ID: 12281

DIAPHRAGM WALLS USED AS PERMANENT STRUCTURES

Evaluation of bond between reinforcement and concrete in connection to project Citytunneln

Jonas Magnusson

Alexandre Mathern

NCC Teknik

Göteborg, 14 Februari 2013

SBUF stödjer
forskning & utveckling
som leder till
praktisk handling

SUMMARY

The aim of this study is to investigate the bond between concrete and reinforcement when casting under bentonite slurry, which is common practice for constructing diaphragm walls. Previous studies reported that the bond strength is lower in such cases than when casting under normal conditions. The reasons for this decrease in bond strength were believed to come either from the slurry, which could lead to a smoother interface between reinforcement and concrete, and/or from the strength of the concrete, which could be lower when cast under slurry.

The present project has been financed by the Development Fund of the Swedish Construction Industry (SBUF) and the Swedish Railway Administration (Banverket). It follows a project about diaphragm walls (ID 11603 och 11796), initiated by the Swedish Road Administration (Vägverket) in 2004, and financed by Vägverket, Banverket and SBUF. The aim of this project was to assess whether diaphragm walls could be allowed as permanent constructions in Sweden. Subsequent to this project, the Swedish Transport Administration (Trafikverket which replaced Banverket and Vägverket) is now accepting diaphragm walls as permanent constructions.

In order to study the influence of casting with the diaphragm wall technique, a series of field tests were carried out at the Citytunnel project in Malmö, where diaphragm walls were built as temporary structures. Pull-out tests were conducted on bars embedded in diaphragm wall panels and in a reference panel to study the bond strength. Concrete cubes cast at the same time as the panels and cores drilled from the diaphragm wall panels and the reference panel were tested to evaluate the concrete compressive strength.

The results of the concrete tests indicated that the concrete cast in diaphragm wall panels reached the same strength as the concrete cast in the reference panel. Therefore it was concluded that casting under bentonite slurry and with the surrounding soil as form did not seem to affect the concrete strength.

The pull-out tests resulted in bond capacities at least 40 % lower in average for the bars in the diaphragm wall panels in comparison to the ones in the reference panel. These results indicate that casting under bentonite can significantly reduce the bond strength. However, the bond capacities obtained for the diaphragm wall panels were consistent with experimental results, from tests carried out without the effect bentonite, reported in the literature. One the other hand, the values from the reference panel were higher than expected compered to results from previous studies.

The results of the experimental study conducted at the Citytunnel project were completed with results from previous tests on diaphragm wall panels and on specimens cast under bentonite conducted in parallel to the Götatunnel project, as well as from other tests on the effect of bentonite from the literature.

Keywords: Diaphragm walls, reinforced concrete, bond capacity, anchorage, bentonite slurry, pull-out tests, ribbed bars, compressive strength

SAMMANFATTNING

Syftet med projektet var att undersöka vidhäftningsförmågan mellan betong och armering vid gjutning under bentonitslurry. Tidigare studier indikerar att vidhäftningen är lägre i detta fall jämfört med när gjutning sker under normala förhållanden. Förmodade orsaker till detta är dels att betongen, på grund av inblandning av stödvätska och/eller smuts får en lägre hållfasthet samt att närvaro av stödvätska kan ge upphov till ett ”glattare”, och därmed vekare, gränsskikt mellan armering och betong.

Detta projekt har finansierats av SBUF och Banverket. Det är en fortsättning av ett stort branschgemensamt utvecklingsprojekt (ID 11603 och 11796) om användandet av slitsmurar i permanenta konstruktioner, som initierats av Vägverket 2004, och som finansieras av Vägverket, Banverket och SBUF. Syftet med det första projekt var att bedöma om slitsmurar kunde tillåtas som permanenta konstruktioner i Sverige, och efter projektets slut accepterar nu Trafikverket slitsmurar som permanenta konstruktioner.

För att studera inverkan av gjutning med slitsmursteknik har fältförsök utförs i samband med Citytunnelprojektet i Malmö, där slitsmurar byggdes som tillfälliga konstruktioner. Utdragsförsök utfördes på armeringsstänger ingjutna i slitsmurspaneler och i en referens-panel för att studera vidhäftning mellan betong och armering. Betongkuber, gjutna samtidigt som panelerna, samt kärnorna, urborrade från slitsmurspanelen och referens-panelen, testades också för att utvärdera betongens tryckhållfasthet.

Resultaten från provningen av kuber och kärnor visade att betongen i slitsmurspanelerna uppnådde samma hållfasthet som den i referenspanelen. Det vill säga gjutningen under bentonitslurry tycks inte ha påverka betongens hållfasthet.

Vidhäftningshållfastheten från utdragsförsöken i slitsmurspanelerna var i genomsnitt minst 40 % lägre jämfört med de i referenspanelen. Dessa resultat tyder på att gjutning under bentonit väsentligt kan reducera vidhäftningshållfastheten. Det är dock värt att notera att värdena på vidhäftningshållfastheten från slitsmurspanelerna ligger i nivå med experimentella resultat, utan inverkan av bentonit, från litteraturen. Däremot var värdena från referenspanelen mycket högre än väntat.

Resultaten från den experimentella studie som genomförts vid Citytunnelprojektet kompletterades med resultat från Götatunnelprojektet, samt med resultat från andra försök som finns redovisade i litteraturen.

Nyckelord: Slitsmurar, armerad betong, vidhäftningshållfasthet, förankring, bentonit-slurry, utdragsförsök, kamstänger, tryckhållfasthet

TABLE OF CONTENTS

PREFACE

The present project has been financed by the Development Fund of the Swedish Construction Industry (SBUF) and the Swedish Railway Administration (Banverket). It follows a project about diaphragm walls, initiated by the Swedish Road Administration (Vägverket) in 2004, and financed by Vägverket, Banverket and SBUF. The aim of the previous project was to assess whether diaphragm walls could be allowed as permanent constructions in Sweden. Subsequent to this project, the Swedish Transport Administration (Trafikverket which replaced Banverket and Vägverket) is now accepting diaphragm walls as permanent constructions.

The experiments in this project were carried out by NCC Construction Sverige AB and Hercules Grundläggning AB in parallel to the Citytunnel Project, for which diaphragm walls were built as temporary structures. Tests on concrete cubes were conducted by Sydsten AB.

The financial support provided by SBUF and Banverket is acknowledged.

Celsa Fundia is acknowledged for providing bars and coupling sleeves.

Göteborg, February 2013

Jonas Magnusson, Alexandre Mathern

1. INTRODUCTION

1.1 Background

Diaphragm walls are underground structural elements in reinforced concrete constructed directly in the ground. The construction technique of diaphragm walls consists in casting concrete in a deep trench excavation using earth as a form. The stability of the excavation is ensured by using a support fluid, often a bentonite slurry. Concrete is then poured in from the bottom of the excavation in order to gradually push out all the slurry.

Tests conducted at the Götatunnel project in Gothenburg (Mahesar and Masuiddin 2004) and other tests reported in the literature (Jones 2004 and 2005) indicate that the bond strength between concrete and reinforcement may be lower and the bond-slip relation weaker when concrete is cast under a bentonite slurry.

The consequence of a bond strength reduction is that the distance between the cracks, and thus the cracks width, increases, which lead to a reduction of the structure’s stiffness. The anchorage length and lap length also need to be increased in design.

Two possible reasons for the bond properties to deteriorate were identified and will be investigated in this study. These reasons are that the casting method causes a reduction of concrete strength and that the presence of residue from the slurry at the interface between reinforcement and concrete results in a weaker and smoother boundary layer.

1.2 Aim of the study

The aim of the study is to investigate the effect of casting with the diaphragm wall technique on the bond between reinforcement and concrete and on the concrete strength.

1.3 Method

The study is based on field tests conducted on diaphragm wall panels at the Citytunnel project in Malmö. Pull-out tests were conducted on diaphragm wall panels and on a reference panel to study the bond strength. Tests on reference cubes cast at the same time as the panels and tests on cores drilled from the diaphragm wall panels and the reference panel were also conducted to study the compressive strength of concrete.

The results of the study are completed with information from the literature and with the results from previous tests on diaphragm wall panels and on specimens cast under bentonite conducted in parallel to the Götatunnel project and at Chalmers University of Technology.

2. BOND AND ANCHORAGE OF RIBBED BARS

2.1 Local bond-slip relationship

Bond properties between reinforcement and concrete depend on the local relation of the relative motion between the materials, so-called bond-slip relation, see Figure 2.1 (Magnusson 2000).

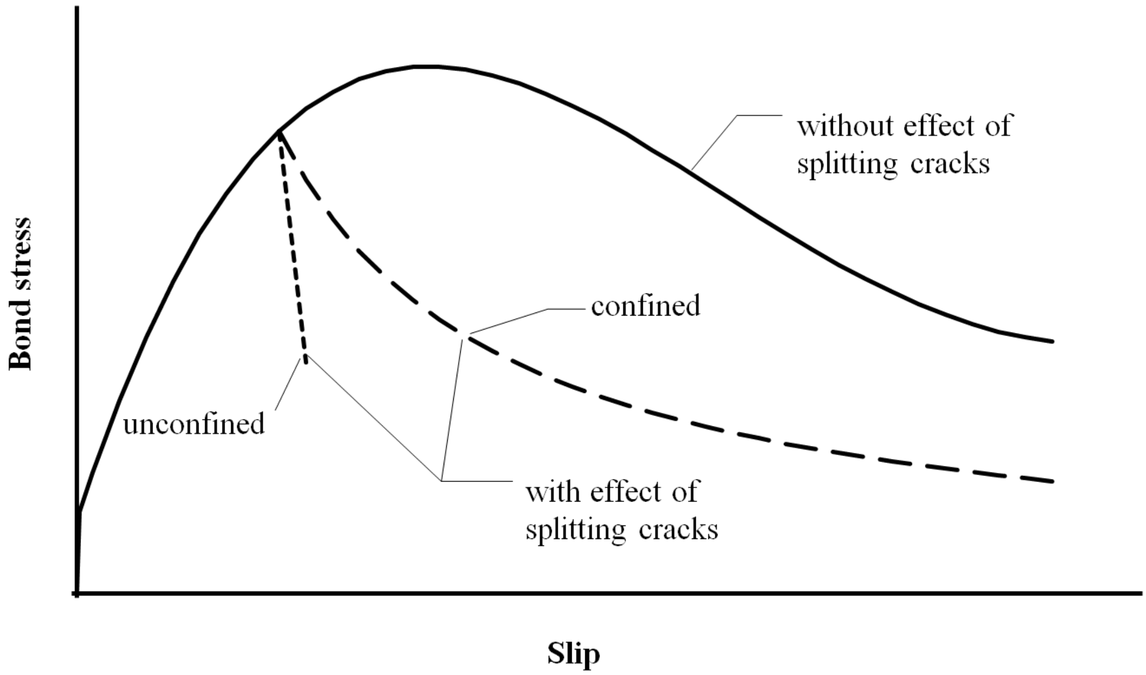


Figure 2.1: Basic overview of the local bond-slip, with and without the effect of splitting cracks and confinement

Initially, the force transmission takes place through adhesion; the slip between reinforcement and concrete being negligible at this stage. When adhesion is released the forces are transferred through friction and mechanical interlocking and some slippage occurs between the reinforcement and the concrete. Locally at the ribs, the forces are important and cause some cracking in the concrete, which in turn increases the sliding between reinforcement and concrete and reduces the bond. The bigger the forces undertaken by the reinforcement are, the wider the cracks and the bigger the sliding are. Crushing of concrete adjacent to the ribs or shearing of concrete between adjacent ribs is crucial for maximum bond strength, see the continuous curve in Figure 2.1.

The transmission of force from the reinforcing bar to the surrounding concrete gives rise to hoop stresses. If the concrete cover layer is small relative to the diameter of the reinforcing bar, these ring stresses can exceed the concrete tensile strength, resulting in splitting cracks. These splitting cracks mean that the bond curve is as the dashed curves in Figure 2.1. If the splitting cracks do not penetrate all the concrete cover, or if there is reinforcement perpendicular to the splitting cracks, it leads to a gradual reduction of adhesion. However, if the splitting cracks penetrate the cover layer and the reinforcement perpendicular to these splitting cracks is not enough, a brittle failure occurs.

The local bond-slip relationship determines how the steel stresses, bond stresses and slip vary along the reinforcing bar, and thus it also determines the anchorage length, the splicing length, the distance between cracks and the crack width.

The two main reasons suspected for the bond properties to deteriorate when casting with the diaphragm wall technique, i.e. lower strength of concrete and possible accumulation of slurry at the interface between reinforcement and concrete, should affect the local bond-slip relationship as follows:

- The stiffness of the relationship between bond stress and slip should decrease due to the lower strength of concrete and to the weaker and less homogenous boundary layer.
- The maximum bond stress and the bond stress for which possible splitting cracks arise should decrease due to the lower strength of concrete.

2.2 Bond and anchorage according to CEB-FIP Model Code 1990

2.2.1 Maximum bond stress

According to CEB-FIB Model Code 1990, the maximum bond stress τ_{max} in confined concrete (leading to failure by shearing of the concrete between the ribs as opposed to splitting of the concrete) and with good bond conditions is determined by:

$$\tau_{max} = 2.5 \cdot \sqrt{f_{ck}} \tag{2.1}$$

where:

f_{ck} is the characteristic value of the cylinder strength of concrete at 28 days

If the bond conditions were not good, the maximum bond stress would be two times less than in Equation 2.1.

As explained by Magnusson (2000), according to the original relation this formula is derived from, it is appropriate to use the following equation in order to do a comparison with experimental results:

$$\tau_{\max} = 2.5 \cdot \sqrt{f_{cm}} \tag{2.2}$$

where:

f_{cm} is the mean value of the concrete cylinder compressive strength.

2.2.2 Design bond stress

The ultimate bond strength f_{bd} is defined in CEB-FIP Model Code 1990 as proportional to the design value of the concrete tensile strength f_{ctd}.

$$f_{bd} = \eta_1 \cdot \eta_2 \cdot \eta_3 \cdot f_{ctd} \tag{2.3}$$

where:

η_1 depends on the type of reinforcement

η_1 = 1.0 for plain bars

η_1 = 1.4 for indented bars

η_1 = 2.25 for ribbed bars,

η_2 depends on the quality of the bond condition and the position of the bar during concreting:

η_2 = 1.0 when good bond conditions are obtained

η_2 = 0.7 otherwise,

η_3 depends on the bar diameter Φ:

η_3 = 1.0 for $\Phi \leq 32$ mm

η_3 = (132 – Φ)/100 for $\Phi > 32$ mm.

2.2.3 Anchorage length

The basic anchorage length $l_{b,rqd}$ defined in CEB-FIP Model Code 1990 corresponds to the required length to transfer the yield force of a reinforcement bar of diameter Φ:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{f_{yd}}{f_{bd}} \tag{2.4}$$

where:

Φ	is the diameter of a reinforcement bar
f_{yd}	is the design yield strength of reinforcement

The design anchorage length l_{bd} is defined by:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \cdot \frac{A_{s,cal}}{A_{s,ef}} \geq l_{b,\min} \tag{2.5}$$

where:

$A_{s,cal}$	is the area of reinforcement required,
$A_{s,ef}$	is the area of reinforcement provided,
α_1	takes into account the form of the bar,
α_2	takes into account the influence of one or more welded transversal bar along the anchorage length,
α_3	takes into account the confinement effect of the concrete cover,
α_4	takes into account the confinement effect of transverse reinforcement,
α_5	takes into account pressure transverse to the plane of splitting along the design anchorage length,
$l_{b,min}$	is the minimum anchorage length

for anchorage in tension: $l_{b,min}$ > max (0.3·$l_{b,rqd}$; 10·Φ ; 100 mm).

2.3 Bond and anchorage according to EN 1992-1-1 (2005)

2.3.1 Ultimate bond stress

The formulation for the design value of the ultimate bond stress in EN 1992-1-1 (2005) is the same as the one in CEB-FIP Model Code 1990 for ribbed bars (Equation 2.4). The design value of the concrete tensile strength in the formula should be limited to the one corresponding to a characteristic value of 3.1 MPa (C60/75) because of the increasing brittleness of high-strength concrete.

2.3.2 Anchorage length

The basic anchorage length $l_{b,rqd}$, as defined in EN 1992-1-1 (2005), corresponds to the required length to transfer the design stress of a reinforcement bar of diameter Φ:

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd}} \tag{2.6}$$

where:

σ_{sd} is the design stress of the bar at the position from where the anchorage is measured from.

The design anchorage length is defined in EN 1992-1-1 (2005) by:

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \geq l_{b,\min} \tag{2.7}$$

The coefficients $\alpha_1 - \alpha_5$ and the minimum anchorage length $l_{b,min}$ are the same as the ones defined in CEB-FIP Model Code 1990 (Equation 2.5).

The design anchorage length defined in EN 1992-1-1 (2005) is therefore equivalent to the formulation of CEB-FIP Model Code 1990 as:

$$\sigma_{sd} \cdot A_{s,ef} = f_{yd} \cdot A_{s,cal} \tag{2.8}$$

2.4 Bond and anchorage according to BBK 04

2.4.1 Crack width, crack spacing and structure stiffness

According to BBK 04 chapter 4.5.5, the crack width characteristic value w_k, crack spacing s_{rm}, and the impact of concrete in tension between cracks ν are determined by:

$$w_m = \nu \cdot \frac{\sigma_s}{E_s} \cdot s_{rm} \tag{2.9}$$

$$\nu = 1 - \frac{\beta}{2.5 \cdot \kappa_1} \cdot \frac{\sigma_{sr}}{\sigma_s} \geq 0.4 \tag{2.10}$$

$$s_{rm} = 50 + \kappa_1 \cdot \kappa_2 \cdot \frac{\phi}{\rho_r} \tag{2.11}$$

The coefficient ν takes into account how bond affects the structure stiffness, and κ_1 takes into account the influence of bond between reinforcement and concrete ($\kappa_1 = 0.8$ for ribbed bars; 1.2 for profiled rods, and 1.6 for plain bars). This means that any negative effect on bond could be addressed by introducing an adjustment of κ_1. This could be done by κ_1 multiplied by a factor which takes into account the effect of casting with the diaphragm wall technique.

2.4.2 Bond capacity of reinforcement

According to BBK 04 Chapter 3.9.1.2, the bond capacity f_b, which is used to determine the required bond length and splicing length of reinforcement bars, can be determined by:

$$f_b = \eta_1 \cdot \eta_2 \cdot \eta_3 \cdot \eta_4 \cdot f_{ct} + \Delta f_b \leq \eta_2 \cdot \eta_b \cdot f_{ct} \tag{2.12}$$

where:

η_1	depends on the surface condition of the reinforcement (bond properties),
η_2	accounts for the position of bars during casting and transverse tensile stresses,
η_3	accounts for bundles of bars,

η_4 depends on the concrete cover and bar spacing,

f_{ct} is the design value of concrete tensile strength (limited to the one corresponding to a characteristic value of the tensile strength of 2.7 MPa),

Δf_b takes into consideration the influence of transverse reinforcement,

η_b specifies the upper limit for pull-out failure and depends on the surface of the reinforcement.

Three of these factors can be affected when casting with diaphragm wall technique: η_1, η_b and f_{ct}. The effect on η_1 and η_b can be handled in the same way as for crack width calculations, that is to say that the design bond is multiplied by a reduction factor (alternatively two different factors, one for η_1 and one for η_b) which takes into account the effects of casting under bentonite slurry. The reduction of the concrete tensile strength due to the casting process is already taken into account in f_{ct} and hence does not need to be considered again.

The same reasoning applies when designing according to Betonghandboken – Konstruktion (1990) chapter 3.9:122.

3. EXPERIMENTAL METHODOLOGY

3.1 Choice of experimental methodology

The necessary input data to study the effects of casting under bentonite slurry on the bond between reinforcement and concrete and on the concrete strength need to be determined by testing.

Three main options for the tests were identified:

- Clean laboratory tests, i.e. experimental samples are manufactured and tested in a laboratory environment.
- Simple field tests, i.e. diaphragm wall panels are constructed so that embedded reinforcing bars can be tested in the field.
- A combination of field and laboratory studies, where for instance samples are sawed from diaphragm wall panels to be transported to a laboratory where the reinforcing bars samples are uncovered and tested.

The choice of performing experiments in a clean laboratory environment, and with casting conditions that differ from the actual manufacturing process, can be justified by the better control over the manufacturing and testing process than in the field. It is usually very difficult to establish a numerical model based on a full scale test in the field. However, it is still desirable to also carry out field experiments to ensure that the results and numerical models established based on laboratory experiments are relevant. The third option, to saw the test specimens and then transport them to a laboratory where reinforcement is exposed and tested is very expensive, because among other things, it requires sheet piling and sawing to extract the samples in the field. Besides, there is a high risk that the samples are damaged during extraction and transport or that the bond between the embedded rods and concrete is weakened due to chocks or vibrations.

For this study, the tests conducted consist of simple field trials performed in parallel to the Citytunnel project for which diaphragm walls were erected as temporary structures.

3.2 Description of experiments

In order to study how the supporting fluid and the surrounding soil affect the bond capacity, pull-out tests have been conducted on both reinforcing bars embedded in diaphragm wall panels and in a reference panel. The diaphragm wall panels were cast under a bentonite slurry with earth as a form, The reference panel was built by a traditional casting method without any slurry, above ground, but without vibrating the concrete (see Figure 3.1 and pictures in Appendix A). Both types of panels were 0.8 m in width. The diaphragm wall panels were approximately 8.6 m in length and were cast using two tremie pipes, while the reference wall was 2.2 m in length and therefore cast using only one pipe (Alén et al. 2006).

The bars intended for the pull-out tests were placed vertically, more information on the experimental assembly are given in Section 3.4. All the pull-out tests were performed in the field at the Citytunnel project.

Figure 3.1: Reference panel (left) and diaphragm wall panels (right)

The results from the tests on concrete have been reported and discussed by Alén et al. (2006).

Concrete samples were examined in fresh and hardened state. In the fresh state, the concrete consistency was checked immediately after casting by measuring the slump value.

The compressive and splitting tensile strengths and the bulk density were determined at 28 days on concrete cubes that had been cast at the same time as the panels.

Tests on cores drilled from the diaphragm wall panels and the reference wall panel have also been performed to determine the compressive and splitting tensile strengths and the bulk density. Other properties studied on cores are the open porosity, the water capillary absorption and diffusion and the chloride migration.

For the Citytunnel project, diaphragm walls have been used as temporary support structures during construction. Compared with permanent constructions, temporary constructions are differently loaded and do not have any crack-width limitations with regard to long-term tightness and resistance. The concrete composition and the amount of reinforcement are therefore different in the two cases, water-cement ratio is higher for the temporary construction (see Section 3.3.1) and the reinforcement amount is lower.

3.3 Material

3.3.1 Concrete

This study focuses on the effect of execution, which means that the diaphragm wall panels and the reference panel were cast with the same concrete mix but with different casting conditions as the reference wall was cast without supporting fluid. In accordance with EN 1538 (2010), the specifications for the concrete used for the diaphragm wall panels and the reference panel were the following (Alén et al. 2006):

- Strength class: C 25/30
- CEM II A/LL 42.5 R

  (cement content: 365 kg/m^3, limestone content: 80 kg/m^3)
- Water/cement ratio: w/c = 0.60
- Consistency class: F5 (slump flow value = 560-620 mm)
- Maximum aggregate size: d_{max} = 16 mm

3.3.2 Reinforcing steel

It is specified in EN 1538 (2010) that the requirements for reinforcing steel in diaphragm walls are the same as the ones defined in EN 10080 (2005) for traditional concrete structures.

The reinforcement bars used in this experiment for the pull-out tests were of grade B500B with a diameter of 25 mm. The mechanical properties that were determined for the rods outside the tests in the project are the yield strength (550 MPa) and the ultimate strength (650 MPa), corresponding to the mean values of three tests.

3.4 Experimental assembly for pull-out tests

A mean bond capacity between reinforcement and concrete is measured indirectly by measuring the force applied to the reinforcing bar in a pull-out test. However this method can only be used with short embedment lengths in order to assume a uniformly distributed bond stress along the embedded length (Magnusson 2000).

For that reason, the reinforcement bars to be tested in the diaphragm wall panels and in the reference panel were placed in electrical PVC pipes slightly larger than the diameter of the bar and sealed at both ends with silicone, in order not for the concrete to penetrate inside during casting (see Figures A.1 to A.4 in Appendix A and drawings in

Appendix B). Only a certain length at the end of each bar was not debonded, corresponding to the embedment length of 125 mm, chosen to obtain anchorage failure. At that end, the bar was supported before casting by a small steel plate welded to the main vertical reinforcement. The other extremity of the bar, at which the tensile force was to be applied for the pull-out test, was threaded and covered with a polystyrene cap to allow access after casting the concrete. Each bar was threaded on the upper side so it could be extended at the time of testing using a coupling sleeve with internal API thread. In this way the coupling was found to be stronger than the reinforcement bar.

At testing, the extended bar was loaded using a hydraulic jack with a manual hydraulic pump and a pressure gauge. The hydraulic jack was installed on a steel platform centred over the bar to be tested (see Figure A.5). As the bars in the reference panel and in the diaphragm wall panels were anchored at a considerable depth under the loading area, it is evident that well-confined conditions were satisfied.

All pull-out tests were performed by the staff of the Citytunnel project. The bars and coupling sleeves were provided by Celsa Fundia.

3.5 Limitations

The pull-out tests are based on only one bond length and only the maximum bond strength is determined. It does not include the bond-slip relation due to the measurement method used.

4. CONCRETE COMPRESSIVE STRENGTH TESTS

4.1 Determination of 28-day strength

Tests conducted on 6 cubes of size 150 mm cast with the concrete used for the reference panel indicate an average 28-day cube compressive strength of 40.8 MPa with a standard deviation of 0.7 MPa, see Table 4.1. These tests were conducted on water-cured specimens.

Tests were also conducted on 28 cubes of size 150 mm cast with the concrete used for diaphragm wall panels of the Citytunnel project. The cubes were obtained from different concrete batches in order to be representative of the variation between batches, but they do not necessarily cover all the diaphragm wall panels where cores were drilled. These tests resulted in an average 28-day cube compressive strength of 39.7 MPa with a standard deviation of 4.2 MPa (see Table 4.1). The values range between 31.3 MPa and 48.3 MPa. These tests were conducted on specimens cured in water during 5 days and in air during 23 days. The results were translated to the situation of curing in water during 28 days using the factor 0.92 according to formula 11.11:17 in Betonghandbok – Material (1997).

The results of the tests are summarised in Table 4.1. The strength of water-cured cylinders is calculated by multiplying the strength of air-cured cubes by 0.76, according to formula 11.11:14 in Betonghandbok – Material (1997). As a consequence, the water-cured cylinder strength in Table 4.1 is equal to 0.826 times the water-cured cube strength (0.76/0.92).

Table 4.1: 28-day compressive strength results $f_{cm,cube}$ obtained from tests on cubes cast concurrently with the diaphragm wall panels and the reference panel, converted to equivalent 28-day strengths of water-cured cylinders $f_{cm,cyl}$

	Diaphragm wall panels	Reference panel
Number of tests	28	6
$f_{cm,cube}$ [MPa]	39.7	40.8
Standard deviation [MPa]	4.2	0.7
$f_{cm,cyl}$ [MPa]	32.8	33.7

4.2 Test on cores from diaphragm wall panels and reference panel

4.2.1 Core strength

Three series of tests were conducted to determine the compressive strength on cores: cores drilled from the reference panel were tested 105 days after casting, and cores drilled from six diaphragm wall panels after approximately 245 days and 275 days. Note that these diaphragm wall panels were different from the ones on which pull-out tests were conducted. The cores were taken approximately 5 m under the ground level, at different positions on the panels, either at the sides or at the middle (Alén et al. 2006).

All the cores tested were 99 mm in diameter and around 100 mm in length. They were prepared from longer samples of 250 mm to 450 mm, drilled around 33 days before testing. Therefore, two to four cores were obtained at each drilling position. The results obtained are summarised in Table 4.2 and plotted in Figure 4.1, see the three series of points called $f_{ci,core}$, and their respective mean value called $f_{cm,core}$.

These results can be directly compared with the values of water-cured cube strength obtained from tests on cubes at 28 days, described in Section 4.1. According to EN 13791 (2007), the strength value obtained by compression tests on cores of diameter 100 mm is equivalent to the one of 150 mm cubes cured under the same conditions, which is confirmed by Bartlett and MacGregor (2003) and True (2003).

The results of the tests on cubes are also plotted in Figure 4.1, see the series of points called $f_{ci,cube}$ for the cubes cast in parallel with the diaphragm wall panels and $f_{ci,cube}$ *Ref. Panel* for the cubes cast in parallel with the reference panel.

Table 4.2: Compressive strength results from tests on concrete cores from the diaphragm wall panels and the reference panel

	Diaphragm wall panels		Reference panel
Approximate time after casting [days]	245	275	105
Number of tests	11	23	6
$f_{cm,core}$ [MPa]	34.2	34.7	32.6
Standard deviation [MPa]	5.3	5.1	2.9

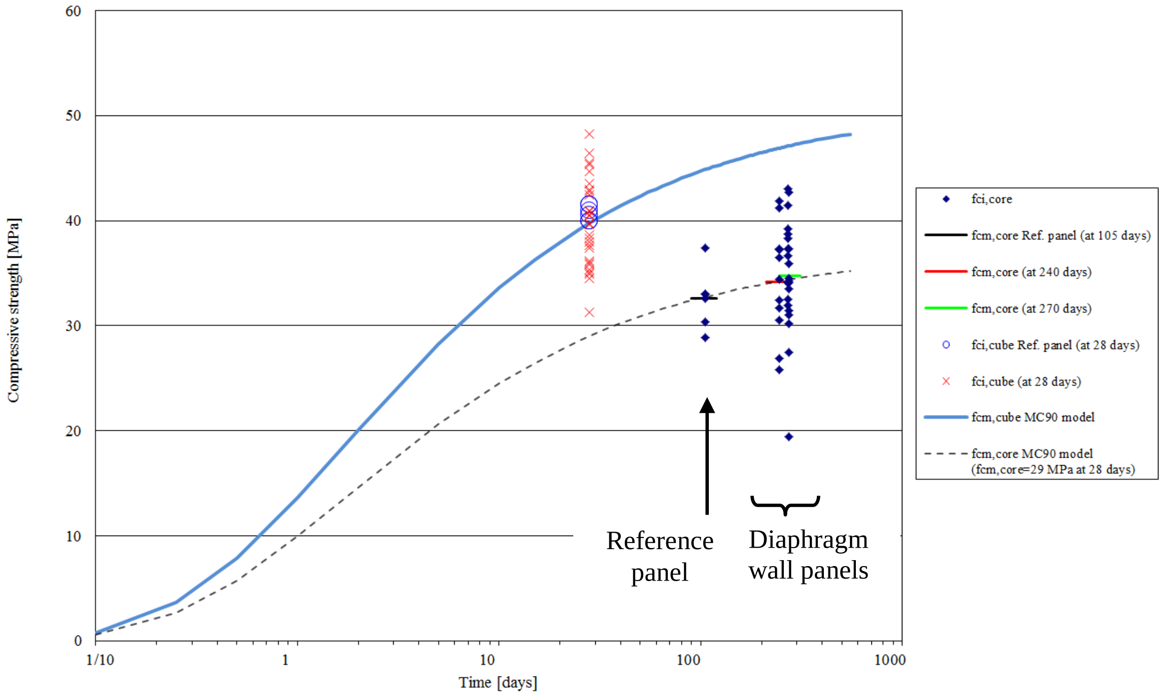


Figure 4.1: Compressive strength from tests on cubes and cores

The first observation that can be drawn out of these results is that the three series of measurements of the compressive strength on cores resulted in average strengths that seem consistent with each others, with a slight increase over time. This almost linear increase between the three average values seems to follow the development of strength with time described by the model of CEB-FIP Model Code 1990 (see the dashed curve in Figure 4.1).

These observations indicate that the conditions of casting with the diaphragm wall method did not influence the development of strength of the concrete. Indeed the cores drilled in the reference panel and in the diaphragm wall panels reached similar strength with time, therefore it can be concluded that casting under bentonite slurry and without real formworks did not affect the concrete strength.

Nevertheless the cores strength values are much lower than the value predicted at the same age by the model of CEB-FIP Model Code 1990 based on a 28-day strength of around 40 MPa, as obtained by tests on cubes (see the continuous curve in Figure 4.1). The majority of the values are even considerably lower than the average 28-day strength itself. The strength development reflected by these values would correspond to the estimation of CEB-FIP Model Code 1990 for a 28-day strength of 29 MPa, as represented by a dashed curve in Figure 4.1.

As the sample panel to determine the 28-day strength on cubes was rather large and spread in time and as the test procedure was well-defined, it is unlikely that a large error is associated to these test results.

The lower compressive strength values obtained by the tests of concrete cores than by the tests on equivalent cubes may be a sign that the in-situ compressive strength of concrete was actually somewhat higher than what was obtained by core testing. The possible reasons for it are numerous, such as damages of the cores due to drilling, the lower bond of cut aggregates, the direction relative to the casting, the moisture content, the presence of reinforcing bars in the cores, etc. These causes are further developed in Section 4.2.2.

It is also possible that the concrete of the diaphragm wall panels did not reach the same strength level as the one of the concrete test cubes tested to determine the 28-day compressive strength, which can result from the high temperature development in the panels during curing. This explanation is further developed in Section 4.2.3.

It should be mentioned, that for the tensile strength of concrete, the tests on the cores drilled in the diaphragm wall panels led to an average value of the splitting tensile strength (3.9 MPa) just slightly higher than the one obtained by the tests on cubes at 28 days (3.5 MPa). Besides, the average value of the splitting tensile strength of the cores from the reference panel was 3.7 MPa. On the contrary to the values obtained for the compression strength of cores, the results for the splitting tensile strength showed very little variation. Reference is made to Alén et al. (2006) for more information.

4.2.2 Evaluation of in-situ cube strength and potential strength

True (2003) indicates that, according to the Concrete Society Technical Report No. 11, the strength of cores drilled vertically is on average 8 % higher than the one of cores drilled horizontally. In this project the cores having been drilled horizontally in the diaphragm wall panels and in the reference panel, this could also explain why the strengths obtained by cores testing are lower than the ones obtained for the concrete test cubes.

Besides, the presence of reinforcing bars in the core can lead to a strength reduction of around 10 % according to Bartlett and MacGregor (2003).

4.2.3 Evaluation of the effect of the temperature on the compressive strength

Simple calculations of heat development in the concrete after casting a wall of 0.8 m with a similar concrete indicate that the maximum temperature reached in the concrete is around 50°C.

Studies evaluated the reduction of the 28-day compressive strength for concrete with byggcement (CEM II 42.5 A/LL), to be around 7 % for a temperature increase of 10°C (Löfgren 2011), and around 15-30 % for a temperature increase of 30°C (Jonasson and Fjellström 2011).

Therefore, it is also possible that the temperature development in the young concrete partly explains the difference between the strength of the cores and the one of the cubes.

5. PULL-OUT TESTS

5.1 Results from pull-out tests

The results from the pull-out tests performed on reinforcing bars embedded in the diaphragm wall panels and in the reference panel are summarised in Table 5.1. As already mentioned in Section 3.4, the diameter of the bars was 25 mm and the embedment length was 125 mm, i.e. five times the bar diameter.

Table 5.1: Results from pull-out tests conducted in the diaphragm wall panels and in the reference panel

	Diaphragm wall panels	Reference panel
Number of tests	8	4
Time after casting [days]	764	730
Range of the force applied at failure [kN]	124 - 192	247 - 266
Average bond stress $\tau_{m,max}$ [MPa]	16.2	26.4 [1)]
Standard deviation [MPa]	2.5	0.9

1) $\tau_{m,max}$ may be higher as failure is probably due to yielding of the steel

The average bond stress at failure is around 40 % lower for the tests conducted on the diaphragm wall panels than for the ones on the reference panel. As failure occurs down in the walls it was not possible to check how it occurred. Nevertheless, it is believed that the failure in the reference panel could be due to yielding of the steel, as the force applied at failure leads to a stress in the bar very close to the yield stress (for a bar of diameter 25 mm with yield strength of 550 MPa, yielding would occur for a force of 270 kN). Failure in the diaphragm wall panels probably occurs in the concrete by shear failure between the ribs of the bar, as the load applied at failure (124-192 kN) is considerably lower than 270 kN.

5.2 Evaluation of results based on previous experiments and codes predictions

A comparison between the experimental results obtained in the project and the ones from other tests conducted in laboratory at three different research institutes is plotted in Figure 5.1, based on a similar comparison presented by Magnusson (2000). These other tests used for comparison were conducted on ribbed bars embedded in different types of concrete cast in a normal way without bentonite. They were performed under conditions considered comparable to those of the pull-out tests of the Citytunnel project, i.e. the load was applied by a hydraulic jack centrally positioned over the bar under well-confined conditions. The names given to the different series in the legend indicate the institute that conducted the tests and the diameter of the reinforcement bars tested. The embedment length was also equal to 5 times the bar diameter in the tests performed at SINTEF (Hansen and Thorenfeldt 1996), while it was equal to 2.5 times the bar diameter in the series of tests performed at Chalmers University (Magnusson 2000) and at Luleå University (Lestander 1993). Therefore, the bond stresses were assumed to be uniformly distributed along the embedment length.

The tests conducted on bars embedded in diaphragm wall panels at the Citytunnel project are represented by the series marked *CTP-P*, while the tests on bars embedded in the reference panel is the series *CTP-Ref.*. The maximum bond stress according to Equation 2.2 derived from the expression in CEB-FIP Model Code 1990, is also represented by the curve called tau_{max} *MC 90*.

The values of f_{cm} used to plot the bond capacity results from the Citytunnel project correspond to the mean 28-day compressive strengths of water-cured cylinders calculated from the results of tests at 28 days on air-cured cubes cast concurrently with the diaphragm wall panels and the reference panel, see Section 4.1. An increase of 20 % of these strength values was assumed as the pull-out tests took place about 2 years after casting. This increase corresponds approximately to the model of CEB-FIP Model Code 1990 for development of concrete strength with time.

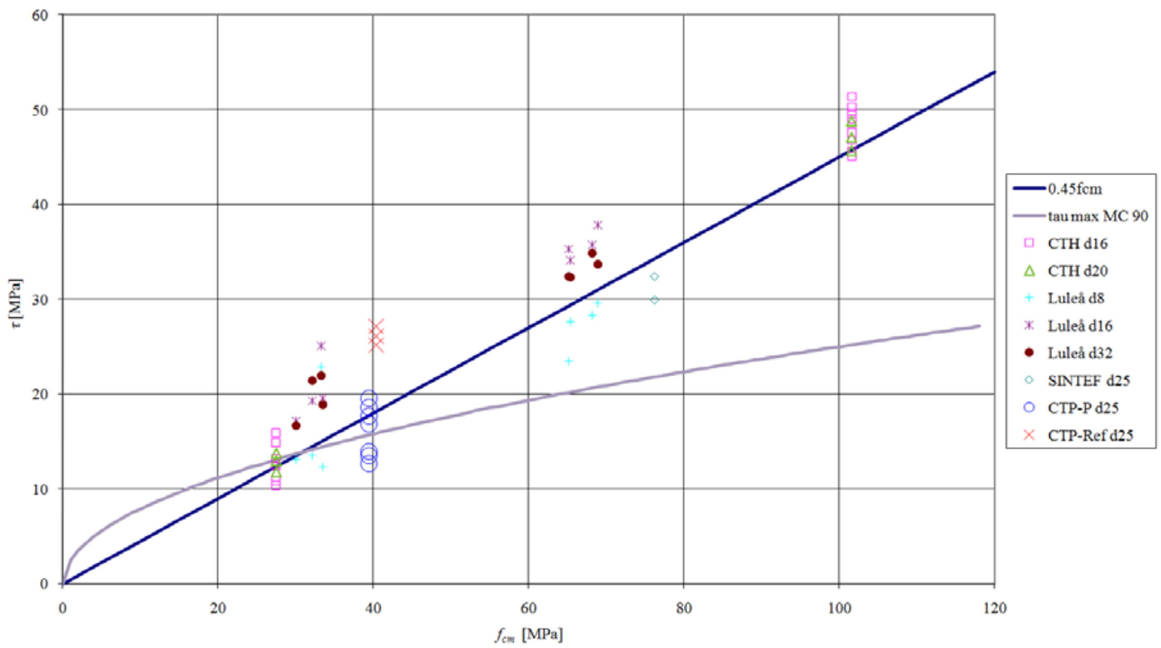


Figure 5.1: Maximum bond stress τ_{max} in relation to concrete cylinder compressive strength f_{cm}

Even if the bond capacity in the diaphragm wall panels appears to be significantly lower than in the reference panel, the bond capacity values in diaphragm wall panels seem to be rather consistent with the other experimental results. The average value for the diaphragm wall panels is just slightly lower than the one given by the equation $\tau_{\max} = 0.45 \cdot f_{cm}$ from (Magnusson 2000) and close to the one given by Equation 2.2 from CEB –FIP Model Code 1990.

According to this comparison, it could also be that it is the test for the bond capacity in the reference panel that resulted in very high values. One reason for it could have been that some concrete penetrated in the electrical PVC pipe during casting in spite of the silicone sealing, resulting in a longer embedment length than the one considered and consequently in a lower bond capacity than the one calculated. However the fact that the results obtained for different bars are very close to each other seems to contradict this explanation.

It should be noted that there are uncertainties on the concrete strength values, as reflected by the important difference between the values obtained by cube testing and the ones obtained by core testing. The values of f_{cm} used to plot the results may have therefore been overestimated. This is illustrated in Figure 5.2 by the series *CTP-P* $f_{cm,core}$ and *CTP-REF* $f_{cm,core}$, for which f_{cm} has been derived from core values instead of cube values. The value used for the core strength was derived from Figure 4.1 (at 2 years) and converted to cylinder strength by the coefficient 0.826 (see Section 4.1). As it is believed that the actual concrete strength lies somewhere in between the values from cubes and from cores, the bond capacity of the bars in the diaphragm wall panels probably correspond to the prediction from other test results or may even be a bit higher.

Besides, the strength values of the diaphragm walls and of the reference panel are very close to each other whether they are obtained by cube testing or by core testing. Therefore the difference in bond capacity results between the bars in the diaphragm wall panels and in the reference panel remains whatever concrete strength results are considered.

It is believed that the lower bond capacity of the bars in the diaphragm wall panels is due to the effect of casting under a bentonite slurry, as it is the fundamental difference in the experimental conditions with the pull-out tests conducted in the reference panel. Residual bentonite may have remained at the surface of the bars and led to a smoother surface or to a reduced interface between the bars and the concrete.

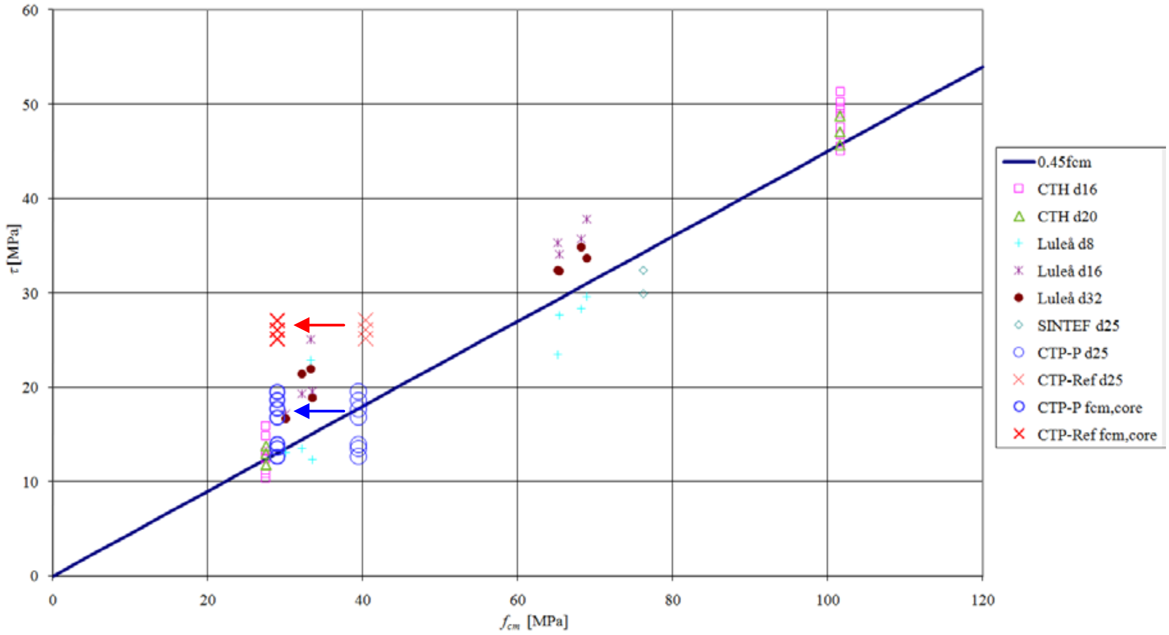


Figure 5.2: Maximum bond stress τ_{max} in relation to concrete cylinder compressive strength f_{cm} – influence of using the strength determined on cores instead of cubes

6. COMPARISON WITH EXPERIENCE FROM THE GÖTATUNNEL PROJECT

6.1 Introduction

In 2004, two series of tests were conducted to study the influence of casting under bentonite slurry: concrete cores from diaphragm wall panels from the Götatunnel project were tested and pull-out tests were performed on reinforcement bars embedded in concrete plates cast with and without bentonite. The plates were cast on site at the Götatunnel project and pull-out tests were conducted in laboratory at Chalmers University of Technology. Therefore the experimental conditions for this series of pull-out tests are less realistic than the ones of the pull-out tests conducted at the Citytunnel project, described in Section 3.4. Reference is made to Mahesar and Masiuddin (2004) for more information on the experiments.

6.2 Tests on diaphragm wall panels

6.2.1 Concrete strength

The tests conducted on 18 cores drilled on two diaphragm wall panels at the Götatunnel project indicate an average concrete strength of 56.2 MPa for cores of 100 mm in diameter, while the average cube concrete strength obtained by testing 4 cubes after 28 days is 48.3 MPa. Even if the time between casting of the diaphragm wall panels and testing the cores is not known exactly, it is considerably longer than 28 days, probably a few months, and much of the difference between the compressive strength of the cores and the cubes can be explained by the increasing maturity of the concrete during that time. Therefore it can be concluded that, as in the case of the experiments conducted at the Citytunnel project, the presence of bentonite does not seem to have led to a reduction of compressive strength for the concrete. However the results obtained at the Götatunnel project differ from the ones obtained at the Citytunnel project in the fact that the strength obtained by core testing never reached the 28-day strength of the test cubes.

6.3 Tests on concrete plates cast under bentonite slurry

6.3.1 Concrete strength

Five cylinders were tested, presumably around 28 days after casting, and resulted in an average concrete compressive strength of 37.6 MPa. Besides 12 cores drilled in the centre of the concrete plate were also tested and had an average compressive strength of 45 MPa. Neither the size of the cylinders, nor the time at testing of each series, nor the curing conditions of the tested specimens are reported. Assuming that the reference cylinders had the normal dimensions of 150 mm in diameter and 300 mm in height, and that the cores drilled had a diameter of 100 mm like the ones drilled in diaphragm wall panels in the same project, would result in an average equivalent cube strength of

45.5 MPa (using the coefficient 0.826, see Section 4.1) which can be directly compared to the strength value obtained for the cores of 100 mm diameter according to EN 13791 (2007). Consequently, in this case the tests conducted on cores and on reference cylinders seem to lead to approximately the same compressive strength of concrete on the contrary to what was found at the Citytunnel project. Nevertheless, it should be noted that the cores were probably older when tested and must have undergone a certain development of the concrete strength during this time difference.

6.3.2 Pull-out tests

Pull-out tests were performed in laboratory on reinforcement bars embedded in concrete plates cast on-site with and without bentonite. The bars were loaded until failure by a hydraulic jack centrally positioned and well-confined conditions were satisfied at the concrete surface around the bars. The relation between the bond capacity and the concrete strength is represented in Figure 6.1 for comparison with other studies. The two series of values from the Götatunnel project are called *CTH-Göta d16* for bars of diameter 16 mm and *CTH-Göta d25* for bars of diameter 25 mm. Different embedment length to bar diameter ratios were used for each test series and for all the configurations anchorage failure was expected before yielding of the bars.

Cases of loose reinforcement before the tests and possible splitting failures in the concrete were not included. The tests on bars located in the corner of the plates were also removed as they exhibited abnormally low failure loads. That could be explained by the method used for casting under bentonite these relatively small size plates, which led to considerable amount of bentonite being trapped in the concrete as observed by the authors. Indeed the top part of diaphragm wall panels cast under slurry is also often not homogeneous, as mentioned in EN 1538 (2010).

Because of the cases of loose bars and the problem of uneven concreting, the question arises whether the results of these pull-out tests are reliable for a comparison with bars embedded in diaphragm wall panels elsewhere than at the very top part of the panels.

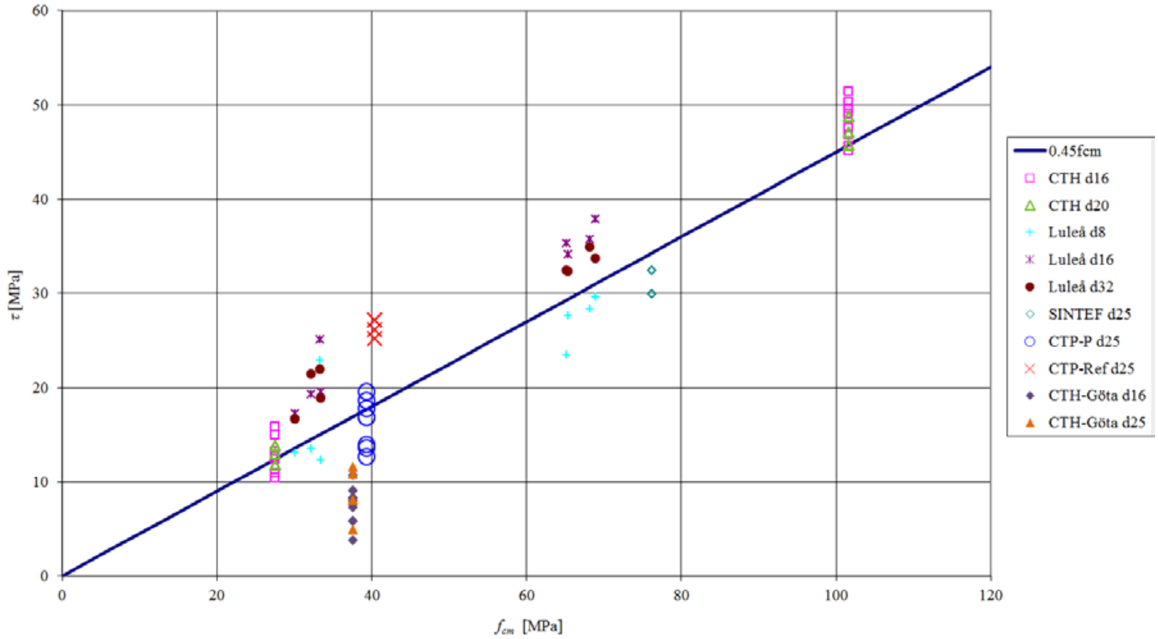


Figure 6.1: Maximum bond stress τ_{max} in relation to concrete cylinder compressive strength f_{cm}

In comparison to the other experimental results, the bond capacity values obtained by pull-out tests on the plates cast at the Götatunnel project appear to be very low.

7. COMPARISON WITH EXPERIENCE FOUND IN THE LITTERATURE

The different test results that are discussed in this section have been found in the summaries of experimental results on the bond capacity of reinforcing bars in concrete cast under drilling fluids done by Jones (2004 and 2005). Reference is made to these two articles for more information.

7.1 CIRIA tests (1967)

This series of tests was carried out after Arup experienced significant cracking in a diaphragm wall cast under bentonite slurry and preliminary tests revealed that at low slip levels, the bond stress induced was considerably lower in bars cast under bentonite than in bars cast in air and that the ultimate capacity was also decreased.

The CIRIA (Construction Industry Research and Information Association) tests were conducted on specimens cast either under air, bentonite or bentonite mixed with clay and sand. Six samples were tested for each of these configurations. The test results are summarised in Table 7.1 for 22 mm ribbed bars loaded in the direction of the concrete flow, with an anchorage length of 152 mm, a concrete cover of 76 mm and a concrete cube strength around 26 MPa, equivalent to a cylinder strength of approximately 21 MPa. As a consequence, bond failure could be expected before yielding of the bars.

Table 7.1: Summary of tests results from CIRIA experiments (1967)

	Air	Bentonite	Bentonite + clay and sand
Average bond stress [MPa]	13.8	8.9	8.8
Characteristic bond stress [MPa]	6.6	0.4	3.4

It can be observed from the test results that the bond capacities are lower for the bars cast under bentonite than for the ones cast in air. The results of the tests with bentonite are also more scattered, as reflected by the very low characteristic value.

Other similar tests were conducted with plain bars and square twisted bars for which no significant loss of bond or stiffness could be observed.

7.2 RLE pile tests (2000)

In 2000, field tests were conducted for RLE (Rail Link Engineering) on piles cast under different drilling fluids (bentonite, two types of polymer and water) and compared to a reference series cast in air. For each drilling fluid two piles were tested with two times 6 bars of diameter 32 mm, anchored on 400 mm at different depth, and with a concrete cover of 75 mm. The concrete cube strengths varied between 42 MPa and 55 MPa, which are equivalent to cylinder strength of around 34-45 MPa. According to the experimental conditions, yielding of the steel could always be expected to occur before anchorage failure.

The tests conducted on specimens cast in air and in bentonite resulted in bond stresses close to each other (around 11 MPa) and higher than the one that would have yielded the bars (9 MPa) at their specified characteristic yield strength of 460 MPa. Even at these stress levels, it can be assumed that failure occurred due to yielding of the bars rather than due to failure at the bond; for an actual yield stress of the steel of approximately 550 MPa. Therefore the only conclusion that can be drawn from these results is that the bond capacity was sufficient to yield the bars under the experimental conditions used, both in the case of the reference piles and in the case of the piles cast under bentonite.

7.3 BRE tests (2001)

Tests have also been conducted at the BRE (Building Research Establishment) on reinforcement in concrete cast under two types of drilling fluids (bentonite and a synthetic polymer slurry) and compared to a reference series cast in air. These tests were carried out in a manner very similar to the CIRIA tests described in Section 7.1. Bars of 20 mm in diameter, were loaded both in the direction of the concrete flow and in the opposite direction. For these tests, the anchorage length was 150 mm, the concrete cover 60 mm, and the concrete had a cube strength of 32-35 MPa, which is equivalent to a cylinder strength of approximately 27 MPa. As a consequence, bond failure was always expected before yielding of the bars.

Table 7.2: Summary of tests results from BRE (2001)

	Air	Bentonite
Average bond stress [MPa]	9.3	9.5
Characteristic bond stress [MPa]	7.2	7.1

On the contrary to what was found in the CIRIA tests, casting under bentonite did not seem to affect the bond strength in this series of experiments.

It can be noticed that the calculated characteristic bond stresses for both specimens cast in air and under bentonite are also similar and closer to the average bond stresses than what was obtained at CIRIA tests, which reflects less scatter in the results.

8. CONCLUSIONS

The following conclusions can be drawn from this study regarding the influence of the diaphragm wall casting method on the concrete strength:

- According to the tests carried out at the Citytunnel project, the concrete cast in diaphragm wall panels reached the same strength as the concrete cast in the reference panel. Therefore it was concluded that casting under bentonite slurry and with earth as form did not seem to affect the concrete strength.
- The tests conducted at the Citytunnel project on cores drilled from the reference panel and the diaphragm wall panels resulted in average to substantially lower compressive strength values than the ones determined at 28 days on concrete test cubes.

  This difference can be partly explained by the fact that the strength of cores is usually a lower bound estimation of the in-situ strength due to reasons like drilling effects, the lower-bound of cut aggregates, the direction relative to casting, the presence of reinforcing bars, etc. A possible additional reason can be that the concrete cast in the diaphragm wall and reference panels did not reach the same strength as the concrete cast in the batches for cube testing due to temperature effects.
- The test results obtained at the Götatunnel project, on the contrary, indicate a similar concrete compressive strength between the cores drilled from diaphragm wall panels and the reference cubes.

The following conclusions can be drawn from the pull-out tests performed on bars embedded in the diaphragm wall panels and in a reference panel:

- The bond capacity of the bars in the diaphragm wall panels at the Citytunnel project was found to be at least 40 % lower in average than the one of the bars in the reference panel. These results seem to indicate that casting under bentonite can significantly reduce the bond strength.
- However, the bond capacity values obtained for the bars in the diaphragm wall panels at the Citytunnel project are consistent with other experimental results from the literature and with code predictions. It seems to be more the values from the reference panel that are especially high.
- The literature study revealed that bond strength was found to be lower for bars cast under bentonite than for bars cast in air in one study (CIRIA tests). However, a following study did not highlight a decrease in bond strength when casting under bentonite (BRE tests).
- In light of these contradictory results, further research is needed to investigate the

influence of casting under bentonite on the bond between reinforcement bars and concrete.

9. REFERENCES

Alén, C., Lindvall, A., Johansson, M., Magnusson, J., and Norén, C. (2006) "Slitsmurar som Permanenta Konstruktioner," SBUF Utvecklingsprojekt 11603 och 11796, Juli 2006.

Bartlett, F., and MacGregor, J. G. (2003) "Equivalent Specified Concrete Strength from Core Test Data," *Concrete International*, V. 17, No. 3, March 1995, pp. 52-58.

BBK 04, Boverkets Handbok om Betongkonstruktioner, 2004.

Betonghandbok - Konstruktion, utgåva 2 (1990) AB Svensk Byggtjänst.

Betonghandbok - Material, utgåva 2 (1997) AB Svensk Byggtjänst.

CEB-FIP Model Code 1990, Bulletin d'Information 213/214. Lausanne, Switzerland, May 1993, 437 pp.

EN 206-1 (2000) "Concrete – Part 1: Specification, Performance, Production and Conformity".

EN 1538 (2010) "Execution of Special Geotechnical Work – Diaphragm Walls".

EN 1992-1-1 (2005) "Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules and Rules for Buildings".

EN 10080 (2005) "Steel for the reinforcement of concrete - Weldable reinforcing steel – General".

EN 12504-1 (2009) "Testing Concrete in Structures - Part 1: Cored Specimens - Taking, Examining and Testing in Compression".

EN 13791 (2007) "Assessment of In-Situ Compressive Strength in Structures and Precast Concrete Components".

Hansen, E. A., Thorenfeldt, E. (1996) "Bond Properties of Deformed Reinforcement Bars in High Strength Concrete," *4th International Symposium on Utilization of High Strength / High Performance Concrete*, Proceedings Vol. 2, 29-31, May 1996, Paris, France, pp. 1105-1114.

Jonasson, J.-E., Fjellström, P. (2011) "Mätning och Modellering av Hållfasthets- och Värme-utveckling med Svenska Cement," Luleå University of Technology, Department of Civil, Environmental and Natural Resources Engineering, Division of Structural and Construction Engineering, Luleå 2011.

Jones, A., Holt, D. (2004) "Design of Laps for Deformed Bars in Concrete under Bentonite and Polymer Drilling Fluids," *The Structural Engineer*, V. 82, No. 18, September 2004, pp. 32-38.

Jones, A. (2005) "Prediction of the Bond Capacity of Bars Cast under Drilling Fluids," *Structural Concrete*, V. 6, No. 2, June 2005, pp. 49-52.

Löfgren, I. (2011) Personal communication, 29 March 2011.

Lestander, U. (1993) "Vidhäftningsegenskaper hos Högpresterande Betong: Utdragsförsök med Statisk Last," Luleå University of Technology, Division of Structural Engineering, Degree project 1993:176E, Luleå 1993.

Magnusson, J. (2000) "Bond and Anchorage of Ribbed Bars in High-Strength Concrete," Chalmers University of Technology, Department. of Structural Engineering and Mechanics, Division of Concrete Structures, Thesis, Göteborg 2000.

Mahesar, A. H., Masiuddin, M. (2004) "Evaluation of Experience from Construction of Diaphragm Walls," Chalmers University of Technology, Department. of Structural Engineering and Mechanics, Division of Concrete Structures, Master's thesis 04:5, Göteborg 2004.

True, G. (2003) "Core Sampling and Testing," in: Newman, J. B., and Choo, B. S. *Advanced Concrete Technology: Constituent materials*. Oxford: Butterworth Heinemann. Chapter 5.

APPENDIX A Pictures of experiments at the Citytunnel project

Pictures of the experimental assembly for the pull-out tests on the diaphragm wall panels and on the reference panel at the Citytunnel project

Figure A.1: Reinforcement cages including reinforcing bars for pull-out tests in electrical PVC pipes and with the end covered by polystyrene caps

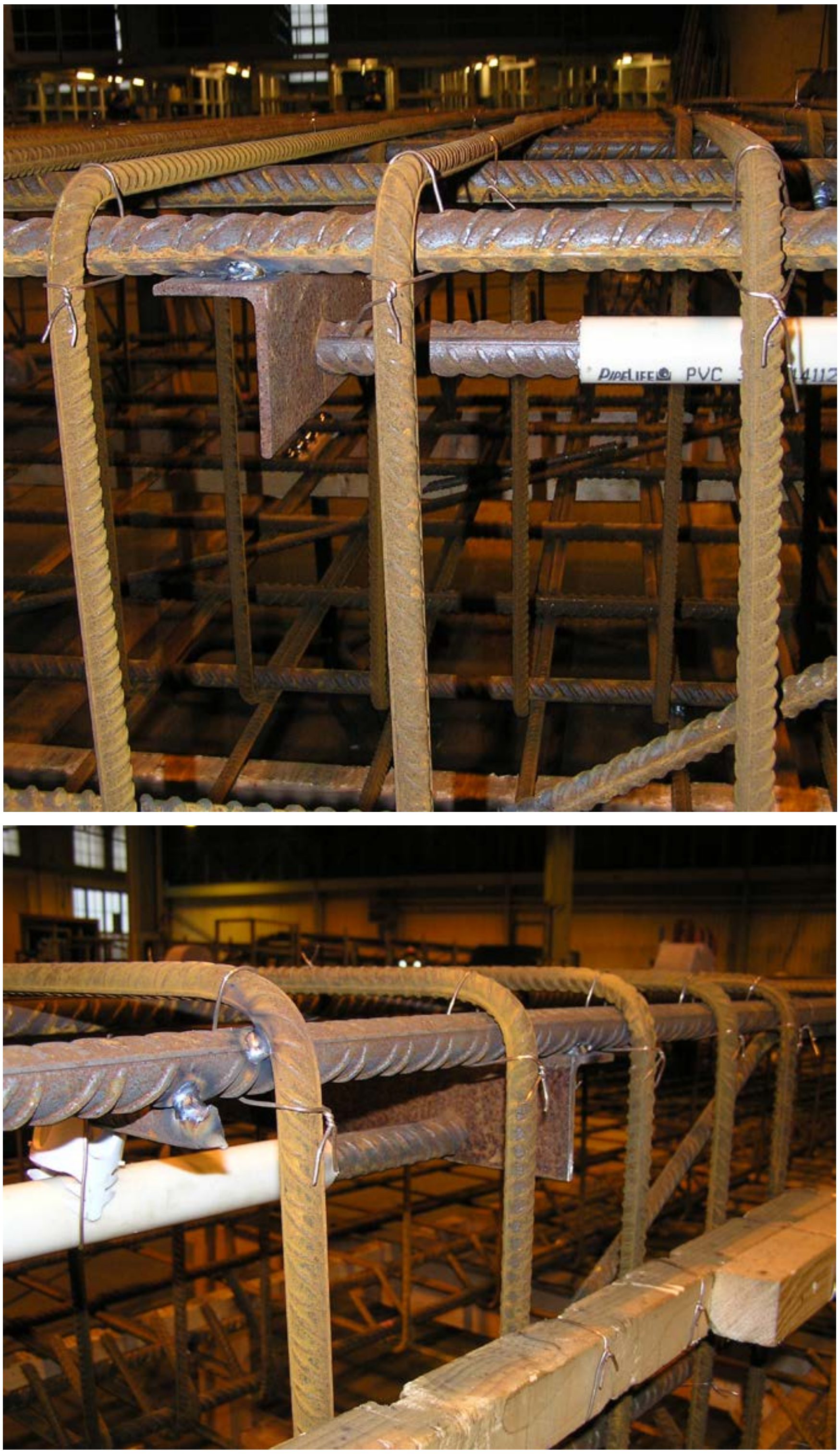


Figure A.2: Part of the bar to be embedded in the concrete for pull-out test

Figure A.3: Reference panel: formwork (top), reinforcement and bars for pull-out tests covered with polystyrene caps (left), reference panel after casting (right)

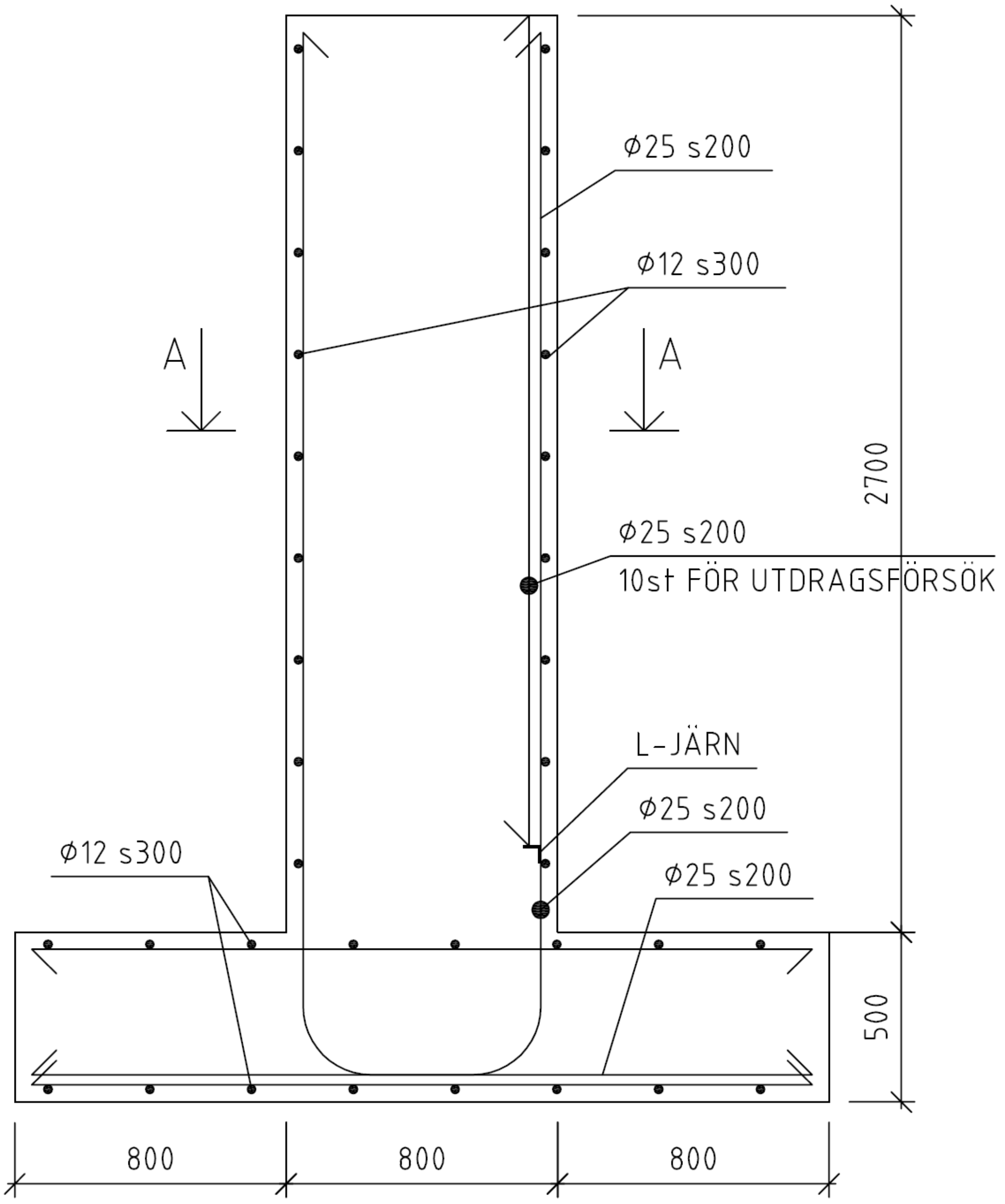
SBUF NCC

Figure A.4: Reinforcement cages and bars for pull-out tests covered with polystyrene caps in diaphragm wall panels

Figure A.5: Diaphragm wall after casting and excavation (top), drilling of concrete cores in a diaphragm wall panel (left), experimental assembly for pull-out test on the reference panel (right)

APPENDIX B Drawings of experimental assembly

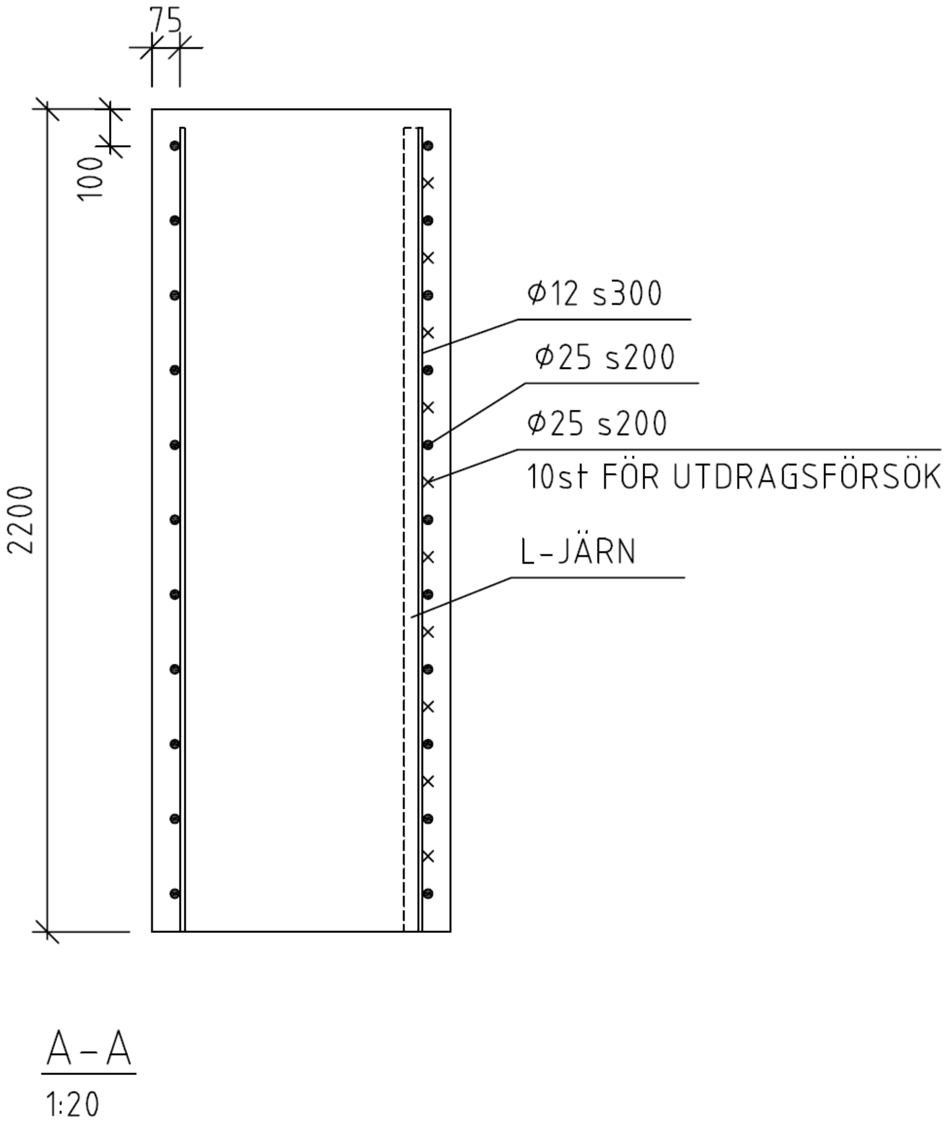
Drawings of the experimental assembly for the pull-out tests on the diaphragm wall panels and on the reference panel at the Citytunnel project

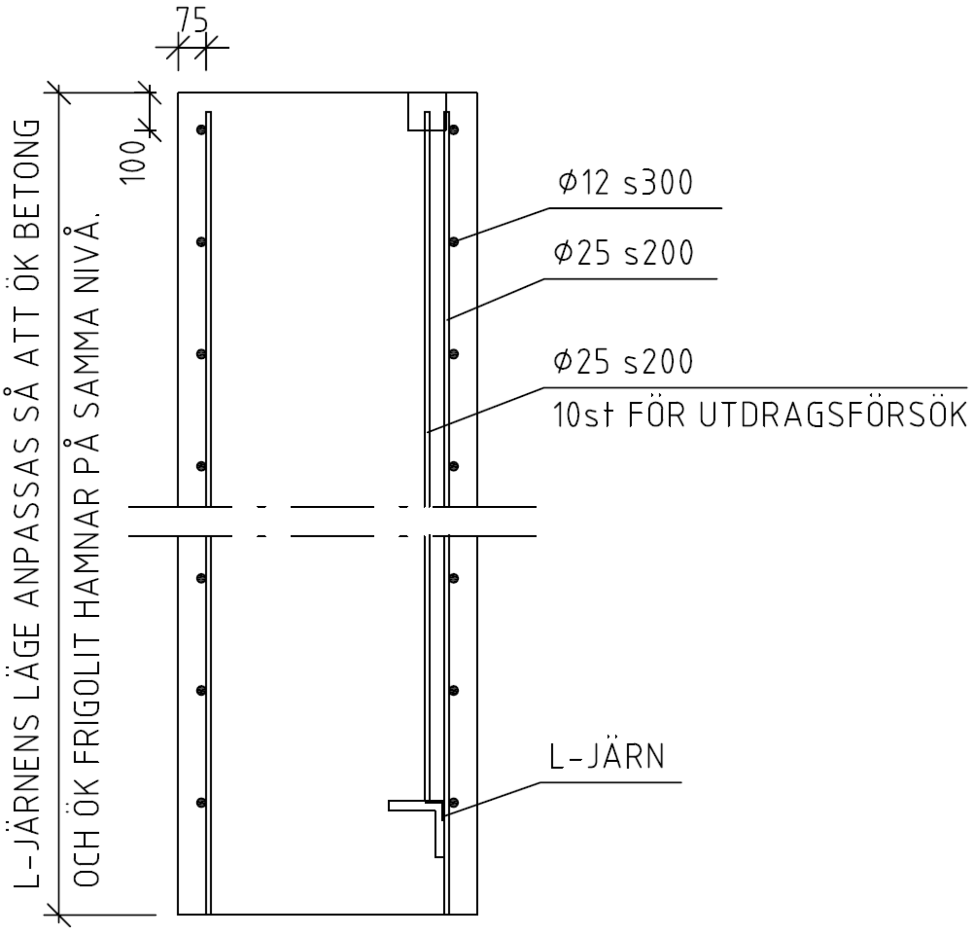


REFERENSPROVKROPP

1:20

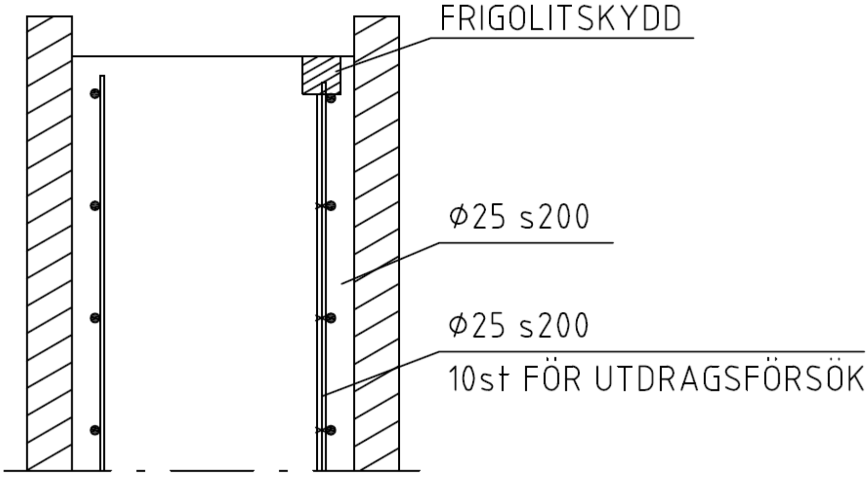
PROVKROPPENS LÄNGD 2200mm


75
100
2200
Ø12 s300
Ø25 s200
Ø25 s200
10st FÖR UTDRAGSFÖRSÖK
L-JÄRN
A-A
1:20



STÄNGER FÖR UTDRAGSFÖRSÖK

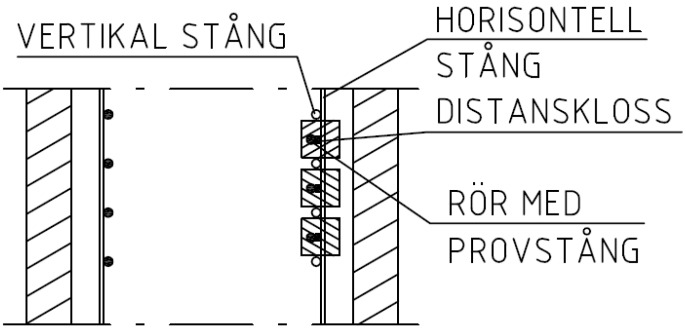
1:20



STÄNGER FÖR UTDRAGSFÖRSÖK

FRÅN SIDAN

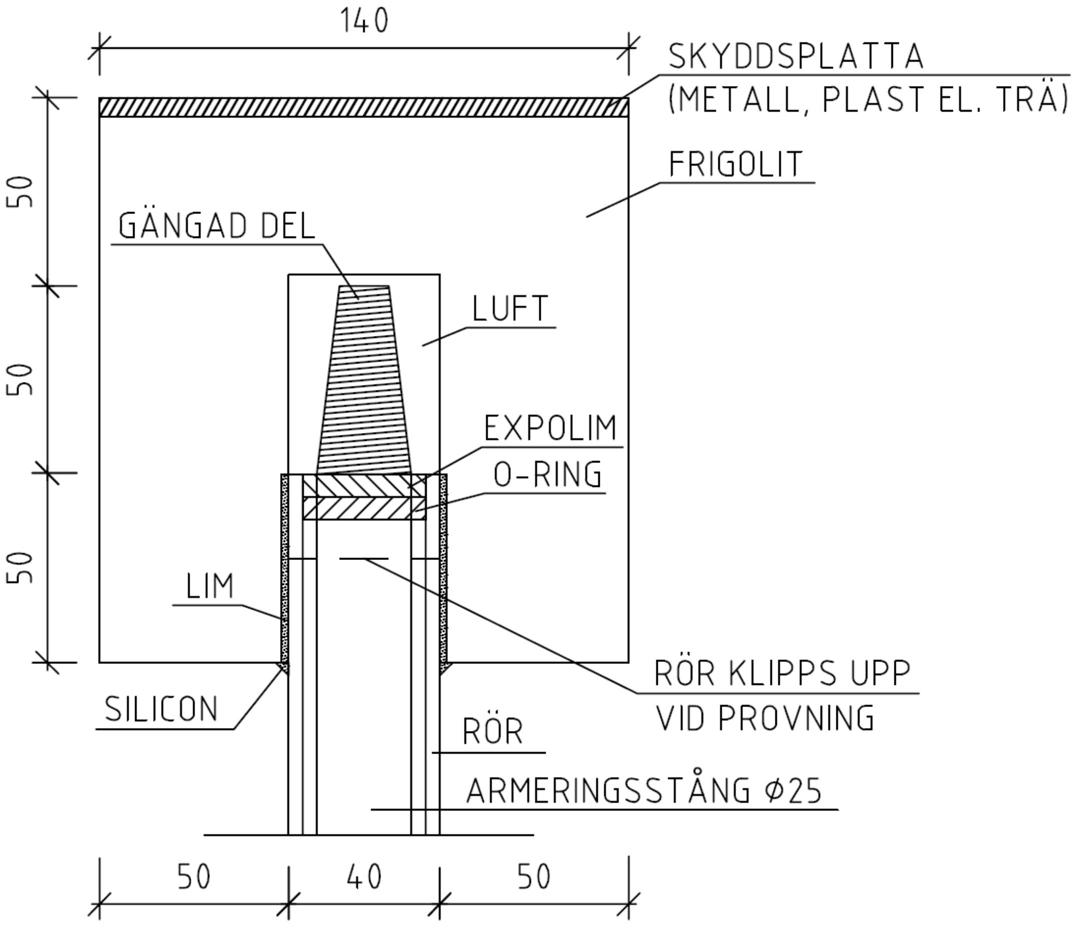
1:20



STÄNGER FÖR UTDRAGSFÖRSÖK

FRÅN OVAN

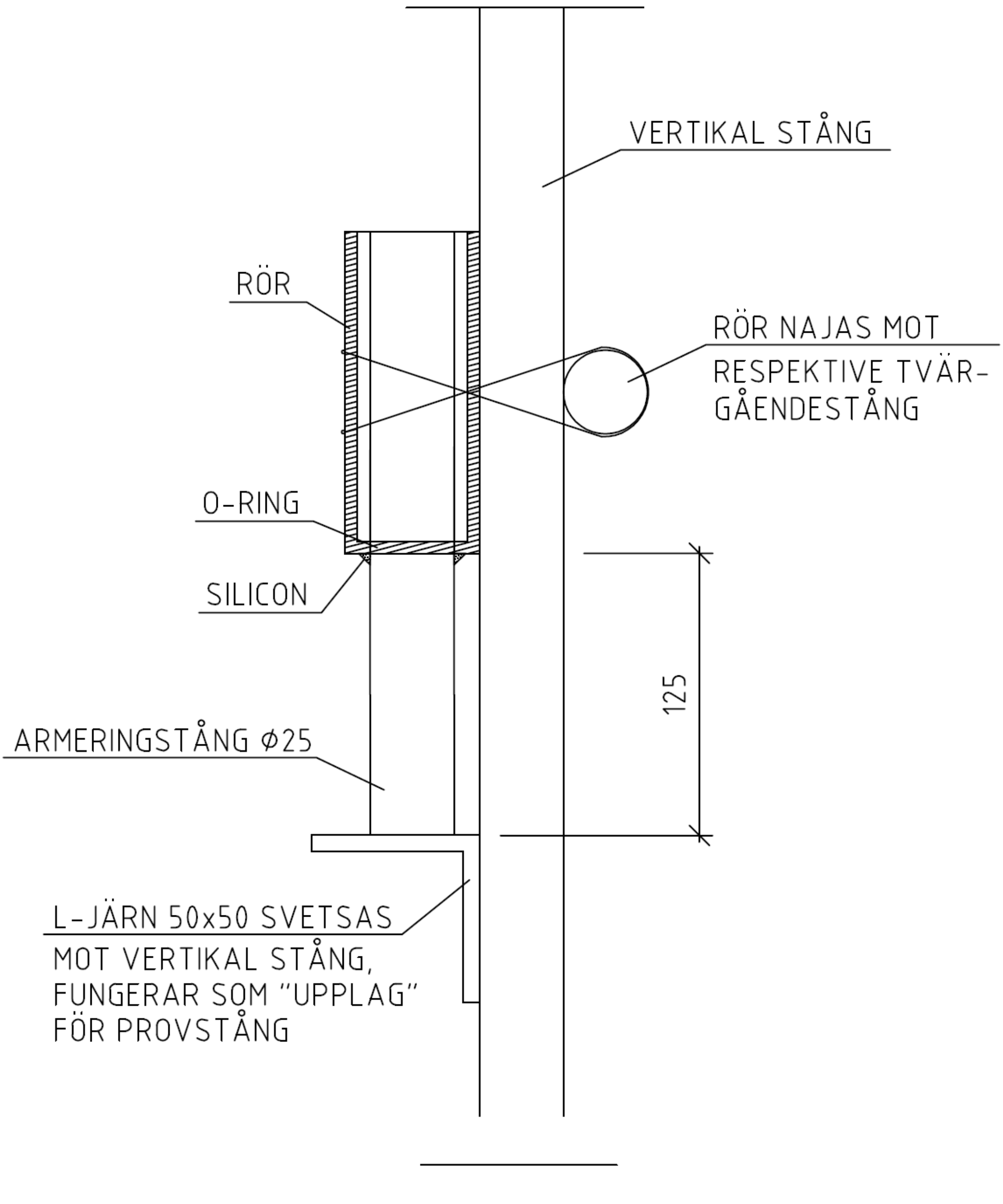
1:20



140
SKYDDSPLATTA
(METALL, PLAST EL. TRÄ)
FRIGOLIT
GÄNGAD DEL
LUFT
EXPOLIM
O-RING
LIM
SILICON
RÖR KLIPPS UPP
VID PROVNING
RÖR
ARMERINGSSTÅNG ⌀25
50
50
50
50
40
50

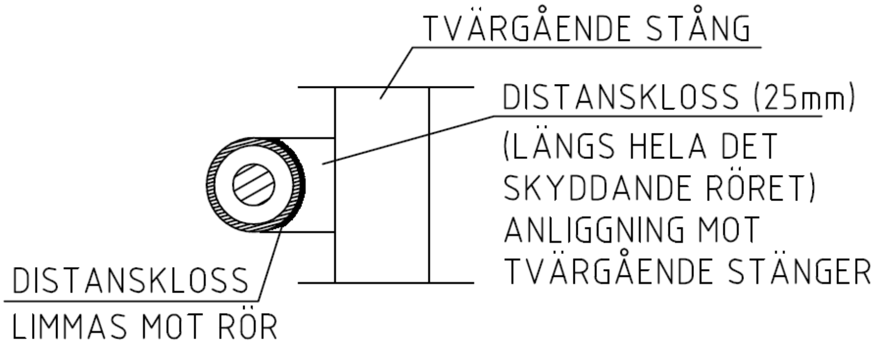
TOPPEN

1:2


VERTIKAL STÅNG
RÖR
RÖR NAJAS MOT
RESPEKTIVE TVÄR-
GÅENDESTÅNG
O-RING
SILICON
125
ARMERINGSTÅNG Ø25
L-JÄRN 50x50 SVETSAS
MOT VERTIKAL STÅNG,
FUNGERAR SOM "UPPLAG"
FÖR PROVSTÅNG

BOTTEN

1:2



BOTTEN – SNITT

1:2

APPENDIX C Tests results from the Citytunnel project

Results from 28-day compressive strength tests, Reference panel

Date of delivery	f_{ci} [1)] [MPa]	$0.826 \cdot f_{ci}$ [2)] [MPa]
2006-02-14	40,0	33,0
2006-02-14	41,5	34,3
2006-02-14	41,5	34,3
2006-02-14	41,0	33,9
2006-02-14	40,5	33,5
2006-02-14	40,0	33,0

[1)] Water-cured cubes

[2)] Water-cured cubes converted to water-cured cylinders (0.76/0.92=0.826)

Results from 28-day compressive strength tests, Diaphragm wall panels

Date of delivery	f_{ci} [1)] [MPa]	$0.92 \cdot f_{ci}$ [2)] [MPa]	$0.76 \cdot f_{ci}$ [3)] [MPa]
2005-06-08	37,9	34,9	28,8
2005-06-15	38,4	35,3	29,2
2005-07-07	42,0	38,6	31,9
2005-07-13	40,9	37,6	31,1
2005-07-15	41,2	37,9	31,3
2005-07-19	43,9	40,4	33,4
2005-07-26	37,5	34,5	28,5
2005-08-02	34,0	31,3	25,8
2005-08-10	38,9	35,8	29,6
2005-08-17	39,3	36,2	29,9
2005-08-23	49,5	45,5	37,6
2005-08-26	44,5	40,9	33,8
2005-08-29	46,5	42,8	35,3
2005-09-06	38,1	35,1	29,0
2005-09-12	39,1	36,0	29,7
2005-09-20	40,7	37,4	30,9
2005-09-27	43,2	39,7	32,8
2005-10-11	48,6	44,7	36,9
2005-10-21	44,4	40,8	33,7
2005-11-03	44,0	40,5	33,4
2005-11-07	43,1	39,7	32,8
2005-11-22	52,5	48,3	39,9
2005-11-24	41,6	38,3	31,6
2005-11-30	50,4	46,4	38,3
2005-12-07	46,7	43,0	35,5
2005-12-13	49,3	45,4	37,5
2006-01-05	47,3	43,5	35,9
2006-01-09	46,2	42,5	35,1

[1)] Air-cured cubes, i.e. first water-cured for five days and then air-cured for 23 days until testing

[2)] Air-cured cubes converted to water-cured cubes

[3)] Air-cured cubes converted to water-cured cylinders

Results from compressive strength tests on cores drilled from the diaphragm wall panels at the Citytunnel project

Panel	Specimen	Depth [mm]	Casting date	Drilling date	Testing date	Age at drilling	Age at testing	f_{ci} [MPa]	Density [kg/m^3]
3-13	1 PM	115	2005-09-27	2006-04-27	2006-05-29	212	244	31,70	2320
3-13	1 TM	100	2005-09-27	2006-04-27	2006-05-29	212	244	26,87	2188
3-13	1 TM	200	2005-09-27	2006-04-27	2006-05-29	212	244	34,38	2254
3-13	1 TV	30	2005-09-27	2006-04-27	2006-05-29	212	244	30,51	2300
3-13	1 TV	180	2005-09-27	2006-04-27	2006-05-29	212	244	41,85	2340
3-13	1 TV	280	2005-09-27	2006-04-27	2006-05-29	212	244	36,54	2329
3-12	2 TM	15	2005-09-28	2006-04-27	2006-05-29	211	243	25,84	2270
3-12	2 TM	115	2005-09-28	2006-04-27	2006-05-29	211	243	32,40	2320
3-12	2 TM	215	2005-09-28	2006-04-27	2006-05-29	211	243	37,31	2316
3-12	2 TV	180	2005-09-28	2006-04-27	2006-05-29	211	243	41,25	2338
3-12	2 TV	280	2005-09-28	2006-04-27	2006-05-29	211	243	37,18	2333
3-11	3 TM	20	2005-08-30	2006-04-27	2006-05-29	240	272	38,32	2372
3-11	3 TM	120	2005-08-30	2006-04-27	2006-05-29	240	272	43,04	2351
3-11	3 TM	220	2005-08-30	2006-04-27	2006-05-29	240	272	39,22	2302
3-11	3 TV	20	2005-08-30	2006-04-27	2006-05-29	240	272	41,38	2292
3-11	3 TV	120	2005-08-30	2006-04-27	2006-05-29	240	272	38,71	2330
3-11	3 TV	220	2005-08-30	2006-04-27	2006-05-29	240	272	31,96	2307
3-10	4 AM	130	2005-08-27	2006-04-27	2006-05-29	243	275	19,35	2202
3-10	4 PH	20	2005-08-27	2006-04-27	2006-05-29	243	275	30,94	2279
3-10	4 TM	140	2005-08-27	2006-04-27	2006-05-29	243	275	33,44	2316
3-10	4 TM	240	2005-08-27	2006-04-27	2006-05-29	243	275	34,12	2306
3-10	4 TV	150	2005-08-27	2006-04-27	2006-05-29	243	275	42,65	2250
3-10	4 TV	250	2005-08-27	2006-04-27	2006-05-29	243	275	37,31	2290
3-09	5 AM	160	2005-08-31	2006-04-27	2006-05-31	239	273	32,47	2281
3-09	5 TM	140	2005-08-31	2006-04-27	2006-05-31	239	273	34,12	2276
3-09	5 TM	240	2005-08-31	2006-04-27	2006-05-31	239	273	37,31	2280
3-09	5 TV	20	2005-08-31	2006-04-27	2006-05-31	239	273	36,67	2307
3-09	5 TV	175	2005-08-31	2006-04-27	2006-05-31	239	273	34,12	2293
3-08	6 KH	10	2005-08-29	2006-04-27	2006-05-31	241	275	34,5	2280
3-08	6 TM	140	2005-08-29	2006-04-27	2006-05-31	241	275	27,39	2186
3-08	6 TM	240	2005-08-29	2006-04-27	2006-05-31	241	275	30,18	2248
3-08	6 TV	20	2005-08-29	2006-04-27	2006-05-30	241	274	31,45	2290
3-08	6 TV	120	2005-08-29	2006-04-27	2006-05-31	241	275	35,91	2284
3-08	6 TV	220	2005-08-29	2006-04-27	2006-05-31	241	275	34,25	2270
Reference	REF TC	10	2006-02-14	2006-04-27	2006-05-30	72	105	37,43	2185
Reference	REF TC	110	2006-02-14	2006-04-27	2006-05-30	72	105	28,87	2250
Reference	REF TC	210	2006-02-14	2006-04-27	2006-05-30	72	105	32,59	2290
Reference	REF TM	10	2006-02-14	2006-04-27	2006-05-30	72	105	33,03	2234
Reference	REF TM	110	2006-02-14	2006-04-27	2006-05-30	72	105	30,38	2272
Reference	REF TM	310	2006-02-14	2006-04-27	2006-05-30	72	105	33,03	2285

[1)] Compressive tests on cores of Φ100mm, correspond to tests on water-cured cubes according to EN 13791 (2007)

Results from pull-out tests on diaphragm wall panels at the Citytunnel project

Casting date	Testing date	Age at testing	Pressure at failure [MPa]	Failure load [kN]	Bond stress [MPa]	Remarks
2006-01-05	2008-02-08	764	36	165	16,8	Westernmost bar
2006-01-05	2008-02-08	764	36	165	16,8	
2006-01-05	2008-02-08	764	42	192	19,6	
2006-01-05	2008-02-08	764	38	174	17,7	
2006-01-05	2008-02-08	764	40	183	18,6	
2006-01-05	2008-02-08	764	29	133	13,5	Easternmost bar
2006-01-05	2008-02-08	764	30	137	14,0	
2006-01-05	2008-02-08	764	27	124	12,6	

Results from pull-out tests on reference panel at the Citytunnel project

Casting date	Testing date	Age at testing	Pressure at failure [MPa]	Failure load [kN]	Bond stress [MPa]	Remarks
2006-02-14	2008-02-14	730	-	-	-	Test interrupted [1)]
2006-02-14	2008-02-14	730	58	266	27,1	
2006-02-14	2008-02-14	730	56	256	26,1	
2006-02-14	2008-02-14	730	54	247	25,2	Southernmost bar
2006-02-14	2008-02-14	730	58	266	27,1	

[1)] Pipe to hydraulic cylinder bent

Experimental characteristics for pull-out tests

Bar diameter, d	25 mm
Anchorage length, l_e	125 mm
l_e / d	5
Force yielding the bar	270 kN

Hydraulic cylinder	NIKE CHF630
Piston length	50 mm
Piston area	47,7 cm2
Capacity	333 kN
Max. oil pressure	70 MPa