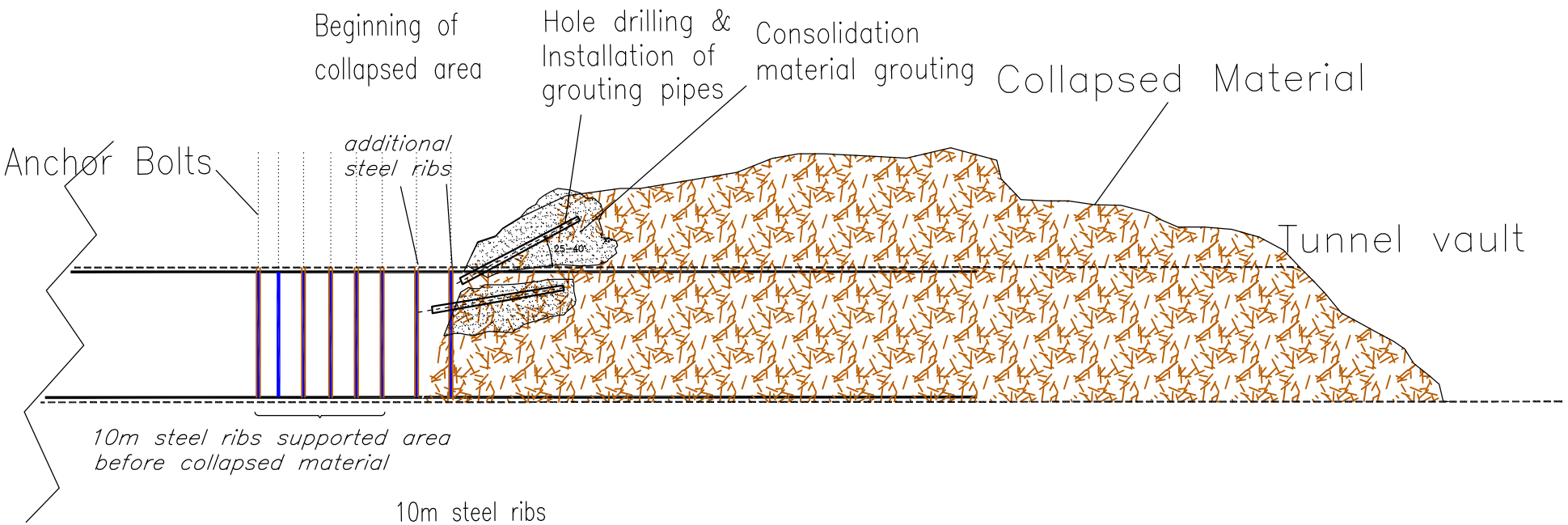


SECTION A–A





Gilgel Gibe III Hydroelectric Power Project							
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS							
Steel Ribs Erection						Size A4	Scale
Fixing of the steel ribs to the vault							Rev.
METHOD STATEMENT n.03				Attachment 3			0
310	TUN	M	SAS	133	Rev. 0	15–May–2007	



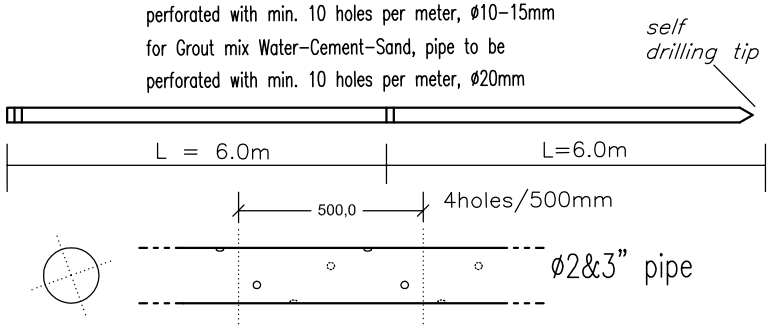
10m steel ribs supported area

Grouting Pipe

Pipe ø2" Lmin = 12m

for Grout mix Water-Cement, pipe to be perforated with min. 10 holes per meter, ø10-15mm

for Grout mix Water-Cement-Sand, pipe to be perforated with min. 10 holes per meter, ø20mm

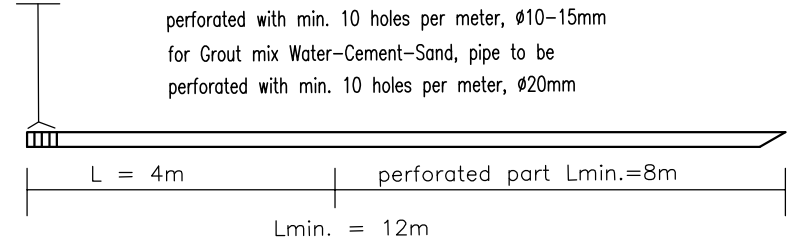


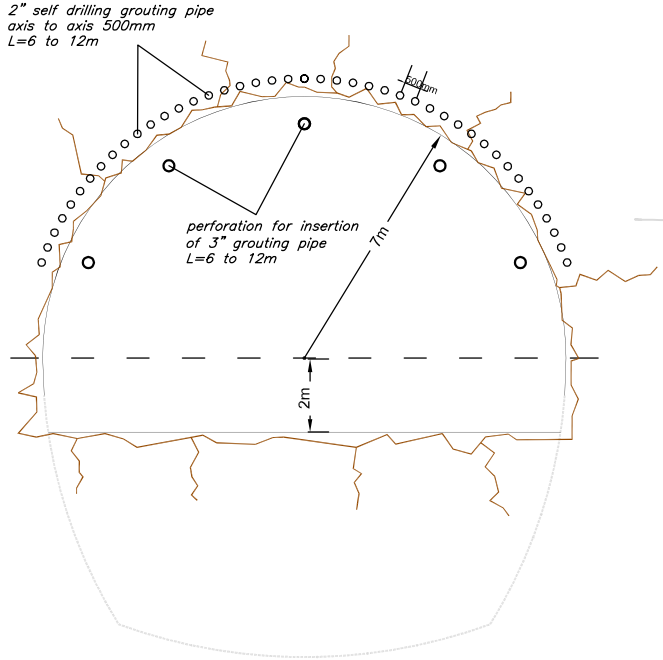
Grouting Pipe

Pipe ø3" Lmin = 12m

for Grout mix Water-Cement, pipe to be perforated with min. 10 holes per meter, ø10-15mm

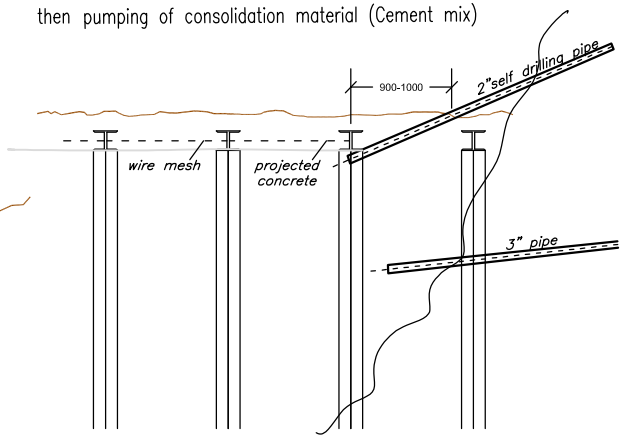
for Grout mix Water-Cement-Sand, pipe to be perforated with min. 10 holes per meter, ø20mm





Cement mix
Water/cement ratio
1:1 to 1:2

Insertion of pipes into the collapsed rock debris then pumping of consolidation material (Cement mix)



Gilgel Gibe III Hydroelectric Power Project

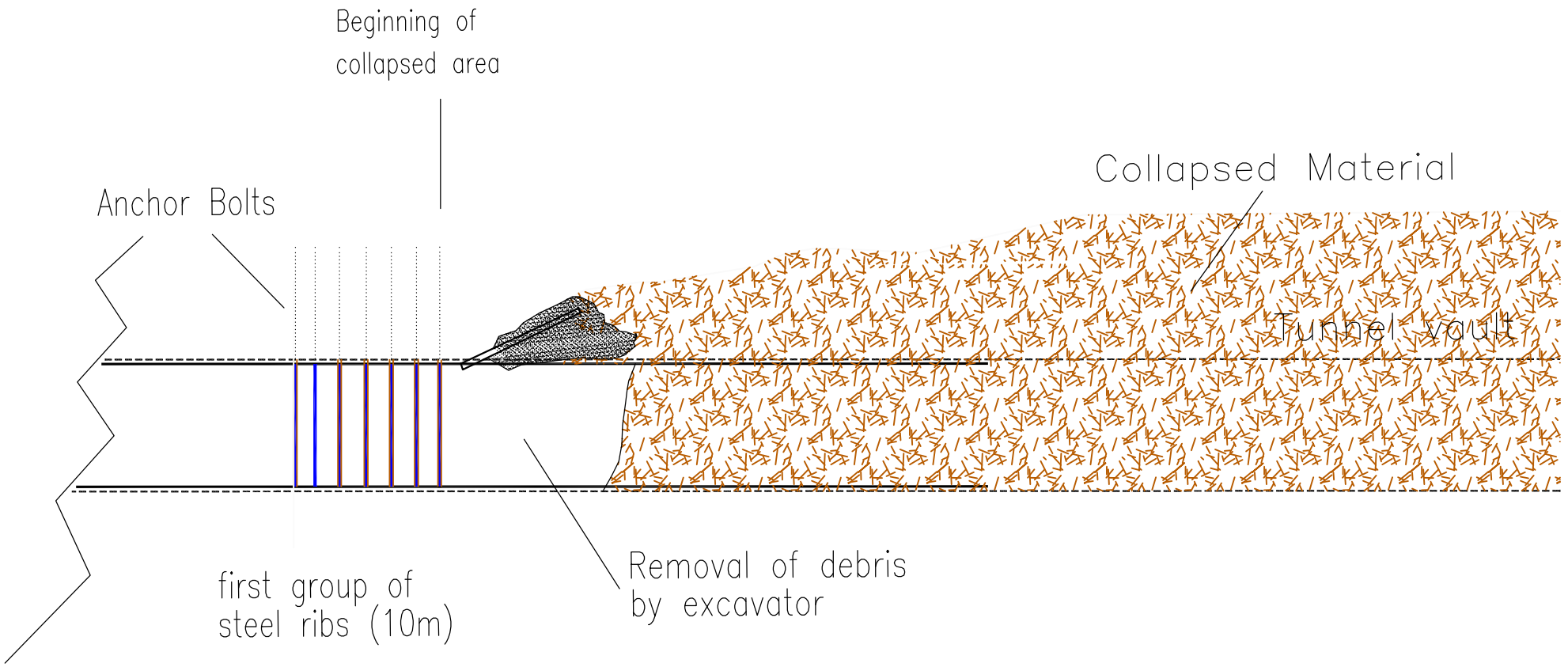
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS

Injection of pipes and pumping of consolidation material

Size A4 | Scale | Rev.

METHOD STATEMENT n.03 | Attachment 4 | 2

310	TUN	M	SAS	134	Rev. 2	13-Jun-2007



Gilgel Gibe III Hydroelectric Power Project

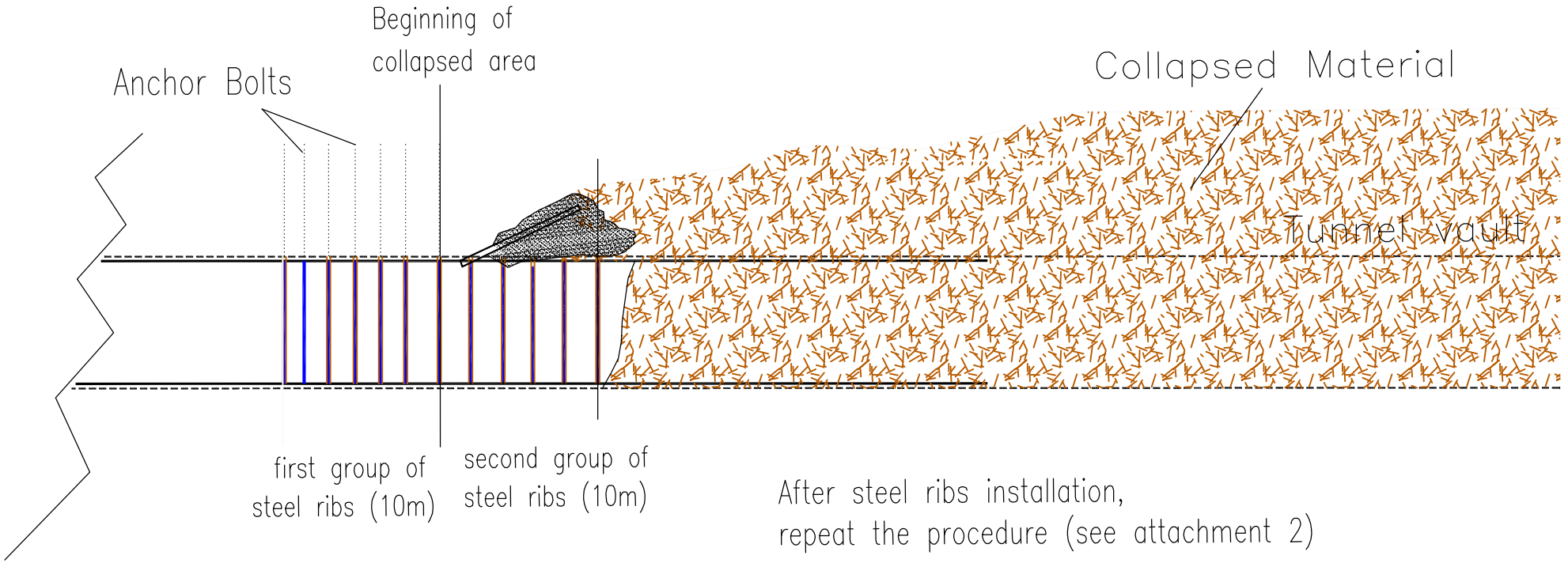
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS

Removal of debris from the tunnel collapsed area

Size A4 | Scale | Rev.

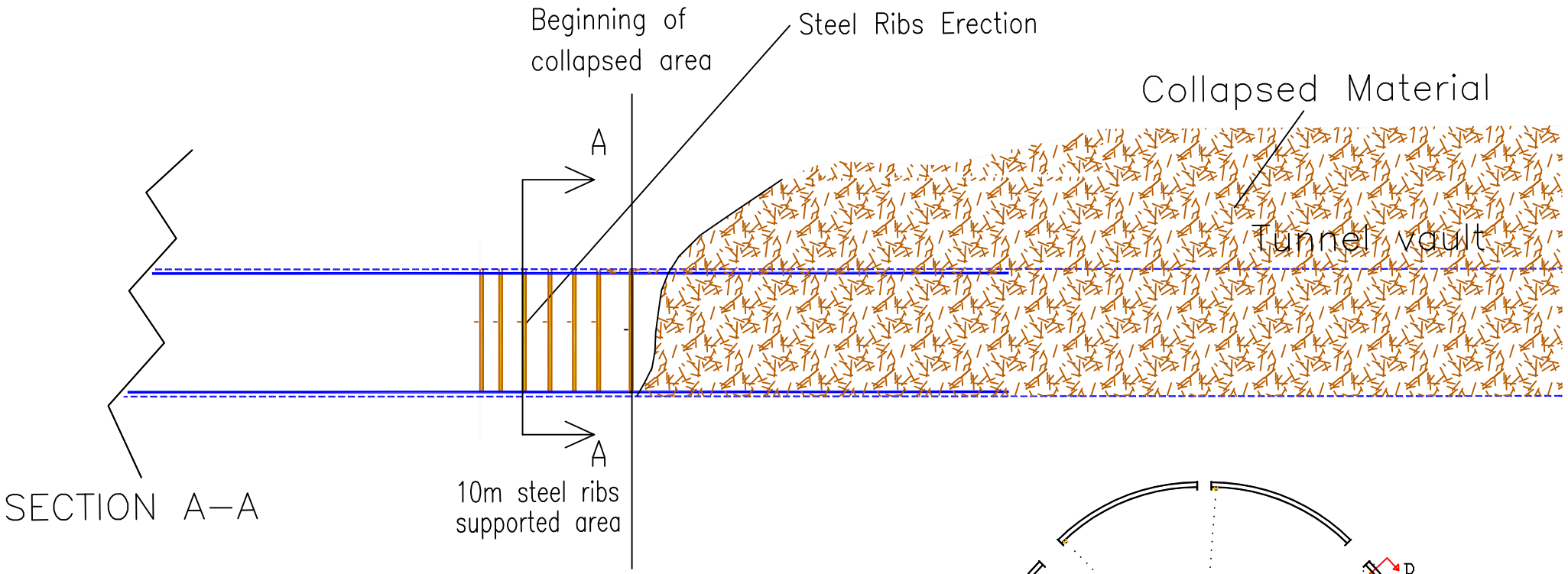
METHOD STATEMENT n.03 | Attachment 5 | 0

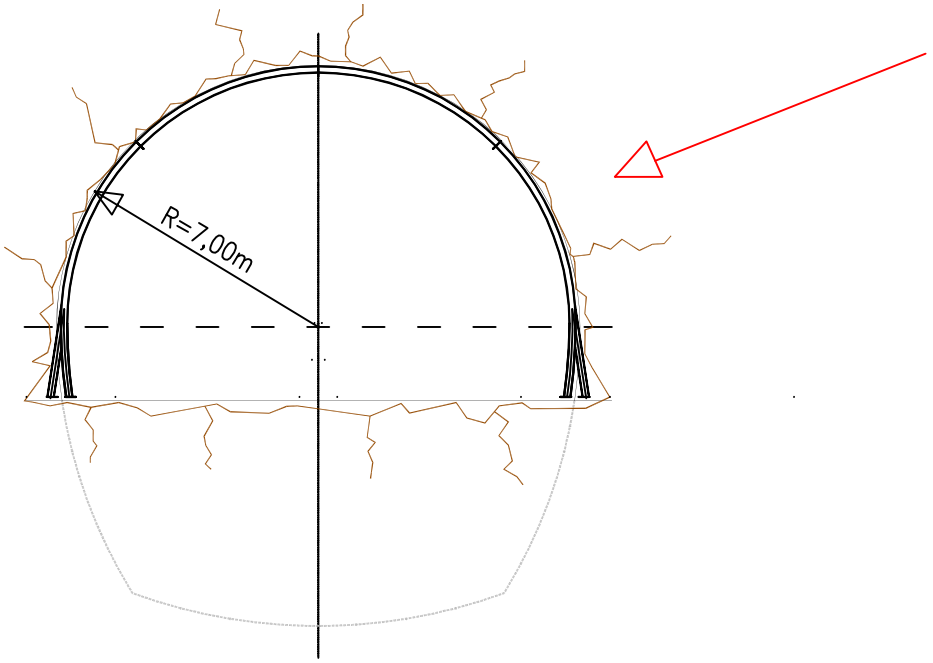
310	TUN	M	SAS	135	Rev. 0	15-May-2007

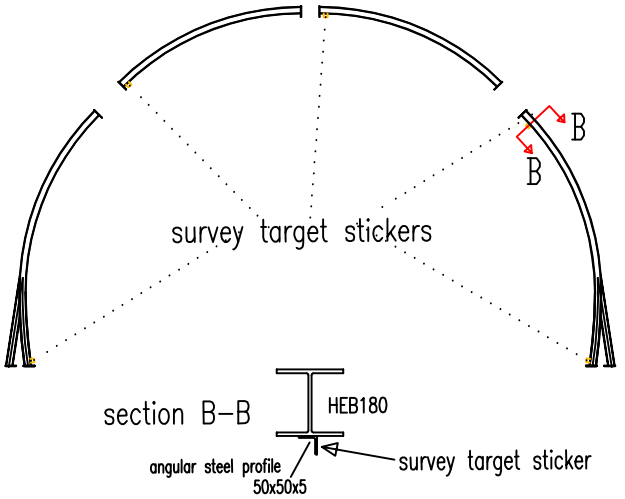


After steel ribs installation,
repeat the procedure (see attachment 2)

Gilgel Gibe III Hydroelectric Power Project							
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS							
Additional support works						Size A4	Scale
New group of steel ribs erection							Rev.
METHOD STATEMENT n.03				Attachment 6			0
310	TUN	M	SAS	136	Rev. 0	15-May-2007	



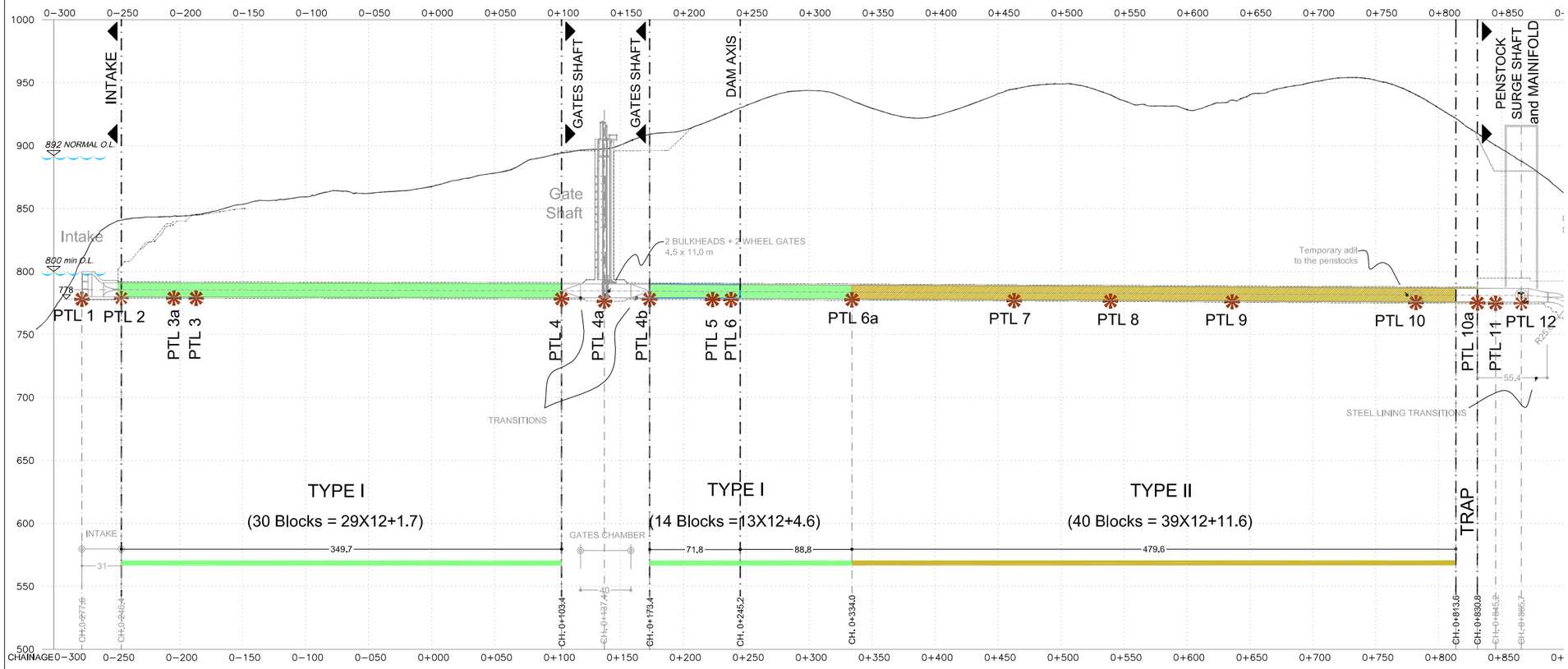


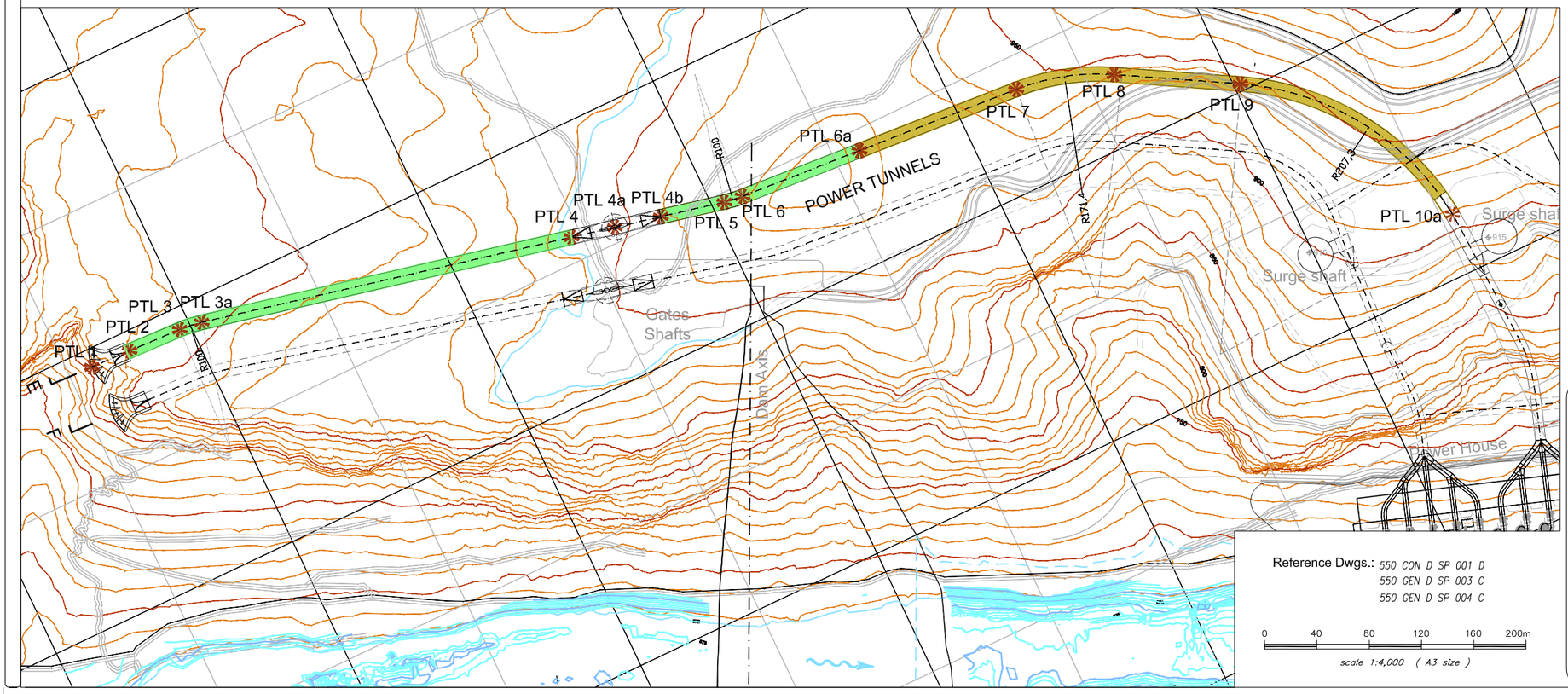


Gilgel Gibe III Hydroelectric Power Project						
DIVERSION TUNNEL'S EXCAVATION REMEDIAL WORKS						
Steel ribs with additional legs support					Size A4	Scale / Rev.
METHOD STATEMENT n.03	Attachment 7					0
310	TUN	M	SAS	137	Rev. 0	30–May–2007

Appendix H: Attachments of Drawings for permanent lining, Consolidation grouting, etc.

PROFILE - sect. **E**





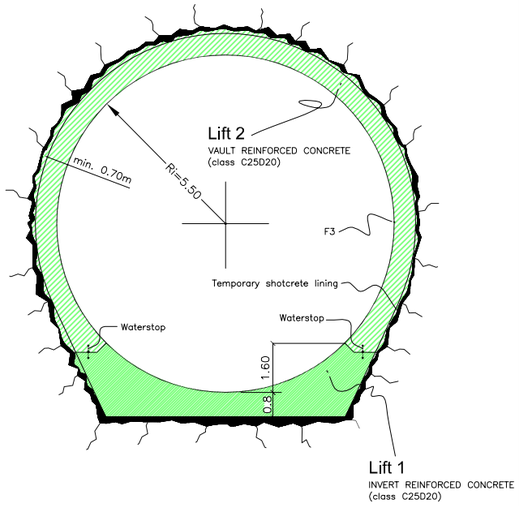
Reference Dwgs.: 550 CON D SP 001 D
550 GEN D SP 003 C
550 GEN D SP 004 C

0 40 80 120 160 200m

scale 1:4,000 (A3 size)

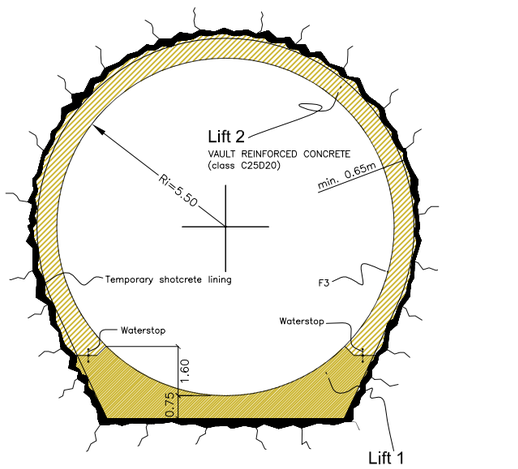
TYPE I

scale 1:500 (A3 size)



TYPE II

scale 1:500 (A3 size)



MATERIAL PROPERTIES

CONCRETE

Levelling concrete

– Class	C20D32
– Cylindric characteristic strength	20 MPa
– Max. size of aggregates	32 mm

Structural concrete

– Class	C25D20
– Cylindric characteristic strength	25 MPa
– Max. size of aggregates	20 mm

CONCRETE SURFACES FINISHING (ACCORDING TO T.S.10.9)

WITH FORMWORK:
- F1 for concrete surface concealed to view
- F2 for concrete surface exposed to view
- F3 for concrete surface in contact with flowing water

WITHOUT FORMWORK:
- U1 for concrete surface to be covered with other concrete
- U2 for concrete surface not permanently concealed
- U3 for concrete surface in contact with flowing water as wall inside floor

STEEL FOR REINFORCED CONCRETE

– Class	ASTM A 615 GR 60
– Yeld strength	fyk = 420 Mpa

First Issue

issue	day	month	year	drawn	checked	approved
A	27	Feb.	2012	E.B.	E.B.	S.S.

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION

project: GIBE III Hydroelectric Project

contractor: salini COSTRUTTORI S.p.A.

designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers

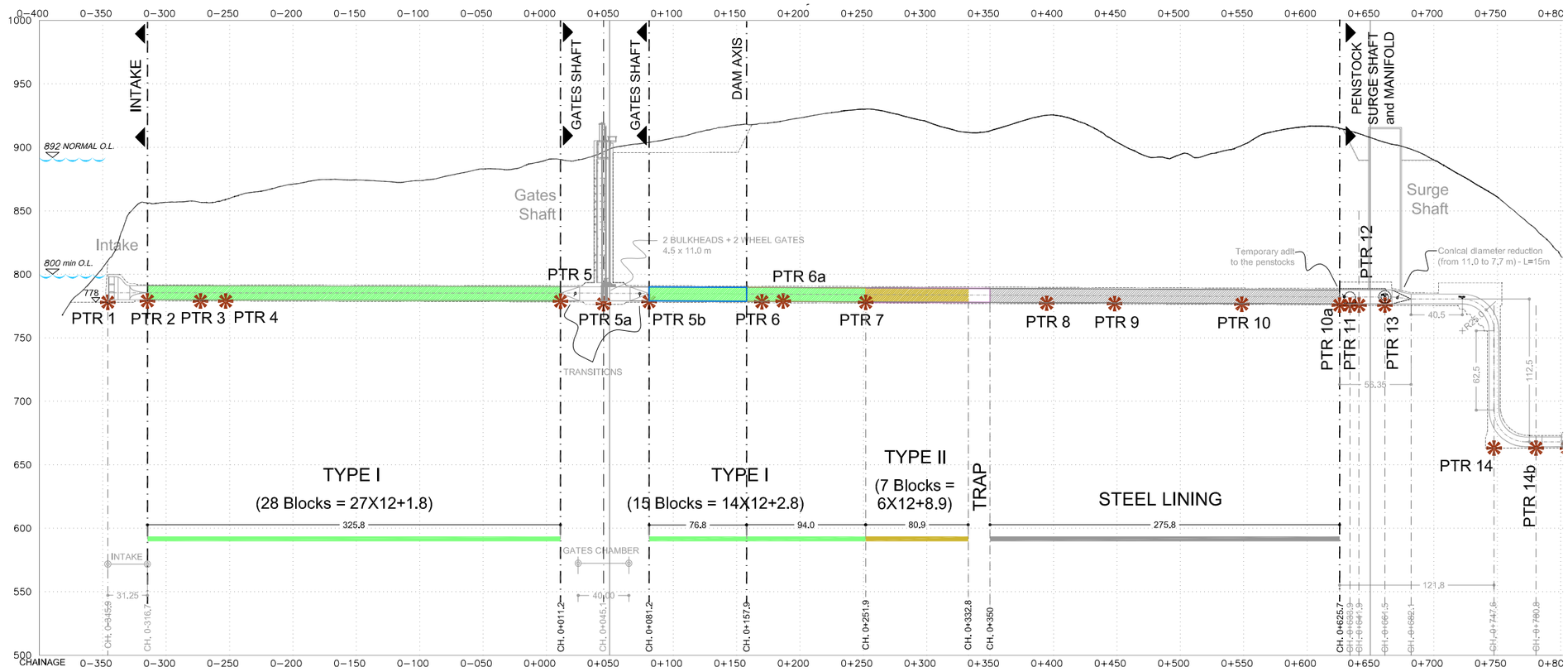
design: Level 2 DESIGN

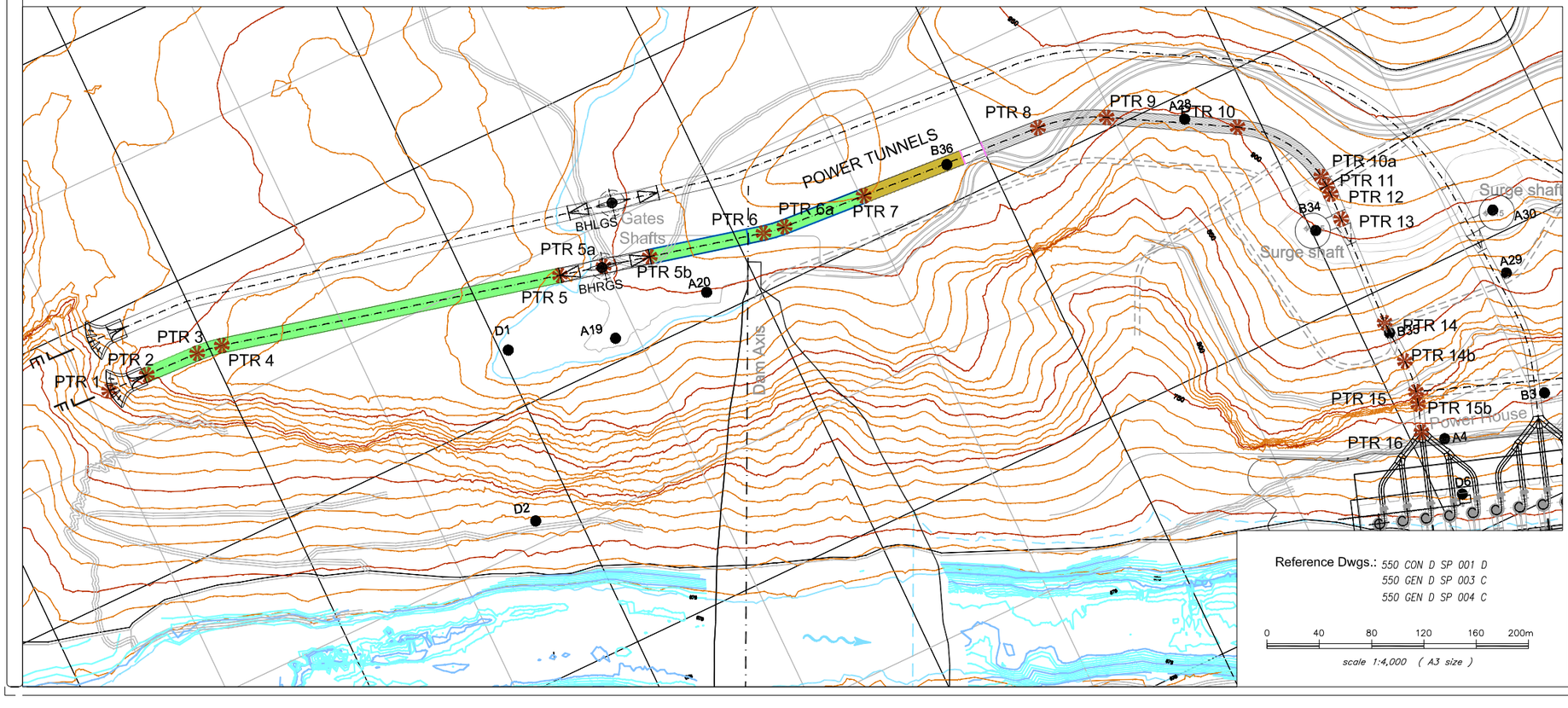
title:

POWER TUNNELS
Left Power Tunnel
Permanent Lining
Plan, Profile and Sections

section	s.sect.	work	office	drawing nr.	rev	date
550	CON	D	SAS	003	A	27 Feb 2012

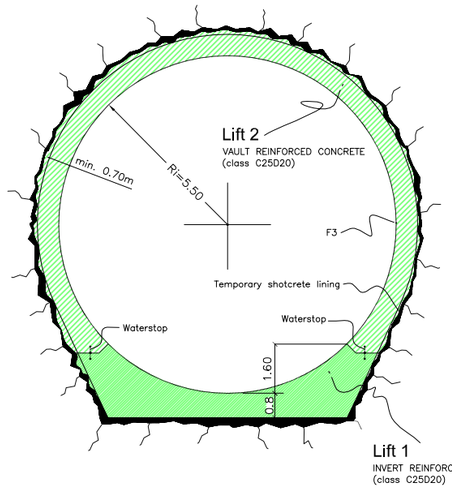
PROFILE - sect. **F**





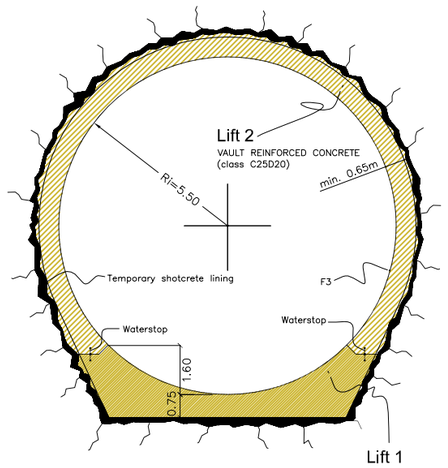
TYPE I

scale 1:200 (A3 size)



TYPE II

scale 1:200 (A3 size)



Reference Dwgs.: *550 CON D SP 001 D*
550 GEN D SP 003 C
550 GEN D SP 004 C

0 40 80 120 160 200m

scale 1:4,000 (A3 size)

MATERIAL PROPERTIES

CONCRETE

Levelling concrete

– Class	C20D32
– Cylindric characteristic strength	20 MPa
– Max. size of aggregates	32 mm

Structural concrete

– Class	C25D20
– Cylindric characteristic strength	25 MPa
– Max. size of aggregates	20 mm

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STEEL FOR REINFORCED CONCRETE

– Class	ASTM A 615 GR 60
– Yeld strength	fyk = 420 Mpa

First Issue

issue	day	month	year	drawn	checked	approved
A	27	Feb.	2012	E.B.	E.B.	S.S.

modifications

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION

project: GIBE III Hydroelectric Project

contractor: **salini** COSTRUTTORI S.p.A.

designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers

design: Level 2 DESIGN

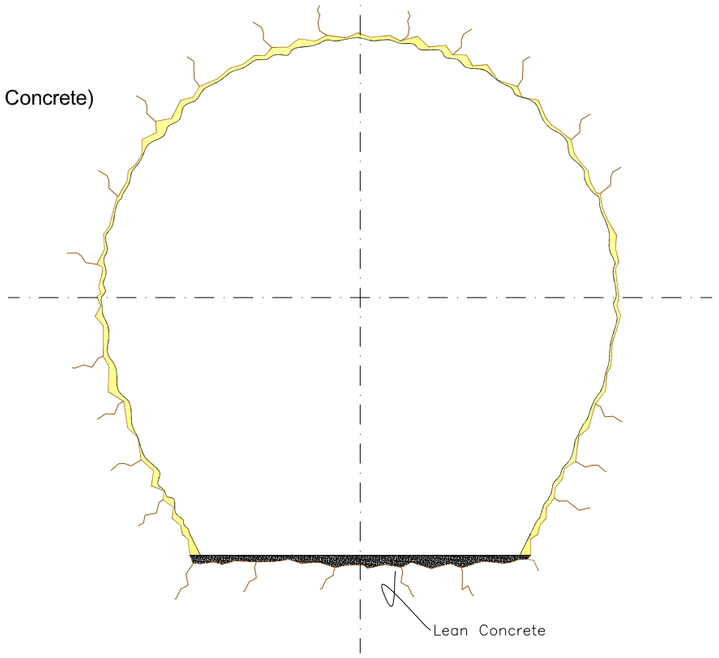
title:

POWER TUNNELS
Right Power Tunnel
Permanent Lining
Plan, Profile and Sections

section	s.sect.	work	office	drawing nr.	rev	date
550	CON	D	SAS	004	A	27 Feb 2012

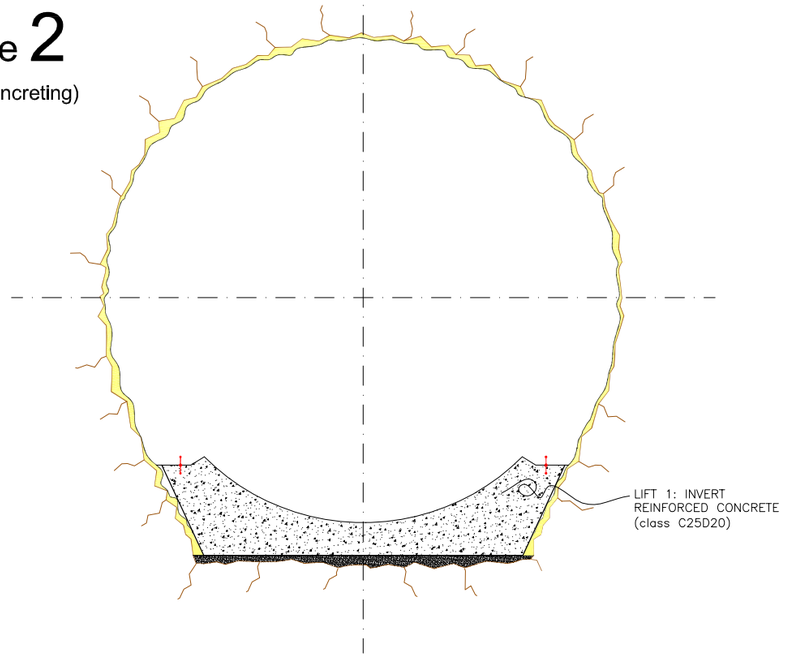
Phase 1

(Cleaning and Lean Concrete)



Phase 2

(Invert Concreting)

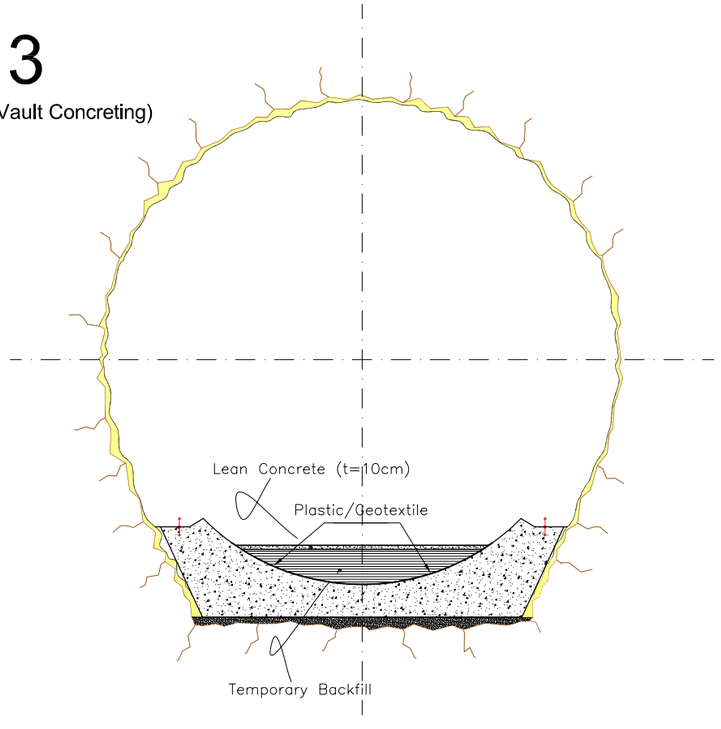


Reference Dwgs.:

550 CON D SP 001 D
550 GEN D SP 003 C
550 GEN D SP 004 C

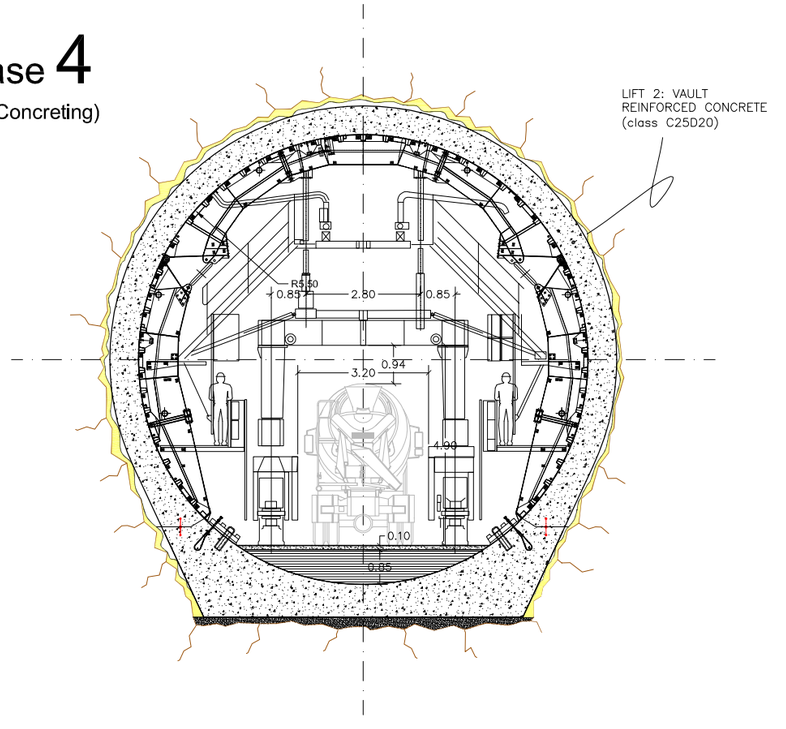
Phase 3

(Invert Preparation for Vault Concreting)



Phase 4

(Vault Concreting)



MATERIAL PROPERTIES

CONCRETE	
Levelling concrete	
– Class	C20D32
– Cylindric characteristic strength	20 MPa
– Max. size of aggregates	32 mm
Structural concrete	
– Class	C25D20
– Cylindric characteristic strength	25 MPa
– Max. size of aggregates	20 mm

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WITHOUT FORMWORK:
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- U2 for concrete surface not permanently concealed
- U3 for concrete surface in contact with flowing water as wall inside floor

STEEL FOR REINFORCED CONCRETE	
– Class	ASTM A 615 GR 60
– Yeld strength	fyk = 420 Mpa

0 1.5 3.0 4.5 6.0 7.5m

scale 1:150 (A3 size)

First Issue

issue	day	month	year	drawn	checked	approved
A	27	Feb.	2012	E.B.	E.B.	S.S.

modifications

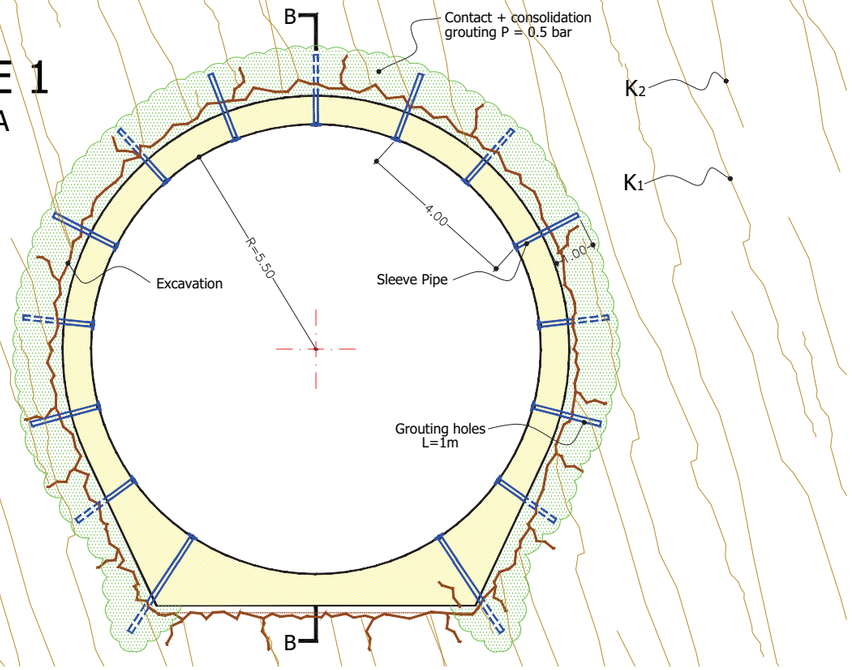
beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: POWER TUNNELS
PTR & PTL
Construction Phases
Typical Sections

section	s.sect.	work	office	drawing nr.	rev	date
550	CON	D	SAS	005	A	27 Feb 2012

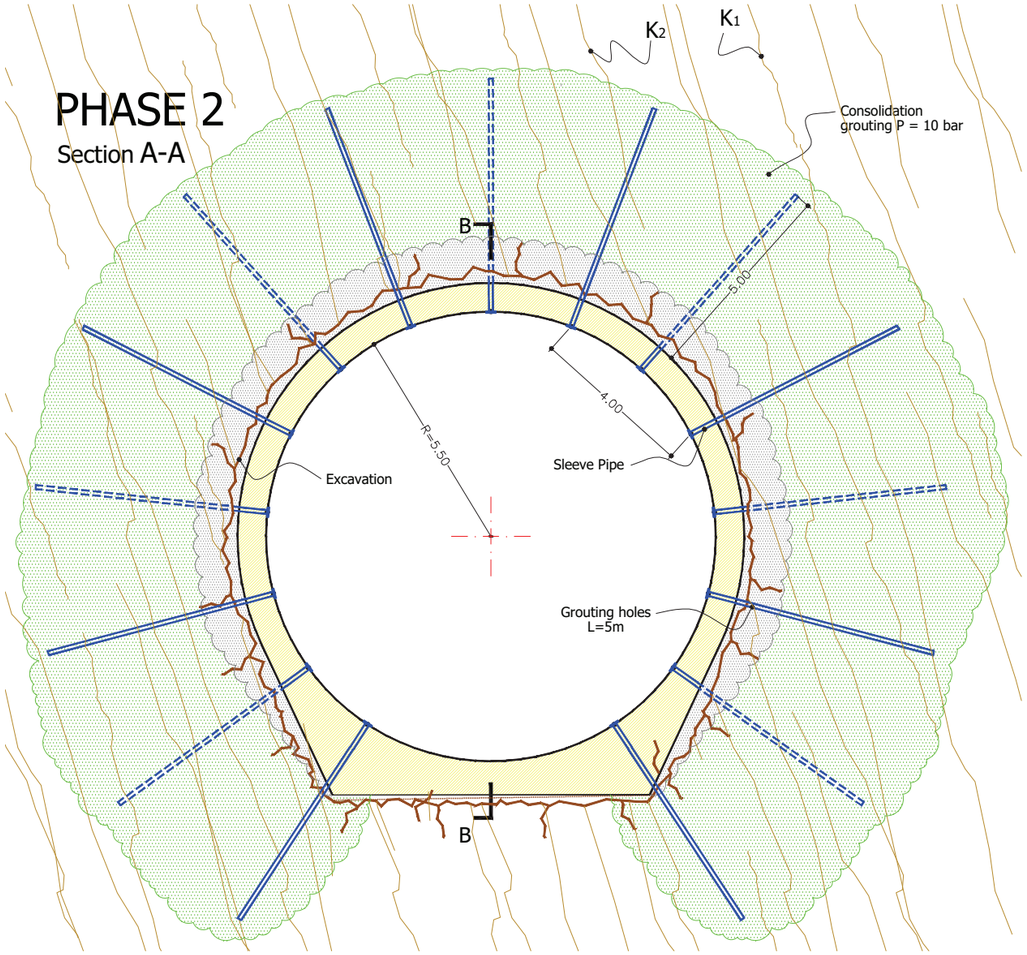
TYPICAL SECTION - PTL 0-246.4 ÷ 0+830.8, PTR 0-316.7 ÷ 0+350.0

Request of dwg:
G3 MoM ER-SAL-SP 06 DEC 2011
ER letter 120120B

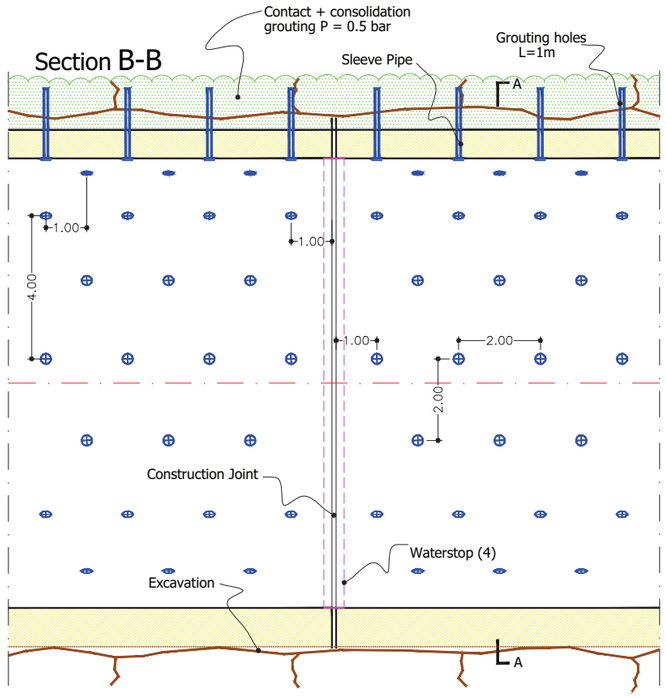
PHASE 1
Section A-A



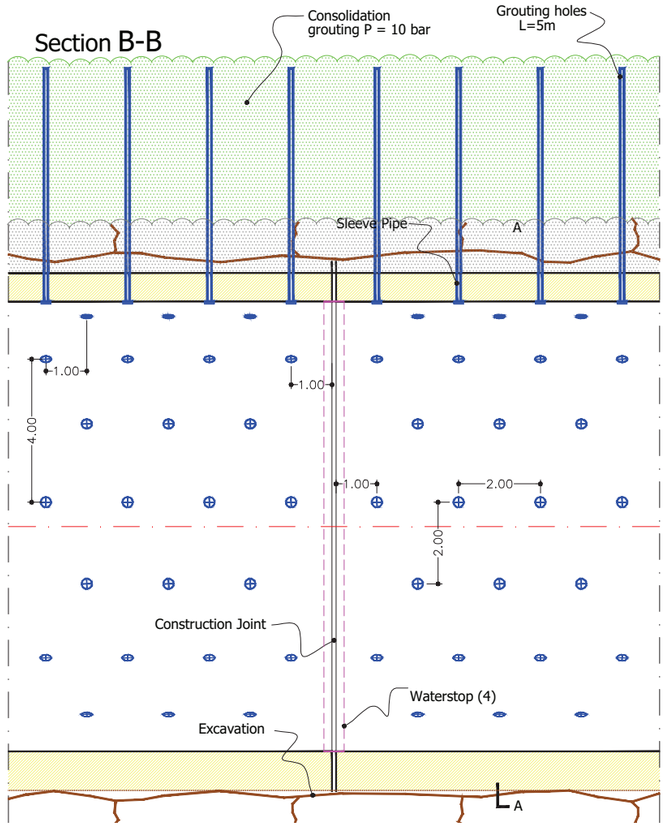
PHASE 2
Section A-A



Section B-B



Section B-B



NOTES:

(1) - Consolidation grouting shall be done systematically all along the tunnel after the placing of lining, in two phases:

- PHASE 1
  - L [m] 1 holes length
  - p [bar] 0.5 grout pressure
  - s [m] 4x1 holes spacing
- PHASE 2
  - L [m] 5 holes length
  - p [bar] 10 grout pressure
  - s [m] 4x1 holes spacing

(2) - The grout shall have the following characteristics:

- W/C [-] 0.7 water/cementitous material
- R [%] 0.5 additive (superplasticizer)

These values can be adjusted during the execution of the works according to the local rock mass conditions, to be evaluated on site.

(3) - Sleeve pipe to be inserted inside lining to avoid perforation of the reinforced concrete at the moment of the grout works.

(4) - Sea details in reference drawing 4.

ref.	title code
4	550 CON D SP 001
3	550 REB D SP 040,041
2	550 RMK D SP 101,102
1	550 EXC D SP 101-104

reference drawings

issue	day	month	year	drawn	checked	approved
Notes 4 and 5 deleted, title changed. Drain removed, one grouting row added	E	23	01	2012	GIB	ROS	ACG
Note 5 added	D	15	12	2011	NEM	ROS	ACG
Consolidation grouting layout updated.	C	08	08	2011	EDS	ROS	ACG
Typical section with membrane added	B	26	04	2010	SCOL	FAV	ACG
First issue	A	08	04	2010	DAL	FAV	ACG

modifications

scale 1:150 (A3 size)

beneficiary: EEPCO ETHIOPIAN ELECTRIC POWER CORPORATION
project: GIBE III Hydroelectric Project
contractor: salini COSTRUTTORI S.p.A.
designer: SP studio ing. g. pietrangeli s.r.l. consulting engineers
design: Level 2 DESIGN
title: 550 = POWER TUNNELS
CON = CONCRETE
CONSOLIDATION GROUTING
CONCRETE TYPICAL SECTION

section	kind works	type	ROMA office	serial n°	rev.	date
550	CON	D	SP	003	E	23 Jan 2012

Z:\GIBE III\500 - LEVEL 2 DESIGN\550 - POWER TUNNELS\04 CON\DWG\550 CON D SP 003 E - Consolidation Grouting - 23Jan2012.dwg gib, sp - Roma 24/JAN/2012 16:21:18

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