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The behaviour of the cast in situ retaining walls of Seville Underground

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ABSTRACT: The Seville Underground now has nearly 19 km of Line 1 in operation crossing the city from East to West. The centre of the Line (old urban area) is built with a double tunnel, excavated with an EPB machine, but the main part of the Line was built with cast-in situ wall solutions. The ground of the city is formed by sandy clayey Quaternary sediments, of medium-low consistence, covering a thick stratum of gravel (8–10 m thick). The near substrate is constituted by a formation of Miocene and fissured stiff clays, that have been studied in previous works. The continuous walls generally cross the upper sediments and gravel and penetrate in the Miocene clays, to achieve interior excavation with little water seepage. These walls are instrumented in several sections. With the results obtained (mainly, horizontal movements) and the corresponding numerical simulations, the behaviour of these structural elements has been studied and the geotechnical properties of the soils affected concluded. This paper provides the analysis and results obtained, that will be used as the basis for designing and building new lines in the Seville Underground.

Between June 2003 and November 2009, Line 1 of the Seville Metro was built. Six years of construction works to execute an infrastructure of more than 18 kilometres, that crosses Seville from East to West, with stretches on the surface in the suburbs and underground stretches in the city, the latter excavated between retaining walls and also using a tunnel boring machine. The technical monitoring and control of the works involved installation of multiple auscultation instruments and taking numerous readings from these, to record the behaviour of the infrastructure built and of the ground and surrounding structures.

Therefore, the real behaviour the retaining walls had during the excavation is known; based on that knowledge, it is established what numerical figures of the ground parameters reproduce that behaviour. In particular, the coefficient that is perhaps the most hermetic of all: the coefficient of subgrade reaction.

The first person to introduce the concept of subgrade reaction in Applied Mechanics was Winkler (1867), in his book on the strength of materials. After him, the first person to use that concept in a practical application was Zimmermann (1888), to calculate the stresses in railway ties resting on ballast over their full length. During the following decades, the theory was expanded to include the calculation of the stresses in flexible foundations, such as continuous footings or rafts. Towards 1920, the theory of subgrade reaction began to be used in cases in which the ground reactions acted in horizontal direction, such as piles and sheet-piles.

Terzaghi (1955) established numerical values for the coefficient of vertical subgrade reaction (k_{30}) for square plates one foot wide (and also for strips of this width and unlimited length), as the basis to calculate the relevant values for rigid footings and mat foundations (k_v). His studies also included piles and sheet piles subject to horizontal loads, using the horizontal subgrade reaction coefficient (k_h), for which he also established numerical values. At the same time, Rowe (1954, 1955, 1956a, b) analysed and published the earth pressures on sheet pile walls and on single piles subject to horizontal loads. Both in cohesionless soils, and based on both theoretical analysis and experimental trials.

In the 1950s, there was a takeoff in construction of continuous retaining walls of reinforced concrete, used to water proof, protect and consolidate excavations (Chadeisson, 1961). In the 60s and 70s, this French engineer developed a computer program to calculate retaining walls in which he used the coefficient of subgrade reaction and Winkler's mathematical model to establish the behaviour of the ground. Based on his experience of analysing retaining walls, he proposed an abacus with numerical values of the coefficient of subgrade reaction. There is no knowledge of the abacus having been published then, but its use appears to have been fairly widespread among French engineers (Monnet, 1994).

Retaining wall calculation is now performed using commercial computer programs. Two of those in most widespread use are Rido and Cype. Both are based on the elasto-plastic spring model characterised by the coefficient of subgrade reaction. As there is a great disparity in figures published for this coefficient, the choice of its numerical value is a particularly difficult task for retaining wall designers. This paper aims to clarify this process, contributing the experience of the Seville Metro works.

1 FIELD WORK

Based on the inclinometer records, the real behaviour of the retaining walls during the different excavation phases is established, on two key sites that characterise overall the Line 1 of Seville Metro: the stations of Plaza de Cuba and San Bernardo.

1.1 *Plaza de Cuba Station*

Plaza de Cuba Station is the deepest of those located to the west of Guadalquivir River. Its excavation depth reaches 20.50 m from street level and was carried out in 6 phases. The reinforced concrete retaining walls are 1.00 m thick and 33.00 m deep.

The real behaviour of the retaining walls during the works in the different excavation phases is represented by the continuous lines in Figure 1.

1.2 *San Bernardo Station*

San Bernardo Station is located to the east of the Guadalquivir River. Its excavation depth reaches 20.50 m from street level and was carried out in 4 phases. The reinforced concrete retaining walls are 1.00 m thick and 32.00 m deep.

The real behaviour of the retaining walls in the works during the different excavation phases is represented by the continuous lines in Figure 2.

2 PRIMARY ANALYSIS

As primary analysis, several calculations are performed at each one of the sites mentioned, using the two programs chosen. In order to cover the whole spectrum, three hypotheses are analysed on the magnitude of the ground-wall friction that conditions the slope of the active and passive earth pressures: $\delta = 0$, $\delta = \varphi/3$ y $\delta = 2\varphi/3$. That is to say, the usual criteria in professional practice are applied, based on the results of the geotechnical surveys. The stratigraphic columns, and the values of the geomechanical parameters of the different ground levels on each site, used for the primary analysis and coinciding with the project, are summarised in Tables 1 & 2.

Table 1. Stratigrafic profile at Plaza de Cuba Station.

	Depth	Bulk unit weight	Cohesion	Friction angle	Subgrade reaction coefficient
	m	kN/m^3	kPa	°	kN/m^3
Fill	0.00–2.00	19	10	25	22,000
Sand	2.00–15.00	21	5	34	40,000
Gravel	15.00–23.00	21	0	37	46,000
Marl	23.00–> 50.00	20	40	28	35,000

Table 2. Stratigrafic profile at San Bernardo Station.

	Depth	Bulk unit weight	Cohesion	Friction angle	Subgrade reaction coefficient
	m	kN/m^3	kPa	°	kN/m^3
Fill	0.00–1.00	19	10	25	22,000
Clay	1.00–10.00	20	15	28	28,000
Gravel	10.00–19.50	21	0	37	46,000
Marl	19.50–> 50.00	20	40	28	35,000

As already mentioned in the introduction, there is a great disparity of published values of the coefficient of horizontal subgrade reaction for retaining walls. There are even discrepancies between different authors in establishing which factors have an influence in the coefficient, and what type of influence might they have.

Terzaghi (1955) establishes that the coefficient depends on the elastic properties of the ground and the dimensions of the area loaded, with a constant value for clay (depending on its consistency), and with a growth in direct proportion to the depth for sand (depending on its compaction).

Monnet (1994) attributes to Chadeisson an abacus in which k_h depends exclusively on the ground, with values depending on the cohesion and the friction angle. These are constant for any depth, both for clays and sands.

Monnet (1994), in his formula, reasonably reproduces the values of the Chadeisson abacus, and introduces the additional factor of the retaining wall rigidity. The subgrade reaction coefficient value rises in relation to the rigidity of the retaining wall, especially for sandy ground.

Schmitt (1995) makes the coefficient of subgrade reaction dependent on the characteristics of the ground and the geometry and rigidity of the retaining wall. The particular feature lies in that, on the contrary to what is concluded by Monnet, the subgrade reaction coefficient value falls as the rigidity of the retaining wall rises, both for clays and sands.

Muzás (2002) states that the k_h values given by Terzaghi (1955) for retaining walls are conservative. He also concludes that, for intermediate soil, the coefficient of subgrade reaction to be considered must be of the trapezoidal type, as a combination of the clay and sand criteria.

Monaco & Marchetti (2004) establish that k_h depends not only on the rigidity of the soil, but also on the excavation depth and the height between strutting on the retaining wall. In sand, their value tends to remain constant after reaching a certain excavation depth. For them, the rigidity of the retaining wall does not have much influence on the coefficient.

At the end of the 20th Century, there have been several authors who have considered the objective of better adjustment of the real behaviour of retaining walls, using different values between active and passive coefficients, different by zones according to whether or not there is nearby strutting or prestressed anchorages, and also different according to whether the tension level is lower or higher than the maximum level previously reached.

Thus, Balay (1984) adapts the formula by Ménard et al. (1964, 1965) to evaluate k_h along the full length of a retaining wall, adopting different values between above and below the excavation depth.

Simon (1995) extends the Ménard formula adapted by Balay (1984), distinguishing k_h by zones. On one hand, there are the zones of free deformation (free height and embedded length of a cantilever wall). And on the other hand, zones of restrained deformation (height between two struts or anchorages and behind a pre-stressed anchorage).

In their method, Becci & Nova (1987) take into account the non linear behaviour of the ground. They calculate k_h based on the E_{ur} module when the tension level is lower than the past maximum stress level, and based on E_{oed} when the stress level exceeds the past maximum.

Muzás (2005) proposes use of three reaction coefficients behind the retaining wall and two coefficients at the front. The ones behind the wall would be: one for unloading from at rest condition to active earth pressure, and two of reloading, from active earth pressure and from the initial at rest condition. And the ones at the front of the wall would be: one for reloading from active earth pressure and another for loading from initial at rest condition up to the passive earth pressure.

With the above considerations in mind, and also considering that use of the Chadeisson values is very widespread among designers, it was decided to take these as the starting point for the primary analysis. An extract of the results obtained in that primary analysis is reproduced in Figures 1 & 2 (dotted lines).

3 DISCUSSION

The comparison between the results obtained using the Rido program and those of the Cype program allow us to conclude, firstly, that both programs predict a behaviour that is practically the same for the retaining walls. There is no difference in results when choosing one program or another.

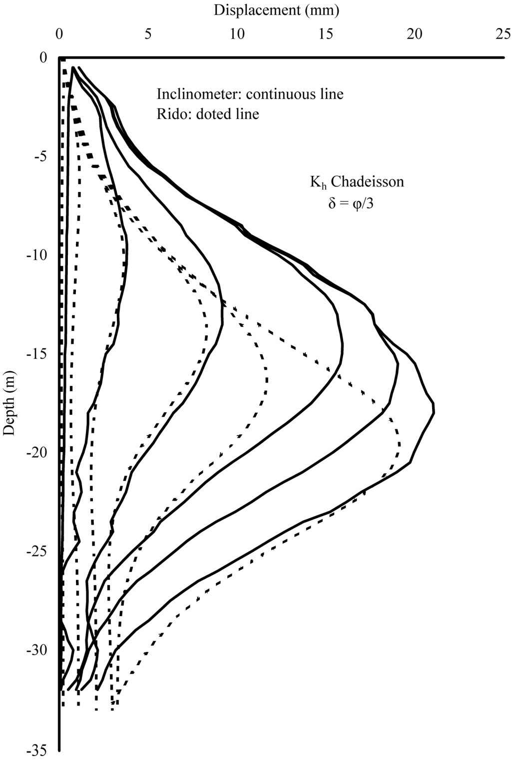

Figure 1. Plaza de Cuba Station. Compared displacements Inclinometer/Rido.

Secondly, the comparison between the results obtained in the calculations using both programs, with the reality recorded on site using instruments shows that the calculations (with k_h by Chadeisson) are not reliable to predict the behaviour of the retaining walls.

In Figures 1 & 2 one may see how in the excavated depth of the retaining walls, all the dotted lines (calculated) are clearly on the left of the continuous lines (measured on site). That means that the calculations are systematically predicting lower movements than those that really occur. The predictions are only 20–40% of the movements recorded by means of the inclinometers. For the deeper excavations, the calculations are nearer to the real maximum displacement magnitude (depending on the value of δ) but they still sub-estimate the displacement at low depths (near to where the foundations of neighbouring buildings may be).

4 PARAMETER ANALYSIS

In the previous discussion, it has been established that the calculations performed as primary analysis are not sufficiently similar to the real behaviour recorded

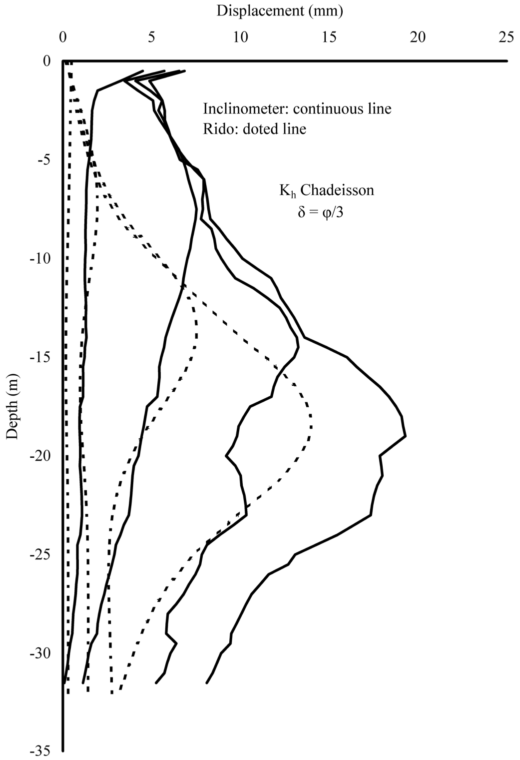

Figure 2. San Bernardo Station. Compared displacements Inclinometer/Rido.

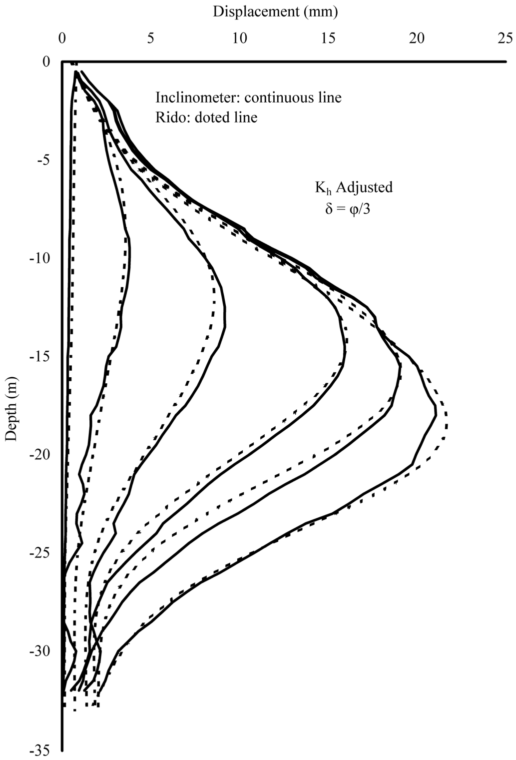

Figure 3. Plaza de Cuba Station. Adjusted displacements Inclinometer/Rido.

on site. Due to this, a parameter analysis has been performed, based on a new series of iterative calculations, prepared by varying the coefficient of subgrade reaction of the different soils involved, until obtaining the values thereof that best reproduce the reality measured.

The first metres embedded in the ground, located immediately below the maximum excavation of a specific phase, are those that establish the magnitude of displacement of the retaining wall. Thus, once this magnitude is known through the inclinometers, it is possible to obtain the numerical value of the coefficient of subgrade reaction in those metres of ground. Moreover, the following considerations have been made.

It was decided not to establish a correlation between the subgrade reaction coefficient (k_h) and the Young's modulus (E), as deriving k_h of the springs from E of the continuum would involve trying to establish a correlation between parameters of different models that do not match.

The scope is for the proposed k_h values for calculation of concrete retaining walls, explicitly excluding sheet piling from that scope. Thus, the size of the area loaded is defined, that in retaining walls such as those of the Seville Metro has a fairly great length (all the continuous face of the retaining wall, tens of metres) and a limited depth of a few metres (those located immediately under the maximum excavation of each phase where the passive earth pressure values are maximum).

Varying the numerical value of the subgrade reaction coefficient according to the rigidity of the retaining wall was rejected due to the fact that the mechanism that links them is not fully understood, and there is not even consensus among the authors as to whether an increase in rigidity makes the values of the subgrade reaction coefficient increase or decrease.

Also rejected was making the numerical value of the coefficient depend on the effective stress, that is thus variable as the successive phases of the retaining walls are excavated with multiple levels of strutting. In addition to it not being easy at all to establish a unique and straightforward correlation between them, in the cases tried it has not been possible to obtain significantly greater adjustments than with constant coefficients.

Adopting different values for active and passive subgrade reaction coefficients was rejected, as well as for the coefficients in virgin compression and recompression. Due to the great dispersion of values proposed for the coefficient by the different authors, it is considered much more important to orient the designer to opt for the correct order of magnitude for a sole coefficient of subgrade reaction, than complicating the matter by choosing two or more different values, especially taking into account the often scarce

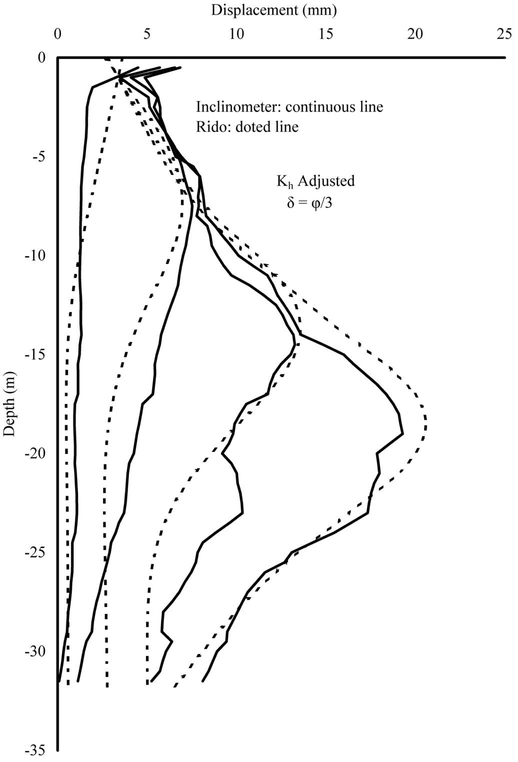

Figure 4. San Bernardo Station. Adjusted displacements Inclinometer/Rido.

degree of knowledge one usually has of the different soils there are on a specific site.

The proposed ground-retaining wall friction to be considered in the calculations falls within the range: $\delta = \varphi/3 - 2\varphi/3$. It is recommended to reject the $\delta = 0$ option because considering zero friction between the ground and the retaining wall makes calculation programs with elasto-plastic springs perform an unreal overestimation of the movements for deep excavations.

4.1 *Plaza de Cuba Station*

The best fit found in the results of the calculations compared with the reality measured on the inclinometers is shown in Figure 3. Plaza de Cuba Station. Adjusted displacements Inclinometer/Rido.

It has been possible to couple the curves fairly well in all the excavation phases: they drop with the same slope in the relevant depths of the excavated zone of the retaining walls and also fit in the embedded zones.

The maximum displacements in the six phases do not show significant differences between the curves of the inclinometer and those from Rido. The maximum differences are of 8%.

The deformation of the retaining wall during the first phases of excavation is that which allows adjustment of the k_h value in the sand level located between the depth of 2 and 15 m. The deformation of the retaining wall during the last excavation phases allows adjustment of the k_h value in the gravel and marl levels located below 15 m.

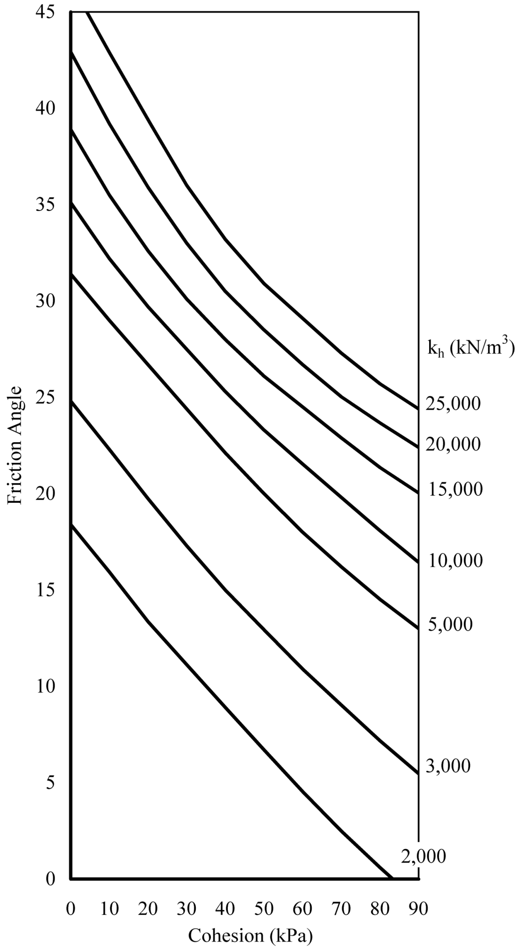

Figure 5. Arozamena's Abacus. Subgrade reaction coefficient for cast in situ walls of Seville Underground.

4.2 *San Bernardo Station*

The nearest fit found in the calculations to the reality measured on the inclinometers is represented in Figure 4. San Bernardo Station. Adjusted displacements Inclinometer/Rido.

It has been possible to couple the all the phases of the excavation fairly well: they drop with the some slope at the depths of the excavated zone of the retaining walls and also adapt to the embedded zones, taking into account the irregularities in the inclinometer curves.

The deformation of the retaining wall during the first phases of excavation is that which allows the k_h value to be adjusted on the clay level located between the depths of 1 and 10 m. The deformation of the retaining wall during the last phases of excavation allows adjustment of the value of k_h at the gravel level located between 10.00 and 19.50 m. The movements of the embedded part of the retaining wall, and particularly those of the foot allow adjustment of the coefficient value of the marl.

5 CONCLUSIONS

What has been performed in the preceding parameter analysis has been to consider the Seville Metro works as if they were a set of "in situ" trials on a large scale in order to determine the values of the horizontal subgrade reaction coefficient for retaining walls. The soils present are sufficiently varied to allow extrapolation of the results obtained.

The results are presented clearly and simply, relating the numerical value of the coefficient to the geomechanical parameters of the ground (cohesion and friction angle) and adopting the format recorded by Monnet (1994) and that he attributes to Chadeisson. This abacus is reproduced in Figure 5.

The designers are recommended to adopt these numerical values. The aim is for their prior predictions of movements to be nearer to reality once the excavation is performed and, thus, their evaluation of damage (arising from such movements) shall be based on results that maintain their validity until the end of the works.

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