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SYNOPSIS: The Paper describes failures of some high tensile strength steel tensioned rock anchors at a deep excavation. The failures are attributed to stress corrosion to which the high tensile strength steel is particularly susceptible. The method which was used to estimate the life of remaining anchors on the project is described. The need for ensuring a high level of care during transportation, storage and installation of such high tensile strength steel bars is emphasised.

INTRODUCTION

The use of ground anchors to provide lateral support for excavations is now a common practice. These anchors can be designed with an adequate margin of safety by using current design methods in combination with well documented data from previous projects, and by exercising adequate quality and testing control during the installation process. In particular the adoption of current methods of design and construction have greatly reduced the possibility of failure of the anchors along the grout-ground, and grout-tendon interfaces. However one aspect of anchor design where great caution still needs to be exercised, relates to the corrosion failure of tensioned steel tendons or the steel anchor head assembly. These failures are seldom reported in literature, although most designers are aware of cases where such instances have occurred in the field.

A detailed case history has been presented by Jurell (1985) who described an anchor failure at an underground machine hall in Sweden. A total of 118 Dywidag 80/105 prestressing bars, 26 mm. in diameter, and with an average length of 12 mm, were stressed to a load of 300kN during 1955. In the spring of 1981, one of the bars failed 2.5 m from the anchor head with such a force that it flew out and landed 8 m away on the floor. Such sudden release of energy apears to be typical of stress corrosion failures. Subsequent investigation revealed that the failure had been triggered by a 5 mm deep primary crack originating from the bottom of a large corrosion pit having a maximum depth of only 0.8 mm. A section of the bar at the corrosion pit where the fracture initiated is shown in Fig. 1.

This paper presents a case history of a deep excavation in downtown Vancouver, B.C. Canada, where several anchors failed during construction despite the fact that corrosion protection for the temporary use of these anchors had been provided. Subsequent investigation on these failures, and the method to estimate the remaining life of anchors to complete the project are also discussed.

The Site

The site, approximately 75 m x 43 m in plan, and 20 m deep was located at a busy intersection. In view of the very close proximity of sensitive structures and major underground utility services including two small tunnels, a vertical excavation was selected. It was further stipulated that the excavation would be maintained at essentially "at rest" (K_o) condition so that the elaxtic deformations could be kept to a minimum.

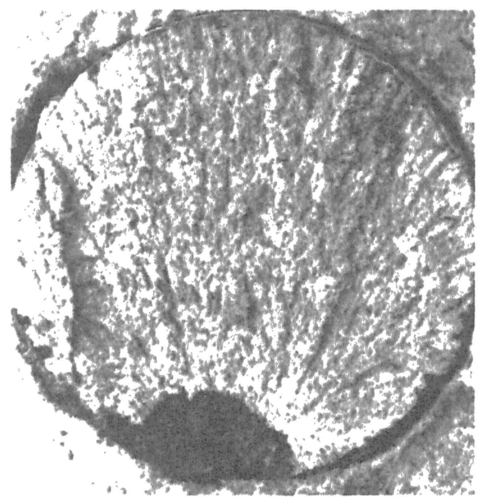

Fig. 1. Fracture surface at corrosion pit (Jurell, 1985)

The subsurface conditions at the site were evaluated from results of 14 drill holes, 10 auger holes, and a number of laboratory tests. Typically, the surficial soils consist of loose silty sands and sand and gravel fill overlying dense sand to an average depth of 3 m. The dense sand is a residual soil derived from weathering of sandstone bedrock. The contact between the dense sand the bedrock is therefore transitionary. The overburden is underlain by sandstone that is generally gradational from fine grained at the top to coarse grained at a depth of approximately 12 m. A 2 m thick mudstone layer underlies the coarse-grained sandstoone at an average depth of 14 m. The sandstone had a typical rock hardness index of R2 (classification after Piteau et al. 1979) The regional groundwater table at the site lies below the excavation floor; however, perched water tables are encountered at several higher elevations especially above mudstone contacts.

Excavation Support System

A method of excavation support using a combination of soldier piles and lagging in the overburden and tensioned temporary soil or rock anchors was judged to be most suitable for the site conditions (Fig 2). All anchors were to be destressed at the end of construction. Current jurisprudence in British Columbia does not permit the use of anchors for permanent support if the bond length intrudes on adjacent property. The permanent lateral suport is provided by the heavily reinforced substation walls and floor slabs. A 75 mm thick shotcrete layer was applied on all exposed bedrock surfaces to minimimize rock weathering and to reduce the risk of local rockfall.

The specifications for the three types of tensioned grouted anchors proposed by the contractor are given in Table 1. It is important to note that despite their temporary nature, the anchors were provided with a corrosion protection. All metal components were coated with corrosion inhibitor and the free length of the bar was enclosed in a grease-filled polyethylene sheath, which was sealed to the bar at the bottom of the free length to prevent ingress of grout in the annular space. Despite low regional groundwater, a perimeter drainage system was designed to relieve pressure from perched groundwater tables. Groundwater around the excavation was controlled by 30 cm diameter vertical perimeter collector drains. A total of 37 drains, at a nominal spacing of 6 m, was installed to the full depth of the excavation. The upper end of the drains, was sealed to prevent entry of additional water from the pervious overburden. At each collector drain, a series of "inclined" seepage drains at a vertical spacing of 3 m were drilled in the bedrock. These drains were cased with 38 mm diameter pre-slotted PVC pipe, and their lengths were varied to drain bedrock at the fixed lengths of the anchors.

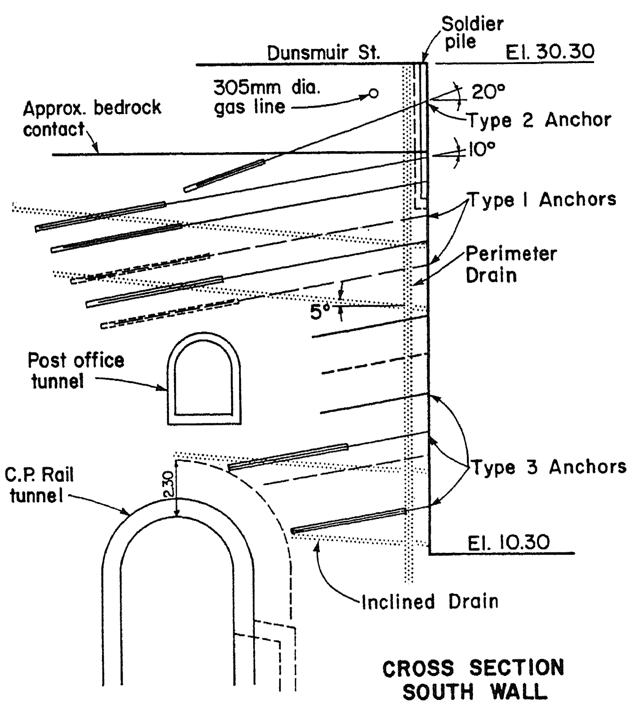

Fig. 2. Excavation support system.

The overburden support was provided by timber lagging retained by bedrock-anchored soldier piles and tiebacks. The support system was optimized using anchors with a maximum working load of 403kN (type II anchors) for the upper tiebacks and anchors with a working load of 659kN (type I anchors) for the two lower tiebacks, for an average horizontal spacing of 2 m. The type I anchors also constituted part of the bedrock support system. Anchor holes were drilled to a diameter of 89 mm for types I and II anchors, and 75 mm for type III anchors. The anchors were designed at a typical grid spacing of 2 m x 2 m to provide an average stress of 150kN/m^2 on the walls. Based on earlier anchor pullout tests, the bond lengths were determined on the basis of working shear resistance of 0.7 MPa at the rock-grout contact. Typical bond lengths for type I and type II anchors were 5 m and 3 m, respectively. Reference should be made to Garga et.al. (1984) for further details on design and construction of the excavation support system for this project.

All anchors were required to be destressed when the horizontal earth pressure could be supported by the rigid perimeter walls and floor slabs of the underground reinforced concrete structure.

Type I Rock Failure Anchors

After all anchors were installed, and during the construction of the perimeter walls and floor slabs, a total of seven randomly distributed sudden failures, occurred in the stressed type I rock anchors. The first tendon to fail under the design load fractured at the interface of the free and bond lengths, aproximately 10.5 m from the face of the excavation. The unbonded portion of the rod was protected against corrosion by a grease-filled polyvinyl sleeve that was taped at both ends. The elastic strain energy stressed in the bar was of such magnitude that it launched the failed portion of the bar 20 m across the site. Fortunately, no injuries or other damage occurred.

Failure Investigation

The failed anchor rod had performed satisfactorily for 13 months with no apparent increase in tension. It was therefore assumed that a delayed cracking mechanism was responsible for the failure. A detailed study of the anchor rod fracture surface using various metallurgical techniques indicated the following:

(i) The fast brittle fracture was initiated by a small elliptical surface crack that was covered with a black magnetite corrosion product (Fe_3O_4). The crack originated in a surface corrosion pit which induced stress and chemical concentration effects. Radial fracture lines covered 98% of the fracture surface and clearly lead back to the fracture origin.

(ii) The defect that initiated brittle fracture was intergranular in nature, and had a maximum depth of 0.86 mm and a maximum length of 3.02 mm (Fig. 3). The striking resemblance to Fig. 1 is obvious.

TABLE 1. High Tensile Steel Anchor Specifications

Type	Use	Dia. mm	Yield Stress N/mm^2	Specified Working Load, Pw kN	Minimum Yield Load, Py (=1.5 Pw) kN	Minimum ultimate capacity, P_{cf} (=1.25Py) kN
Type I	Rock support	36	1080	659	989	1236
Type II	Overburden support	32	835	403	605	756
Type III	Local rock support in front tunnels	25	835	103	195	244

Fig. 3. Fracture surface from first anchor failure. Arrow indicates originating defect. Magnification x 2.5

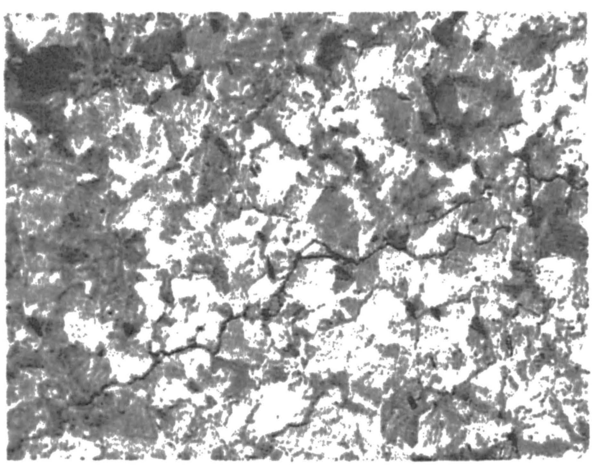

Fig. 4. Micrograph showing intergranular nature of crack. Magnification x 200.

(iii) A small intergranular crack was found immediately above the rod fracture face. This crack also originated in a surface corrosion pit and had a maximum depth of 0.5 mm (Fig. 4). The general condition of the bar was good, with no evidence of gross surface pitting.

The evidence indicated that the fracture-originating defect was probably caused by a form of stress corrosion cracking. Samples of construction materials and ground-water were analyzed but the corrosive medium that caused the cracking could not be identified.

Material Evaluation

Mill certificates supplied by the steel manufacturer indicated that the rock anchor steel was a high carbon, high silicon type with a vanadium addition (type ST 1080/1230). Metallographic examination of the steel revealed a microstructure of nearly 100% pearlite. The yield strength of the rod material was 1100 MPa, which is 40% above the apparent stress in the rod when the failure occurred. Fracture toughness testing was therefore initiated to determine the material performance in the presence of surface defects.

Fracture toughness testing was completed on test samples of the steel bar cut from areas adjacent to the fracture surface. All testing was performed in air at room temperature (20°C). These tests showed that the fracture toughness value (K_{Ic}) for the rock anchor material was aproximately 30 MPa $\sqrt{m}$ in air.

The first broken anchor rod performed satisfactorily for a period of 13 months. At failure, the defect had a maximum depth of 0.86 mm and a circumferential length of 3.02 mm. No other data relating to the rate of crack growth is available. A defect analysis was next performed using the three-dimensional case of semielliptical surface defects in finite plates (Paris and Sih 1965). The maximum normal working stress of 690 MPa was used for the purpose of calculating the K_{Ic} value which corresponded to the measured critical defect size. For the measured critical flaw size in the field environment, the following equation applies:

$$K_{Ic} = [1 + 0.12(1-a_c b)] . \frac{\sigma(\pi a_c)^{1/2}}{\phi} . \left| \frac{(2t)}{\pi a_c} \tan \frac{(\pi a_c)}{2t} \right|^{1/2} \quad [1]$$

where a_c = critical defect depth = 0.86 mm; b = 1/2 defect length = 1.52 mm; σ = working stress = 690 MPa; t = bar diameter = 36 mm; ϕ = elliptic integral = 1.37. The value of K_{Ic} = from eq [1] is calculated to be 3.65 MPa $\sqrt{m}$.

A comparison of the K_{Ic} value determined in the laboratory by testing in air and that back-calculated for the site environment from the measured defect size on failed anchor rod thus showed an order of magnitude difference.

Remaining Life Estimates

The velocity of the growth of stress corrosion cracks depends on the environment, the stress intensity, and the material properties. Both the environment and the stress intensity could vary thoroughout the life of the bar. The nature of the cracking found immediately below the fracture face indicated that the cracks were growing in a stable fashion. Stress corrosion cracks of this type develop in three stages. Crack initiation is usually followed by a rapid growth over a very short period of time (stage I), followed by cracking at a steady crack growth velocity (stage II). The final stage comprises of unstable crack growth at a very rapid rate. The best estimate of crack growth velocity i.e. assuming that the defect grew at a constant rate, may be obtained from:

$$\nu = \delta \ell / \delta t \quad [2].$$

For t = 13 months; ℓ = 0.86 mm (initial defect depth), a crack velocity $\nu = 2.71 \times 10^{-8}$ mm/s is obtained. The value of K remains constant in a given environment. The critical crack size which would permit unstable fast fracture to occur can therefore be calculated for a variety of stress conditions, by using Eq.(1). To estimate the minimum remaining life, of the anchors the bars in the excavation can be proof loaded to a higher load after which they can be "locked off" again at the original working load. The survival of the bars during the proof loading process provides a direct confirmation that the defect did not yet attain the critical depth. It should be noted that the higher proof stress results in a smaller critical depth (Equation 1). The critical defect size at the working load is known from examination of the fracture surface. The difference between the measured value of defect size at the normal working load of 690 kN and the calculated value of the defect size at some higher proof stress can be transformed into time by dividing by the estimated stable crack velocity. As an example, for a working stress of 690 MPa, a critical defect size, a_c, of 0.86 mm was mesured in the field. If the stress during proof loading is increased by, say, 20%, then a new critical defect size can be back calculated for Eq.[1]. In this case, for the higher stress of 828 MPa, a defect size, a_c, equal to 0.66 mm is calculated. The difference in critical crack size is therefore 0.20 mm. Assuming that the rate of crack propogation remains constant at 2.71×10^{-8} mm/s as determined from Eq.[2], the additional time gained by proof loading to 827 MPa is given by 85 days (0.20mm/1.71×10^{-8} mm/s). The same argument can, of course, also be aplied to a decrease in stress level in the bar. A lower stress value will result in a larger critical defect size. The difference between this value and the measured critical defect size at the normal working load can also be transformed into a minimum time to failure.

At the time of the first anchor failure the stress level in the anchor rods could not be reduced since the floor slabs were not yet fully constructed, and were therefore not capable of resisting the ensuing increase in lateral stress. It was therefore decided that a statistically significant number of anchor rods should again be proof stressed above the normal working stress. As explained int he preceeding paragraphs, fracture toughness data was used to generate a proof load versus time gained curve as shown in Fig. 5. This curve was based on the average load of 700 kN initially reported for the rock anchor system. A total of 125 type I anchors were proof-loaded to 850 kN to obtain a minimum theoretical remaining life of 110 days. The work was resumed at the site with this knowledge.

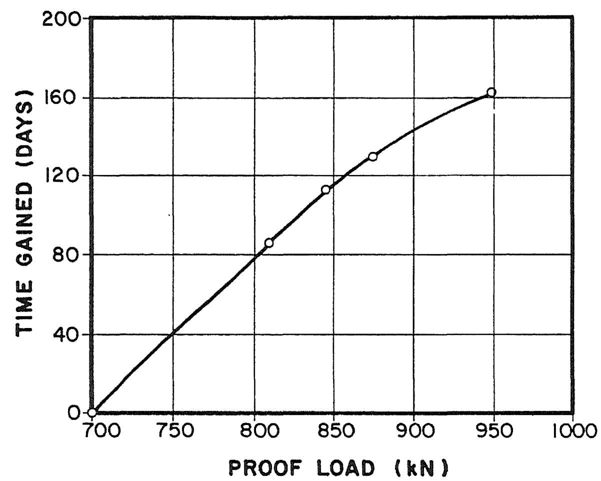

Fig. 5. Proof load versus time gained.

Approximately 70 days after proof loading was completed, a second rock anchor in the south wall failed suddenly. The fracture features were identical to those found on the first anchor failure. This anchor failed at the taped interface between the corrosion protection sleeve and the unprotected end of the rod immediately below the anchor plate. At the time the second failure occurred, it was learned that the type I anchor rods in the system were tensioned at loads varying between 620 and 838 kN (average value 700 kN). Since further failures appeared likely, it was decided to partially destress all type I rock anchors because some lateral load could now be carried by the lower floor slab and perimeter walls. The strength of the installed concrete substation walls allowed destressing to an equivalent load of 620 kN on all type I anchors. New calculations were made to estimate the time to be gained by both proof loading to 850 kN, and unloading to 620 kN immediately thereafter. Table 2 shows the calculated values of time gained versus the original recorded load in the bar. These results are shown graphically in Fig. 6.

Table 2. Remaining life estimates

Initial Load in bar (kN)	Critical Defect depth a_c (mm)	Proof Loading to 850 kN (1) a_c (mm)	Proof Loading to 850 kN (1) Time gained (days)	Unloading to 620 kN (2) a_c (mm)	Unloading to 620 kN (2) Time gained (days)	Total time gained (1) + (2) (days)
600	1.14	0.56	234	---	0	234
620	1.07	0.48	205	---	0	205
650	0.97	0.38	162	0.10	43	205
680	0.89	0.30	129	0.18	76	205
700	0.84	0.25	108	0.23	97	205
750	0.71	0.13	54	0.36	151	205
800	0.64	0.05	22	0.43	184	205
850	0.56		0	0.51	216	216
900	0.51		0	0.56	233	233

Two additional type I rock anchors failed during proof loading to 850 kN. Proof loading was an effective means of identifying anchor rods containing defects approaching the critical defect size. The combined total time gained by proof loading and partially unloading the type I anchor bars provided a minimum of 205 days of safe working time at the Cathedral Square site. No further rock anchor failures occurred during construction after partial unloading was completed. During final destressing of the rock anchors after all the concrete was placed, three tendons failed just below the anchor nut as the tensile load was being applied to the anchor for "lift-off."

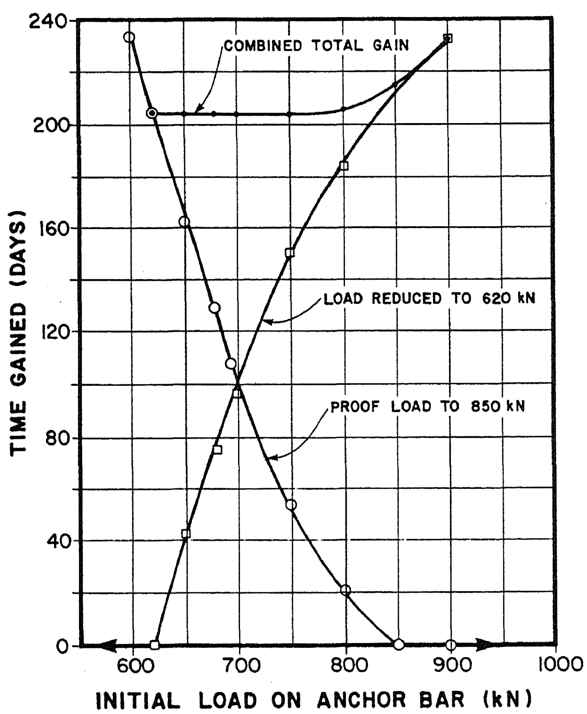

Fig. 6. Time gained by proof loading and load reduction.

Discussion

High tensile strength steels of the type used in this project are produced with a pearlite microstructure These steels have been developed to maximize tensile strength, but at the expense of toughness. As witnessed at this site, surface defects of seemingly insignificant depth can initiate catastrophic brittle failure in bars that are stressed to normal working stress levels. The bars at the site were provided with corrosion protection consistent with their temporary use, and yet failures occurred aproximately 13 months after installation.

It is important to emphasise, since it is not commonly appreciated, that high tensile strength post-tensing bars and accessories require an extraordinary care during transportation, storage and installation. Often the level of care demanded cannot be guaranteed even on well managed construction sites. For example, the specifications for such steels often contain requirements to the following:

- -Steel must be transported dry.
- -Any damage to the surface such as notches, abrasions, etc. must be absolutely avoided.
- -Steel bars must not be thrown or dumped from a truck.
- -The steel must be stored in a dry place, and sufficient ventilation must be provided to avoid condensation of water. In other words, direct contact of plastic sheet with steel is not permitted.
- -The steel bars must not come in contact with the ground during storage.
- -Hot welding sparks may initiate failure.

It is difficult to contemplate the enforcement of the above requirements, on an average construction project in North America.

Conclusion

1. After experiencing anchor failure, proof loading of anchors coupled with a careful examination of the fractured anchor bars was an effective means of determining the remaining life of the anchors.

2. The high strength steel of the type used in this project develops a high tensile strength at the expense of toughness. Hence, even minor defects on the surface of the bar can initiate catastrophic failure. Such steels require an exceptionally high level of attention during all phases of transportation, storage and installation. The Engineer must satisfy himself whether such level of care can be guaranteed on the project.

REFERENCES

Garga, V.K., Carey, E.I. and Milne R.W. (1984), "The Cathedral Square Substation Anchored Excavation, Vancouver, British Columbia". Canadian Geotechnical Journal, Vol. 21, No. 4, pp. 621-633.

Jurell, G. (1985), "Investigation into the Failure of a Prestressed Rock Anchor," Water Power and Dam Construction, February, pp. 45-47.

Paris, P.C. and Sih, G.C. (1965), "Stress Analysis of Cracks". Fracture Toughness Testing and its Applicatons, ASTM, Special Technical Publication,. STP 381, pp. 30-83.

Piteau, D.R. et al. (1979), "Rock Slope Engineering Reference Manual". Federal Highways Administration, Washington, D.C., Report No. FHWA-TS-79-208.