Stability Analysis of Vertical Shaft and associated Design of Support system

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Abstract—Unlined/Shotcrete lined pressure tunnel and shafts in hydropower project is economically attractive project. In Norway, the concepts and design principles behind these conduits were successfully implemented in planning design, construction and operation. In an unlined tunnel, the confining pressure from the rock mass should be able to counteract the water pressure for the safety of unlined tunnel against hydraulic jacking. Water leakage potential from the tunnel can also determine based on this principal. The required length of steel lining are determined based on minimum in situ principal stress and Maximum dynamic Hydrostatic pressure along the alignment of the tunnel.

Geological Data of Dudhkoshi Storage Hydroelectric Project are used for the study of this paper. The Norwegian design concepts and criteria were applied. This criteria shows Vertical Shaft is safe with Norwegian Design Criteria but not to state of art design criteria. To design engineering structures in rock mass, requires knowledge of the prevailing in-situ stress are most essential also requires to know geology and topographic features. Semi empirical, analytical and numerical method were conducted during the research works. From Semi Empirical and Analytical Method, deformation and plastic zone radius around excavation were measured and selected the proper support system to minimize the plastic failure of Rock mass. The result obtained are verified from Numerical Analysis Method using Phase2 software

Keywords— Hydraulic jacking, Vertical Shaft, Shotcrete lining, Unlined, Steel lining, Leakage, Norwegian Design Criteria

I. INTRODUCTION

A tunnel is an underground passageway and may be for foot, for road traffic, or for a conveyance hydraulic fluid with or without applying pressure. Due to requirement and existing topographic condition a tunnel made for different ways like horizontal, inclined or vertical direction. When tunneling works to be done vertical or near vertical than it called shaft (Bhawani Singh et al., 2006). The most commonly used shaft design in modern day excavation is actually of circular or elliptical shape which in itself is self-supporting and accommodating if a concrete lining is being utilized. In unfavorable ground conditions, the construction of circular shafts is required. The circular shafts avoid stress concentrations in corners and benefit from arching action in the supported material. Similarly, in comparison to the elliptical shape it is construction easy. The depth of shaft varies with its purpose. Some shafts are very deep (H>1000 m) with depth, the excavation and the supporting problem also increases [8].

Unlined pressure Tunnel/Shaft in Hydropower schemes are becoming popular worldwide due to cost effectiveness compared with lined with concrete or steel pipes. (Basnet and Panthi,2018). Unlined tunnels are relatively easy for construction and take lesser timer for construction in favorable condition of topography, geology and geotechnics. The unlined of Norway is the valuable example in the world than one constructed in other countries. The earliest attempts to use unlined pressure Tunnel in Hydropower projects with surface powerhouse in Norway was already in 1920s, which is already 100 years back from now. Today Norway has over 230 underground Powerhouse and over 4300 Km unlined tunnels and shafts. Experience gained in design construction and operation of such waterway systems has led to the development of different design criteria for unlined tunnels (Broch, 1982).The principal behind the idea of unlined pressure tunnel concepts is that rock mass itself works as a natural concrete against the pressure exerted by water column in the tunnel (Selmer-Olsen, 1969)

II. GEOLOGY OF THE STUDY AREA

The study area is considered of Dudhkoshi Storage Hydroelectric Project, is located in Dudhkoshi river between the boundaries of Okhaldhunga and Khotang Districts in Eastern Development Region of Nepal. The Dam site is located in a gorge nearly one kilometre downstream of the confluence between Dudhkoshi River and Thotne River. The Dudhkoshi Dam site, Powerhouse and headrace tunnel are in the lesser Himalaya zone. Stratigraphically, the lesser Himalaya is divided into three subgroups: Upper, Middle and Lower. The majority of the rocks in the project area are characterized by a low metamorphic grade, such as quartzite, phyllite, mica schist, limestone, gneiss (both schistose and granite).The project has two powerhouse, one is near Sunkoshi River and another is at dam toe on the right bank. The study area of this paper is of dam toe power house lies near Dam site of the project. The main lithotypes are phyllites, quartzites and Phyllite/Quatzite . The length of Headrace tunnel of toe powerhouse is 553 m having diameter of 6.4 m, Penstock of length 110 m having diameter of 4.5 m and Surge Shaft of 128 meter length having diameter of 15 meter

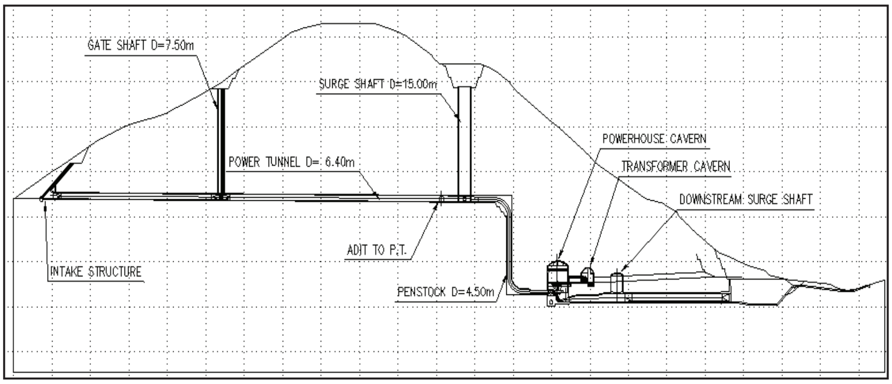


Figure 1 Longitudinal Section of Pressure Tunnel, shafts, Powerhouse and Tailrace tunnel of Toe option of Dudhkoshi storage Hydroelectric Project.

III. BRIEF HISTORY OF UNINED PRESSURE TUNNEL/SHAFTS

Norway started to implement unlined pressure shaft and tunnel concept for hydropower projects in 1920s (Basnet and Panthi 2018). Initially four projects were implemented, three out of four hydropower schemes with unlined pressure shafts were operating perfectly after initial problem were fixed, it took almost 40 years to beat the world record of static water head of 152 m with unlined high pressure shafts of Svelgen hydropower project. The Tarfjord K3 hydropower project with static head of 286 m was the one to beat this record, which was successfully put into operation in 1958 (KK panthi, CB basnet 2018). After the construction of this project the hydropower industry in Norway achieved confidence in the application of unlined pressure shafts and tunnels. This design approach limits the steel lined part of high pressure shafts only near the powerhouse (mostly not exceeding 75 m).This too is to make sure that there is no leakage path reached from unlined pressure shaft to the underground powerhouse cavern. More than 100 km of the tunnels were excavated every year. The experience gained through underground excavation for hydropower schemes made it possible to develop advanced tunneling technology in both excavation and support philosophy. Innovative ways of thinking and their realization in real design and construction followed over the years. One such Norwegian innovation is use of unlined high pressure tunnels and shafts as waterway systems, and another is the development of the so called air cushion surge chamber that has replaced in certain topographically difficult hydropower schemes

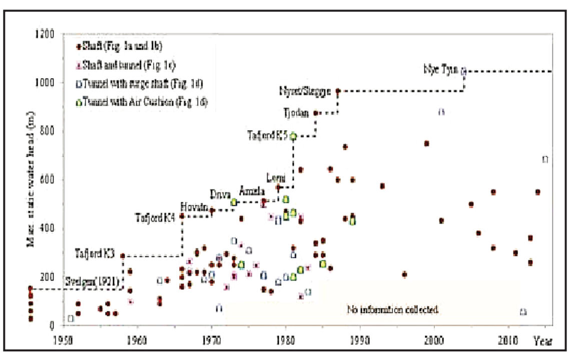


Figure 2 Development of unlined high pressure shafts and tunnels in Norway since 1920

IV. NORWEGIAN DESIGN CRITERIA

The design criteria developed is based on the principal that both vertical and lateral rock cover should be sufficient to confine the pressure given by static water head at the tunnel.

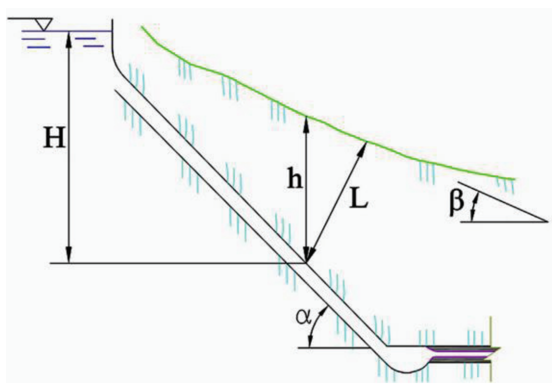


Figure 3 Definition of Minimum Rock Cover in Empirical Design Criterion of Bergh- Cristensen and Dannevig [1]

In 1970 Selmer-Olsen presented revised rule in which the inclination of the tunnel with respect to the slope was indicated.

$$h > \frac{\gamma_W . H * FS}{\gamma_R . \cos\alpha} \qquad (1)$$

Another empirical criterion, proposed by Bergh-Christensen and Dannevig in 1971 (Broch 1984), at each point along the pressure-tunnel alignment minimum rock cover,

$$L > \frac{H * \gamma_W * FS}{\gamma_r * \cos\beta} \qquad (2)$$

Where, h is vertical overburden, L is the shortest distance between the surface and the point of study (m), γ_W is the density of water taking value 9.81 KN/m3, γ_R is the density of rock mass taking value 26 KN/m3, α is the inclination of shaft, H is the static water head at the point of the tunnel under consideration, FS is the factor of safety

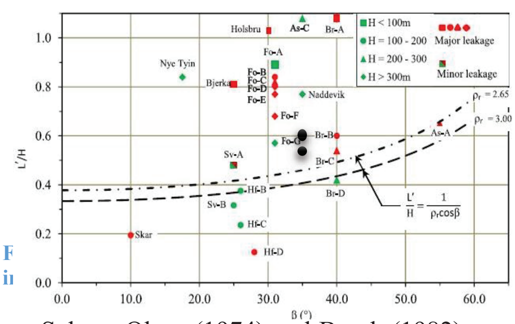
The above equation 2 is no longer valid if the shaft has an inclination more than 600. In such situations, the shaft should be placed inside the line representing minimum depth for 45° shaft (11).

TABLE 1 CALCULATION OF REQUIRED LATERAL AND VERTICAL ROCK COVER ACCORDING TO NORWEGIAN DESIGN CRITERIA AT DIFFERENT LOCATION OF SHAFTS

	Location				
	Vertical Shaft			Surge Shaft	
	Bottom	***Center***	***Top***	***Bottom***	***Center***
Available Vertical Rock Cover, m	200	150	100	130	75
Lateral Rock Cover, m	164.02	121.4	82.00	108.16	61.03
According to Norwegian Design Criteria					
Required Vertical Rock cover (H), m	122.19	95.35	68.51	68.51	34.25
Required lateral Rock cover (m), m	105.47	82.31	59.14	59.14	29.57
Factor of Safety					
Criteria 1 (Equation 1)	1.64	1.57	1.45	1.90	2.19
Criteria 2 (Equation 2)	1.55	1.47	1.39	1.83	2.06

From the above table find that, the available Lateral and Vertical rock cover is satisfied to Norwegian design criteria for the construction of unlined Tunnel.

To evaluate the reliability of the Norwegian rule many cases have been presented in literature (Broch, 1982,) Fig. below shows a selection of existing unlined pressure shafts/tunnels, where major leakages or damages occurred or not. In the figure the lower curve represents the limit defined by the Norwegian rule assuming an average unit weight of 26.5 KN/m^3 for the rock-mass.



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Selmer-Olsen (1974) and Broch (1982) among the others, that the unlined tunnel is safe against hydraulic splitting or jacking only if the minor principal stress is greater than static water pressure in the tunnel [1]. Mathematically, this criterion can be expressed as follows:

$$\sigma_3 > h_w \ * \gamma_w \qquad (3)$$

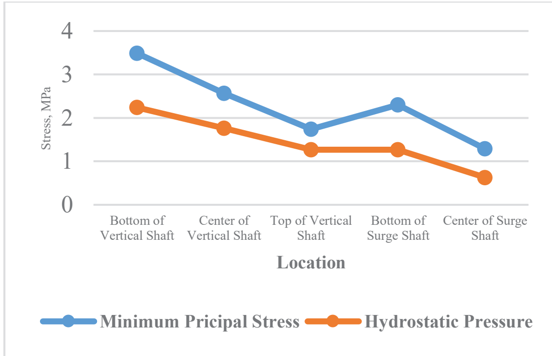


Figure 5 Chart of minimum principal stress and static hydrostatic pressure at different location of shafts.

V. ASSESSMENT OF THE STRESS STATE IN THE RANDOMLY ORIENTED ROCK JOINT INCLINED IN DIFFERENT ANGLE

It is important to consider that the minimum principal stresses are not relevant unless hydro fracturing occurs. In more probable case of hydro jacking, the stresses normal to the various discontinuities are relevant. Joint inclination higher than natural slope is more favorable than having a joint lesser than the natural slope from the potential of sliding and hydro jacking which cause the slope instability [4]. According to the infinite slope theory, the normal stress acting on a joint as a function of its orientation angle α is shown in figure and can be expressed as in equation

$$\sigma_\alpha = (\frac{1+K}{2} - R.cos(2\omega - 2\alpha)) * \gamma_R * h$$

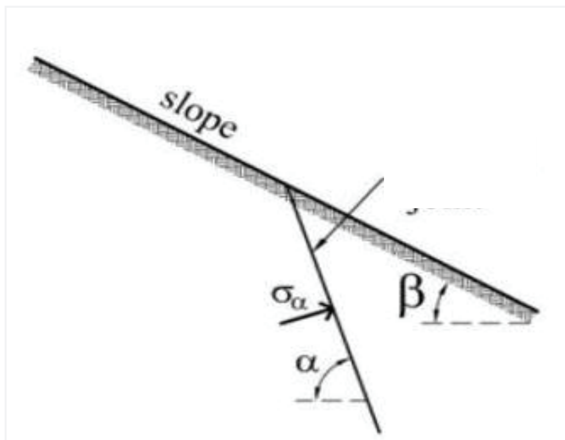


Figure 6 Figure shows the natural slope of β° and joint present in rock inclined at α^0 with horizontal

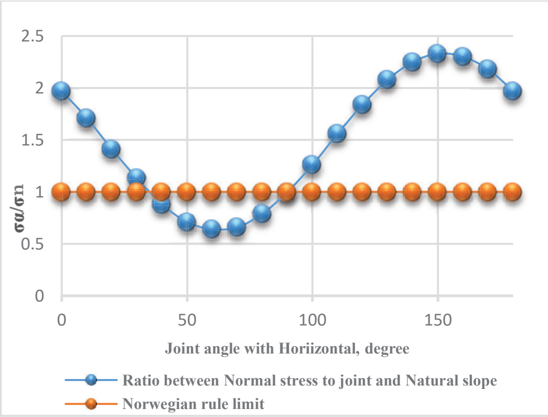


Figure 7 Variation of the stress normal to a randomly oriented joint with respect to the stress σN normal to the slope (β = 35°).

VI. Determination of Steel Lining Length for Pressure Tunnels

The criterion for deciding when a tunnel should be steel lined is when the minimum principal stress in the rock mass falls below the maximum dynamic water pressure in the tunnel. This is a function of the maximum static head of water in the tunnel, the operation of the gates and the characteristics of the turbines. An allowance of 20% over the maximum static head is usually considered adequate for a pressure tunnel associated with the operation of a Pelton wheel since this does not induce large pressure fluctuations. In the case of a Francis turbine, larger pressure fluctuations can be induced and an allowance of 30% above the maximum static head is normally used [9].

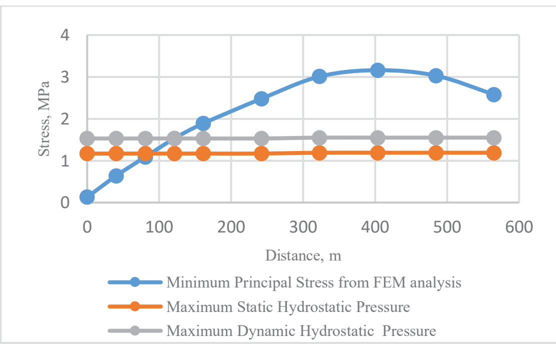


Figure 8 Chart of calculated values of Static and dynamic Hydrostatic pressure and minimum stress analysis along Headrace Tunnel.

From above chart, steel lining is required on the HRT in initial 121.07 m where minimum principal stress is lower than the maximum dynamic hydrostatic pressure. For the Francis turbine, Maximum Dynamic Hydrostatic Pressure is determined with additional 30% of Maximum Static Hydrostatic pressure.

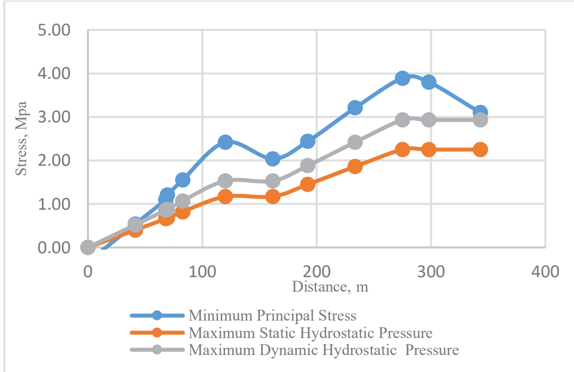


Figure 9 Chart of calculated values of Static and dynamic Hydrostatic pressure and minimum stress analysis along different chain age of Vertical and Surge Shafts.

From above chart, Minimum Principal Stress is less than Dynamic Hydrostatic pressure only on the top of the surge shaft but practically steel lining is not provided in the Surge Shaft.

VII. Analysis Using Semi Empirical Method

Labasse (1949) has considered the design of shaft linings through horizons that are assumed to have zero cohesion (William Hustrulid, 1984). The required dimensions of a shaft lining depend naturally on the forces to which it is subjected. If the ground withstands the elastic stresses developed as a result of sinking then support is unnecessary since the ground will stand alone.

If the ground is relaxed, a lining becomes essential in order to prevent the fall of dislodged rock, to arrest dilatation of the latter, and finally to prevent any deformation of the shaft that cannot be tolerated because of hoisting installations.

The shaft radius (X) after the development of the relaxed zone can be found using

$$X = R[1 - K_o(1 - \frac{a^2}{R^2})^{1/2}$$

The development of the plastic zone can be obtained using following formula:

$$P_i = \sigma_H(1 - sin\emptyset)(\frac{a}{R})^{\alpha}$$

Where, Ø is the friction angle, a is the radius of Shaft, R is the radius of the development of relaxed zone and □ can be calculated using relation $\frac{2sin\emptyset}{1-sin\emptyset}$

Friction angle at the bottom of shaft is 38.610, lining were installed before initiation of the relaxation R=a, the support required of the lining if rock failure is to be prevented should be a maximum. The lining must be capable of resisting

$$P_{i(max)} = (1 - \sin\emptyset) * \sigma_H$$

The allowable external pressure on the shaft $P_i(allowable)$ would be

$$P_i(\text{allowable}) = \sigma\varphi' * \left(\frac{Ri^2 - Ro^2}{2 * Ri^2}\right)$$

Where, Ri is the internal radius of Shaft, Ro is the outer radius of shaft, $\sigma\varphi'$ is the maximum tangential stress, which can safely be taken by the lining.

TABLE 2: RESULT OF CALCULATION FROM LABASSE METHOD

Required Pressure to prevent relaxation around the shaft	2.05 MPa
Factor of Safety	1.5
Compressive strength of Shotcrete	25 MPa
Thickness of Shotcrete	300 mm
The allowable external pressure on the shaft P_i(allowable)	2.37 MPa

Result: The allowable pressure is greater than the required pressure to prevent the relaxation around the shaft. So lining thickness 300mm with M25 concrete is suitable for that particular section

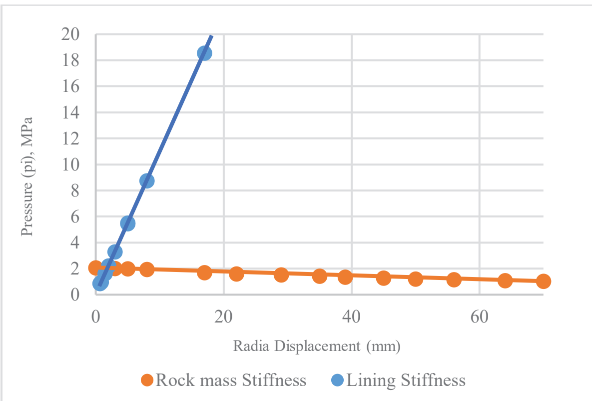


Figure 10 Lining Pressure--Radial Wall Displacement (for Equilibrium) for the vertical shaft section below 229 m from existing ground surface.

VIII. ANALYTICAL ANALYSIS

A. Convergence confinement method (CCM)

Carranza-Torres and Fairhurst (2000) concluded that CCM has three basic components viz. the Longitudinal Displacement Profile (LDP), the Ground Reaction Curve (GRC) and the Support Characteristics Curve (SCC) [10].

1) Ground Reaction Curve (GRC)

GRC is the relationship between decreasing internal pressure pi and increasing radial displacement of tunnel wall ur. The relationship depends upon mechanical properties of rock mass and can be obtained from the elasto-plastic solution of rock deformation around an excavation (Carranza-Torres and Fairhurst, 2000).

2) Support Characteristics Curve (SCC)

Support characteristic Curve is the plot between increasing pressure Ps on the support and increasing radial displacement u_r of the support.

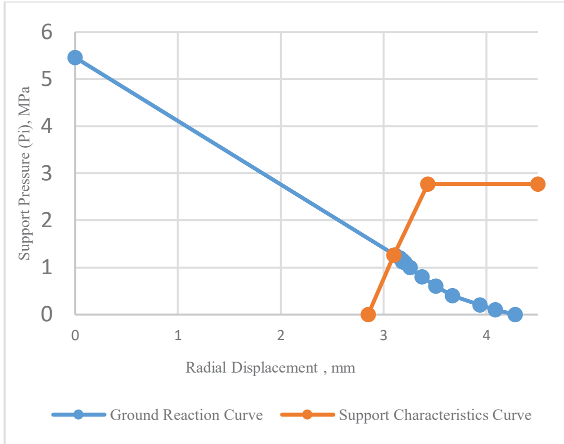


Figure 11 Rock-Support interaction plot.

TABLE 3 INPUT PARAMETERS AND OUTPUT FROM ANALYTICAL METHOD AT THE BOTTOM OF VERTICAL SHAFT.

Input	Output
Tunnel Radius = 2.55 m	Critical Pressure = 1.262 MPa
In situ stress = 5.46 Mpa	Plastic Zone Radius = 3.428 m
Distance from Tunnel Face = 2 m	Max Plastic zone/ tunnel radius = 1.34
UCS = 50 Mpa	Maximum tunnel displacement = 4.26 mm
Rock mass shear Modulus = 2543.78 Mpa	Distance from tunnel face/ tunnel Radius = 0.78

B. Calculation of available support

The maximum support pressure developed by concrete or shotcrete lining van be calculated from the following relationship which is based on the theory of hollow cylinders.

$$(P_{smax}) = 0.5 * \sigma_{conc}\left[1 - \frac{(r_i - t_c)^2}{r_i^2}\right]$$

Available support for Concrete or Shotcrete Linings The stiffness constant Kc is as follows:

$$K_s = \frac{E_s(r_i^2 - (r_i - t_c)^2)}{(1+\vartheta_c)((1-2\vartheta_c)r_i^2 + (r_i - t_c)^2)}$$

Where, σ_{conc} is the compressive strength of shotcrete, r_i is the internal diameter of shaft, t_c is the thickness E_s is the Young's Modulus, ϑ_c is the Poison's ratio of shotcrete lining

TABLE 4 CALCULATION OF SHAFT LINING PROPERTIES AT THE BOTTOM OF VERTICAL SHAFT

Radius of Shaft	2.55 m
Young's Modulus of Lining	30000 MPa
Thickness of Lining	300 mm
Compressive Strength of Lining (Shotcrete)	25 MPa
Maximum support pressure of shotcrete lining	2.77 MPa
Stiffness of Lining	4735.8 MPa/m

IX. Finite Element Method

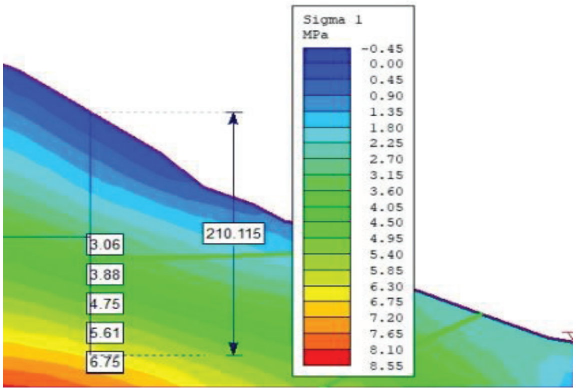
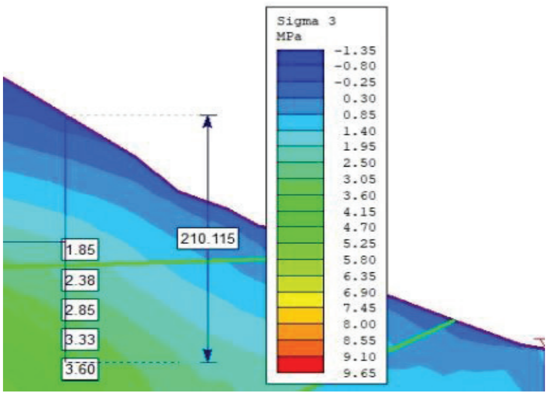


Figure A



FigureB

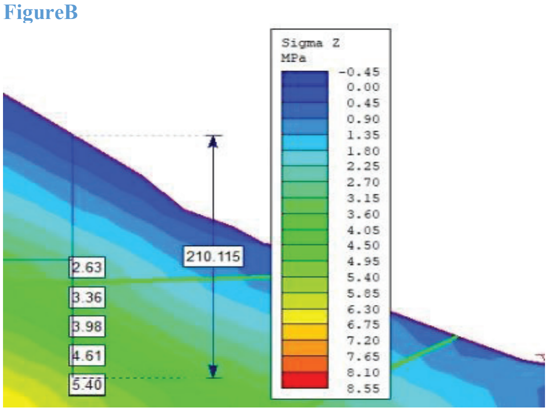


Figure C

Figure 12 Figure A, B and C are the Stress Analysis along the vertical Shaft

From the above figure of stress analysis along the alignment of vertical tunnel, at the bottom of vertical shaft, Sigma 1, Sigma 3, Sigma Z value have been found as 6.75 MPa, 3.6 MPa, 5.4 MPa respectively.

For the stress analysis at the bottom of vertical tunnel, stresses of sigma 1, sigma 3 and Sigma Z value are taken as 5.4 MPa, value 3.6 MPa, value 6.75 respectively.

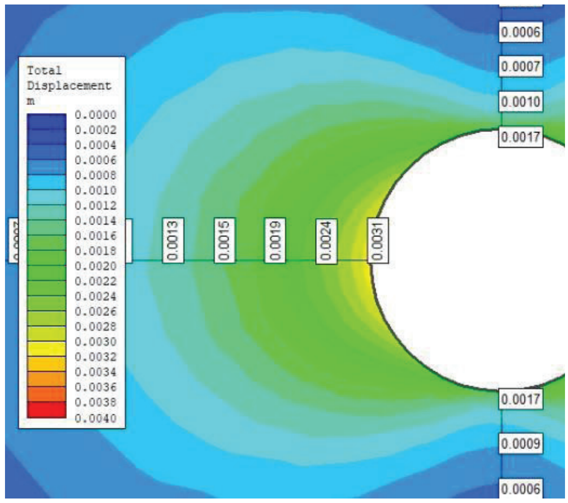


Figure 13 Displacement contour in a bottom of vertical.

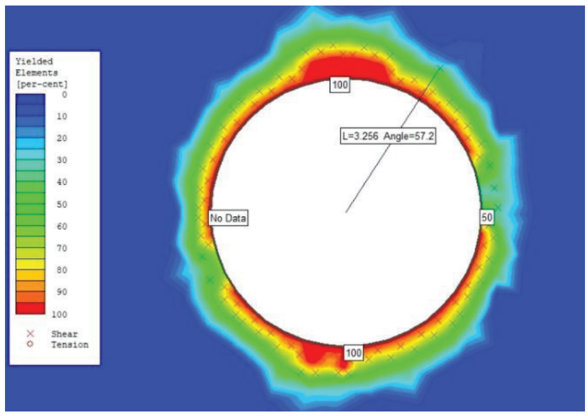


Figure 14 Yielded Element Contour in percentage in bottom of vertical shaft

From above output figure from phase2 Software shows the Plastic zone formed around the excavation, the maximum radius of Plastic Zone is 3.256 m. The displacement is maximum near excavation and reduced along the radial direction.

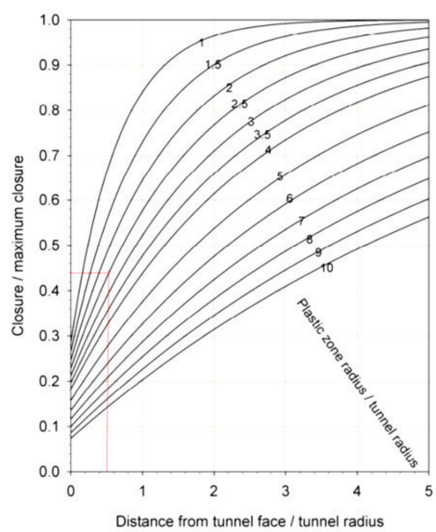


Figure 15 Maximum closure chart (Vlachopoulos and Diederichs, 2009)

From the above chart and figures

Radius of Plastic Zone (Rp) = 3.256 m,

Distance from Tunnel Face (X) = 1.8 m, and

Maximum Displacement (umax) = 3.13 mm.

The Distance from tunnel face/tunnel radius = 0.78.

The Plastic zone radius/tunnel radius = 1.28

Closure at the time of support installation = 0.74*3.16 = 2.3 mm

Practically installation of shotcrete lining at 1.8 m distance from tunnel face is not practically justified for the vertical shaft. The support can install after completion of excavation due to very low Maximum Displacement after excavation at bottom of vertical shaft.

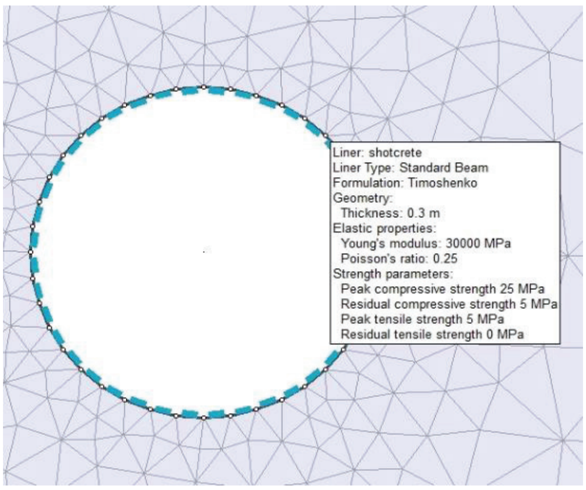


Figure 16 Phase2 Model after lining Installation

The above figure shows the Excavation Model in Phase2 Software after Support installation, Shotcrete lining having thickness 300mm, Poison's ratio 0.25, Compressive Strength 25 MPa is installed in this model.

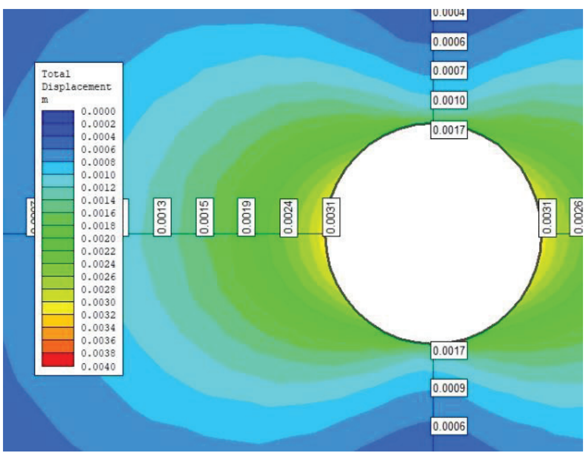


Figure 17 Displacement contour in a bottom of vertical shaft after Lining installation.

From the displacement contour, displacement at the face of excavation is reduced to 2.12 mm from 3.13 mm after support installation.

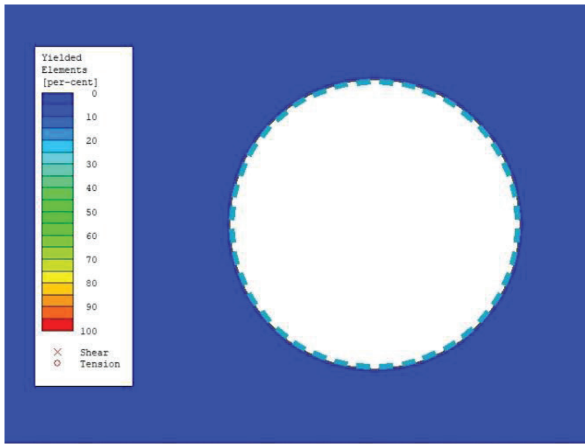


Figure 18 Yielded Zone Contour at the bottom of vertical shaft after installation of Support where no yield Element.

This figure shows that there is not any yielded zone around exaction after support installation.

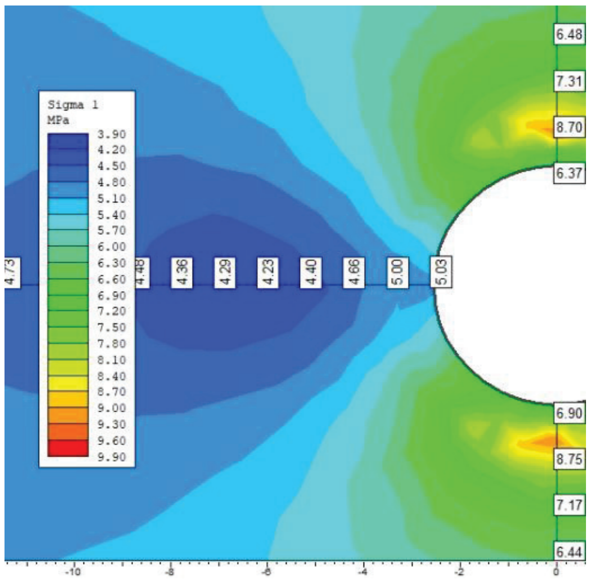


Figure 19 sigma 1 before lining installation at the bottom of vertical shaft

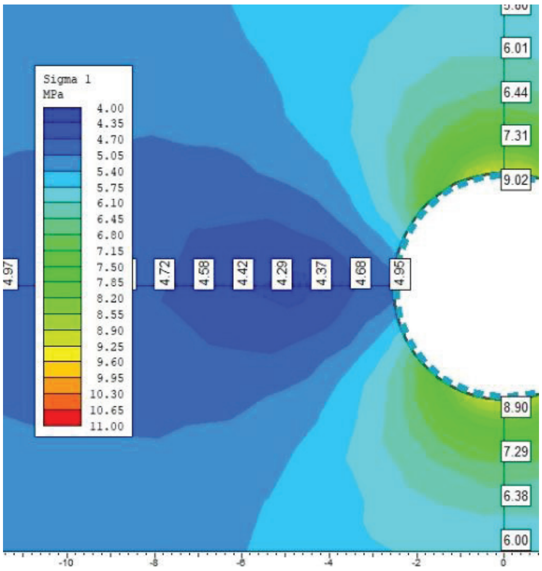


Figure 20 Sigma 1 after lining installation at the bottom of vertical shaft

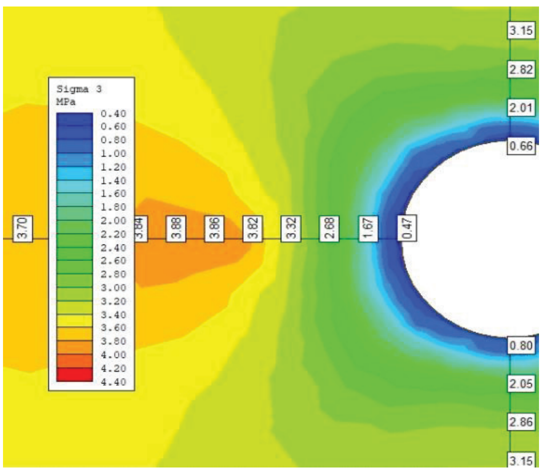


Figure 21 sigma 3 contour lining installation at the bottom of vertical shaft

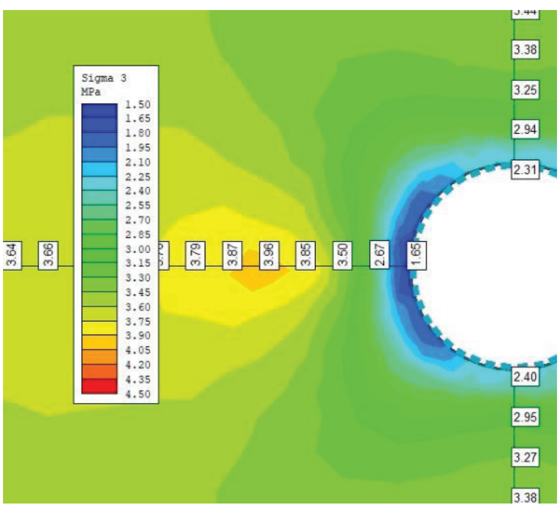


Figure 22 Sigma 3 contour after lining installation at the bottom of vertical shaft.

In the figure above, major and minor principal stress around the tunnel in radial direction is increased due to Support Installation.

TABLE 5 OUTPUT SUMMARY OF NUMERICAL ANALYSIS METHOD USING PHASE2 SOFTWARE

Plastic Zone Radius before and after support Installation	3.256 m, 0 m
Shotcrete lining Properties	
Thickness	300 mm
Compressive Strength	25 MPa
Poison's ratio	0.25
Young's Modulus	30000 MPa

X. CONCLUSION

The available lateral and vertical rock cover seems satisfied a Norwegian Design Criteria. The minimum principal stresses are not relevant unless hydro fracturing occurs. In more probable case of hydro jacking, the stresses normal to the various discontinuities are relevant. Steel lining along a pressure tunnel and shafts can be determined where the dynamic hydrostatic pressure exceeds minimum principal stress. The Analysis at the bottom of Vertical Shaft was done by semi empirical and Analytical method and the results were very close to the results of Numerical Analysis method using Phase2 Software. Number of Yielded elements on the joints is increasing with decreasing the friction angle of joint.

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