

Use of passive anchors in increasing the stability safety factor of downstream slope of spillway in Shahriar Storage Dam

Ali Nemati Hayati, Mohammad Mehdi Ahmadi
Department of Civil Engineering, Sharif University of Technology, Tehran, Iran

Aliakbar Kashighandi
Tablieh Construction Company, Tehran, Iran

Mehdi Hajjar
Department of Mining Engineering, Tarbiat Modares University, Tehran, Iran

Mehdi Dadashi
Department of Civil Engineering, Khaje Nasir University of Technology, Tehran, Iran



ABSTRACT

A hanging block in the right flank under the spillway of Shahriar Dam, Mianeh, Iran with minimal factor of safety of 1.37 is addressed in the present study. The safety factor is less than the admissible according to design specifications. The following paper aims at presenting a solution to increase the existing safety factor to the allowable limit by installation of passive rock bolts without berm removing. The latter is not only cumbersome due to topological conditions, but also may be detrimental to the spillway structural concrete and rock-mass foundation. After plain failure analyses, it is found that passive anchors not only provide the required safety factor, but also are much more feasible considering construction limitations.

RÉSUMÉ

Un bloc de suspension dans le flanc droit en vertu de l'évacuateur de crues du barrage de Shahriar, Mianeh, l'Iran avec le facteur minimal de sécurité de 1,37 est abordé dans la présente étude. Le facteur de sécurité est inférieure à la recevable selon les spécifications de conception. Le document qui suit vise à présenter une solution pour augmenter le facteur de sécurité en vigueur à la limite permise par l'installation de boulons d'ancrage passif sans berme enlever. Ce dernier est non seulement lourde en raison de conditions topologiques, mais aussi peut être préjudiciable à l'évacuateur de crues en béton de structure et de fondation rocheuse de masse. Après des analyses de défaillance plane, il est constaté que les ancrs passive est non seulement le facteur de sécurité requis, mais ils sont aussi beaucoup plus facile tenant compte des limites de construction.

1 INTRODUCTION

A hanging block in the right flank under the spillway of Shahriar Storage Dam, Mianeh, Iran with a minimal factor of safety of 1.37 is addressed in this paper, which does not meet the required safety factor of 1.50 for static conditions according to the design criteria (JVSE (2005a)). An initial solution consisting of 10-m-height berm removing and subsequent installation of anchors in the area below the spillway had been proposed, which was not practical due to topological conditions. Furthermore, blasting was detrimental to the spillway structural concrete and rock-mass foundation.

The following paper aims at presenting a solution to increase the existing safety factor to the allowable limit by installation of passive rock bolts without berm removing. This method is both safe and economical and alleviates the difficulties involved in anchoring perpendicular to the slope face.

2 PROBLEM DESCRIPTION

The spillway is founded on moderately jointed sound to slightly weathered rock mass in the downstream of right flank. Fig.1, reproduced from JVSE (2006), clearly shows the rock mass of the slope below the spillway together with major joint sets. Fig. 2 illustrates a schematic representation of the joints with respect to the slope face in GG section. As seen in the figures, there are two joint sets with dip angles lower than the slope face, which may be important in view of sliding along joint planes. Both of these joint sets have dip directions in the range of the slope-face dip direction and are described in Table 1. Joint set 4L has maximum friction angle lower than the minimum contemplated friction angle of 38° for the rock mass, thus possibility of plane failure along this joint is trivial. Joint set 7L is rather parallel to the slope, daylighting in the slope, and has the highest frequency among joint sets (JVSE (2007)). Due to the aforementioned, this joint set is most significant in plane failure analysis.

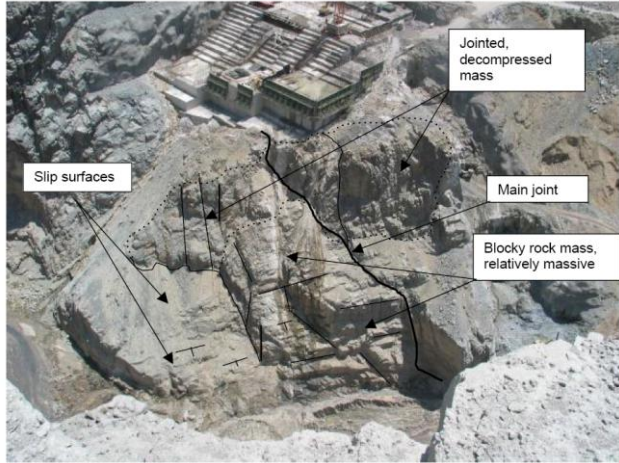


Fig. 1. Plunge pool area below the spillway (JVSE (2006))

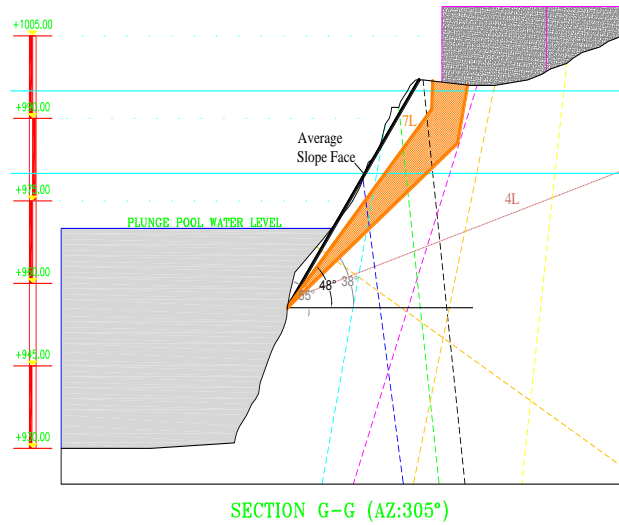


Fig. 2. Schematic representation of joint sets in the slope

Table 1. Joint sets with strike in the range of the slope face (JVSE (2007))

Joint Set	Dip		Dip Direction	
	Min	Max	Min	Max
4L	5	30	165	345
7L	35	55	285	315

3 ANALYSIS

3.1 Overview and Assumptions

The shear strength as well as rock mass parameters of the joint set 7L in plane failure analyses are assumed to be identical to the ones adopted in JVSE (2006): $c=50\text{kPa}$, $\phi=38^\circ$ and the dip angle of the joint set equal to 45.17° . The average slope face angle in the 40m height of the slope is also the same as JVSE (2006), being 55° (Fig. 2).

The only joint set along which sliding may stimulate further sliding of the underneath packages and therefore cause

some foundation problems on the spillway structure is 7L. Table 1 clearly elucidates without the need to plot stereograph that the most prominent failure mode along this joint set is plane failure. Since the slope stability considering tension crack is more critical than without tension crack, all the analyses have been undertaken considering tension crack with the angle 85° , according to JVSE (2006), that results in a minimum safety factor.

The slope stability analysis under the above conditions reveals safety factor of 1.37, which is insufficient for static conditions according to JVSE (2005a). To increase the safety factor, 10m berm excavation had been firstly proposed to lessen the active thrust on failure plane; however, this was not practical without blasting, which had potential side effects on structural concrete as well as probable activating of underneath sliding blocks. Therefore, an alternative stabilizing solution by means of $\Phi 32$ fully grouted passive rock bolts perpendicular to the upper face of the slope is of interest in the present paper. It will be shown that four rows of such rock bolts will provide enough apparent additional cohesion to result in sufficient safety factor for all load combinations in Table 2. In this regard, plane-failure analyses are implemented using RocPlane v. 2.029. Three load combinations for excavated slopes adjacent to critical structures (above full supply level & downstream of the dam) with the allowable safety factors in Table 2 are contemplated. Due to the possibility of the slope being frozen in winter, another load case, namely D, with different water distribution and safety factor of 1.3 is also assumed.

Table 2. Design load cases for excavated slopes adjacent to critical structures (above full supply level & downstream of dam) (JVSE (2005a))

Combined load case	Safety Critical Slopes			Factor of Safety		
	Structural load (if present)	25% water pressure on joint	Earthquake 0.32g	1.5	1.3	1.0
A	✓			✓		
B	✓	✓			✓	
C	✓		✓			✓

3.2 Additional Cohesion by Fully Grouted Passive Rock Bolts

Unwieldy topological condition of the slope makes it rather impractical to establish a platform for drill wagon to drill boreholes through the slope face to the joint set to install tensioned rock anchors. Therefore, in order to foster the required safety factor, fully bonded passive rock bolts are contemplated, which contribute to resisting force, and have no effect on driving force. Main defect of using passive rock bolts is rather large extent of displacements required to mobilize the additional shear strength. This could imperil serviceability requirement in certain conditions. However, for the present situation, where the stability of the block is of concern solely to prevent progressive sliding of the underneath packages, it may be

justifiable to make allowance for the displacements to reach the favorable safety factor.

Design of passive rock bolts and evaluating their effect on cohesion (so called 'apparent cohesion') dates back to 1974, when Bjurström (1974) presented a method for incorporating the effect of fully grouted passive rock bolts in increasing the resisting force by introducing additional apparent cohesion. The method is acknowledged in the technical reports of the project, as in JVSE (2005a). Thus, main reference has been made to this method to estimate the additional cohesion; even though, comparisons are made with state-of-the-art design practices or articles dealing with the subject.

3.2.1 Bjurström Method (1974)

Bjurström (1974) categorizes joint planes based on roughness into 'flat' and 'rough' joints and gives an equation for calculating the apparent additional cohesion in each case. Parameters in the equations are cross-sectional area of passive rock bolts, A_s , area of failure surface, A , shear strength of the steel, $\tau_a = 0.6F_y$, uniaxial compressive strength of the intact rock, σ_{ci} , and yield strength of the steel, F_y .

Rough joints contribution to apparent cohesion is loftier than flat joints due to stimulation of combined tension-shear stress in the reinforcement.

If we consider four rows of fully grouted passive steel bars crossing the failure plane (7L) with in-plane spacing of 1m, the apparent additional cohesion values for flat and rough joints would be those given in Table 3.

Table 3. Apparent additional cohesions for flat and rough joints in Bjurström Method, $F_y = 400\text{MPa}$

	nA_s [m ² /m]	A [m ² /m]	$\tau_a = 0.6F_y$ [MPa]	σ_{ci} [MPa]	c_a [MPa]
Flat Joint	3.21E-3	48.2	240	100	0.008
Rough Joint	3.21E-3	48.2	N/A	N/A	0.021

Table 3 reveals that roughness has great influence on the apparent cohesion imparted by the rock bolts. Therefore, proper value of the roughness for the joint set should be incorporated.

Geology reports (JVSE (2007)) assigned a joint roughness coefficient (JRC) of 8 to most joint sets, which is classified as slightly rough. If we assign $JRC = 0$ to flat joints, and $JRC = 20$ to rough joints, we may interpolate the relevant cohesion for $JRC = 8$ in the following manner:

$$c_a = \frac{(c_a)_{rough} - (c_a)_{flat}}{20} \times JRC + (c_a)_{flat} = 0.013 \text{ MPa}$$

Eventually, the cohesion used in stability calculations, c_{eq} , is equal to sum of the joint cohesion and the apparent cohesion by rock bolts:

$$c_{eq} = c_{joint} + c_a = 0.050 + 0.013 = 0.063 \text{ MPa}$$

The analysis results in subsections 3-3 to 3-6 are based on this value of c_{eq} .

3.2.2 Simplified Method by LRPC Regional Ponts et Chaussees Laboratory (1994)

In this method (Asroun et al. (1994)), contribution of a passive bar to cohesion of a non-dilating or minimally-dilating discontinuity is calculated in two following simple ways:

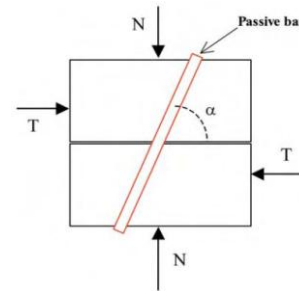


Fig. 3. Schematic view of shear box test

- If α in Fig. 3 is less than approximately 75 degrees, then the contribution taken into account is equal to two-thirds of the tensile yield strength of the reinforcement bar;
- If the inclination is less favorable, the bar contribution would be set to half the bar yield strength

In our case, where direction of reinforcement bars makes an angle of $128^\circ < \alpha < 138^\circ$ with the discontinuity plane (Fig. 9), the bar inclination is considered as less favorable and the according bar contribution is equal to $0.5F_y = 200\text{MPa}$. Therefore, the apparent additional cohesion in this case becomes:

$$c_a = \frac{n\pi \cdot 0.032^2 / 4 \cdot (F_y / 2)}{A} = \frac{4\pi \cdot 0.032^2 / 4 \cdot (400 / 2)}{48.2} = 13\text{KPa}$$

The above is the cohesion induced by rock bolts crossing joint flat joint. As observed in the previous subsection, roughness of the joint has considerable effect on the shear strength. Thus, the real cohesion imparted by rock bolts is considerably higher than 13KPa due to roughness of the joint.

Large-sized shear box tests by Bidaut et al. (2006) ascertain that the simplified method discussed herein provides safety factors of at least 2.0 for all tests.

3.2.3 Large-sized Shear Box Tests (2006)

Patrick Bidaut et al. (2006) designed a shear machine that enabled conducting full-scale shear tests on rock discontinuities reinforced by passive rock bolts. Fully grouted steel bars with diameter varying between 20 and 40 mm with different inclination angles to the shear plane were sheared up until failure. The applied forces and displacements were recorded.

Table 4, which is reproduced from Bidaut et al. (2006), obviously shows that the maximum contribution by passive anchors is greater than the tensile strength of the bars, ranging from 1.37Ty for $\alpha = 45^\circ$ to 1.01Ty for $\alpha = 120^\circ$.

Table 5 elucidates that the safety factor acquired in the design by simplified method by LRPC is greater than 2, according to experiments.

Table 4. Influence of bar inclination ($\phi 25\text{mm}$)(tensile yield strength of 246 KN) (Bidaut et al. (2006))

Bar inclination (angle α in degrees)	Maximum tangential force T (kN)	Angle of friction ϕ (degrees)	Normal force N (kN)	Maximum contribution C_s (kN)	Displacement u_i at failure (mm)
45	1017	36	936	338	12
60	1071	37	985	331	11
75	962	36	932	285	16
80	969	35.5	965	280	20
90 (two tests)	940/922	37/38	862/810	291/288	39/42
100	1000	37	980	262	46
120	863	37.5	802	248	96

Table 5. Safety factors acquired in the simplified method according to experiments (Bidaut et al. (2006))

Angle α (degrees)	Ultimate measured contribution (kN)	Simplified method (kN)	Safety coefficient
45	338	164	2.06
60	331	164	2.02
75	285	123	2.32
80	280	123	2.28
90 (two tests)	291/288	123	2.32
90 (N = 0.55 MN)	270	123	2.20
90 (N = 0.30 MN)	290	123	2.36
100	262	123	2.13
120	248	123	2.02

Eventually, it is worthwhile to re-emphasize that by considering $c_a = 13\text{KPa}$, we have assumed $\text{JRC}=8$ in Bjurkström method and neglected the roughness in the simplified method. Comparison with Full-scale experiments reveals the reasonable safety factor of the adopted value.

3.3 Load combination case A

Fig. 4 shows schematic view of plane-failure analysis of the stabilized slope in case A, under static dry conditions. Safety factor is 1.52 as deemed necessary.

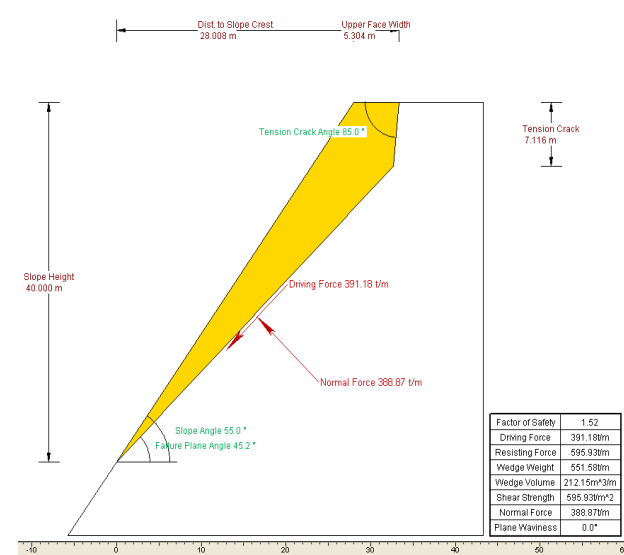


Fig. 4. Safety factor in load case A after stabilizing by rock bolts

3.4 Load combination case B

Fig. 5 shows schematic view of plane-failure analysis of the stabilized slope in case B, under static conditions with 25% water pressure on joint. Safety factor in this case is 1.40, which is above the minimum 1.3.

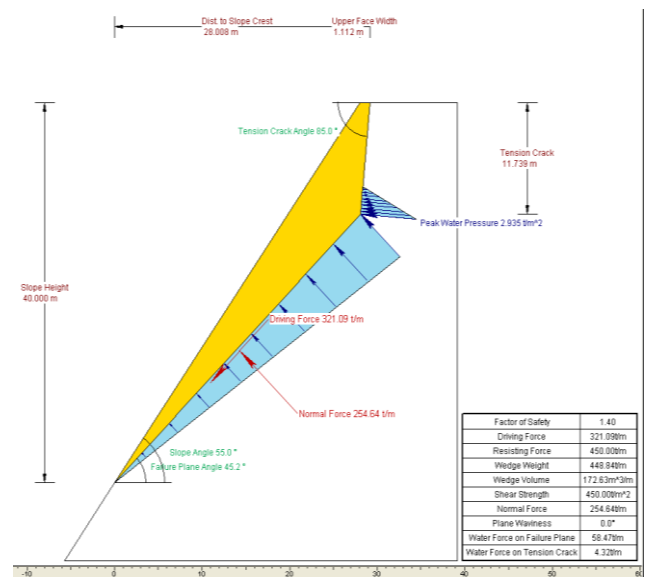


Fig. 5. Safety factor in load case B after stabilizing by rock bolts

3.5 Load combination case C

Fig. 6 shows schematic view of plane-failure analysis of the stabilized slope in case C, under MDL earthquake with 0.32g acceleration. Safety factor in this case is 0.97, which is negligibly below the allowable safety factor of 1.

- Referring to JVSE (2005b), it becomes clear that maximum persistence of joints is rarely beyond 15m--while it is assumed to be more than 48 m here;
- cohesion values of 120KPa for joints beyond 15m due to 30% rock bridges are contemplated—while 50KPa has been used here.

Based on the above, it may be reasonably inferred that the actual safety factor is much beyond the current value for all cases.

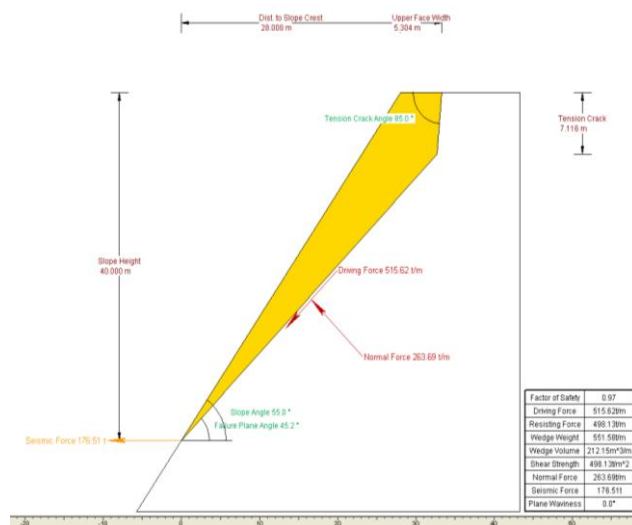


Fig. 6. Safety factor in load case C after stabilizing by rock bolts

3.6 Load combination case D

When slope face freezes, the water pressure in the joint does not become zero in the slope toe, as demonstrated by Fig. 7. Instead, it may be assumed that the water pressure increases linearly from tension-crack base to the toe in the slope. Safety factor in this load combination is calculated as 1.38 (Fig.7).

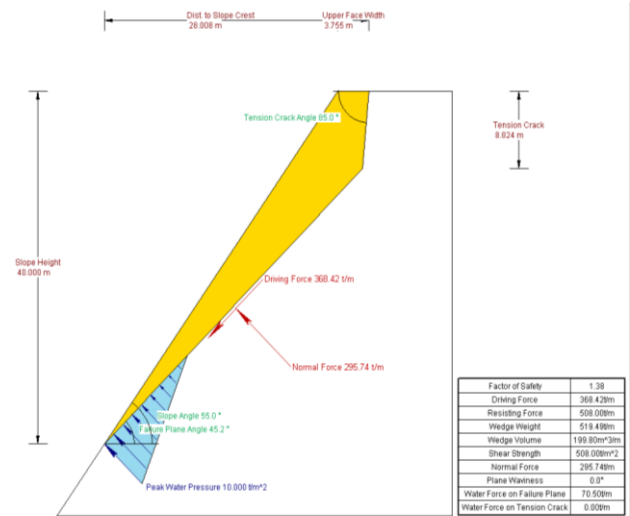


Fig. 7. Safety factor in load case D after stabilizing by rock bolts

3.7 Sensitivity analysis and stabilization scheme

The above analyses verified that installation of four rows of $\Phi 32$ fully grouted passive steel rock bolts of $F_y = 400\text{MPa}$ with the arrangement shown in Fig. 9 provides the required safety factor for all cases of load combination. Main requirement for the rock bolts is to be long enough to cross the failure plane (7L). Dip angle range of this failure plane is between 35 and 55 degrees (JVSE (2007)). Even though, dip angles lower than the friction angle (38°) do not cause plane failure. A sensitivity analysis on joint (failure) plane angle reveals that for dip angles between 38° and 48° , safety factor is below the allowable limit of 1.5 (Fig. 8). Thus, a proper arrangement and length of rock bolts should be that the rock bolts cross all these joint planes. Besides, sufficient penetration depth of the rock bolts in the lower block should be determined.

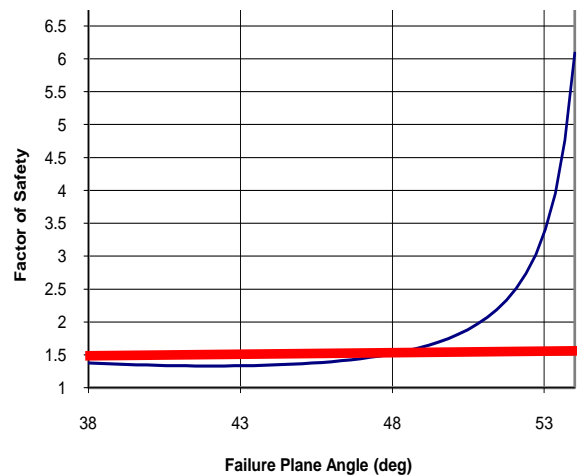


Fig. 8. Failure plane angle (deg) vs. factor of safety

To calculate this depth, an estimate of tensile strength of cement used in the grout is necessary. The cement is pozzolanic with 28-day compressive strength of $f_c = 28\text{MPa}$. If we suppose the tensile strength of the cement, f_t , to be 10% of the uniaxial compressive strength, it will be estimated as $f_t = 2.8\text{MPa}$. Applying a safety factor of 2 to f_t , length of penetration into the lower block, l , may be calculated as

$$l = \frac{\pi \frac{0.032^2}{4} \times F_y}{0.032\pi \times \frac{f_t}{\text{S.F.}}} = \frac{\pi \frac{0.032^2}{4} \times 400}{0.032\pi \times \frac{2.8}{2}} = 2.3\text{m}$$

Thus, rock bolts shall be extended for 2.3m into the lower block.

The devised array of rock bolts, lengths ranging from 16.9m to 19.5m in Fig. 9, is based on the above considerations.

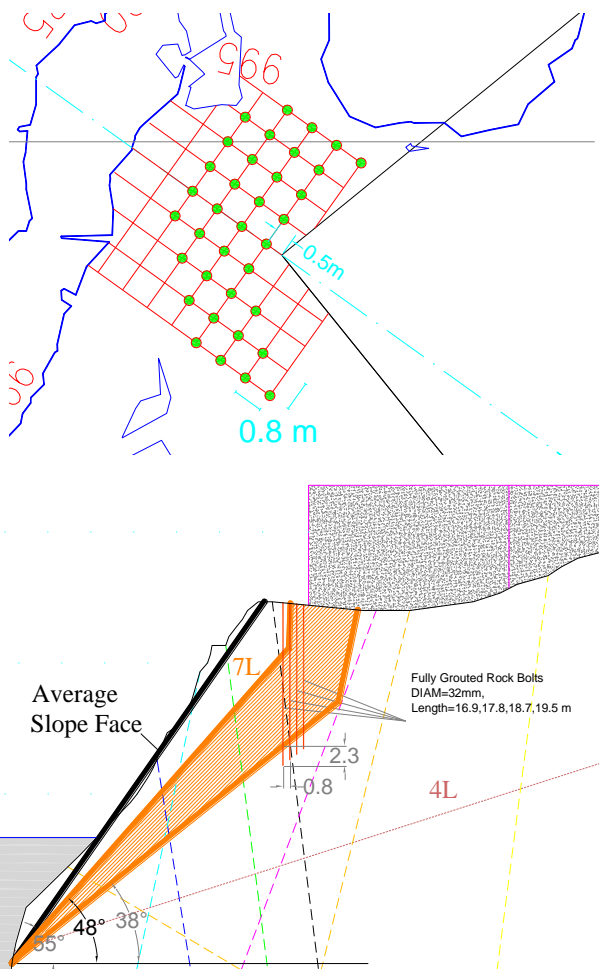


Fig. 9. Arrangement of rock bolts

Prior to the installation of rock bolts, application of 100mm thick reinforced concrete is necessary to homogenize the surface zones, prevent weathering effects and prohibit water percolation to the failure plane, tension crack and bore holes to restrain possible corrosion of the steel bars.

Moreover, to suppress water filtration to the tension crack and failure plane and its detrimental effect on stability, it is recommended to drill some drainage boreholes from the drainage gallery below the spillway to the slope face. This is not only effective in suppressing water accumulation in tension crack/failure plane, but also in stifling the formation of frozen ice on the slope face in winter and the resulting water pressure in the slope toe.

4 SUMMARY

In this paper, a stabilization method by means of passive rock bolts installed on the slope crown was proposed to increase the present safety factor of the slope under the spillway of Shahrivar Dam, Mianeh, Iran. This method removes the need for berm removing—which is only possible by using blasting due to the very hard structure of the rock mass. Moreover, it is economical and time-saving as opposed to the traditional method of anchoring perpendicular to the slope, which is cumbersome due to the special topology of the slope.

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