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David R. Beadman
Tony Gee and Partners,
Cobham, formerly Arup
Geotechnics, London, UK

Eurocode design of underground metro structures

D. R. Beadman MA CEng FICE MIStructE

Eurocodes were used for the design of a new metro, for which the contractor's design was undertaken largely between 1996 and 1999, when the Eurocodes were in their infancy. This paper discusses the experience, the difficulties encountered and the benefits gained by the project, with particular reference to the geotechnical design of the underground stations. Eurocodes 1, 2 and 7 were used for the design of the retaining walls for the deep stations. Two design parameters created challenges for the structural design of the reinforced concrete embedded retaining walls: a restriction on the maximum allowable crack width, and a large cover to the reinforcement, required by the construction method. Benefits were gained by using the observational method.

1. INTRODUCTION

Eurocodes were used for the design of a new metro, under a design and build contract. The contractor's design was undertaken largely between 1996 and 1999, and this was the first occasion on which many members of the design team had used the Eurocodes. This paper discusses the experience, the difficulties encountered and the benefits gained by the project, with particular reference to the geotechnical design of the underground metro structures.

2. EUROCODES

The Treaty of Rome in 1975 paved the way for the development of the Eurocodes to eliminate technical obstacles to trade. The Eurocodes were gradually developed, and by 1995 a number of the key Eurocodes were available as European pre-standards. The metro client recognised the need for the involvement of international contractors in the construction of the proposed metro, and therefore specified the use of Eurocodes to try to ensure equal opportunities for potential contractors. The local national application document was not available at the time, so the client's engineers wrote a project application document to specify the key data needed for the Eurocodes.

The main Eurocodes used on this project applicable to the geotechnical design were:

(*a*) Eurocode 1: Part 1[1] (EC1). Covers the basis of design and the densities, self-weight and imposed load actions due to fire, snow, wind, thermal loads, loads during execution and accidental actions. This document has now been replaced by EN1990 *Basis of design* and EN1991 *Actions on structures* for loading.
(*b*) Eurocode 2: Part 1[2] (EC2). Contains the general basis for the design of structures in reinforced concrete, with specific rules for building structures. This document has been substantially revised in the development of the current version (2004).
(*c*) Eurocode 7: Part 1[3] (EC7). Contains the general design basis for the geotechnical aspects of the design of building and civil engineering works, including the geotechnical design of spread footings, retaining walls and piles and calculation rules for actions originating from the ground (soil and groundwater pressures).

The text of the Eurocodes defines Principles (EC7 Clause 1·3), designated by the letter P, which are general statements and definitions for which there are no alternatives, and Application Rules, which are examples of generally recognised rules that follow the Principles and satisfy their requirements. Alternative rules to those defined in the documents are permitted, provided they can be shown to satisfy the relevant Principles, although a greater degree of justification is required for such alternatives. Such designs may not be considered to be fully compliant with the Eurocodes.

3. EMBEDDED RETAINING WALLS

The embedded retaining walls on the project consisted of diaphragm walls and piled walls, which were a combination of hard/soft secant piles in the glacial materials, reducing to contiguous piles in the underlying limestone. The primary piles were formed using a 1180 mm diameter casing, which was extended only as far as the limestone, approximately halfway down the depth of the stations. In the limestone the piles were excavated at 1050 mm diameter, without a casing, dimensioned to suit a tool size to fit inside the 1180 mm diameter casing. The constant-diameter reinforcement cage was designed to fit inside the 1050 mm diameter pile, and therefore extra cover was provided within the 1180 mm diameter section.

4. SPECIFICATION

The client defined a 100-year design life for the structures, which his engineer translated into a number of specific requirements including a minimum concrete grade, minimum cover requirements and a maximum crack width, presented in a Project Application Document. In comparison with the usual requirements for a transportation project in the United

Kingdom, designed in accordance with BS5400,[4] these requirements were more onerous. The comparison is shown in Table 1.

5. DESIGN DOCUMENTATION

At the start of the project, each party had a different view concerning the design documents that were to be presented to justify the design, based largely on their previous *modus operandi*. Eurocode 7 states as a Principle that a Geotechnical Investigation Report (GIR) (EC7 Clause 3·4) and a Geotechnical Design Report (GDR) (EC7 Clause 2·8) are produced, and includes a suggested list of topics that should be included. The client's engineers applied this list as part of their checking criteria for the submitted documents, and some confusion was caused initially when it was unclear whether a submitted document was a GIR or a GDR. The early submissions were focused on allowing piling work to start in order to achieve the proposed construction programme, and therefore the full design was incomplete at this stage. It was useful to have the checklist to ensure that no subjects were missed, but the fact that the early submissions were unable to cover all of the items on the list caused some initial delays to the design approvals.

6. SOIL–STRUCTURE INTERACTION

The local normal design approach for embedded retaining walls was to use the Brinch Hansen method,[5] involving the use of plastic hinges to assess an ultimate limit state collapse mechanism at each construction stage. With the specified requirement to limit crack widths, it was necessary to investigate the development of the wall section forces as the excavation progressed, and to introduce the props into the calculation in sequence, which could not be achieved with the Brinch Hansen method. The Brinch Hansen method would have risked underestimating the bending moments and shear forces in the retaining wall by ignoring the development of member forces from previous construction stages. It was proposed to use the soil–structure interaction program WALLAP,[6] a pseudo finite element program, to analyse the embedded retaining walls, which allowed the construction sequence to be modelled.

Design Case A (EC7 Clause 2·4·2) was checked for each structure to ensure against flotation. A tension force was developed in the retaining walls where additional holding-down resistance was required from the embedment below excavation level. The design of the embedded retaining walls considered Cases B and C (EC7 Clause 2·4·2) for the ultimate limit states and a serviceability limit state loadcase to assess the crack width design and deflections. It should be noted that recent revisions to Eurocode 7 have amended the Case A, B and C terminology. The key design parameters are summarised in Table 2.

All three analyses were run using the WALLAP program, using the construction sequence for the temporary excavation stages as shown in Fig. 1. The design ultimate bending moments and shear forces were taken as the maximum values from Cases B and C, applying a model factor (EC7 Clause 2·4·2(17)) to the results of the Case B analysis to give ultimate bending moments and shear forces. A model factor of 1·35 was used (EC7 Table 2·1).

Case C analysis is to check the overall stability (in addition to checking the strength of the structural sections), and could have been applied to the maximum excavation stage in isolation, which is the most critical one in terms of the stability of the

Requirement	BS 5400	Metro specification
Design life: years	120	100
Concrete grade:* N/mm^2	$f_{cu} = 35$	$f_{ck} = 30$
Minimum cover for durability for retaining wall members, as concrete cast in non-aggressive ground: mm	35	50‡
Minimum cover for structures cast against the ground: mm	75	90‡
Maximum crack width:† mm	0·25	0·2‡

* The concrete grade is specified in terms of the cylinder strength (f_{ck}) in EC2, compared with the cube strength (f_{cu}) in BS 5400. Cylinder strength of 30 N/mm^2 is approximately equivalent to cube strength of 40 N/mm^2.

† The contract was based on this crack width being assessed at the minimum cover from reinforcement rather than the actual cover. The minimum cover was defined as the minimum cover required for the durability, independent of the method of casting the concrete (see Fig. 4).

‡ Note that the minimum cover for durability, the minimum cover for structures cast against the ground and the maximum crack width were specific values defined by the Project Application Document rather than by EC2.

Table 1. Comparison between BS 5400 and the metro specification (the Project Application Document)

Loadcase/design parameter	Serviceability loadcase	Ultimate loadcases	
		Case B	Case C
Partial factor on soil shear strength, c_u/c'	1·0	1·0	1·6
Partial factor on soil friction, $\tan \phi'$	1·0	1·0	1·25
Over-dig allowance	0	Lesser of 0·5 m or 1/10th of the depth below the last prop	
Factor on resulting bending moments and shear forces	1·0	1·35 (applied to the output as a model factor; EC7 Clause 2·4·2(17))	1·0

Table 2. Key design parameters

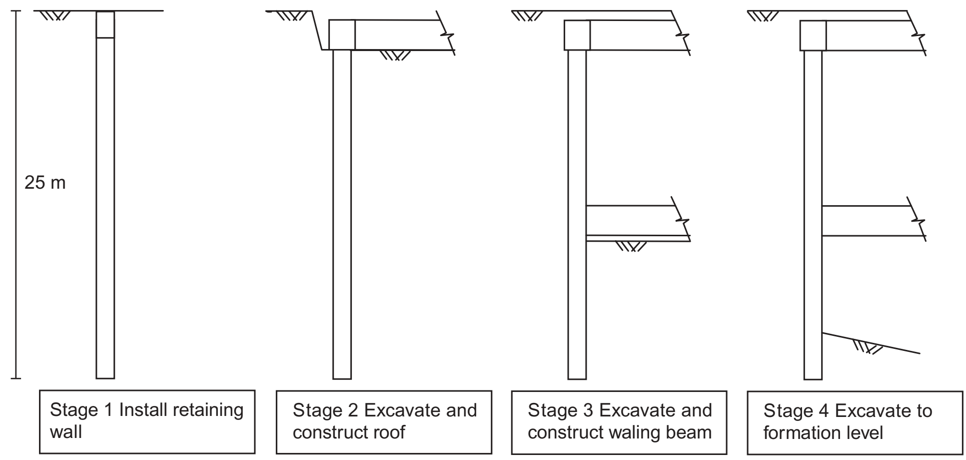


Fig. 1. Temporary construction stages analysed for Cases B and C

embedded section of the wall. The assessment of Case C for every construction stage was unnecessary when only the final stage was the critical one to check the toe depth of the retaining wall, although the code requires it to be checked for every stage. Fig. 2 illustrates the limited analysis required to satisfy Case C, without the need to model the stress history of the wall through each construction stage. In other words, Case C is satisfied by considering each stage in isolation. However, the WALLAP program proved a simple means of assessing Case C, following the excavation stages as shown in Fig. 1 and applying the factored strength parameters throughout. The ultimate bending moment and shear forces from Case C were less than those generated from Case B (once the model factor was applied) at all stages except for the maximum excavation stage.

There was some difficulty in explaining the above design approach to the client's engineers, but this was due to their historic use of the Brinch Hansen method and their lack of familiarity with a soil–structure interaction approach, rather than any difficulty with the application of the Eurocodes.

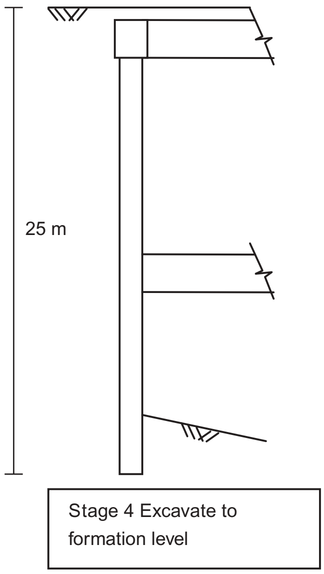


Fig. 2. Case C only applied to final excavation stage

7. STRUCTURAL DESIGN

The use of Eurocode 2 was initially difficult because of the inevitable learning curve and the need to adapt the parties' design methods, for example from BS 8110[7] and BS 5400[4] for the parties from the UK. Section design programs and standard spreadsheets had to be adapted to allow for Eurocode 2 requirements and methods. The writing and checking of the proprietary section design programs proved time-consuming, and eventually designer's own spreadsheets were written to cover the rectangular and circular section analyses needed to design the retaining walls to ensure that the design programme could be achieved.

A further difficulty was the limitation of Eurocode 2 Part 1 as a code for buildings rather than a code for heavy civil engineering. At the time of the design, further Parts 3 and 6 were planned to include *Concrete foundations and piling* and *Massive civil engineering structures* (EC2 Clause 1·1·3). This is now not the case, and these parts are no longer planned. This caused some difficulty with the design of the embedded retaining walls, which had to be cast underwater using a tremie pipe without the ability to use mechanical vibration of the wet concrete. To ensure adequate compaction, specific minimum bar spacing requirements are recommended in prEN 1536.[8] The practical design of the reinforcement cages to ensure competent concrete with the need to satisfy the onerous specification requirements resulted in the use of large-diameter bars in bundles. The limiting design criterion was the 0·2 mm maximum crack width. The stress in the reinforcement was low in order to limit the crack width, so a large area of longitudinal reinforcement was required to resist the applied bending moments. The option of increasing the pile diameter above the chosen nominal 1200 mm diameter was not adopted, for the following reasons:

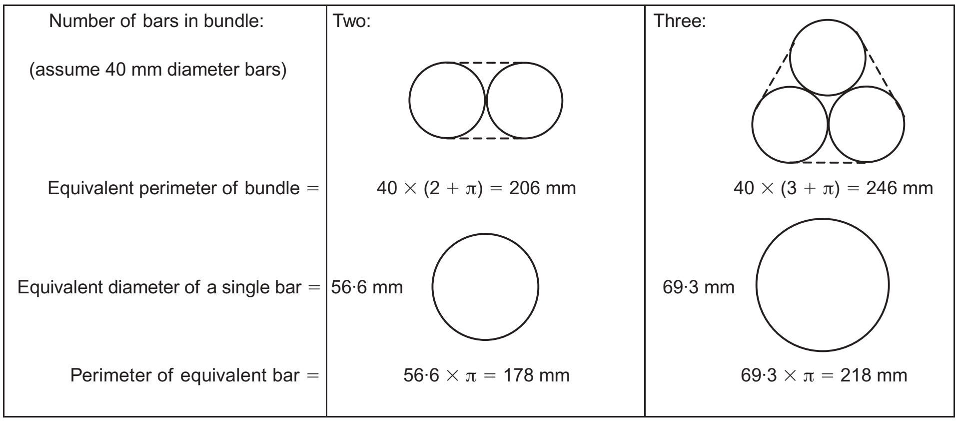
(*a*) An increased pile diameter gave an increased pile spacing, which increased the applied bending moment, negating the benefit gained from the increased effective depth.
(*b*) The increased pile stiffness would have attracted additional bending moments.
(*c*) The available space on the sites to build the station was tight, and the additional width to accommodate larger piles would reduce the clearance to the surrounding buildings, their foundations and underground services.
(*d*) The piling equipment for a larger diameter was not readily available.

Three specific problems were encountered with Eurocode 2, for which concessions were sought and granted on this project:

(*a*) Eurocode 2 (EC2 Clause 5·2·6·3) did not permit the lapping of bars greater than 32 mm diameter as a Principle. The proposed cage used bundles of two, and in some cases three, 40 mm diameter bars. The use of couplers within the cage pushed the bars in the bundles further apart, reducing the clear space between the bundles. This was detrimental to the flow of concrete through these gaps into the cover zone, which contained a large volume of concrete due to the specified cover to the reinforcement. Lapped bars at 40 mm diameter were permitted because of the low working design stress in the bars. Note that the current EC2, updated in 2004, does allow the lapping of such bars in sections greater than 1 m thick if the stress is less than 80% of the design ultimate strength, f_{yd}. Designed links are required in the lap zone (EC2 Clause 8·8.(4), 2004).

(*b*) Eurocode 2 (EC2 Clause 5·2·7·1) did not permit the use of a bundle of equivalent diameter greater than 55 mm, which is less than two 40 mm bars (56·6 mm equivalent diameter). This is not stated as a Principle. Bundles of three 40 mm bars were permitted in this case because of the low working design stress in the bars. This restriction still applies in the current code (EC2 Clause 8·9·1 (2), 2004).

(*c*) Eurocode 2 (EC2 Clause 5·2·7·1(2)) states that a bundle of bars should be considered as a notional bar having the same sectional area (also not a Principle). The equation for the calculation of the crack width (EC2 Clause 4·4·2·4) considers the bond between the bar and the concrete. A bundle of bars has considerably better bond characteristics than the nominal bar of equivalent area, as illustrated in Fig. 3. The equations were adjusted to suit the use of the correct perimeter. This limitation in the crack width calculation still applies in the current code, with some relaxation for the two-bar situation.



Number of bars in bundle: (assume 40 mm diameter bars)	Two:	Three:
Equivalent perimeter of bundle =	40 × (2 + π) = 206 mm	40 × (3 + π) = 246 mm
Equivalent diameter of a single bar =	56·6 mm	69·3 mm
Perimeter of equivalent bar =	56·6 × π = 178 mm	69·3 × π = 218 mm

Fig. 3. Comparison between the perimeter of a bundle and the perimeter of an equivalent bar

Obtaining concessions for Application Rules is clearly easier to achieve than for Principles, provided suitable justification is presented to prove that the relevant Principle is satisfied. In this instance, a concession was obtained for one Principle, as noted above.

There was also a difficulty using the Eurocode 2 equations to check the crack width at the nominal cover. The Eurocode 2 equations are based on the area of concrete in tension rather than the distance between the face of the tension bar and the point of assessment of the crack (as used in BS 5400, for example). The area of the concrete in tension was reduced as illustrated in Fig. 4, ignoring the extra concrete outside the nominal cover, which also shows the additional cover in the 1180 mm diameter pile due to the cage being dimensioned to suit the 1050 mm diameter section. There is no agreed method of applying the effective concrete tension approach to circular sections.

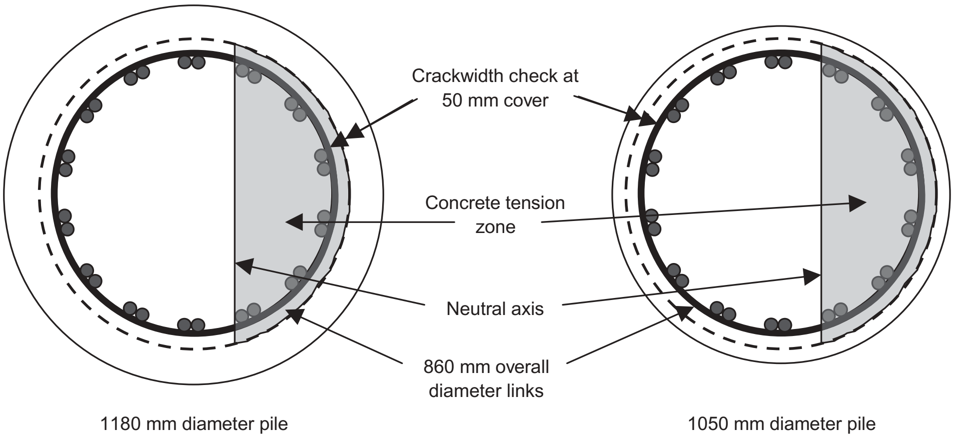


Fig. 4. Crack width calculation parameters adopted for 1180 mm diameter piles

8. OBSERVATIONAL METHOD

Eurocode 7 permits the use of the observational method (Nicholson[9] and EC7 Clause 2·7) as one method of designing geotechnical elements. The observational method was used during the construction of one of the deep stations to demonstrate that one of the propping levels was not required, thereby saving considerable cost and reducing the programme for this particularly critical element of the works. The four requirements for the use of the observational method as defined in Eurocode 7 were met in full:

(*a*) The limits of behaviour of the retaining wall were defined in terms of acceptable deflections.

(*b*) The range of possible behaviour was identified from measurements made during a similar deep excavation for a station, taken previously on the same project.

(*c*) A monitoring plan was devised to record the wall deflections, using both electronic inclinometer strings and manually read inclinometers. Various hold points during the excavation were defined to compare the predicted movements with the measured movements, to give ample warning of the possible need to install the props.

(*d*) The contingency plan was to install the extra level of props. To reduce any delays if the contingency measure had to be used, the props were fabricated and the site team was trained in their installation. In the event, the props were not required.

9. CONCLUSIONS

The use of Eurocodes on this project brought a number of benefits.

(*a*) They ensured a consistent approach to the design.
(*b*) They provided a common language for the different nationalities to discuss the design.
(*c*) The use of the observational method was accepted.

Inevitably there were inefficiencies at the start of the design process as the designers learnt about the new code and the design tools were created. Geotechnical engineers are not used to working according to a defined process, and some of the application rules within Eurocode 7, used as strict checklists, caused some frustration. The use of a partial suite of new codes led to some detailed design discussions owing to their inapplicability to parts of the design.

The project provided the opportunity for the design team to experience the use of Eurocodes in advance of their formal introduction.

10. ACKNOWLEDGEMENTS

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