Proceeding of the Institution o
Civil Engineers
Geotechnical Engineering 15
January 2002 Issue 1
Pages 71-78

Paper 12405
Received 14/12/2000
Accepted 21/02/2001

Keywords
Field testing and monitoring / retaining walls / pressuremeter

J.Monnet
Assistant Professor of Civil Engineering, Lirigm, Joseph Fourier University, 38041, Grenoble, France

D. Allagnat
Chief Engineer, Scetauroute 38180, Seyssins, France

Design of a large soil retaining structure with pressuremeter analysis

J. Monnet and D. Allagnat

The A49 motorway runs along the Isère River near Tèche, between Grenoble and Valence in France and a large retaining wall was needed. It is an anchored wall. The design was made with the help of pressuremeter measurement. The pressuremeter tests were done with lantern probe, which was pushed in the soil by dynamic driving. A new pressuremeter theory was used to determine the internal angle of friction of the gravel. The results allowed the best adjustment of the wall to the geotechnical environment. The A49 motorway has been used since 1991 without trouble.

NOTATIONS

a	borehole radius
b, c	external radius of the first plastic area
r	radius
z	depth of the test
u_a	radial displacement at the borehole wall
p	radial pressure applied at the borehole wall
D_{60} D_{30} D_{10}	grain size for 60% 30% 10% passing the sieve
G	Elastic shear modulus
γ	unit weight of the soil
λ and μ	Lame coefficient
K_0	Coefficient of earth pressure at rest
Φ	internal angle of friction
Φ_μ	interparticle angle of friction
ψ	Dilatancy angle
P_l	Limit pressure
σ_1, σ_2, σ_3	Main stresses
σ_r , σ_θ , σ_z	Radial, circumferential, vertical stresses
ε_1, ε_2, ε_3	Main strains
ε_r , ε_θ	Radial, circumferential strains

1. INTRODUCTION

The design of a large retaining structure is a balance between safety rules, which are used to prevent failure or large displacements and the building cost of the wall. In such a case, the shearing parameters of the soil are very sensitive so that small variations on cohesion or internal angle of friction give large variations on forces applied on the structure. The usual way to work is to extract intact and representative samples, which are tested with laboratory equipment. Unfortunately it is not possible to use this technique for gravel because the maximum graduation of the soil is larger than the diameter of the sample on the triaxial test.

The pressuremeter analysis is the technique used to measure the internal angle of the gravel. It is assumed that the soil is non cohesive. As the pressuremeter curves are linked with the elastic shear modulus G and the internal angle of friction Φ, an unloading reloading sequence is made to evaluate the elastic behaviour of the soil alone and to point out the influence of the shear modulus G. The internal angle of friction Φ is measured from the linear relation between the logarithm of the radial stress and the radial strain at the borehole[1]. The mechanical characteristics are controlled by the comparison between the experimental and theoretical pressuremeter curves as well as the experimental and theoretical limit pressure[2,3]. The design of the retaining wall of Tèche on the A49 Grenoble Valence motorway is made with the friction angle determined by pressuremeter test, which allows the adjustment of the wall to the geotechnical environment.

2. PROJECT DESCRIPTION

The A49 motorway (Grenoble, Valence) allowed the connection of the alpine motorway, managed by the AREA Company, to the Rhône valley motorway. A49 follows the Isère River on the right side and passes along a 1300 m hill-side in the area of Tèche. The hill slope is 30 to 40° with a regular topography except for a few 3 to 5 m. deep ravines. The two roadways lie on two different levels (figure 1). A 10 to 20 m. high vertical excavation is stabilized upside by anchors and nails. A 6 to 15 m high embankment down side is made of reinforced earth. The 6 m high mid bank between the two roadways is made of reinforced earth. A railway passes upside the slope, and it is essential to insure no displacement at all; 900 anchors, 1100 nails and 14000 m^2 of reinforced earth were used for a total cost of 13.7 M€.

2.1 Tèche geological environment

The site geology is quite simple; the sandstone substratum (liocene) is covered by stream glacial deposits with variable thickness, which comes from the "Saint Marcel les Valences" terrace. Down side, a stream terrace is found along the Isère River.

Fig. 1: The Tèche retaining wall

2.2 Tèche geotechnical environment

The soil is heterogeneous on the upper part, and sandstone is homogeneous on the lower part. The variations of the upper part proceed from the history with many remoulding (channelling, substratum destruction, various deposits) linked to the variations of the level of the Isère River after the last glaciation. Six different soils are found which are red gravely clay (0/150 mm) near the surface, sandy gravel (0/150 mm) with some clay and sometimes large blocks for 75% of the soil, grey gravely clay (0/150 mm) which is sometimes found inside the previous soil, conglomerate of various cohesion (0/150 mm) with some fine sand pockets (0/0,5 mm), gravel (0/150 mm or 0/60 mm) which is found near conglomerate, sand and fine gravel of 0,20 to 0,40 m thickness, homogeneous grey sand.

The sandstone substratum is made up of yellow cemented sand with occasional fine deposits of beige clay (1 to 10 mm thick) and levels of cohesive soil (0,15 to 0,20 m thick). The slope of the upper surface is 10 to 15° north west and sometimes a few 3 to 5 m. deep ravines can be found. There are cracks spaced by 3 to 5 m but no fault.

For the hydrogeology one may notice the presence of ravines on the slope, which are connected to local temporary water stream during heavy rains. The main aspects of the site are the level of the sandstone substratum, the constitution and mechanical characteristics of the gravely upper covering.

2.3 The geotechnical investigations

The geotechnical explorations were made in different phases and different ways. They consisted of electrical and seismic studies, destructive drilling with recording of mechanical parameters, sampling, Menard pressuremeter and cyclic test, excavation with mechanical shovels to identify the soil. The results of these studies are shown below (table 1). The gravely upper covering cannot be sampled because the particles are too large for laboratory tests. The pressuremeter test is used to determine mechanical characteristics because it is an in situ test which does not modify the structure of the soil. It can be made in a gravel soil with large particles. It allows the measurement of the elastic shearing modulus G and of the internal angle of friction ϕ. It can be carried out at any point in the horizontal plane and at a selected vertical depth.

The pressuremeter is a well-known apparatus[4]. It is widely used nowadays for foundation engineering[5-7]. The usual interpretation of the test is to derive the pressuremeter modulus, which is obtained in the range of the linear relationship between the pressure and the volume in the pressuremeter, and the limit pressure which is the pressure applied by the probe when the cavity is twice the initial one. Pressuremeter modulus and limit pressure are used in empirical formulas to respectively estimate settlement and bearing capacity of foundations.

If we consider the pressuremeter test as a shearing test[8], it shows a lot of qualities. The pressuremeter is an apparatus which measures soil deformability and shear resistance. It is an in situ test, which can be carried out in any soil, without sampling. Thus, there is no problem for grain size distribution, or change in consolidation, or remoulding of sample as occurs in laboratory test sampling.

In this paper we use a method of interpretation of a pressuremeter test to obtain the Young modulus of the soil and the angle of internal friction when the soil is granular. These values are stress strain parameters, and are not linked to the probe type or the way that the borehole is drilled. They can be used in design without empirical rules, and for other works than foundation, for example retaining wall, slope stability analysis and tunnel modelling. In such cases, in situ soil parameters allow the design to be adapted to the soil and cost to be saved for the project.

Lithography	Pressuremeter tests			Mechanical drilling parameter	Seismic parameter
	Pressuremeter Modulus (Mpa)	Standard Deviation (Mpa)	Number of tests	Drilling velocity (m/h)	Seismic velocity (m/s)
Sandy gravel	43	26	256	100-150	850-1500
Grey sand	94	21.8	7	50-150	850-1500
Loose gravel	2.6	1.3	7	>250	300-600
Loose sandstone	15.4	6.9	17	20-200	1200-1400
Hard sandstone	458	253	66	<50	1900-2200

Table 1 : Results of the geotechnical exploration

This method uses an experimental process with a cycle of unloading reloading in the "linear" behaviour range before the so-called creep pressure is reached. This cycle is performed to obtain the elastic shear modulus, which is linked to the elastic Young modulus through an assumption on the value of the Poisson ratio. This cycle erases the larger part of the plastic deformation. The test procedure requires more readings in the creep part of the test so that the analysis of the shearing parameters can be more precise.

The test readings are reduced by hydraulic and mechanical corrections described in patent[9] so that the reaction pressure of the soil and the deformation of the borehole are precisely defined. The test results are used in an elasto-plastic theory for the expansion of the pressuremeter probe to find the shear strength or the angle of internal friction of the soil.

2.4 Theoretical study

2.4.1 *Assumptions.*

The soil behaves with a linear elasticity at low level of shear with a constant shear modulus G. For a higher level of shear, a non standard plasticity appears with the angle of internal friction Φ and the dilatancy angle Ψ. The dilatancy is a function of the interparticle angle of friction Φ_μ [10]:

1	$\Psi = \Phi - \Phi_\mu$

The Mohr-Coulomb relation gives the failure of the soil:

2	$F(\sigma) = (\sigma_1 - \sigma_3) - \sin\Phi \, . \, (\sigma_1 + \sigma_3)$

The non-associated flow rule is given by the relation:

3	$d\varepsilon^p = \xi \, . \, dH(\sigma) / d\sigma$

with the undefined scalar ξ and the plastic potential:

4	$H(\sigma) = (\sigma_1 - \sigma_3) - \sin\Psi \, . \, (\sigma_1 + \sigma_3)$

Three different areas of soil are considered from the borehole wall to the infinite radius

As plastic shear may appear between the radial stress σ_r and the circumferential stress σ_θ in the horizontal plane, a first plastic area may extend between radii a (borehole wall) and b (external radius of the first plastic area) and there is a linear relation between σ_r and σ_θ :

5	$\sigma_\theta / \sigma_r = N = (1 - \sin\Phi) / (1 + \sin\Phi)$

The plastic flow is given by the non-associated rule:

6	$d\varepsilon_r^p / d\varepsilon_\theta^p = -n = (1 - \sin\Psi) / (1 + \sin\Psi)$

A plasticity may appear in the vertical plane between the vertical stress σ_z and the circumferential stress σ_θ in an area between radii b and c (external radius of both plastic areas)
An elastic area extends beyond radius c

The stress and strain are negative in compression.

2.4.2 Equilibrium condition

In the horizontal plane, the equilibrium of an element of soil is given by:

7	$\sigma_r - \sigma_\theta - r \, . \, d\sigma_r / dr = 0$

In the vertical plane, the equilibrium condition is:

8	$d\sigma_z / dz = \gamma$

2.4.3 Global equilibrium with plasticity between σ_θ - σ_r and between σ_θ - σ_z

The general equilibrium condition between stress and strain, which is the general form of the pressuremeter equation with two plastic areas, is[3]:

$$Ln\left[\frac{u_a}{a}.(1+n) - C_1\right] = \alpha.Ln(p) - \alpha.Ln(\gamma.z) + Ln\left[(1-K_0).\gamma.z.\frac{(1+n)}{2.G} - C_1\right] \qquad (9)$$

The continuity of the stress between the three different areas allows the C1 constant to be determined with :

$$\alpha = \frac{1+n}{1-N} \qquad (10)$$

$$C_1 = \frac{n.\left(\frac{u_a}{a}\right)(1+n)\left(\frac{z.\gamma}{p}\right)^{\alpha} + (1+n)(N-K_0).\frac{\gamma z}{2.G}}{1+n.\left(\frac{z.\gamma}{p}\right)^{\alpha}} \qquad (11)$$

The value of C1 is very small and can be neglected, so relation (9) shows a linear relation between the logarithm of the radial strain at the borehole wall and the pressure applied by the pressuremeter. The proportionality between these two variables was found previously[1]. Such a relation allows determining the slope α of the straight line between the variables, which is a function of Φ, the angle of internal friction, and Φ_μ, the interparticle angle of friction. The knowledge of Φ_μ and α allows Φ the angle of internal friction of the soil to be inferred.

2.4.4 Global equilibrium with plasticity between σ_θ - σ_r

The general equilibrium condition between stress and strain, which is the general form of the pressuremeter equation for one plastic area, is[3]:

12	$$Ln\left[\frac{u_a}{a}.(1+n)-C_1\right]=\alpha.Ln(p)-\alpha.Ln\left[\frac{2.K_0.\gamma.z}{(1+N)}\right]+Ln\left[K_0.\gamma.z.\frac{(1-N).(1+n)}{2.G.(1+N)}-C_1\right]$$

with:

13	$$C_1=\frac{K_0.z.\gamma.(1-N).(n-1)}{2.G.(1+N)}$$

The proportionality between the axial strain at the borehole wall and the pressure applied by the pressuremeter is also obtained.

The difference between the two cases is linked to the value of the radial stress for the external radius of the plastic area c. For the second case, the value of the radial stress must be smaller than the vertical stress $\sigma_{rc} < \sigma_z$ and this leads to the condition:

14	$$K_0 \geq \frac{1}{(1+\sin\phi)}$$

This relation was obtained previously (WOOD and al. 1977) for a single plastic area.

2.4.5 Ménard conventional limit pressure

For the two cases, we can find the limit pressure P_l when we assume that the volume of the probe is double the initial volume. The radial strain at the borehole is then equal to $\sqrt{2}-1$.

This particular value of the radial strain is put in relation (9) and then we find the conventional limit pressure for two plastic areas:

15	$$P_l=\gamma.z.\sqrt[\alpha]{\frac{[(1+n)(\sqrt{2}-1)-C_1]2.G}{[(1-K_0)(1+n).\gamma.z-2.G.C_1]}}$$

In this relation, we find proportionality between P_l and the value of the vertical stress γ.z, which seems to be natural because the vertical stress is normal to the horizontal plane where the shearing takes place.

The particular value of the radial strain is put in relation (10) and then we obtain the value of the limit pressure for one plastic area:

16	$$P_l=\frac{2.K_0.\gamma.z}{(1+N)}\sqrt[\alpha]{\frac{[(1+n)(\sqrt{2}-1)-C_1]2.G.(1+N)}{K_0.\gamma.z.[(1-N)(1+n)-2.G.C_1.(1+N)]}}$$

We find here that the limit pressure is proportional to the vertical stress and linked to the shear modulus G and the internal angle of friction with the variable N.

3. EXPERIMENTAL STUDY

3.1 Soil used

The soil used in the experiment is a gravely sand. It is a gravel of glacial origin and a grain size distribution between 0.1 mm and 150 mm: D_{60} is 30 mm, D_{10} is 0.2 mm, ratio D_{60} / D_{10} is 15, 43% is under 20 mm, 14% under 2 mm, and 6.5% under 0.08 mm.

3.2 The triaxial test

Tests are carried out on remoulded cylindrical samples, 7 cm in diameter and 14 cm in height. The particles larger than 5mm are removed and the soil was compacted until a dry density of 1.80 is reached. The soil is saturated and a lateral pressure is applied. Four hours later, the consolidation is reached and the process of shearing begins with a velocity of 0.11mm/min. Drainage is free along the triaxial test and the volume variation of the sample is measured inside the sample.

The results of the triaxial tests are shown on figures 2 and 3. The mechanical characteristics measured in the laboratory are shown below (Table 2).

3.3 Pressuremeter tests

3.3.1 *Pressuremeter process*

Tests are performed with a push-in pressuremeter probe and a slotted tube of 800 mm slit length, 60mm outside diameter and 49 mm inside diameter. The probe is made of guard cells of 110 mm length inflated by air pressure and a measurement cell of 420 mm length inflated by water pressure. Two calibration tests are made for each borehole, the first one, on volume loss, is made by inserting the probe vertically into a walled steel tube, the second one, on pressure loss, is done by the measurement of the pressure needed for the expansion of the probe outside the borehole[12]. Additional correction on pressure measurements is made to take into account the influences of difference on the radius inside the slotted tube where the water pressure is applied and the radius outside the slotted tube where the soil pressure reacts. A correction on volume measurements is made to take into account the influence of the slotted tube shape under deformation, and a mean value is found along the 210mm of the measurement cell [9].

3.3.2 Pressuremeter analysis

Equations (9, 12) show that pressuremeter curves are linked with the elastic shear modulus G and the internal angle of friction Φ, and it is necessary to separate the influence of the elastic shear modulus G from the internal angle Φ. An unloading reloading sequence is performed to evaluate the elastic behaviour of the soil and to point out the influence of the shear modulus G (Fig. 5). The internal angle of friction Φ is measured from the linear relation (Fig. 4) between the logarithm of the radial stress and the radial strain at the borehole, which was previously found[1]. Equations 7 and 10 show that this linear relation is linked to the internal angle of friction Φ and the interparticle angle of friction Φ_μ.. The measurement of Φ_μ is made on remoulded sample on the triaxial test (see Table 2) thus a unique value of the slope α leads to a unique value of the internal angle Φ. The mechanical characteristics are controlled by the comparison between the experimental and theoretical pressuremeter curves (see Fig. 5) as well as the experimental and

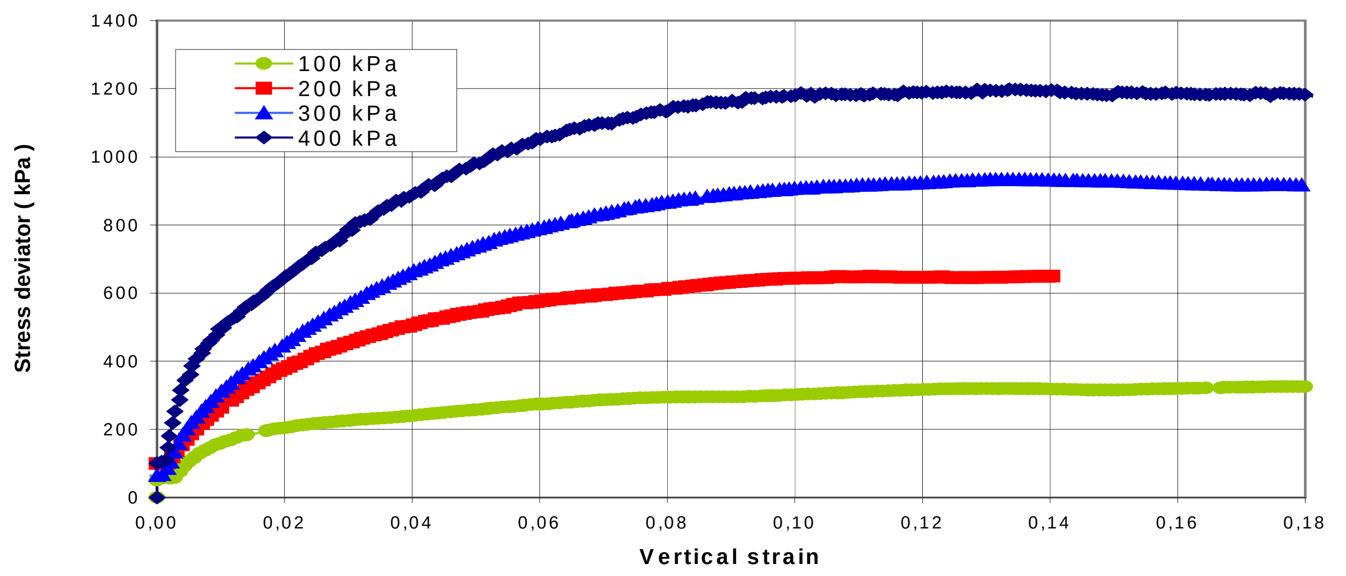

Fig.2 Shearing behaviour of the soil on the triaxial test

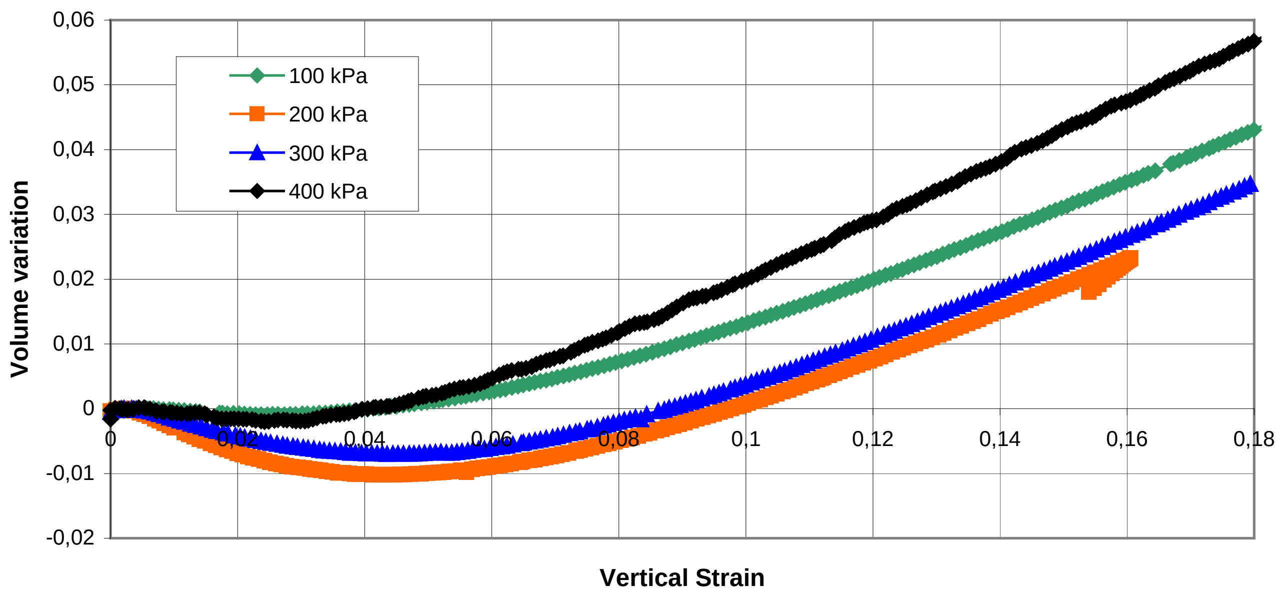

Fig. 3: Dilatancy behaviour on the triaxial test

Lateral pressure	100 kPa	200 kPa	300 kPa	400 kPa	Mean value
Young modulus (kPa)	2840	16040	15390	19540	13220
Poisson's ratio	0.406	0.284	0.340	0.317	0.344
Interparticle friction angle	29.4°	30.0°	30.0°	27.6°	29.2°
Cohesion					0
Internal angle of friction					38.9°

Table 2: The mechanical results on triaxial test

theoretical limit pressure[3]. The pressuremeter results are shown on Table 3. The control of the mechanical characteristics is made by the comparison between experimental and theoretical limit pressure, which show a mean difference of 3%. So it can be considered that the internal friction angle is quite accurate. With no cohesion, a mean value of 44.2° with a standard deviation of 6.1° is found.

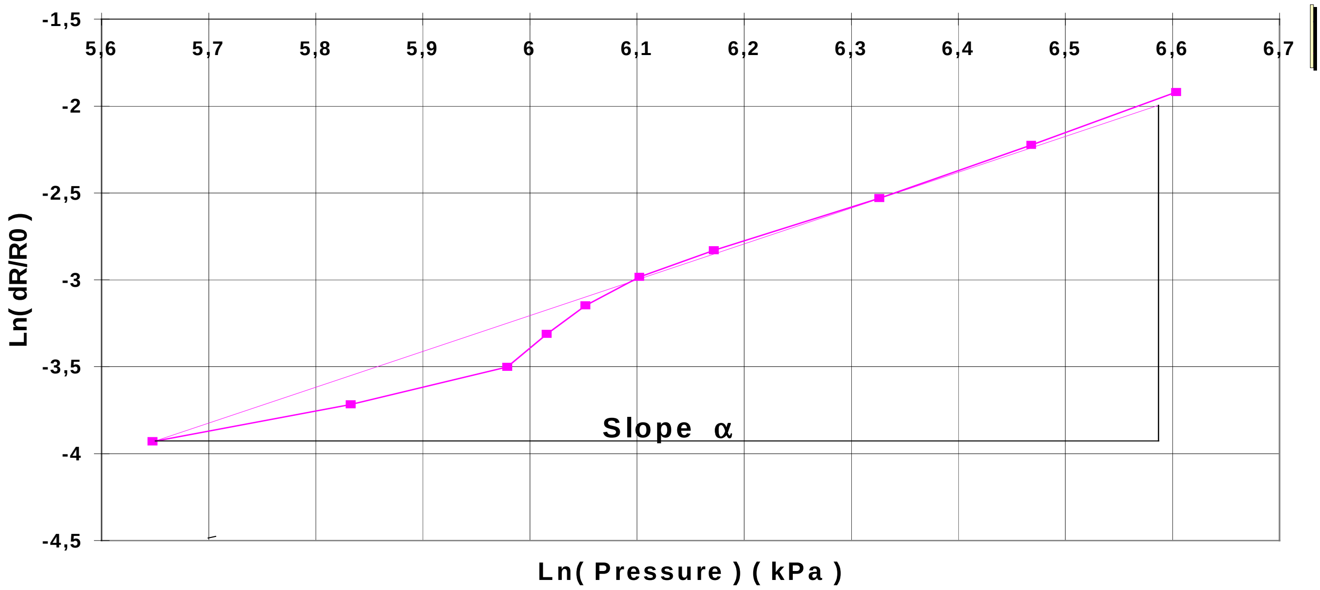

Fig. 4: Measurement of the internal angle of friction by the slope of the linear relation between the logarithms of radial strain and stress at 9m depth

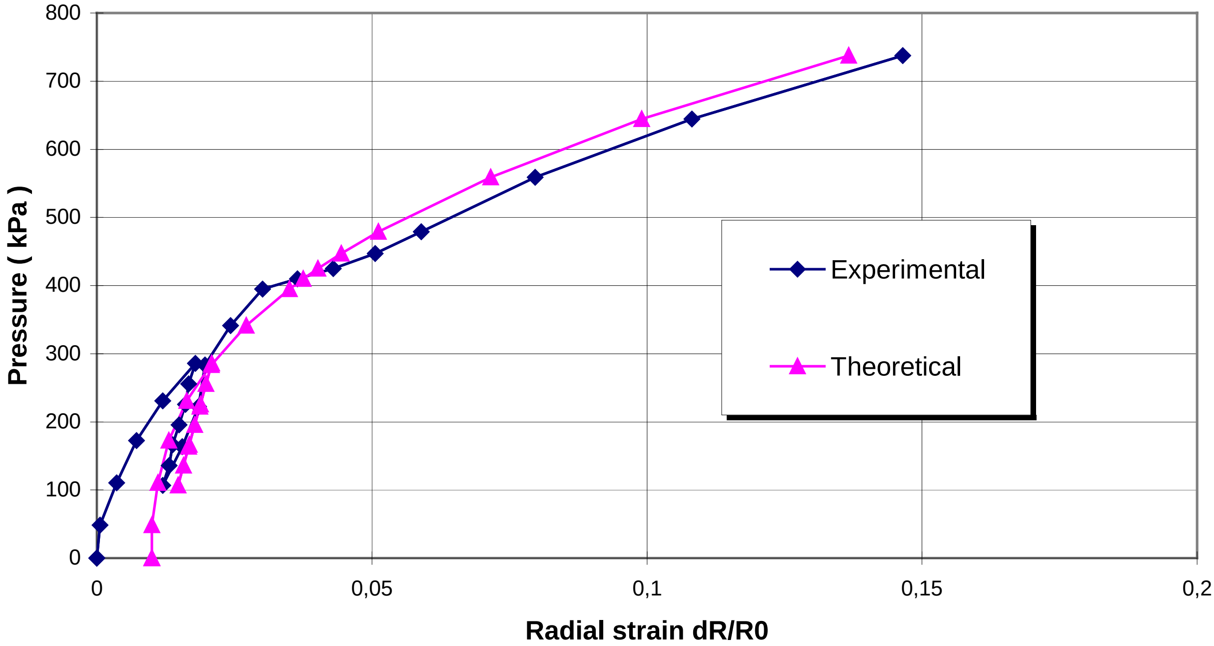

Fig. 5: Control of the mechanical characteristics by comparison between theoretical and experimental pressuremeter curves at 9m depth

4. DESIGN OF THE RETAINING WALL

The various probing allowed 7 sorts of soil on the upper part to be sampled. But these soils are generally in a pocket shape and it is not easy to discern these different materials by their mechanical characteristics. All the mass can be assumed as sandy gravel with some clay. The intact sampling is not possible with such large particles. So the shearing characteristics were measured from special triaxial tests and the use of the pressuremeter analysis shown above. The results give dispersion on the internal angle, which is linked to the heterogeneous composition of the soil. It was [saut ici]

Borehole	Deepness m	Elastic Modulus kPa	Pressuremeter Modulus kPa	Limit Pressure kPa	Theoretical Limit Pressure kPa	Internal Friction Angle
P214	2	21400	6300	1230	1200	52°
	12	134600	39580	4440	4235	42°
	14	87600	25775	3525	3515	41°
	16	77700	22850	3980	4145	44°
	17	101000	29705	3375	3360	37°
	19	156400	45990	4580	4850	39°
P214 bis	9	57200	16815	2150	2170	42°
	14	67000	19700	2945	2865	43°
	15	31180	9170	1090	1135	31°
P211	2	67520	19585	2470	2470	52°
	4	76050	22365	4220	4400	56°
	6	109580	32230	3870	3725	47°
	8	80180	23580	3865	4000	49°
	10	30100	8845	1945	2030	44°
	14	46240	13610	2865	2980	44°

Table 3: Results of the pressuremeter analysis

decided to use the mean value (44.2°) reduced from the standard deviation (6.1°), which gives 38°. For the general stability of the retaining wall, a value of 35° was used (without cohesion) to take in account the possibility of a horizontal pocket of clay to be present near the retaining wall. The computation does not take in account the water table because it cannot be observed. A preventing drainage was made by sub horizontal drains along the wall. The factor of safety against sliding by Bishop method and perturbation method is close to 1.65 (Fig. 6). The area considered around the wall is threefold the total height.

The anchors are made to ensure the provisional stability of the retaining wall with a safety factor of 1.4. Thus the excavating phase n is carried out under the protection of the upper level of anchor n-1. Each excavating phase is 2.5 m in height. The excavation for the reinforced earth support is also controlled with the same safety factor.

For the highest retaining wall (20 m) the prestressed strength is 2000 kN/ml. The retaining wall is made of 3 to 5 levels of horizontal reinforced concrete beams stabilised by anchors. The beams are 20 m in length with 1.5 to 0.7 m of thickness. The length of the anchors varies between 15 to 25 m and the nominal force varies between 500 to 1000 kN; 900 anchors were wade on a total surface of 10000 m^2.

The stability calculation does not take in account the hydraulic pressure, and the geotechnical campaign does not show any water table. Some streams can appear in the gravel levels and at the top of the sandstone when a heavy rainfall occurs. The injection of the anchors and nails and the surface of the concrete wall can alter the circulation of water. So sub-horizontal drains were drilled over 10 to 20 m and spaced to 8 to 10 m at each level of anchor.

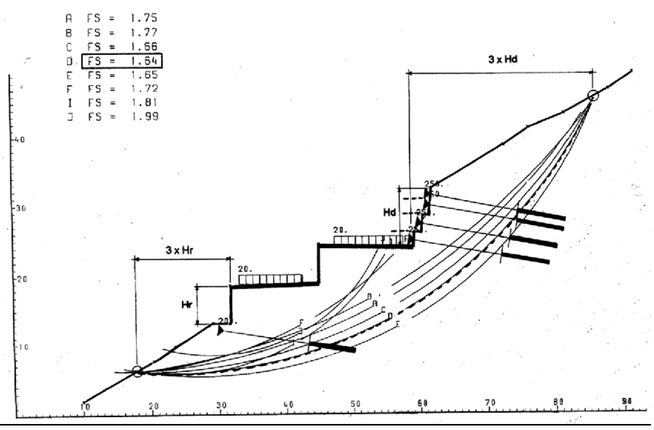

Figure 6: Control of stability against sliding by Bishop method

The SNCF company, which manages the railway above the wall, has not given a maximum available displacement but imposed an inclinometric survey from the early stage of the construction. It was decided to used prestressed anchors rather than passive nailing to control displacements.

For the long-term control of the retaining wall, special measurements are operated. That is the strength on 10% of the anchors by dynamometric apparatus, inclination measurement in 4 upside boreholes to study the stability of the railway, topographic measurement of the vertical surface of the wall, volumetric flow of the drainage, annual visual observation. All these measurements are carried out in a surveying organisation with a fixed frequency of controls and threshold displacement values to ensure. Nowadays after 10 years of measurements there is no significant displacements of the retaining walls.

5. CONCLUSION

The pressuremeter test allowed the mechanical characteristics of the gravel soil of Tèche to be measured, in a soil mass where no sampling is possible. The site is heterogeneous from the geological history, influencing the results of the pressuremeter analysis, and the pressuremeter study gives variability on the internal angle of friction and on the elastic shearing modulus. To prevent any damage, it was decided to use the mean value of the internal angle of friction find by pressuremeter analysis decreased from the standard deviation. The design of the retaining wall was made with such a result; 10 years after the end of the building, the surveying of the wall does not shown any significant displacements.

REFERENCES

1. Hughes J.M.O., Wroth C.P., Windle D., Pressureter tests in sand, *Geotechnique*, 1997, **27**, N°4, 455-477
2. Monnet, J., Theoretical study of elasto-plastic equilibrium around pressuremeter in sands, *3rd International Symposium on Pressuremeters*, Oxford, 1990, 137-148.
3. Monnet J., Khlif J., Etude théorique et expérimentale de l'équilibre élasto-plastique d'un sol pulvérulent autour du pressiomètre, *Revue Française de Géotechnique*, 1994, **67**, 3-12.
4. Ménard L. Pressiomètre, French Patent, n° 1.117.983, Paris, 1955
5. Ménard L., Mesures des propriétés physiques des sols, Annales des Ponts et Chaussées, 1957, **14**, 357-377
6. Gambin M., Vingt ans d'usage du pressiomètre en Europe, Congrès Européen de Mécanique des Sols et des Travaux de Fondation, Brighton, 1979.
7. Amar S., Clarke B.G.F., Gambin M., Orr T.L.L, The application of pressuremeter test results to foundation design in Europe – European Regional Technical Committee N°4, Pressuremeter, A.A.Balkema, 1991 1-23.
8. Clarke B.G., Gambin M., Pressuremeter testing in Onshore ground investigations, Report by the ISSMGE Committee TC16, 1rt Int. Congress on site Characterisation, Atlanta, Ed. Balkema, 1998, Vol.2
9. Gaiatech, Procédé d'essai de forage, French Patent, n° 89 09674, Lyon, 1989.
10. Monnet J., Gielly J., Détermination d'une loi de comportement pour le cisaillement des sols pulvérulents, Revue Française de Géotechnique, 1978, **7**, 45-66.
11. Wood D.M., Wroth P.C., Some laboratory experiments related to the results of pressurementer tests, Geotechnique, 1977, 27, N°2, 81-201.
12. French Norm NF P 94-110, Essai pressiométrique Ménard, AFNOR, 1991.

Please email, fax or post discussion contributions to the secretary: email: mary.Henderson@ice.org.uk; fax +44(0)20 7799 1325;
Or post to Mary Henderson, journals department, Institution of Civil Engineers, 1-7 Great George Streey, London SWIP 3AA