# Special design and treatments of swelling clays and other marginal materials in a Spanish motorway

## Definition especial et des traitements des argyles gonflants at des autres materiaux marginales dans une Autoroute Espagnole.

C. Oteo – *Dr. Ing. C.C. y P. Prof Polytechnic Univ. of Madrid, Spain*
A. Domingo – *Madrid Regional Goverment, Spain*
J. Velasco – *Ing. C.C. y P. Trados - 45. Madrid. Spain*
J. Castanedo – *Ing. C.C. y P, Madrid Spain*

ABSTRACT: Following the usual methodology in the execution of earth structures, a swelling clayey matrix embankment reinforced with lime has been built along Section II of the M-45 motorway, with a length of almost 2 km and heights of up to 10 m. Moreover, remains from demolition works have been used for making embankments in this same work.

RESUME: Avec l'utlisation des techniques habituelles dans la construction des terrassements pour routes, on a realise a remblais avec des argilles gonflants reforceès avec de la chaux, dans la section II de l'autoroute M-45 (Madrid, Espagne). Aussi on a utilisé des rests de demolition pour des autres remblais dans la même chantier.

## 1.-INTRODUCTION

The M-45 is a motorway owned by the Madrid Regional Government, being built under a concession system. It is going to link up the N-II with the N-VI.

The materials present in the section II of M-45 consist of clays with high plasticity and gypsums, corresponding to the Central Facies of the Madrid Miocene. These materials have traditionally considered as inadequate, and the original plan was therefore to take them to a dump. There also exist in this area various waste heaps and refuse dumps where for the last thirty years remains of demolition works, earth materials from excavations, inorganic waste, etc., have been dumped (fig. 1).

On starting construction work on section II of the M-45 Motorway, it was possible to confirm that there was an absence of quality materials for borrowing. Also the environmental conditions prevented the dumping of surplus materials from the excavations to be performed. The need therefore arose to exploit all these existing materials as much as possible.

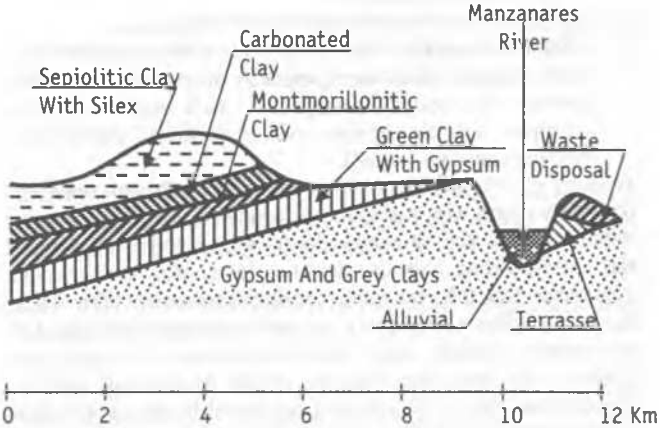


Fig. 1.- Geological profile of the zone II of the M–45 motorway.

## 2.- SEPIOLITIC MATERIALS

Found among the clayey deposits of these facies were sepiolitic clays, with the presence of silex adges and diffuse silications in the clayey structure. In the area, the layers of which that were most abundant in the clayey mineral were being worked on the industrial scale. Sepiolite is a clayey mineral belonging to the smectites group, which means that sepiolitic clays have the following general characteristics: a) Very low dry densities. b) Medium to high plasticity. c) High capacity for water absorption even in a very dry state. Because of these properties, it was decided that this clayey material could be used in the execution of a partially disintegrated clay embankment with reinforcement of its matrix using lime. The main geotechnical properties are:

- Variable fine content: 23-99.5 %. The predominance of sands in some samples is due to the existence of lithification or cementation by silica in the clays
- Liquid limit (fig. 2): LL = 49-97% (excepcionally: 90-160%) / Plastic limit: LP = 32-70% (excepcionally 55-117%)
- Bulk dry density: 700 to 1100 kg/m$^3$. Void ratio, $e_o$ = 1,05 – 3,03
- Natural water content: 20-49%
- Unconfined compressive strength, $q_u$, :200 to 380 KPa. Effective Cohesion, c': 0-100 KPa . Effective angle of friction : 30-48°
- $SO_3$ : 0.01-0.10% and $CO_2$ : 0.35-24%
- Expansivity: Swelling pressure: 1-170 KPa . Lambe Swelling: Marginal to critical.

## 3.- CONDITIONS OF USE OF SEPIOLITIC CLAYS

When it came to compacting these clays, it was sought to obtain a disperse structure with a percentage of clayey matrix greater than 50%. With these conditions. and given the range of variation of the Standard Proctor optimum water content, depending on the nature or plastic limit of the clays and on the degree of crushing obtained, the risk is sometimes taken of excessive moistening of the material in order to make sure that it stays on the wet side. This can produce a fall in the carrying capacity of the subgrade in some cases, which would mean hold-ups to the work in order to wait for the material that had been laid in an excessively wet state to dry out. This risk was not acceptable in the case of a work like the M-45, where the movement of a large volume of earth in a minimum time had to be guaranteed.

In order to prevent the risk of loss of the carrying capacity of the

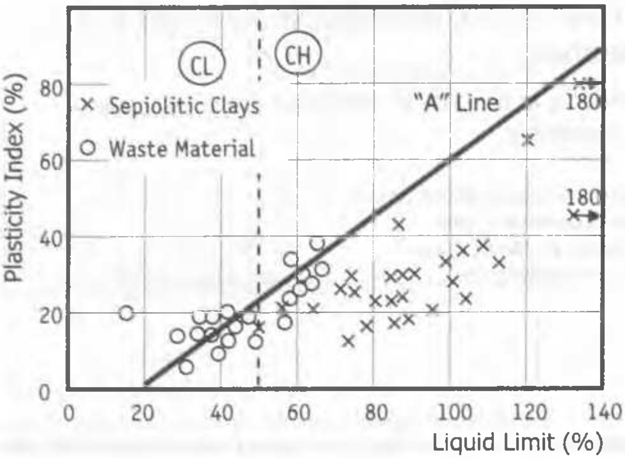

Fig. 2.- Plasticity of the materials.

roadbed during construction, and in addition guaranteeing an absence of risk of swelling of the clayey material once laid, it was decided to reinforce the matrix of the clays with slaked lime.
The various standards in force (Little 1995; LCPC, 1997, etc.), basically consider the use of lime in order to fulfil two purposes:

- Stabilisation of the upper layers of the embankment crest or subgrade, using the lime to improve their bearing capacity.
- Improvement of the bearing capacity of very soft layers that prevent the normal passage of work machinery (for example, in the bottom of depressions, before the embankments are started).

Working on the wet side, (respect the standard Proctor optimum conditions), and with a disperse structure in the clayey matrix, stable roadbeds can be achieved with high contents of wetness and corresponding bulk dry densities lower than the optimum (remember that the maximum density decreases notably when the laying wetness increases), as can be deduced from the US experiences. Initially, and before the process of curing the mixture with lime, the capacity of the treated soil will be lessened but, due to the effect of the actual binder, it will then increase, first of all during a period of 24 hours in what is known as "primary hardening" and which is of a crystalline nature, and after that by means of secondary hardening owing to the hydration of colloids, a period that starts from 24 hours and can end over a year later, though at least half the final strength will have been reached within a period of one month.
In our case, it was considered to expand this experiment to a reinforcement of the clayey mass of the embankment by treatment with lime, not in order to stabilise it but instead – and treating it with the usual compaction philosophy in an embankment – to try to reinforce the clay with an additional cementation. Various compaction tests were conducted in laboratory (Table 1), both for untreated material and for several percentages of slaked lime. In order to obtain the undrained shear strength, $C_u$, the sample was left to mature for 24 hours in a wet chamber in order to permit setting of the clay-lime mixture. Starting from 24 hours, the hardening of the treated soil was extended up to a minimum of 28 days. From Table 1 it can be deduced that, with the lime, values of $C_u$ were obtained of the order of 25-40% greater than those of untreated clay.
From these tests it can be deduced that the primary maturing of the lime that takes place within 24 hours depending on the weather conditions produces a sufficient CBR index for the passage of machinery, even with percentages of lime of less than 2%. The undrained shear strength tests and expansivity tests, with different proportions of lime, gave the results included in Table 2. To conclude, and as was expected, it is more recommendable to go for compaction on the wet side with which sufficient strengths are obtained in the medium term, even for wetnesses of around 10% above the standard Proctor optimum water content.

Table 2.- EVOLUTION OF THE STRENGTH AND EXPANSIVITY WITH WETNESS (CaO: 2.49%; Wopt= 55%)

| Maturing Time (days) | Massed Wetnes (%) | Undrained Strength | | Swelling Pressure (Kpa) | Collapse in with $\sigma_v$ = 240 Kpa% |
|---|---|---|---|---|---|
| | | Cu (Kpa) | CBR | | |
| 46 | 15 | 130 | 5.62 | 71 | 3.27 |
| 53 | 15 | 118 | 5.10 | 31 | 3.64 |
| 55 | 15 | 123 | 5.32 | 71 | 1.67 |
| 57 | 15 | 162 | 7.00 | 10 | 2.51 |
| 62 | 15 | 158 | 6.81 | 10 | 0.00 |

## 4.- STRUCTURE OF THE EMBANKMENT CONSTRUCTED WITH SEPIOLITIC CLAYS

According to the results of the laboratory tests and the field embankment test, an embankment was earth structure designed in which the lime reinforced the clayey material of the fill, differentiating between foundations, core, shoulders and base course (fig.3). The thickness of layer, once compacted, is 25 cm, worth the percentages of lime indicated in the fig. 3:

- In the core: 1,8% of the lime (refered to the dry weight).
- In the foundations and shoulders: 3,2%.
- In the crest or base course, same as in the foundating and shoulders.

The mixing and crushing of the material was done by means of employing a grill harrow (in the base course crest) or plough (in the core, foundations and shoulders), and "goat's foot" compacting. It has been confirmed that the effect of the goat's foot is not just valid for crushing the material but also for mixing it with the lime, with the designed thicknesses of layer. Both the addition of water and that of powdered lime was done between the runs of the compactor, which were 5 doubles. In order to check the penetration of the lime inside the layer, some undercuts were made in the experimental embankments, and the evolution of the strength was measured with a pocket penetrometer. Control of the layers that were laid was done by means of a mixed system (finished product and by procedure). It was found that: the water content of the disintegrated material was above the expected margin, which corresponds to a texture in the matrix higher than the plastic limit, and that the percentage of sizes less than 2 mm in the laid material has to be greater than 65% in order to ensure that the fines are perfectly enveloped in the matrix. In order to control the finished product, continuous measurements were made of the bulk dry densities by the nuclear method, and plate bearing tests and the Swiss of the deformation modulus of the second cycle ($EV_2$) of greater than 100 MPa in the core of the embankment and around 200 MPa in the shoulders. The total hardening due to the lime does not take place until one month after laying the layer. After this period the "sut" test gave a value about 1-3 mm. And the second cycle deformation modules reach 150-330 Mpa.
Also Borro's type dynamic penetration tests were conducted (which gave values of 20-30 blows/20cm), along with taking of samples on layers that had been laid in order to check the undrained shear strength $C_u$, (results greater than 100-150 KPa), swelling pressure (which was less than 5 KPa) and collapse settlement (which was less than 0.2%). Various tests conducted comparing the use of the rotavator with the compaction by "goat's foot" roller in the crest did not reveal any improvement with the rotavator and, nevertheless, the subgrade was left drier, which could give rise to expansivity problems.

Table 1.- LABORATORY TESTS

| Trial pit | Km Point | L.L. | W% | Without LIME | | | | 1,25% of CaO | | | | 1,70% of CaO | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CM-15 | 2-600 | 58 | 49,4 | 1,01 | 57 | 150 | 5.52 | 0.99 | 61% | 166 | 6.20 | 1.01 | 57 | 1.72 | 6.47 |
| CM-16 | 3+100 | 112 | - | 0,75 | 93 | - | | 0.80 | 85 | 192 | 7.34 | 0.78 | 87 | 2.87 | 11.4 |
| CM-18 | 5+600 | 64,8 | 25,92 | 1,21 | 42 | 135 | 4.8 | 0.99 | 61 | 167 | 6.26 | 1.025 | 56 | 1.87 | 7.13 |

LL: Liquid limit

W: Natural water content

$W_{opt}$: Optimum water content % (St. Proctor)

$C_u$: Undrained shear strength (KPa)

$\gamma_{opt}$ : Maximum dry density in $Tn/m^3$ (St. Proctor)

CBR: Deduced CBR index starting from $C_u$

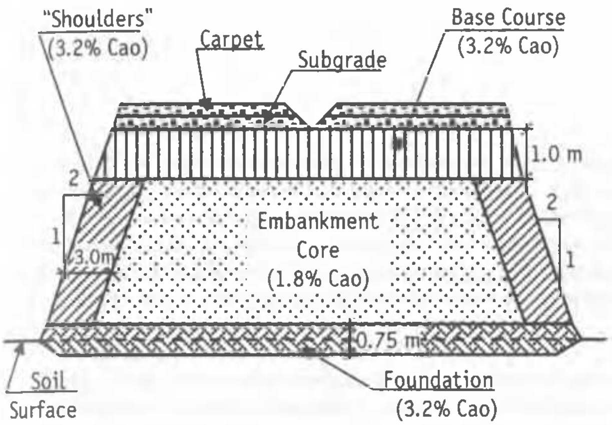


Fig. 3.- Lay-out of the embankment with sepiolitic clays reinforced with lime.

5.- REUSE OF RUBBLE

For the feasibility study on the exploiting of dumps, the phases followed were: a) Photohistoric analysis of the evolution of the volume of waste and its distribution in zones. B) Sinking of boreholes and trial pits. From the first analysis it was deduced that the materials from the dump in San Martín de la Vega would in all likelihood be able to be exploited since they met the following conditions: 1.-Predominance of sandy-clayey soils. 2.- Scarcity of organic waste (refuse). 3.- Stabilisation period of the waste (after passing through the recent topmost level which was more organic) of more than 10 years. With the passage of time and cycles of dampness/drought, soils dumped in the waste heap tend to achieve an equilibrium dampness which, although it is variable due to the hetefogeneity of the soil, is normally around the suction minimum for a flocculated structure, with a water content between - 3 to +1 with respect to Standard Proctor optimum water content conditions.

In 9 out of the 11 boreholes that were sunk – with detailed core sampling - geophysical tests was also carried out measuring natural gamma radioactivity, density and porosity. With this technique it is possible to check the existence of materials that are clearly unsuitable due to a greater predominance of organic remains and of urban refuse (by monitoring the gamma radioactivity), and to check their continuity and presence in the entire dump or in partial zones of it. A search can also be made of zones in which there is a predominance of thick rubble from demolition work (porosity control).

Due to needs of reuse, the partial re-exploitation of the material in the dump was considered, using the following for laying in the body of the embankments: soils coming from excavations and remains of bricks and concrete from demolition work, with large blocks, plastic, paper, plaster partitions, etc., being removed. In order to carry this out, it was sufficient to use mobile vibrating screen equipment, with a bar separation of 15 cm; no crushing equipment was necessary. In 60% of the have been represented in figure 2. This material (with these geotechnical properties) is calified in the Spanish Road Rules as a "adequate materials" for embankments. The base course and shoulders of the embankment containing this rubble material were produced using a material with the lowest possible content of fines and organic matter.

The field control tests carried out during the embankment construction give favourables results (principally the "rut" and the plate bearing tests) in accordance with the normal specifications requerided in normal embankments construction controls.

6.- ACKNOWLEDGEMENTS


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