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THE FOUNDATION OF THE CROZET BRIDGE. A CASE STUDY

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Abstract: The Crozet bridge is located on the Grenoble-Col du Fau motorway, in the Grenoble–Sisteron intinerary, fifteen kilometres south of Grenoble, Frace. It crosses a 350 m wide valley and the RN75 national road. The adaptation of the bridge to the lanscape has involved an arch design with three bays (direction of Grenoble–Sisteron) and one bay (direction of Sisteron–Grenoble). The supports of the structure were difficult to build because of a huge horizontal force and a low displacement tolerance. The low stiffness and strength characteristics foreseen led to a geotechnical investigation by cyclic pressuremeter tests with a friction angle and cohesion interpretation. The foundation calculations were carried out by the CESAR-LCPC program to determine the support rigidity. A complete computation of the bridge was done with the calculated support rigidity which showed that displacements of the arches were lower than the tolerance limit. The bridge is located in the area of low seismic activity and the design takes into account the maximum foreseeable magnitude. The soil liquefaction risk was analysed as well.

Monitoring carried out for completion of the bridge in 1999 and along the surveying shows displacements lower than tolerance values. Since 1999, the bridges has withstood huge service weights without any difficulty.

LIST OF SYMBOLS

a – radius of the borehole,
$a_{\max}$ – static horizontal acceleration equivalent to a seismic excitation,
a_N – nominal acceleration,
C_r – correction coefficcient from Seed function of K_0,
c_u – undrained cohesion,
E_M – pressuremeter modulus from the Ménard test,
E_e – Young's modulus,
F – safety coefficient,
Φ' – effective friction angle,
Φ_μ – interparticle angle of friction,

g – acceleration of gravity,
γ – unit weight of soil,
G – shear modulus of the soil,
h – depth of the sample,
k_{hx}, k_{hy}, k_s, k_v, $k_{\theta 1}$, $k_{\theta 2}$ – stiffness of the soil subjected to displacements and rotations,
γ_{sat} – unit saturated specific weight,
n – number of cycles
p – pressure applied at the borehole wall by the pressuremeter,
p_l – limit pressure of the Ménard test,
Ψ – dilatancy angle,
r_d – reduction coefficient from the Seed function of depth,
σ'_d – effective deviatoric stress,
σ'_3 – lateral stress in triaxial test,
σ'_{v0} – effective initial vertical stress,
τ_{eq} – equivalent shearing stress,
τ_l – cyclic shearing stress,
u_a – displacement at the borehole wall,
z – depth of a pressuremeter test or the sample,
V, H, M – forces applied to the foundation.

1. INTRODUCTION

The Crozet bridge is located on the A51 motorway Grenoble-Col du Fau along the Grenoble-Sisteron itinerary, fifteen kilometers south of Grenoble. It spans the distance of 350 m across a small valley where the national road RN75 is built. An embankment was initially foreseen, but a bridge has now been built with two separate decks lying, for the first one, on a single arch (Eastern way Sisteron–Grenoble) and for the second one, on three arches (Western way Grenoble–Sisteron). The arch design was chosen, from an architect point of view, in such a way that the bridge could be integrated with a natural site of the Crozet valley. Establishing the foundations on simple supports is a conventional method (drilled piles), but the design of the foundations of the arches is rather complex due to a low stiffness and low strength characteristics of the soil.

This paper presents an original geotechnical approach to the soil investigation, the design of the foundation, design of the bridge, which takes into account the displacements of the structure and the soil, and the arrangements used for the foundation construction. The measurements of foundation displacements are made in order to monitor the behaviour of the bridge in this seismic area.

2. DESCRIPTION OF THE BRIDGE

The Crozet bridge (figure 1) runs from northeast to southwest and is made up of two decks, i.e. 313 m eastern deck and 335 m western deck (figure 2). The arch design is chosen in such a way that the bridge is integrated with a natural site of a small Crozet valley. Each deck supports three lanes. This paper is focused on the eastern deck (figure 3) made up of spans whose length ranges from 13 to 20 m (locally 28 m for RN75 crossing) and whose radius of arch reaches 120 m, and the length is 143.5 m. This deck is supported by two end columns and six central columns, each 1 m in diameter. In the case of the western deck, land surveying and architectural control led to a three-arch design: two arches are 101.5 m in length, and the third is 87 m in length. This deck is supported by 6 and 5 small columns. A total height of the bridge is 30 m above the Crozet stream.

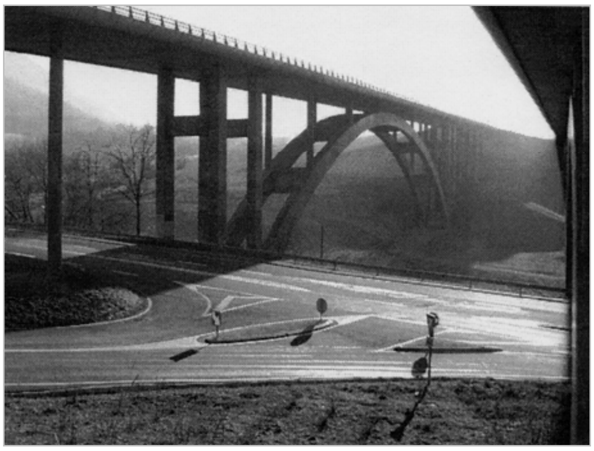

Fig. 1. General view of the bridge

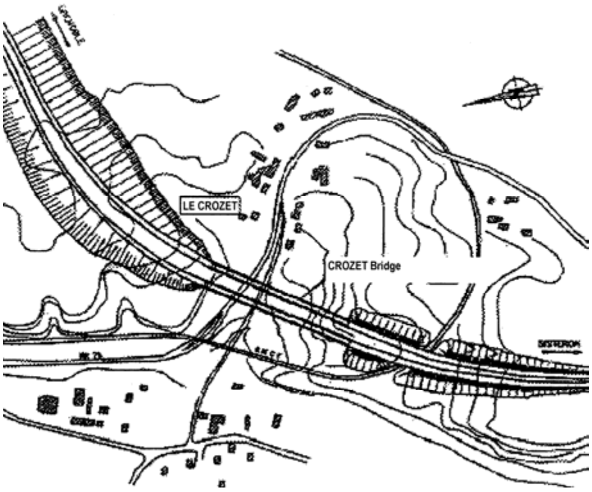

Fig. 2. Plane view of the bridge

3. GEOTECHNICAL ENVIRONMENT

3.1. GEOLOGICAL AND GEOTECHNICAL ENVIRONMENT

The geology of the Crozet valley is quite simple. It is a fluvial-glacial terrace deposit (figure 3), which is modelled by a thalweg with gentle slopes. At the bottom a small stream is flowing. The site is classified as the area of low seismic activity.

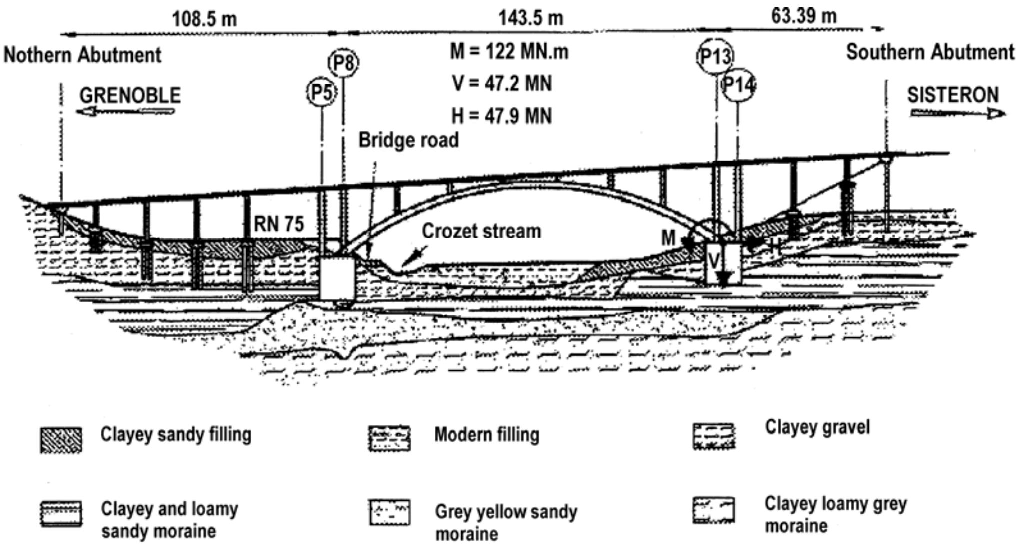

Fig. 3. Geological situation and elevation view of the bridge

Different investigations reveal quaternary alluvial and glacial deposits whose thickness exceeds 10 m. The bredock is made of "black marl" and has never been reached by the boreholes of 40 m depth. Recent deposits are made up of gravely clay and modern alluvial deposits found at the depth ranging from 7 to 10 m. The former alluvial and glacial deposits can be categorized as four different families of soils, depending on their depth:

• Family F1: *clayey gravel* with sandy levels of decimetre thickness. The thickness of this loose layer ranges from 8 to 15 m. This thickness is 2 m in the thalweg due to channeling of the recent alluvial deposits. This stratum forms the part of the Würm Formation.

• Family F2: *clayey and loamy sandy moraine*, grey, very stiff, with scarce sandy and gravely levels. This formation is chanelled by the family F1, and its thickness is differentiated.

• Family F3: *grey yellow sandy moraine* with some clay; it is very stiff and its thickness varies from 15 to 8 m from north to south; this soil disappears under the south side of the thalweg.

• Family F4: *clayey loamy grey moraine*, very stiff, overconsolidated, of thickness greater than 25 m, very homogeneous.

The last three levels are the part of Riss Formations.

3.2. RESULTS OF INVESTIGATION AND TESTS

Investigations comprised a lot of *in situ* and laboratory tests:

• Sixteen classical pressuremeter boreholes.

• Three boreholes with undisturbed samples for laboratory tests.

Two boreholes were drilled in order to carry out fifty-nine high-pressure pressuremeter tests with three cyclical loadings undertaken in the four old formations (F1–F4). Several triaxial tests were performed to measure the shearing characteristics of the soils. The statistical analysis allows determination of mean values shown in table 1.

Table 1

Results of the Ménard pressuremeter tests

Deposit	Family of soil	Description	E_M (MPa)	P_l (MPa)
Recent deposits		Man deposit filling	3	0.3
		Colluvium	5	0.5
		Modern alluvial deposits	6	0.7
Old deposits	F1	Clayey gravel	30	2
	F2	Clayey and loamy sandy moraine	60	5.4
	F3	Grey-yellow sandy moraine	100	6.5
	F4	Clayey loamy grey moraine	60	6.2

3.2.1. TRIAXIAL TESTS ON DRAINED CONSOLIDATED SOIL SAMPLES

Tests were carried out in the LIRIGM laboratory of the Joseph Fourier Universty in Grenoble on remoulded samples, which match to the *in situ* effective lateral pressure with a pore pressure of 100 kPa. There was a lateral drainage during the consolidation phase. Dimensions of the samples are as follows: 70 mm in diameter and 150 mm in height, without any antifriction system on each end. The samples tested were drained at a speed of 0.01 mm/mn. The final consolidation was reached in 90 minutes and the test lasted for 22 h, which was 15 times as long as the consolidation duration. The measurement of the volume variation is done based on the measurement of the inner volume of the sample. The results of triaxial tests are shown in table 2, and physical characteristics of samples are given in table 3. The interparticle angle of friction is the minimum value of the friction angle. It allows measurement of the friction between two different particles of soil. The Revised Public Road System classification (HRB classification) is given in table 3.

Table 2

Results of triaxial tests conducted on intact samples of drained consolidated soils

Family of soil	Description	Depth (m)	Number of tests	Φ_μ (degree)	c_u (kPa)	Φ (degree)
F2	Loamy sandy moraine (Riss)	10	3	33.8°	7	41.5°
F2	Clayey sandy moraine stiff (Riss)	15.7	3	21.5°	174	24°
F3	Grey-yellow sandy moraine (Riss)	29	3	29.5°	0	37.5°
F4	Clayey loamy grey moraine (Riss)	31	3	31.9°	4	38.5°

Table 3

Results of a physical classification of soils

Family of soil	Description	Depth (m)	Per cent of soil passing through 75 μ sieve	Classification HRB	Classification USCS
F2	Loamy sandy moraine	12.5	90.5	A6	CL
F2	Clayey sandy moraine	20.8	94.3	A6	CL
F3	Grey-yellow sandy moraine	25	30	A2-4	SM
F3	Grey-yellow sandy moraine	32.5	33.5	A2-4	SM
F4	Clayey loamy grey moraine	32	55.2	A6	CL

3.2.2. CYCLIC PRESSUREMETER TESTS

Pressuremeter tests were carried out with a slotted tube of 63 mm external diameter and 1090 mm length; the slots are 915 mm in length, and the thickness of the tube reaches 2 mm. The interpretation of the tests was carried out using the Gaiatech patent [6], which takes into account the corrections of volume and pressure based on the French standard [14], but also takes into account the influence of the shape of the probe under pressure (FAWAZ et al. [4]), the non-uniformity of the pressure distribution along the probe (BASUDHAR and KUMAR [1]), and the difference between the external and internal radii of the probe.

The process leads to the determination of the mechanical characteristics in four different steps:

- Determination of the angle of interparticle friction in the samples (MONNET, GIELLY [10]) of soil that relates to the results of triaxial test on consolidated drained soil samples. The measurement of the volume variation is carried out.
- Determination of the elastic shearing modulus along with the unloading–

reloading cycle of the pressuremeter test. The cycle is carried out in the so-called «linear» range of the soil behaviour of the pressuremeter test.

• Determination of the angle of internal friction of the soil by measuring the slope of the straight line representing the relation between logarithms of the pressure and the radial strain of the pressuremeter results (MONNET, KHLIF [12], MONNET, CHEMAA [13]).

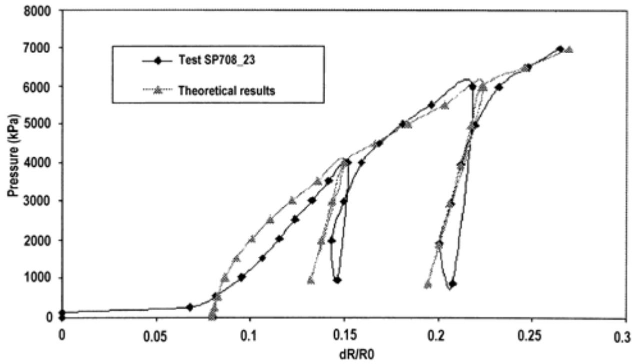

Fig. 4. Control of mechanical characteristics by comparison between experimental and theoretical curves: test 23 m depth into borehole SP708

• Control of the mechanical characteristics of elasticity (shear modulus) and strength (cohesion of friction angle). The GaiaPress program [5] is used to check whether the comparison between experimental and theoretical pressuremeter curves (figure 4) as well as experimental and theoretical limit pressures is correct. The shear modulus value is controlled by the correspondence between experimental and theoretical cycles. The values of friction angle and cohesion are controlled by the correspondence between experimental and theoretical pressuremeter curves above the creep pressure. The theoretical curve representing the granular soil assumes its elastoplastic behaviour, with effective stress, a non-standard dilatancy and a three-dimensional equilibrium (MONNET, KHLIF [12]):

$$\log\left[\frac{u_a}{a}.(1+n)-C_1\right]=\delta.\log(p)-\delta.\log(\gamma.z)+\log\left[(1-K_0).\gamma.z\frac{(1+n)}{2.G}-C_1\right] \quad (1)$$

with

$$\delta=\frac{1+n}{1-N} \quad \text{and} \quad C_1=\frac{n.\left(\frac{u_a}{a}\right)(1+n)\left(\frac{\gamma.z}{p}\right)^{\delta}+(1+n)\,(N-K_0)\,\frac{\gamma.z}{2.G}}{1+n.\left(\frac{\gamma.z}{p}\right)^{\delta}}, \quad (2)$$

with

$$N = (1 - \sin\phi')/(1 + \sin\phi') \tag{3}$$

and

$$n = (1 - \sin\Psi)/(1 + \sin\Psi)\,, \tag{4}$$

$$\Psi = \Phi' - \Phi_u\,. \tag{5}$$

The theoretical curve representing the cohesive soil assumes its elastoplastic behaviour, with total stress, no volume variation when plasticity occurs and a three-dimensional equilibrium (MONNET, CHEMAA [13]):

$$\log\left[\frac{u_a}{a} + \frac{c_u}{2.G}\right] = \frac{p}{c_u} - \frac{\gamma.z}{c_u} + \log\left[(1 - K_0).\frac{\gamma.z}{2.G} + \frac{c_u}{2.G}\right]. \tag{6}$$

The results of pressuremeter test are shown in table 4. In the fluvial-glacial deposit, the value of the ratio of elastic modulus to pressuremeter modulus (E_e/E_M) ranges from 1.37 to 4.77, amounting on an average to 3.07. The ratio is 3.31 in the grey clay and 2.06 in the grey sand. The value of elastic modulus is high and greater than 80 MPa. The undrained fluvial-glacial deposits and grey sand moraine (F1 and F2) have a huge cohesion without friction angle. Interpreting the results obtained for the family F3, we assume lack of cohesion, and in all cases the shearing strength of the soil is calculated with a friction. For the grey sand, this leads to a friction angle of 40° to 45°, which is in the range of the values expected.

Table 4

Results of cyclic pressuremeter tests

Family of soil	Description	Elastic modulus E_e (MPa)	Ratio of E_e/E_M	c_u (kPa)	Φ (degree)
F1	Clayey gravel	150–200	3.07+/–1.70	900–1200	0
F2	Clayey sandy moraine, very stiff	80–150	3.31+/–0.88	600–1000	0
F3	Grey-yellow sandy moraine, stiff	150–220	2.06+/–0.67	0	40°–45°

3.2.3. CYCLIC TRIAXIAL TESTS

These tests were conducted in the LIRIGM laboratory of the Joseph Fourier University in Grenoble. Samples from the family F2 were selected to study liquefaction under seismic excitation. The "Association Française de Génie Parasismique" (AFPS) calculation was followed to define the test procedure.

Samples, 70 mm in diameter and 140 mm in height, were used. The cell is connected to a cyclic hydraulic press of 20 kN maximum load. This press is connected to two hydraulic systems, one for the axial force, the other one for the lateral pressure of the triaxial cell. For a constant lateral pressure, the number of cycles, axial displacement, axial force and pore pressure are recorded.

The bridge is located in the area of a low seismic activity, but is classified as the category C: construction with a high level of risk due to frequent use and huge economic importance. This leads to a nominal acceleration a_N equal to 2 m·s^{-2} (tables 5 and 6).

Table 5

French classification of the site effect

Class of the site	I_A	I_B	II	III
a_N (m/s^2)	1.5	2.0	3.0	4.0

Table 6

French classification of by building type for I_B class

Under class for I_B	*B*	*C*	*D*
a_N (m/s^2)	1.5	2.0	2.5

The current sample is tested under the equivalent seismic shearing:

$$\tau_{\text{eq}} = (2/3).\gamma_{\text{sat}}.z.(a_{\max}/g).r_d = 36 \text{ kPa}, \tag{7}$$

$$[\tau_1]_n = C_r.[\sigma'_d / 2.\sigma'_3]_n.\sigma'_{v0} = 78.3.[\sigma'_d / \sigma'_3]_n \text{ kPa} \tag{8}$$

with C_r and σ'_{v0} equal to 0.6 and 261 kPa, respectively.

A safety coefficient *F* is defined as the ratio of the laboratory stress, which leads to liquefaction phenomena, to the equivalent stress calculated at *n* cycles:

$$F = \frac{\sigma_d}{0.459\sigma}. \tag{9}$$

The experimental conditions are:

$$\sigma'_3 = 100 \text{ kPa}, \quad \sigma'_d = 45.9.F \text{ kPa}, \text{ with a 0.5 Hz frequency.} \tag{10}$$

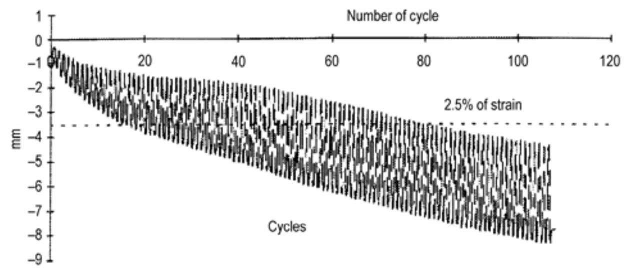

Fig. 5. Results of triaxial liquefaction tests: displacements versus number of cycles

We have carried out a test with σ'_d equal to 58.4 kPa. In the case of a liquefaction phenomenon that appears in the fifth cycle, the safety coefficient is 1.27. If a deformation criterion (2.5% of strain) is applied (figure 5), liquefaction does not appear in the first five cycles (17 cycles). The risk of liquefaction is consequently very low.

4. FOUNDATION DESIGN

A general design of the bridge was carried out in such a way as to integrate the foreseeable stiffness of the foundation into the structural analysis (according to an organization of the calculation indicated in figure 6).

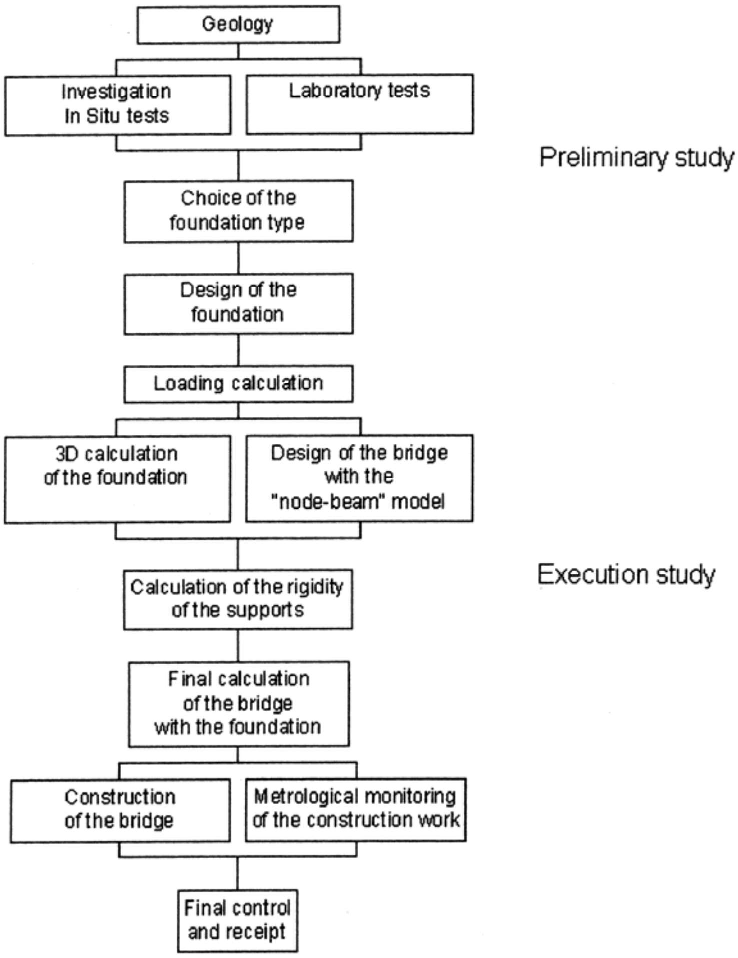

Fig. 6. Hierarchical organisation used for the bridge design

4.1. GENERAL DESIGN OF ARCH FOUNDATIONS

Preliminary studies in order to design the principal supports of the arch led to massive and rigid foundations made of a unit of orthogonal bars organised in a box with three webs. The continuity of the reinforcements had to be ensured. This solution was regarded as the basis of Construction Company tenders for the project. As deformation of the structure had to be limited, it was necessary to employ a very rigid foundation. The solution of inclined large piles was not studied because it is technically difficult to drill such foundations at 40° of slope.

As there was no major hydraulic constraint, the Construction Company proposed foundations resting on elliptic boxes. For the arch of the eastern deck, the boxes are 13 m in length, 10 m in width at a maximum depth of 16 m. These hollow boxes are filled with soil to ensure stability and rest on a reinforced concrete raft foundation, 2 m thick (figure 7). The creep effect was taken into account by the reduction of an elastic modulus of soil.

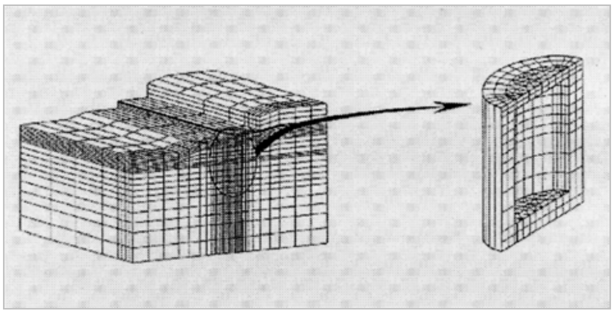

Fig. 7. Mesh used for finite element calculations and the mesh used for the foundation well

The well foundation excavated by 1 m steps is followed immediately by a reinforced concrete lateral support and an elliptic truncated cone formwork. After installation of a reinforcement cage, the second form is placed in order to make the final walls 2 m thick. After being filled with granular soil, a concrete roof slab, 1m thick, is poured to close the foundation box.

A cement grouting system with pipes uniformly distributed perpendicular to the lateral supporting walls permits improvement of the contact between the surrounding soil and the foundation. For the supports that receive the force of a single arch, the horizontal component is very large and equal to the vertical component (close to 48 MN for support P13/P14 of eastern deck) with a stabilizing moment of 122 MN.m for the permanent service load. The arch is settled on the foundation with a joint that allowed prestressing of the arch by jacks. The contact between the arch and the foundation is protected by reinforced cables which can be removed in the case of modification of the prestressed efforts of the arch. The elliptical shape of the foundation improves its rigidity as the long axis of the ellipse is in the direction of the horizontal component. The aim is to keep horizontal displacement within a two-centimeter threshold.

4.2. ORGANIZATION OF THE CALCULATION

The method of the bridge structural calculation used in general seismic calculation of the bridge requires knowledge of the stiffness matrix of the soil mass around the foundations. The model suggested by the Construction Company for the foundation of the P5/P6 supports of the eastern and western arch decks is an elastic «node-beam», where the elliptical well foundation is modelled (figure 8) by a vertical beam (along the z-axis) and two orthogonal horizontal beams (along the x- and y-axes). Along the vertical beam, there are continuous horizontal springs k_{hx} and k_{hy} so that the horizontal reactions are produced along the body of the well foundation. At the ends of the horizontal beams, there are vertical springs of the rigidity k_s that represent the side friction generated along the side of the well foundation. A vertical spring of rigidity k_v is placed at the lower end of the vertical beam to represent the ground vertical stiffness under the foundation, reduced by the stiffness k_s of four springs. The model also has local differential springs of the stiffness $k_{\theta 1}$ and $k_{\theta 2}$ to represent the difference in vertical pressure due to the anchorage along the foundation; they are linked to the rotation of the horizontal beams.

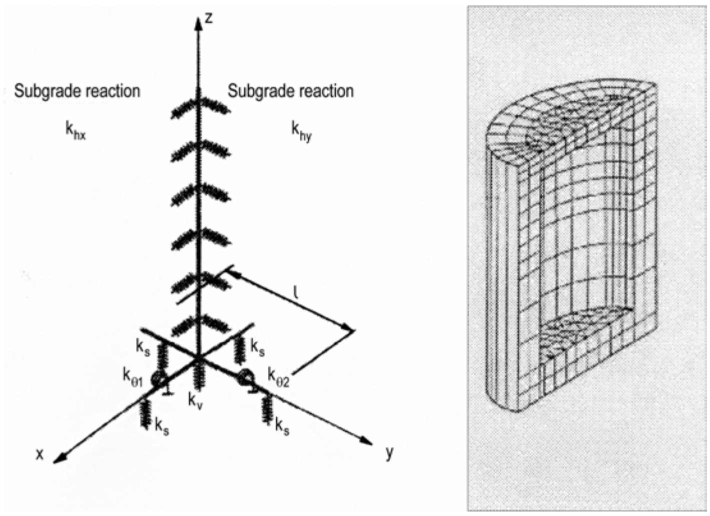

Fig. 8. The node rod model used for the foundation calculation

A 3D model that is used for the foundation support is calibrated based on finite element calculation (figures 7 and 8) with the LCPC CESAR program. The calibration of the «node-beam» model is carried out in the following manner:

• $k_{\theta 1}$ and $k_{\theta 2}$ are calculated from the model of axial rigidity of a deep foundation (Fascicule 62, 1993) and from the inertia of the foundation base (elliptic surface).

• k_{hx} and k_{hy} are calculated from the results of pressuremeter tests $E_M(z)$ (Fascicule 61, 1971).

• k_s is found based on the calculated settlement of the foundation well, by the finite element program CESAR-3D.

• k_v is found based on the calculated result of the rigidity at the top k_{v0}, by the finite element calculation, reduced by the rigidity k_s:

$$k_v = k_{v0} - 4.k_s. \tag{11}$$

• The length of beam l is based on the results of calculation CESAR-3D so that a similar rotation is obtained.

Taking into account a huge sensitivity of this type of structure (arch bridge) to the foundation stiffness, the latter was investigated by means of parametric calculations with characteristics raised by 30% (stiff ground) or undervalued by 30% (soft ground).

4.3. RESULTS

4.3.1. THE CESAR 3D MODEL

The model of the southern arch foundation was 45 m in depth, 110 m in width and 110 in length comprising more than 6500 elements (LAC [9]). The linear elastic calibration of the «node-beam» model was carried out without contact elements. However, the model did include an intermediate layer between the external side of the foundation and the soil in order to take into account decompression of the near-field soil due to excavation of the well foundation.

A 3D finite simulation allows determination of the elastic settlement of the foundation and its rotation. The settlement obtained for this support is 7 mm in the center of the foundation, with a rotation of $6 \cdot 10^{-4}$ rad (settlements at the two ends of the box are 2 mm and 10 mm). The ground under the raft is under compression (σ_{max} = 450 kPa) except the end of the long axis where tension of 50 kPa occurs. Complementary parameter calculations show that the separation is sensitive (extent and value of the tensile stress) to an increase in the stiffness of the soil under the raft.

4.3.2. THE «NODE-BEAM» MODEL

An explicit structural «node-beam» model along the (x,y,z-axes) was applied to all supports by taking into account the results of the 3D model, especially for the settlement of the box and the magnitude of the rotation. This calibration allows a lateral compression of the box to be taken into account. For the south support of the eastern deck, the results at the permanent loads are shown in table 7.

Table 7

Displacements found by means of finite element analysis

Hypothesis	Settlement (mm)	Horizontal displacement (mm)	Rotation (1000 rad)
Stiff hypothesis	3.2	8.6	0.5
Soft hypothesis	5.9	15.9	0.9

5. CONSTRUCTION AND MONITORING

5.1. FINAL CONTROLS

The final controls were focused on three different objects:
- Stratification of the soil.
- Methods used for the construction.
- Displacement of the foundations under the loading forces.

5.1.1. STRATIFICATION OF THE SOIL DURING EXCAVATION

During construction of the foundation box (figure 9), excavation of the foundation well allowed us to confirm the position of soil layers and the geological and geotechnical models used in the design. Many boreholes were made and the knowledge gained during excavation did not show any significant difference with respect to the geotechnical model of the preliminary study. This control also confirmed the stiffness of the Riss formations, which were used for direct support of the foundations.

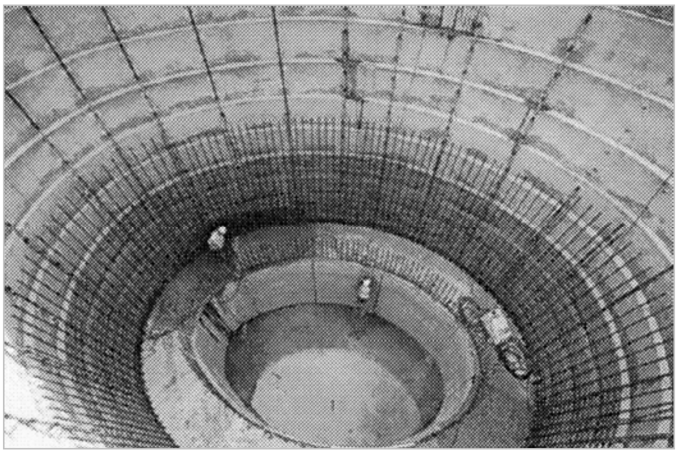

Fig. 9. General view of the foundation well

5.1.2. EFFECTS OF THE METHOD OF CONSTRUCTION

The control used during the construction checked the amount of decompression of the soils. This limitation was ensured by the following special methods of construction:

- Excavation and construction of the retaining wall (of the well foundation) made with a very short delay. Thus, the construction management led to a depth of excavation of 1 m per stage, with a delay of 18 h to built the retaining wall. This wall was made of concrete sleeves of 0.3 m thickness.
- Excavation made by mechanical power (high power shovel).
- Decompression of the soil restrained by injections after concrete work of the foundation well. These injections were made into the interface between soil and foundation under low pressure with one injection point per 1 m^2. The construction work did not exhibit special difficulties. The injected volumes remained on a low level.

5.1.3. DISPLACEMENTS OF THE FOUNDATIONS UNDER THE LOADING FORCES

Along with the construction phases, special measurements were made by means of the following special techniques:

- Measurement of forces on the jacks supporting the arches by the knowledge of the hydraulic pressure.
- Measurement of relative displacements between the foundations and the basis of the arches by four gauges.
- Measurement of foundation displacements and rotations by topographic surveying of targets with a motorised theodolite, and by clinometers fixed on top of the foundation.
- Measurement of the displacements of the arch by topographic targets.

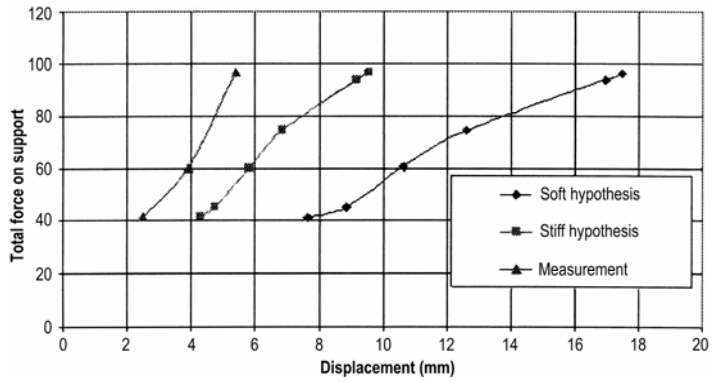

Fig. 10. Achievement control in the support stability

During the construction of the deck topographic measurements were carried out. New periodical controls are planned in order to observe the long-term behaviour of the foundations and to ensure a creep smaller than tolerance limit. These measurements allowed us to plot the curve representing an experimental loading and to compare it with the theoretical ones (figure 10) being drawn for two different stiffness values of the soil. In figure 10, one can see that the value of soil stiffness is higher than hypothetical one. Long-term measurements will be carried out to measure creep of the soil under permanent loads. Topographic observations were planned in 2002.

5.2. PERFORMANCE MONITORING AND SURVEYING

The Crozet bridge crosses the south-east motorways in France, and depends on AREA company (société des Autoroutes RhônE Alpes), which provides a sufficient service level to each construction under conditions of use in accordance with its functions and with a safety which remains the main priority. Bridges, usually longer than 100 m, allow deep and wide breaches to be crossed. They are essential not only for the continuity of the motorway traffic, but also for the stability of economic exchanges in the area crossed. Their supervision and monitoring are of a main priority. Moreover, maintenance and repair of these bridges are of critical importance because of their length and because each working site causes a great constraint to traffic over a very long time.

The management of AREA construction works is based on periodic detailed inspections as defined in ITSEOA (Instruction Technique de la Surveillance et de l'Entretien des Ouvrages d'Art, 1995). Specialists carry out these detailed inspections every 6th year. The corresponding reports that check the damage observed visually give a state index based on IQOA (Image de la Qualité des Ouvrages, 1994) proposed by the French Road Authority. It allows the emergency of the maitenance work or repairs to be emphasized, particularly those related to user's safety.

To improve the monitoring of construction work, each Maintenance Centre distributed in the motorway network visits each year the construction works located in their sector. Thus this short-time management allows the evolution of the damage to be observed for the detailed inspections, a correct realisation of maintenance repair works to be checked and an abnormal behaviour of the construction to be detected in time.

The Crozet bridge was built in 1998 and was for the first time inspected in 1999 before its use. The inspection report described a construction in good condition (index IQOA: 1). The next detailed inspection is planned in 2005.

Because of its particular structure, decks supported by prestressed concrete arches founded on a soil with a medium bearing capacity, it is necessary to check the movements of the arches precisely, especially the separation between the arch foundations.

Calculation shows that the bridge stability can disappear when the arches move apart from one another. For the eastern arch of 140 m, a maximum displacement of 5 cm is allowed. For the western deck, which is supported by three arches, the displacement of the central supports it not taken into account because it is supposed to be compensated for by the efforts of the two end arches, so that the maximum allowable displacements are 1.5 cm and 2.5 cm for the arch (87 m in length) in the Grenoble direction and for the arch (102 m in length) in the Sisteron direction, respectively.

When these displacements are reached, special execution processes with recovery of the hydraulic jacks of the arches will be carried out. The process is designed in such a way that it is possible or not to tighten the prestressed cables of the foundation boxes. Topographical reference targets were fixed on the construction so that displacements can be measured:

- Reference targets of medallion type on each of foundation boxes, 0.8 m above ground level.
- Reference targets of target type fixed at the top of the columns and on arches.
- Reference targets of rivet type fixed on the pavements of the bridge.

Finally YXZ measurements were carried out on May 28th, 1999. A new series of measurements was carried out on August 17th, 2002. No significant displacement was found.

6. CONCLUSION

The Crozet bridge is an arch viaduct founded on soil, which is not a stiff subgrade for this structure, with strict displacement limitations. The calculation, which was used for the design, took into account both characteristics of structural deformation and subgrade deformation with an iterative process. The stiffness of the soil was the function of displacements and displacements modified the forces exerted on the foundation, in a total analysis both in static and dynamic mode so that seismic risk was considered. To achieve this ambitious aim, geotechnical recognition was used by means of cyclic pressuremeter tests with slotted tube and laboratory cyclic triaxial tests for liquefaction analysis.

The achievement controls made at each step of the construction and for the first time used for the bridge show that displacements are lower than we expected. The *in situ* tests may underestimate the behaviour of soil at low deformation.

The design and the grouting of the space between the well foundation and the soil allowed achievement of a foundation with high horizontal force but low horizontal displacement. The bridge has been in use since 1999 without any trouble. This design method is now available for other arch bridge founded on soft soil.

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