Geoengineering aspects of the Lavarak hydro-power cavern in very soft rock, Iran

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Abstract: Soft rocks are the third type of geotechnical material for which their stress/strength features are related to the overlapping spectrum between soil and rock mechanics. These materials and hard soils exhibit, at failure, a behaviour that is intermediate between that of hard soils and soft rocks. This appears to be the case for materials of sedimentary origin as well as for grouted sands that can be considered as artificial soft rocks or hard soils depending on the cementing agent, the density of the sand and the efficiency of grouting. On the other hand, a rock mass is considered to be weak when its in-situ uniaxial compressive strength is less than about one third of the in-situ stress acting upon the rock mass through which a tunnel is being excavated.

The Lavarak hydro-power cavern with a 47 MW electricity potential is located about 50 km north-east of Tehran, Iran. This cavern, along with its two deep and inclined shafts, has been designed and constructed within Plio-Pleistocene soft and massive conglomerates. There are also two great reverse and thrust, high seismic potential faults (Mosha and north Tehran) in the vicinity of the cavern. The rock mass has a low shear strength (very weak) and a high deformability potential which, from in-situ tests and extensive monitoring programmes in trial chambers, it was concluded that the rock masses' geomechanical parameters are highly stress dependant. With increasing confining pressure (to a certain extent), the deformation modulus due to compressibility and consolidation of conglomerate, will increase at least two times. Accordingly, the orientation of the cavern was selected and constructed perpendicular to the maximum in-situ stress.

Résumé: Les roches molles sont le troisième type de matériel géotechnique pour lequel leurs dispositifs de stress/strength sont liés au spectre de recouvrement entre le sol et la mécanique des roches. Cet matériaux et objet exposé dur de sols, à l'échec, un comportement qui est intermédiaire entre celui des sols durs et les roches molles. Ceci semble être la caisse pour des matériaux d'origine sédimentaire aussi bien que pour les sables scellés au ciment qui peuvent être considérés comme roches molles artificielles ou sols durs selon l'agent de cimentage, densité du sable et efficacité du jointoiement. D'autre part, une masse de roche est considérée comme faible quand son résistance à la pression uniaxiale in-situ est moins qu'environ un tiers de l'effort in-situ agissant sur la masse de roche par laquelle un tunnel est excavé.

La caverne d'hydro-électricité de Lavarak avec un potentiel de l'électricité de 47 MW est située environ 50 kilomètres de nord-est de Téhéran, Iran. Cette caverne, avec ses deux profonds et axes inclinés, a été conçue et construite dans les conglomérats mous et massifs de Plio-Pléistocène. Il y a également deux grands renversé et pousse, les défauts potentiels séismiques élevés (Mosha et Téhéran du nord) à proximité de la caverne. La masse de roche a une basse résistance au cisaillement (très faible) et un potentiel élevé de déformabilité que, des essais in-situ et des programmes de contrôle étendus dans les chambres d'essai, on a conclu le que les paramètres geomechanical des masses de roche sont fortement personne à charge d'effort. Avec l'augmentation de la pression d'emprisonnement (dans une certaine mesure), le module de déformation dû à la compressibilité et à la consolidation du conglomérat, augmentera au moins deux fois. En conséquence, l'orientation de la caverne a été choisie et a construit la perpendiculaire à l'effort in-situ maximum.

Keywords: Classification, Laboratory tests, Site investigation, Plate-bearing tests, Shear tests, Monitoring

INTRODUCTION

Underground hydro engineering projects such as power station chambers and different types of "conduits" supplying water to the station, such as pressure tunnels and shafts are seldom situated within soft rocks. The main criteria for locating projects in soft rocks are the suitability of the type and structure of the rock mass with respect to: heterogeneity; discontinuities; anisotropy, rock strength and ratio of the field stress to the strength of rock mass.

The first task is to define what is a soft rock. Two main features of these materials are as follow:

- Soft rocks are rocks can fail under their in situ stress or $P_o / \sigma_{cm} > 1$.
- Soft rocks are materials with low UCS and described as geomaterials with properties soils and rock.

With regard to soft rocks' uniaxial compressive strength, (σ_{ci}) ISSMFE (1985) has classified soft rocks with $\sigma_{ci} = 0.5 - 25$ (MPa) and Brown (1981) also classified the geotechnical materials as in Table1.

Table 1. Soft rock materials

σ_{ci} (MPa)	Description
0.25 - 1	Extremely weak
1 – 5.0	Very weak
5 – 25.0	Weak

ISO (1997) have been determined the boundary of soil and rocks as $\sigma_{ci} < 0.6$ (MPa). In Anon (1977) classification, also, the overlap spectrum of very weak rocks/ hard soils is presented as $\sigma_{ci} < 0.6 - 1.25$ (MPa).

The facts indicate that the characterization of soft rocks is a complex interdisciplinary process. Therefore, tests in the natural in situ state are extremely important and their results represent actual influences on the rock mass. Different laboratory tests are helpful and supplement the results of field stress.

In this paper, the authors present a case history of large- scale tests of the most important mechanical properties of a soft conglomeratic rock mass, i.e. shear strength tests, in situ deformability tests and monitoring in trial chambers, for the design and construction of a cavern, deep shafts and access tunnels.

GEOLOGICAL FEATURES OF THE ROCK MASS INVESTIGATED

The masses investigated are composed of Plio-Pliostocene massive non-cemented, clayey-silty matrix conglomerate with very wide spaced master joints (S>5m) which are undulating; rough – planar smooth, with clean to stiff clay infill and a high persistence (L>10m) which in the Hoek et al. (2002) classification correspond to a massive good rock mass, GSI=75 and Q=26. This conglomerate with about 300 m thickness occurs in a large area in the north east of Tehran (Figure1).

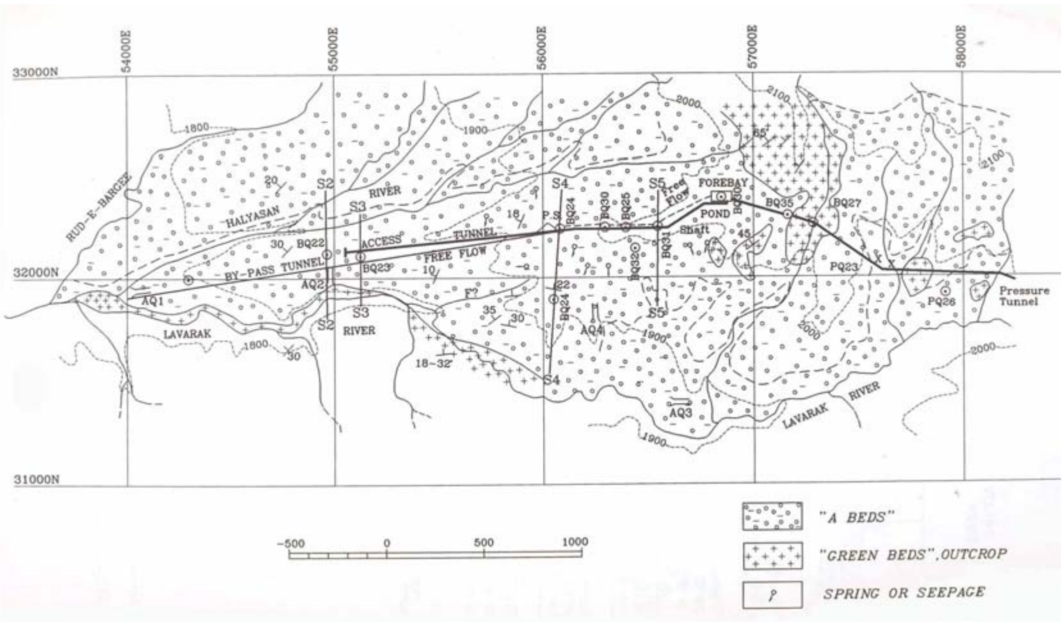

Figure 1. Geological conditions of Lavarak hydro project North- East of greater Tehran. The two great seismic reverse faults of Mosha and North Tehran with 80° dip toward N to N30E, situated in vicinity of cavern and causes a green tuff (Eocene) to be in contact with the Hezar Dareh conglomerate.

GEOTECHNICAL INVESTIGATION

Investigations were performed in the following stages:

- The six deep (65 to 279 m) exploratory bore holes totalling 965m.
- A series of exploratory chambers for access to the cavern, in situ tests, monitoring and 20 shallow bore holes (15 to 30m, totalling 512m). These chambers and tests were conducted by the Japanese organisation Kuma Gami (1976-78) and Iranian (1994) companies (Figure 2).
- Laboratory rock and soil mechanics testing on 54 to 146 mm cores.

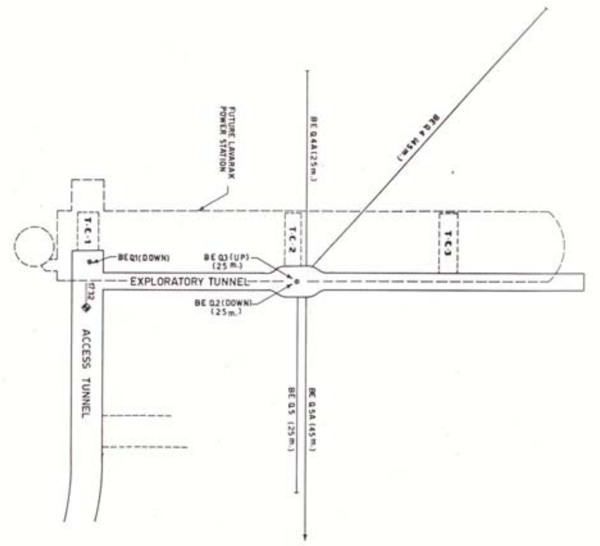

Figure 2. Location of openings for access and in situ testing of the cavern rock mass.

Data interpretation

The drill holes data indicated that:

- Depth of conglomerate (H) is below the cavern base or H>250m.
- No lithological variation and a homogeneous rock mass.
- The permeability is 10^{-9} to 10^{-11} cm/sec and a water- table at 18.0 m from surface.
- No joint induced blocky behaviour and a RQD of 90 – 100 %.

Laboratory tests

The result of laboratory testing in consolidation drained (CD) and saturation state (SCD) are presented in Tables 2 and 3.

Table 2. Physical properties of conglomerate.

Soil type	Atterberg limit			Natural density γ (t/m^3)		Specific gravity G_s (t/m^3)		Dry density γ_d (t/m^3)		Porosity n(%)		Water content w(%)		Longitudinal wave velocity v_p (Km/s)
	LL	PL	PI											
GC	30 - 38	16 - 18	14 - 20											
SM	30 - 38	18 - 20	12 - 18	2.12 – 2.36	2.241	2.56 – 2.68	2.61	2.04 – 2.33	2.2	11.3 – 21.1	16.8	1.42 – 15.7	4.88	1.78 – 1.8

The data indicated that the proposed mass in a dry state is very weak and in a saturated state (σ_{ci} <1.0 MPa) is in the extremely weak rock class (Anon 1977). The strength and deformability of intact rock is strongly related to water content:

$$\sigma_{ci} = 4.413\, e^{(-0.1997(w\%))}$$

$$\sigma_{ci} = 25.245\, (w\%)^{-1.2553}$$

$$E_{Si} = 0.7548\, e^{(-0.3388(w\%))}$$

$$I_{S50} = 11.687\, e^{(-19.223(w\%))}$$

Table 3. Mechanical properties of conglomerate.

σ_{ci} (MPa)	E_{si} (MPa)	I_{s50} (MPa)	Direct shear test		Triaxial consolidated drain (CD) *		Triaxial saturated consolidated drain (SCD)		p - q			
									SCD		CD	
			C (MPa)	ϕ (Deg.)	C (MPa)	ϕ (Deg.)	C (MPa)	ϕ (Deg.)	C (MPa)	ϕ (Deg.)	C (MPa)	ϕ (Deg.)
0.11 – 10.1; 2.7	6 - 900; 335	0.33 – 1.32; 0.53	0.125 – 0.416; 0.273	11 – 31.5; 20.4	0.1 – 0.8; 0.43	25.4 – 38.6; 32.1	0.1 – 0.5; 0.15	21.8 - 31; 25.4	0.38	23.5	0.7	33.1

* Pore back pressure B= 1- 3- 6- 8 (bar) and σ_3 = 1- 3- 6- 8 (MPa)

The effect of water content has a reverse relation with degree of compaction and consolidation of the silty clay (CL) matrix and grain fabric of the conglomerate. The best strength achieved is in sutured and concave- convex grain contact. The above features indicated the stress induced behaviour of the young conglomerate.

In situ tests

The in situ tests in Lavarak cavern were limited to plate loading and dilatometer tests; the results of which for a 200m overburden are summarized in Tables 4 and 5.

Table 4. The result of jacking tests in trial chambers for deformation modulus.

Test No.	Test location	Test orientation	E_m (MPa)		E_S (MPa)	E_t (MPa)	Creep factor* (%)
			Right face	Left face			
1	Exploratory adit 1	Horizontal	2290	2220	-	-	-
2	Exploratory adit 1	Horizontal	1790	1600	-	-	-
2A	Exploratory adit 1	Vertical	2145	2220	-	-	-
1	Trial chamber No.1	Roof	710		880	960	16.67
2	Trial chamber No.1	Floor	1970		2850	3000	14.29
3	Trial chamber No.1	Right wall	2180		3000	3470	9.1
4	Trial chamber No.1	Left wall	960		1384	1400	17.19

* CF= $\frac{\delta_c}{\delta} = \frac{\text{creepdisplacement}}{\text{totaldisplacement}} \times 100$. Plate Diam. = 30 cm

Table 5. The result of dilatometer test in trial chambers (T.C.), P_{max} = 5-12 MPa

Test No.	Location	E_m (MPa)
1	T.C. No.1	1500 - 4000
2	T.C. No.2 (left)	260 – 330
3	T.C. No.2 (right)	1550 – 2720
4	T.C. No.3 (left)	1730 – 6890
5	T.C. No.3 (right)	500 - 1500

The jacking test conducted without extensometer, in 4 cycles of 1.5- 3.0- 4.5- 4.7 MPa with the 4th cycle having loading time of 789 minute (horizontal.) and 765 min. (vertical) used for rock creep. The deformation modulus was computed by:

$$E_m = \left(\frac{1-\upsilon^2}{2a}\right).\frac{\Delta F}{\Delta W} = \frac{\Pi.a(1-\upsilon^2)}{2}.\frac{\Delta\sigma}{\Delta\delta}$$

where;

υ = Poisson ratio
a= Plate radius
ΔF = Loading step
ΔW = deformation step

$\Delta\sigma$ = Stress step

$\Delta\delta$ = Deformation step

The dilatometer test was conducted by a Menard pressuremeter at depths of 0.5 to 4.5 m in 86mm bore holes where the mass deformation is calculated by:

$$E_m = (1+\upsilon)\, d \frac{\Delta P}{\Delta d}$$

where;

d= Borehole diameter

$\frac{\Delta P}{\Delta d}$ = Slope of the stress- deformation curve

The data indicated that there is a relatively large variation in E_m values and this variability occurs more in the pressuremeter than the jacking tests. Due to lack of a clear trend in E_m value, parallel and across the cavern axis and from rock surface to depth, it is concluded that, the degree of consolidation, compaction and moisture content (Fig.3) are the main controlling parameters of conglomerate deformability. It should be noted that, due to the stress controlled nature of the aforementioned parameters, the geotechnical behaviour of the young conglomerate is also controlled by its compressibility nature which is related to the value of the confining pressure (Fig4).

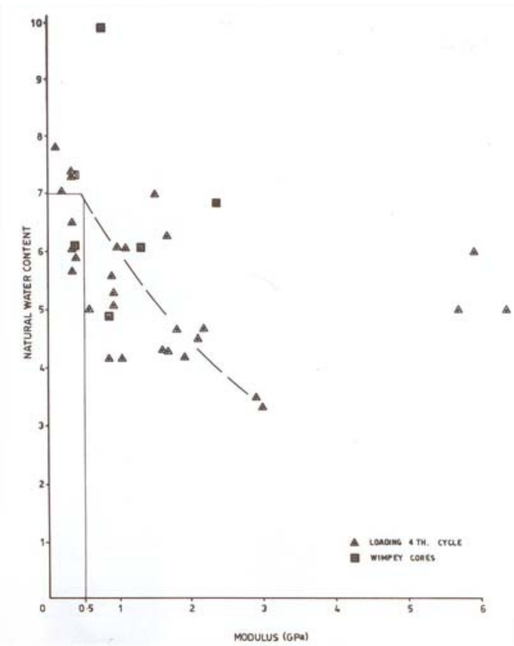

Figure 3. Effect of moisture content on the deformation modulus of conglomerate.

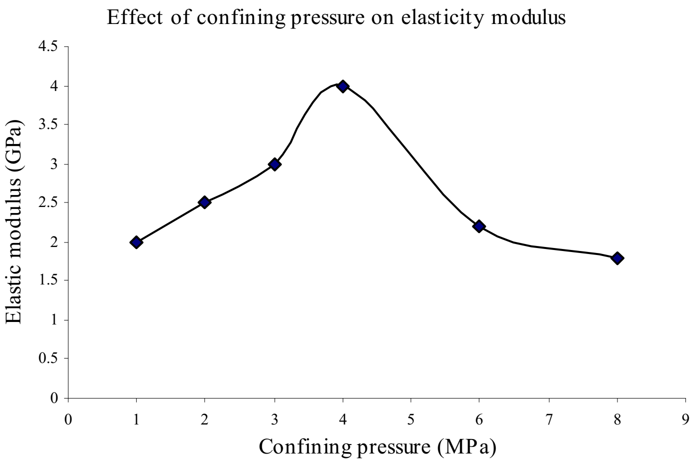

Figure 4. Effect of confining pressure on elasticity modulus of conglomerate in "SCD" triaxial test.

The comparison of Table 3 with Tables 4 & 5 and Fig.4 regarding confined and unconfined elasticity modulus indicated a three-fold increase in confined (2.5 – 4 GPa) relative to the unconfined (0.335 GPa) state. This explains how the rock mass parameters could be higher than intact rock. The influence of the stress state on the stress – strain behaviour of rock can be summarized in three model cases based on Lionco & Asis (2000) (Figure 5).

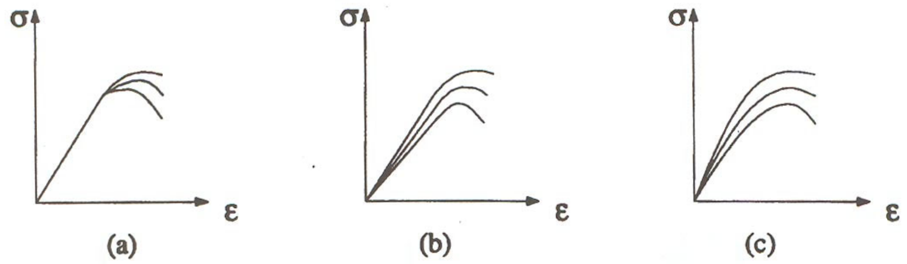

Figure 5. Influence of the stress state on the modulus: (a) The modulus is insensitive to the variation of the stress state; (b) Dependence only on the confining stress; (c) The modulus depends on the confining and deviator stresses (Lionco & Assis 2000).

The stress – strain curves of conglomerate [$(\sigma_1 - \sigma_3)$ versus $\varepsilon_v \times 10^{-3}$] for each confining stress in triaxial tests are shown in Figure 6.

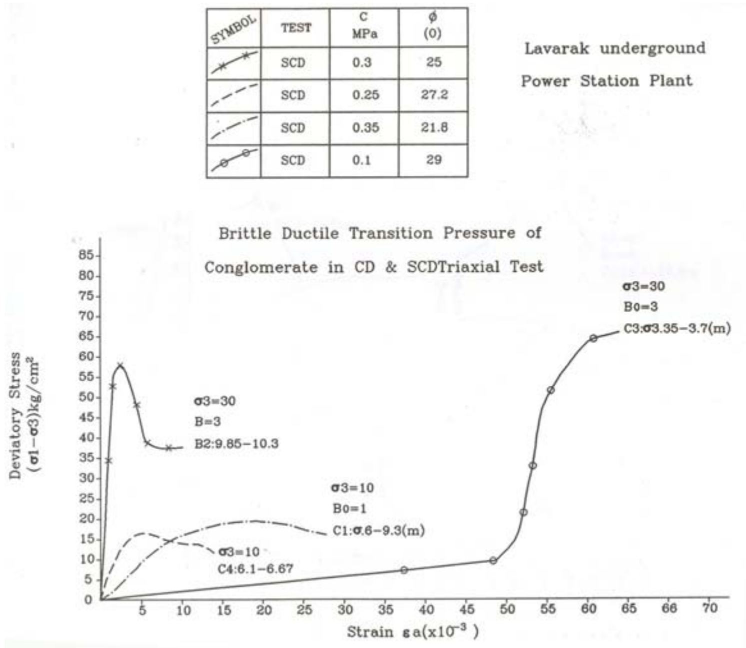

Figure 6. Stress – strain curves in triaxial tests.

As shown, for each confining stress there is different peak strength and behaviour. This variable behaviour of rock is a specific feature of soft rock masses, and in this case, for he cavern conglomerate, as determined by in situ tests. A single value of the elastic or deformation modulus could not be used in analysis.

Shear strength parameters

As shown in Table 3, direct shear tests and CD – SCD triaxial tests were performed on a number of intact samples at their natural water content in order to determine the shear strength of rock material and the comparison of intact material to mass parameters, the failure criteria, special to soft and weak rocks were used (Table 6).

The shear strength parameters above indicate good agreement with range of cohesion obtained from direct shear tests which clears by itself the high degree of equality of intact rock and rock mass. This is a special feature of un-jointed soft rocks.

The reason of dispersion on friction angle (ϕ) of conglomerate (Table 6) is related to the arrangement of clast grains (fabric) to each other which, in turn, depends on stress related compaction property of conglomerate.

Table 6. Shear strength parameters of conglomerate by several failure criteria.

Method	Equation	GSI	σ_{ci} (MPa)	C (MPa)	ϕ (Deg.)
Stress dependence of Hoek et al. (1992)	$\sigma_1 = \sigma_3 + \sigma_{ci}[m_b.\frac{\sigma_3}{\sigma_{ci}}]^a$ $m_b / 14 = \exp(\frac{1}{25}(RMR - 100))$ $a = \exp(-\frac{1}{53}RMR)$	47	2.6 – 3.0	0.13	7
		60		0.28	27
Carter et al. (1991)	$\frac{\sigma_{cm}}{\sigma_{ci}} = \exp(\frac{1}{25}(RMR - 100))$ $\frac{\sigma_1}{\sigma_{cm}} = (1 - \frac{\sigma_3}{\sigma_{co}})^R,\ \sigma_{co} = 0.1MPa$ $R = \exp(-0.3 - 0.0025RMR)$	54	2.6 – 3.0	0.183	7.1
Bieniawski (1993)	$\frac{\sigma_{cm}}{\sigma_{ci}} = \exp(\frac{1}{24}(RMR - 100))$ $\frac{\sigma_1}{\sigma_{cm}} = (4(\frac{\sigma_3}{\sigma_{cm}})^{0.6} + 1)$	54	2.6 – 3.0	0.251	18
Johnston (1985)	$\frac{\sigma_1}{\sigma_{ci}} = [(\frac{M}{B})(\frac{\sigma_3}{\sigma_{ci}}) + 1]^B$ $B = 1 - 0.0172(\log\sigma_{ci})^2$ $M = 2.065 + 0.231(\log\sigma_{ci})^2$		2.6 – 3.0	0.9	21
Hoek et al. (2002)	$\tau = A(\frac{\sigma_n - \sigma_{tm}}{\sigma_{ci}})^B$ $\sigma_{tm} = \frac{\sigma_{ci}}{2}(m_b - \sqrt{(m_b^2 + 4S)}$ $m_b = m_i \exp(\frac{GSI - 100}{28})$ $S = \exp(\frac{GSI - 100}{9})$ If GSI>25 S=0 If GSI<25	47 - 60 | 54	2.6 – 3.0	0.27 – 0.3	21 - 26

Orientation and design parameters of cavern

According to numerical back analysis of monitoring data and the main structural geology of the cavern area (faults and folds), the in situ stress condition of the proposed area is concluded as being as follows:

- Maximum horizontal stress orientation *N005 E* to *N 015 E*
- Vertical stress σ_v =4.0 (MPa)
- Major horizontal stress σ_H = 3.8 (MPa)
- Minor horizontal stress σ_h = 2.15 (MPa)

With the stress related strength feature of the conglomerate being the relationship between in situ stress to rock mass strength close to unity (σ_0 / σ_{cm}), as a prevention against rock mass failure, it was decided to orient the cavern axis at 60 degree to main and at 30 degree to minor in situ stress (fig.7) which is *N305 – N125*.

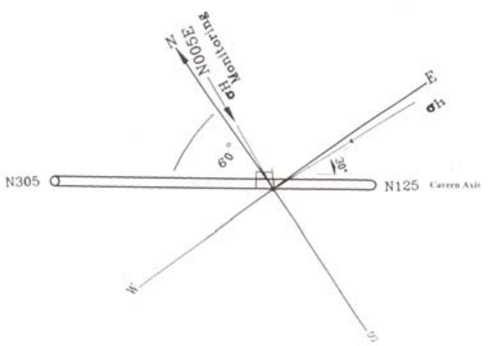

Figure 7. The cavern orientation with respect to *in situ* stress.

For numerical stability analysis and structural design data, the following confining stress related geotechnical parameters are presented according to the evaluation of laboratory and field tests in one hand and engineering judgment (from 16 km water conveyance tunnels in this conglomerate) on the other hand (Table 7).

Table 7. Geotechnical parameters of conglomerate for design.

Natural density γ (t/m^3)	**C_m (MPa)**		**ϕ_m (Deg.)**		**E_m (MPa)**		**υ**
2.3	**Dry**	**Sat.**	**Dry**	**Sat.**	**Dry**	**Sat.**	0.3
	0.27 – 0.3	0.1 – 0.15	21 - 25	15 - 18	3000	1200	

CONCLUSIONS

The results of the wide geotechnical investigations for design and construction of Lavarak cavern in massive soft rock (conglomerate) demonstrated the rock mass behaviour due its to compressibility is confining stress related. Increasing the stress, until close to mass strength (4 MPa), would cause the reduction of pores, void spaces, compacting the silty clay matrix and convert it to stiff clay and finally, and rearranging the clast grains from floating to long and suture contact. Thus, the cavern axis was oriented relatively perpendicular to main major stress (σ_H) and σ_h 30° to structure long axis.

Due to structural homogeneities and non-joint controlled behaviour of the conglomerate, the intact and mass properties of rock are very close to each other. The unconfined compressive strength of rock (2.7 MPa) is smaller than confined strength (σ_{cm} = 4.0 MPa).

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