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The Behaviour Under the Influence of Soil Creep Pressure of the Concrete Bridge Built at Klosters by the Rhaetian Railway Company, Switzerland

Comportement sous l'action du fluage des terres du pont en béton construit à Klosters par la Cie des Chemins de Fer Rhétiques, Suisse

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Summary

In spring 1938 several cracks appeared in the reinforced concrete viaduct. This deformation was caused by creeping movement in the left side of the valley where one of the two abutments and approach viaducts are situated. After three years' study of the creep process (1939–51) a proposed structure for safeguarding the bridge was developed. The principal part of the reconstruction is a horizontal shore (stiffening beam) between the two abutments, designed to resist the estimated maximum value of creep pressure. It was fitted with a measuring device, which permits the recording of the gradual increase and the yearly fluctuations of the creep pressure. The purpose of the present paper is to record the results of measurements up to this time (1939–52) so far as creep movements and creep pressure are concerned, as well as the observations and experiments carried out on the rebuilt bridge and its surroundings.

Sommaire

Plusieurs fissures ont apparu dans le viaduc en béton armé de Klosters au printemps 1938. La cause de cette déformation est à rechercher dans le mouvement de fluage de la rive gauche de la vallée dans laquelle la culée ouest est fondée. Les résultats de l'étude des mouvements de fluage recueillis pendant une période de 3 ans (1939–41) ont permis d'élaborer un projet de construction assurant la stabilité et la conservation de l'ouvrage. L'élément principal de la reconstruction consiste en un étançon de béton armé reliant les deux culées, dont les dimensions ont été fixées de façon à ce qu'il puisse supporter la poussée maxima de fluage estimée. Cet étançon a été muni d'un dispositif de mesure permettant d'observer l'augmentation graduelle et les fluctuations saisonnières de la pression de fluage. Le présent article résume les mesures du mouvement et de la pression du fluage des terres effectuées à ce jour (1939–52), ainsi que les observations et expériences faites sur l'ouvrage restauré et le terrain avoisinant.

Introduction

A railway bridge in reinforced concrete was built over the river Landquart at Klosters in 1930 in accordance with a project of the late Engineer *R. Maillart* of Geneva. It consisted of a centre arch with stiffening girder of a span of 30 m and arch height of 7.90 m. The axis line of the railroad makes a horizontal curve with a radius of 125 m (Figs. 1 and 2). The approach spans on both sides are built as stiff frames and the foundations of all parts of the structure had to be built on talus. The left slope of the river bank consists of a loamy landslide mass that is rich in dolomite boulders and which is subject to creep (see also *Terzaghi*, 1951). The right bank of the river is formed of detritus material which has been deposited by the river Landquart (for general configuration of the terrain see Fig. 8).

In the spring of 1938, cracks caused by a movement of the left abutment appeared in most parts of the structure. The measurements taken at that time led to the conclusion that only slow movements were taking place (Fig. 1a). On account of the formation of these cracks and after the opening—with a chipping hammer—of an expansion joint 26 mm wide on the left bank between the stiffening girder and the girders of the stiff frames, the stresses in the structure were released. The expansion joint, however, closed immediately afterwards.

By the end of 1938, the management of the Rhaetian Railroad Company entrusted the Laboratory of Hydraulic Research and Soil Mechanics at the Swiss Federal Institute of Technology with the task of determining the extent and the direction of the deep-seated movements and also their cause.

For this purpose, the Swiss Federal Topographic Service installed a number of benchmarks on the structure itself and on the surrounding ground which were then periodically observed. In addition, two shafts were excavated below groundwater level for the purpose of determining creep movements by means of plumb lines (see Fig. 5). Proposals based on the observation of the creep movements during three years (1939–1942), were made for the reconstruction of the bridge, with a view to preserving the entire structure (*C. Mohr, R. Haefeli, L. Meisser, Fr. Waltz, W. Schaad,* 1947). The knowledge gained in Snow Mechanics has been of inestimable importance in the design, notwithstanding the fact that essential differences exist between the behaviour of snow and soil. Several publications containing references to the relation between snow and soil are mentioned in the paper quoted above.

The fact that the right bank was practically stable and that the left bridge abutment showed only relatively minor vertical movements led to the decision to adapt the structure to the movements of the soil. This was carried out by shoring up the moving left abutment against the adequately reinforced right abutment by constructing a horizontal pressure beam, while the foundation of the stiff frame on the left bank was protected by a horseshoe-shaped retaining wall (Fig. 1b).

We refer to the publication above mentioned in which the observations made up to 1946, the proposals for the reconstruction, as well as the principles of the calculations for estimating the expected beam pressures and also the geotechnical circumstances are described in detail. We are limiting ourselves in the present paper to the discussion of the measurements results which were regularly carried out until 1952.

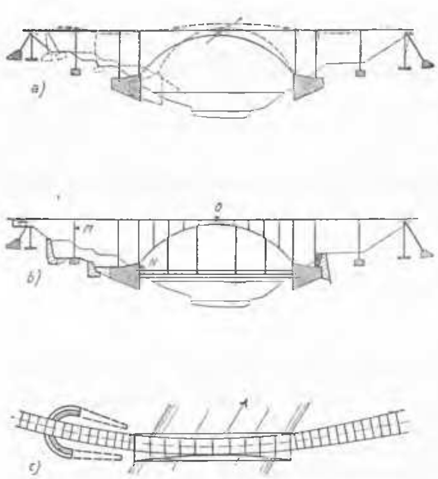

Fig. 1 Deformations of the Bridge, December 1938, Scheme (*a*) Strengthening Work in Longitudinal Section (*b*) and Plane (*c*)
Déformation du pont, décembre 1938, schéma (*a*), schéma des travaux confortatifs, coupe (*b*) et plan (*c*)

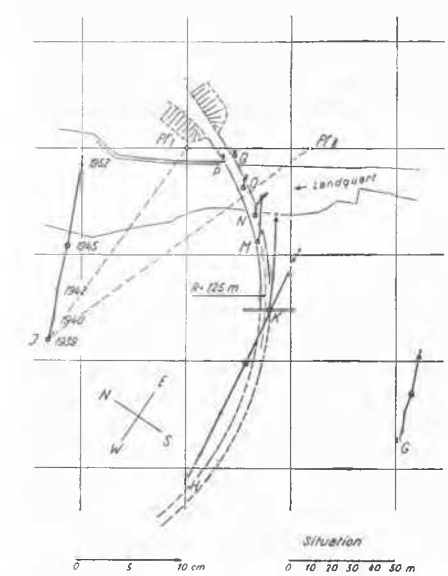

Fig. 2 Situation of Benchmarks (Pf_I and Pf_{II}) and Horizontal Projection of Measured Displacements 1938–1952
Situation des repères (Pf_I et Pf_{II}) et projection horizontale des déplacements mesurés de 1938 à 1952

Results of Measurements and Observations

Under the direction of *W. Lang* and Prof. Dr. *F. Kobold*, the Swiss Federal Topographic Service and later on the Geodetic Institute at the Swiss Federal Institute of Technology took altogether five control measurements over an observation period of 13 years. The first three measurements (points of time A^*, D^* and E^* 1938–1942) were taken before, and the last two (F^* and G^*) after the reconstruction of the bridge (see Figs. 3 and 4).

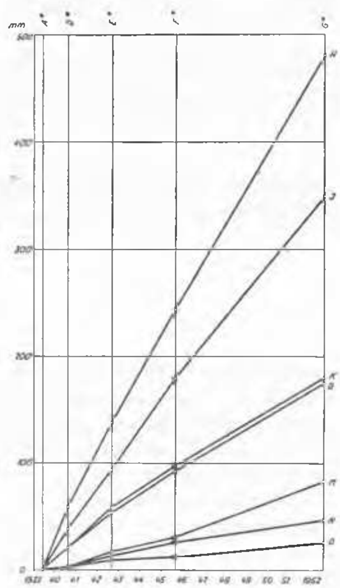

Fig. 3 Horizontal Displacements in Relation to Time
Déplacements horizontaux en fonction du temps

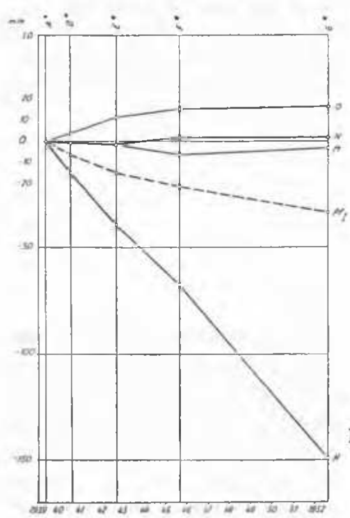

Fig. 4 Vertical Displacements in Relation to Time
Déplacements verticaux en fonction du temps

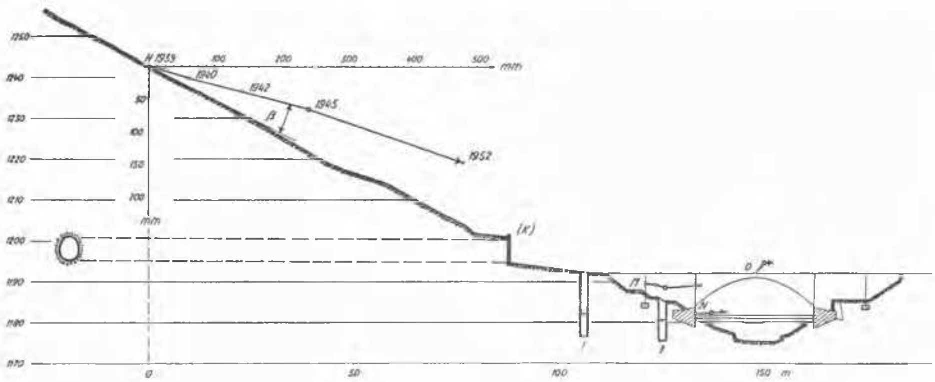

Fig. 5 True Displacement Vectors and Longitudinal Section (H–K–O, see Fig. 2)
Vecteur du déplacement réel et profil du terrain (H–K–O, voir Fig. 2)

The situation of the benchmarks and the horizontal projections of the measured displacements are visible from Figs. 1 b and 2. The components of the displacements in the horizontal and vertical directions are represented in Figs. 3 and 4 in function of time and the true displacement vectors of some characteristic points are given in the longitudinal section in Fig. 5.

The measurements show that during the entire observation period, the movements of the various groundpoints in the horizontal projection (G, H, J and K) and of the points on the bridge (M, N and O) practically all took place in the same direction. As can be seen in Figs. 3 and 4 it is furthermore evident that the velocity of creep of all points on the ground was practically constant for the duration of the measurements. Thus the reconstruction has had no noticeable effect on creep movements in the vicinity of the bridge.

The biggest horizontal displacements were observed at the highest point on the ground H. In the course of 13 years this point moved a total distance of 480 mm, i.e. 37 mm per year. The vertical displacement of benchmark H is considerably smaller and during the observation period was displaced only 14.9 mm. The true movement thus took a flatter course than that of the ground-surface at this profile (angle of creep negative, see Fig. 5). The measurements carried out in 1939–1940 with plumb lines, which reached as far as 12 m below ground surface, showed that the velocity of creep was fairly evenly distributed over the entire depth of the observation shaft. This led to the conclusion that the stable subsoil must be at a relatively great depth and that underpinning of the bridge foundations was out of the question. The fact that this creep is a deep seated movement is evidenced by the deformations and effects of pressure which had occurred meanwhile, in the near-by tunnel, Figs. 2 and 5. The fact that the tunnel is acting as a drainage channel and is thus exercising a favourable effect upon the stability of the slope has to be taken into consideration. More recent observations have confirmed the earlier interpretation, according to which the creeping movement of the landslide masses which is ending in a wedge downhill and is subject to compression, is similar to the movement of a glacier tong (without ablation = wear of glacier surface by climatic action).

The benchmarks established on the structure on the left side of the valley, show considerably smaller displacements than the points on the ground. They are moving mainly in a horizontal direction and, as expected, the moment of closure of the pressure beam between the two abutments of the arch, which took place on the 4 November 1944, is very apparent.

The benchmarks P and Q on the right abutment remained practically unchanged until 1945 and show only in the last 6 years, i.e. since the increase of pressure in the pressure beam, a horizontal displacement of 6–7 mm. Benchmark 0, which during the years 1939–1942 showed clearly a raising of the apex of the arch, has shown since the completion of the reconstruction work, a slight horizontal displacement which corresponds closely to that of the right abutment.

By 1946 sliding planes could clearly be observed beside the left abutment, which indicates that the landslide masses were creeping underneath the abutment.

Determination of the Beam Pressure

In order to check up the safety of the structure and to verify the calculations which had been carried out for the purpose of estimating the pressure expected on the pressure beam, an attempt was made to establish the actual pressure on this beam. The stresses on the cross beam were established by an indirect method based upon the measured compression of the concrete. The method of these measurements was published in the paper mentioned above. They were carried out in such a way that only the disturbing effects of temperature changes and of shrinkage were eliminated but not the effects caused by the creep of the concrete itself.

Under the direction of Prof. Dr. *M. Roš* and Dr. *A. Vcellmy*, the Swiss Federal Institute for Testing Materials made parallel tests with test-prisms for the purpose of estimating this effect. For the concrete used in the pressure beam (mix *PC* 300, crushing strength of the prisms 20/20/60 cm on the 28th day: W_d = 268–320 kg/cm²; on the 180th day: W_d = 352–384

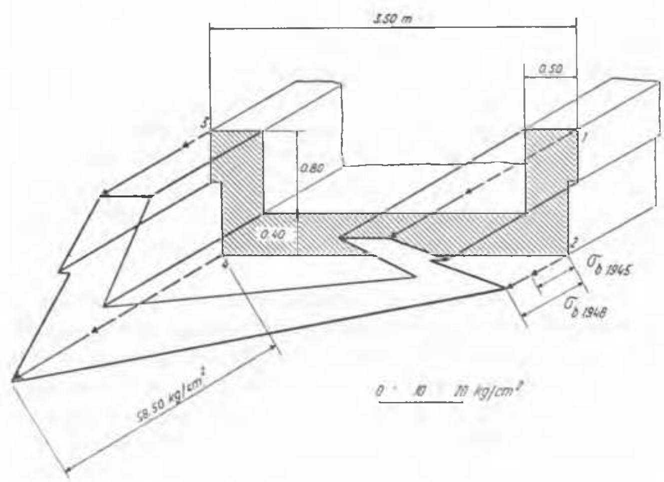

Fig. 6 Stress Distribution over the Cross Section of the Beam at the Moment of Maximum Pressure (V_b = 200.000 kg/cm²) August–September 1945 and 1948 (see Fig. 7)
Répartition des contraintes au moment des poussées maxima dans le verrou (V_b = 200.000 kg/cm²), août–septembre 1945 et 1948 (voir Fig. 7)

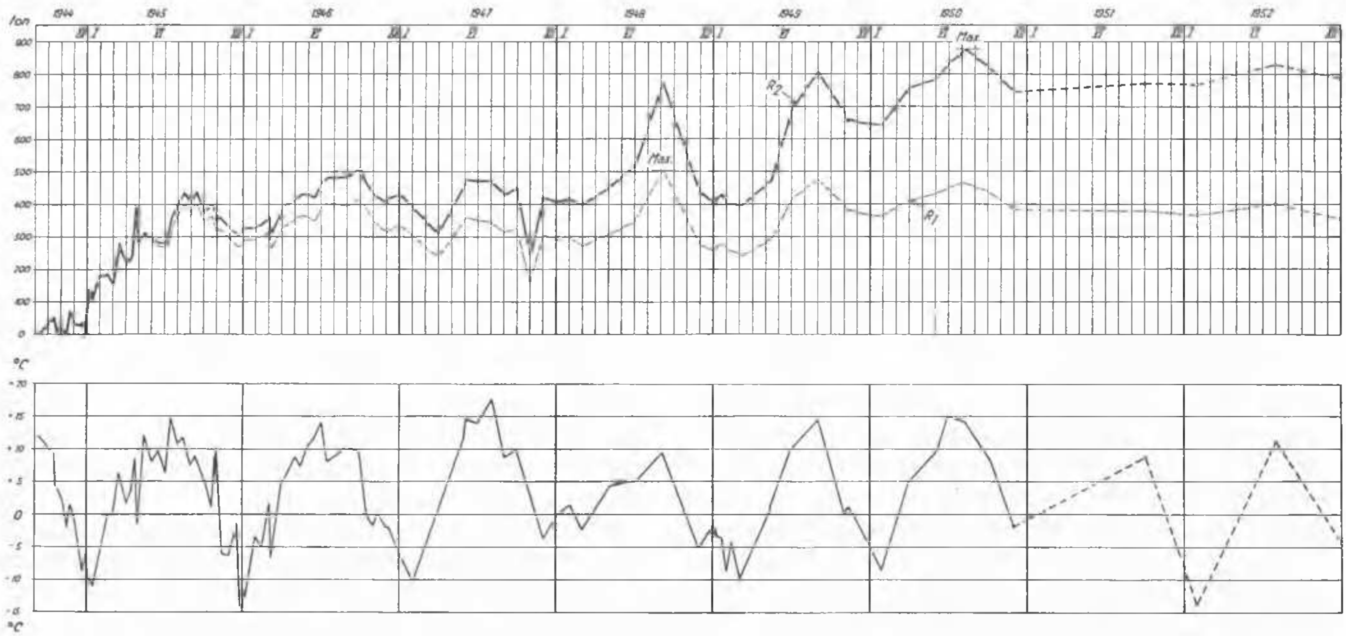

Fig. 7 Diagram of Measured Beam Pressure and Average Air Temperature. Curve R_1 for a Variable Deformation Modulus V_b of the Concrete (Creeping) and Poisson Coefficient $m = 5$, R_2 for $V_b = 200{,}000$ kg/cm², $m = 5$
Diagramme des poussées mesurées dans le verrou et de la température moyenne ambiante. R_1 pour un coefficient de déformation du béton V_b variable (fluage) et coefficient de poisson $m = 5$, R_2 pour $V_b = 200.000$ kg/cm², $m = 5$

kg/cm²; bending strength = 30–52 kg/cm²) under a constant compression stress of 45 kg/cm², the following reduction of the deformation modulus V_b in relation to time was obtained:

Age of concrete in days	*Deformation modulus V_b in kg/cm²*
1	215,000
190	200,000
1,000	150,000
1,750	120,000

Taking these values into consideration and based on deformation- and temperature measurements taken during approximately 8 years, the course of the beam pressure as represented by the curve R_1 in Fig. 7 is obtained. The actual beam pressure may be slightly higher than curve R_1, because the respective deformation for an average pressure below 45 kg/cm² diminishes less in relation to time than was established in the tests. On the other hand, if the creep of the concrete is partly neglected, by calculating with a constant deformation modulus $V_b = 200{,}000$ kg/cm² (see *Mohr*, *Haefeli*, *Meisser*, *Waltz*, *Schaad*, 1947), a rather too high beam pressure R_2 is obtained. The actual beam pressure can therefore be assumed to lie between the two boundary curves R_1 and R_2 given in Fig. 7.

The measured stress distribution over the cross-section of the beam at the moment of maximum pressure in August 1945 and 1948 is given in Fig. 6 for $V_b = 200{,}000$ kg/cm² (R_2). The torsional stresses, which are created on account of the elastic suspension of the beam on the main arch and on account of the effects of temperature variations, are clearly visible.

As had been expected, the pressure in the beam increased gradually with yearly fluctuations and hitherto reached a maximum value in August/September, diminishing again with lower temperatures. After the sliding planes had become established round the abutment (observed beside the abutment 1946) it seems that a certain period of inactivity followed as can be concluded from the pattern of the curves R_1 and R_2 in Fig. 7. The maximum pressure amounts to 500 tons, respectively 880 tons, in accordance with the deformation modulus used in the calculations (curve R_1 1948, respective R_2 1950). The cross beam was designed to resist a pressure of 1,650 tons at the centre of gravity of the cross-section, corresponding to a top pressure of 90 kg/cm².

The evaluation of this theoretical maximum beam pressure of 1,650 tons was based on the assumption that beside and underneath the left abutment, which is embedded in the creeping slope, a sliding plane is developing. For the apparent friction angle acting in the sliding plane (i.e. along the underside of the abutment foundation of the arch) a top limit of $\varphi_s = 45°$ was assumed (tg $\varphi_s = 1.0$). The comparison between calculation and actual observation shows that after the establishment of the sliding plane (1946), the determining coefficient of friction is considerably smaller than 1.0 and is fluctuating between approximately 0.3 and 0.6 according to the value of the deformation modulus. The explanation for this relatively small friction value may be found among other things in the exceptionally small sliding- or creep velocities. As more recent investigations show (*A. Casagrande* and *S. Wilson*, 1951), this statement must not be generalized.

Conclusions

The observations and measurements made between 1944–52 on the strengthened bridge over the river Landquart show that it is possible—under favourable circumstances—to control the creep pressures acting upon individual parts of a structure. In order to arrive at a constructive solution where safety and economy factors are concerned, it is however desirable, if not necessary, to take creep measurements over a considerable period of time which must precede the planning.

It is advisable to promote the formation of favourable sliding planes between the fixed structure and the creeping subsoil by structural means.

Fig. 8 General View: (1) Bridge; (2) Slide Area; (3) Airal Railway Gotschna; (4) Railway
Vue générale: (1) pont; (2) zone de glissement; (3) téléférique de Gotschna; (4) chemin de fer

The evaluation of the creep pressures can be made based on relatively simple assumptions. The measurement of the creep pressures in the completed structure shows, with the exception of yearly fluctuations caused by temperature variations, a slow increase of the pressure up to a maximum value, which in the case discussed above was reached after approximately 5 years. If such a measurement is based upon the deformation of parts made of concrete or reinforced concrete, it is then necessary to consider also the effect of the creep process in the construction material itself (concrete). This point is also of a certain importance in the design, calculation and observation of concrete dams, especially for arch dams.

Acknowledgment

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