An Investigation on the Interface Shear Resistance of Twinwall Units for Tank Structures

J. S. Rana[1]; C. Arya[1]; J Stehle[2]; C. Crook[2]; A. McFarlane[2]; and T. Bennett[3]

[1]Dept. of Civil, Environmental and Geomatic Engineering, Univ. College, London WC1E 6BT, U.K. E-mail: j.rana@ucl.ac.uk; c.arya@ucl.ac.uk
[2]Engineering Excellence Group, Laing O'Rourke, Dartford DA2 6SN, U.K. E-mail: jstehle@laingorourke.com; ccrook@laingorourke.com; amcfarlane@laingorourke.com
[3]ARUP, Cardiff CF10 4QP, U.K. E-mail: tim.bennett@arup.com

Abstract

Hybrid precast twinwall concrete units, mainly used in basements, core and crosswall construction, are now being adopted in water retaining tank structures. Their use offers many advantages compared with conventional in-situ concrete alternatives. However, the design could be optimised further via a deeper understanding of the unique load transfer mechanisms involved. In the tank application, twinwall units, which consist of two precast concrete biscuits connected by steel lattices and an in situ concrete core, are subject to bending. Uncertainties about the degree of composite action between the precast biscuits and hence flexural performance of the units necessitated laboratory tests to investigate the interface shear resistance. Testing was also required to assess both the leakage performance and buildability of a variety of joint details. This paper describes some aspects of this novel approach to the design/construction of tank structures as well as selected results from some of the tests that were carried out.

INTRODUCTION

In 2009, Tamesis, a joint venture between Laing O'Rourke and Imtech, was awarded the contract to construct extensions to the Beckton Sewage Treatment Works which included a number of large rectangular water-retaining 'aeration' tanks, originally designed with in-situ reinforced concrete.

An alternative hybrid twinwall system (Whittle and Taylor, 2009) combining the benefits of in-situ and precast concrete, including faster construction times, reduced on-site activity, robust continuous jointing, improved quality control, reduced waste and less mess, was developed by Laing O'Rourke (LOR), and proposed to the client via the 'Expanded' site assembly division of LOR. The twinwall system, produced by the 'Explore Manufacturing' division of LOR in a new state-of-the-art facility, has been used before by LOR for water retaining applications but no examples of use for tank applications could be identified anywhere in the world, nor could use at such a large scale in a water-retaining application be found. The proposed use of this system raised a number of questions regarding water tightness and structural performance, which had to be addressed prior to construction. For example, at the outset it was not clear if the twinwall design would be able to

© ASCE

This is a preview. Click here to purchase the full publication.

meet the stringent performance criteria demanded by the Civil Engineering Specification for the Water Industry (CESWI, 2004) which requires crack widths to be limited to 0.2mm and also for the rate of leakage to be less than a drop in water level of 10mm in a 7 day period. Moreover, it was unclear if the maximum available wall thickness of units (i.e. 400mm) would be sufficient to resist the forces due to the retained water.

To date, twinwall panels have been commonly used as load bearing elements connected to in-situ or precast lattice slabs (Explore Manufacturing, 2010). A typical twinwall panel essentially consists of two precast reinforced concrete biscuits, which are connected by shear connectors in the form of 3-dimensional triangular steel lattices, partially embedded in the inner faces of the biscuits (Figure 1). The core is filled with concrete and splice bars on site.

The planned use of these units to form the walls of tank structures meant that the units would be subject to magnitudes of flexure not normally applied. Uncertainties about the degree of composite action between the precast biscuits and hence flexural performance of the units meant that laboratory tests were necessary to validate the models used to check the interface shear resistance. Before describing the tests that were undertaken, the following gives more design information on the scheme.

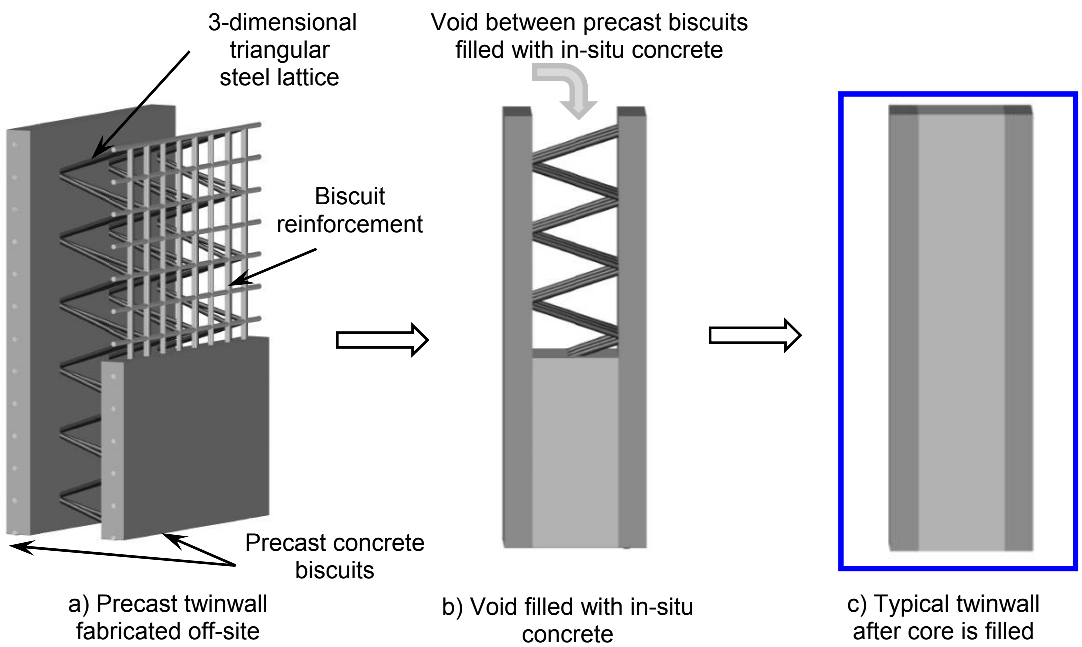

Figure 1 – Components of a typical twinwall

DESIGN

The project requirement was for 2 banks of 3 tanks, as shown in Figure 2. The aeration tank process requires an approximately uniform linear velocity flow, and thus rectangular tanks were deemed necessary, with internal baffle walls provided to effectively lengthen the path of travel within each tank.

© ASCE

This is a preview. Click here to purchase the full publication.

Each tank is approximately 80m long, 38m wide and 8m deep. Each bank of tanks has a total width of approximately 113m. Thus the total volume of sewage to be contained is approximately 144,000 m^3 or 144 Megalitres.

The original in-situ reinforced concrete tank design required a 700mm thick wall at its base, tapering to 525mm thick at its top. The need for such a thick wall was due to the applied bending moment, which for a cantilever varies according to the water depth cubed.

The maximum overall thickness of twinwall unit available from Explore Manufacturing was 400mm. Clearly such a thin wall would not work in cantilever action; hence, buttresses and tie beams were introduced into the design, not unlike the approach used for walls supporting the lateral thrust of arched cathedral roofs. The structural behaviour was thus modified to one of a 'propped' cantilever, which resulted in more than a 50% reduction in the peak bending moment. This approach, which is better suited to precast rather than in-situ construction due to the complexity of the formwork system that would be required due to the presence of buttresses, resulted in approximately 10% savings in the volume of concrete in the structure.

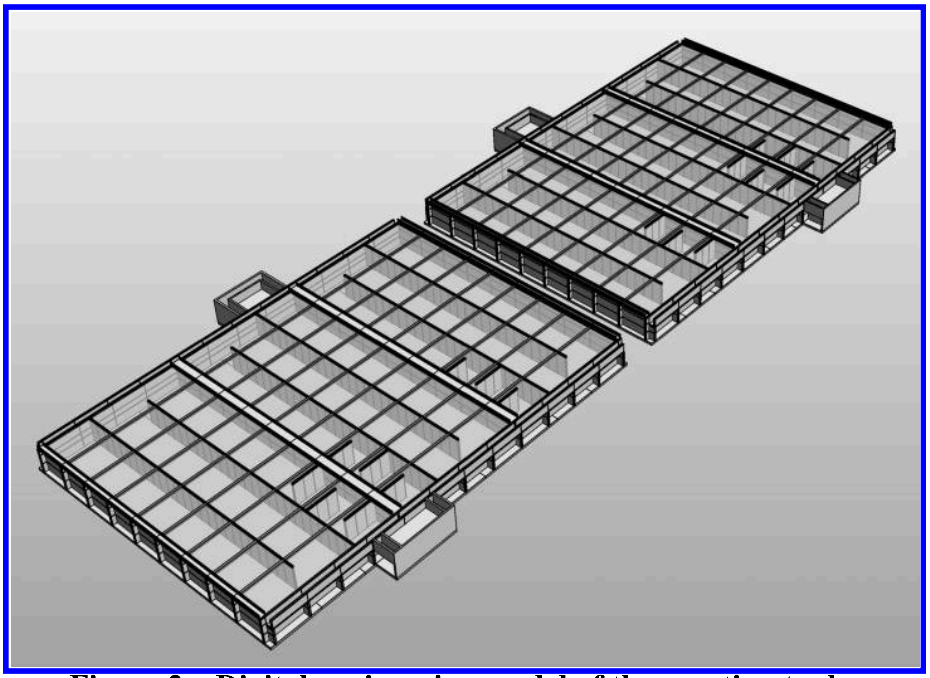

Figure 2 – Digital engineering model of the aeration tanks

TESTING

Due to the innovative nature of the design and construction technique some preliminary testing was necessary. This included building a trial tank as well as carrying out laboratory tests on interface shear resistance as elaborated below.

Trial tank

The internal volume of the trial tank was approximately 9m long by 3m wide by 8m high. The panels on one long face were constructed in a vertical orientation

This is a preview. Click here to purchase the full publication.

and on the other side in a horizontal orientation, as shown in Figure 3. The tank was primarily used to assess leakage performance and buildability of a variety of joint details.

To assess leakage, the trial tank was completely filled with water and leakage rates were monitored by recording water levels against a control leak-free contained water volume subject to the same rainfall and evaporation. Some damp patches and cracks were observed, but these were within acceptable criteria. Interestingly, although half the tank was provided with water bars at the base whereas the other half was not, there was no correlation with the extent of the damp patches i.e. the use of water bars did not seem to provide any noticeable benefit. Nevertheless, they were used in the final design.

With regard to the drop in water level, the loss (corrected for rainfall and evaporation) was less than the 10mm criteria required over a 7 day period after the tank had been filled for a month.

A critical feature of the design that was identified at an early stage was the ability of vertical bars and horizontal splice bars to transfer their loads to reinforcement embedded in the precast biscuits of twinwall elements. The magnitude of the bar forces is dependent on the joint location relative to the bending moment diagram, and thus the bar size and bar length are affected. The ease of which splice bars of different sizes and orientation could be installed in the trial, as well as panel handling and manufacture lessons learnt were used to inform the twinwall elementisation in the final design implemented at Beckton.

Figure 3 – Construction of 'trial' twinwall water-retaining tank at Laing O'Rourke's manufacturing facility near Worksop

© ASCE

This is a preview. Click here to purchase the full publication.

Interface shear resistance

As previously noted, twinwalls have to date principally been used as wall elements but in water tanks the walls are subject to greater levels of flexure. The bending strength is a function of the degree of composite action between the two outer biscuits, which is in turn a function of the interface shear resistance. Section 6.2.5 of Eurocode 2: Part 1.1 (BSI, 2010), hereafter referred to as EC2, can be used to predict the interface shear capacity, v_{Rdi}, which is given by:

$$v_{Rdi} = c\, f_{ctd} + \mu\sigma_n + \rho\, f_{yd}(\mu \sin\alpha + \cos\alpha) \leq 0.5\, v\, f_{cd} \quad \text{-------------[Equation 1]}$$

where,

- f_{ctd} design tensile strength of concrete
- f_{yd} design yield strength of reinforcement
- σ_n stress per unit area caused by an external normal force across the interface
- ρ $= A_s/A_i$
- A_s area of reinforcement crossing the interface, with adequate anchorage at both sides of the interface
- A_i area of the joint
- α angle between reinforcement and slip surface, and $\alpha \geq \pi/4$
- c coefficient of cohesion
- μ coefficient of friction

Note that the cos α term in equation (1) is ignored for $\alpha > \pi/2$ according to section D.2 of BS EN 13747 (BSI, 2005).

The cohesion and friction coefficients depend on the nature/roughness of the finish at the biscuit/in-situ concrete interface. EC2 further implies that the design tensile strength in equation (1) should be based on the 'weaker concrete'. Also, use of equation (1) assumes that the steel is fully anchored on both sides of the interface but EC2 does not directly provide any information on embedment depths and hence it was decided that the relationship between the shear connector embedment depth and interface resistance would be of interest. It should be noted that BS EN 13747: 2005 (BSI, 2005) requires a minimum 10mm of embedment, but it is not clear if this requirement guarantees full strength development.

To investigate some of these issues research, including small scale physical interface shear tests, was instigated at UCL. The key aspects investigated were:

1) Surface roughness of the concrete at the core/biscuit interface
2) Compressive strength of core and biscuit concrete
3) Embedment depth of shear connectors
4) Diameter of shear connectors.

A direct way of estimating the interface shear capacity of twinwall is by carrying out push-out tests, which was the approach used in this work. The following gives details of the experimental procedure that was followed.

EXPERIMENTAL PROCEDURE

Figure 4 shows details of the test specimens used in this work. The two outer biscuits (1 and 2) (530 x 100 x 40 mm) were reinforced with 4mm diameter mild steel

© ASCE

This is a preview. Click here to purchase the full publication.

bars (f_{yk} = 250 N/mm^2) placed mid-depth at 40 mm centres in the longitudinal direction and 75 mm and 100 mm centres in the transverse direction. The lattice girder was 80 mm deep overall and the diagonals had a pitch of 39° and 141°. These angles are slightly outside the recommended limits for Equation 1.

The bottom bars were 3mm diameter and the top bar 5 mm diameter, all made of grade 250 steel. The lattice was attached to the reinforcing bars in the biscuits using steel tie wires prior to casting.

The casting procedure for the control specimens (Type "A", Table 1) involved placing the bottom half of the reinforcement cage supported on spacers in a timber mould and pouring sufficient concrete to form Biscuit 1. In the control specimens, the lattice was embedded 20mm into the concrete biscuit, where the embedment depth refers to the 'overall' dimension of the embedment.

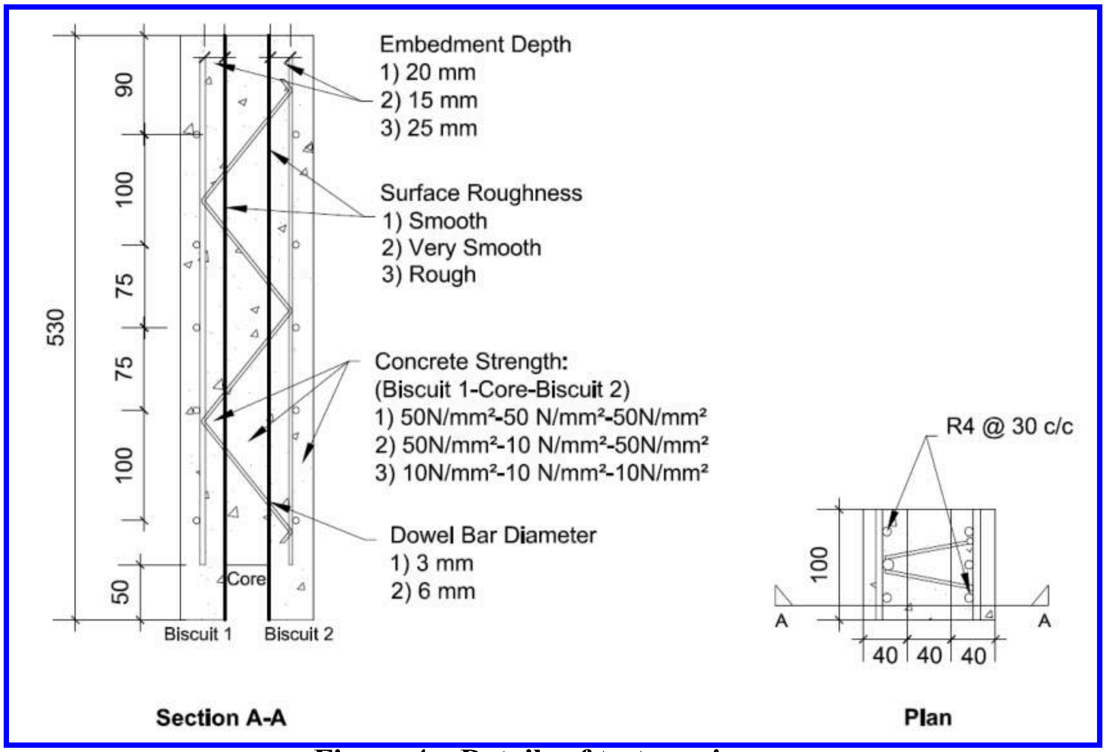

Figure 4 – Details of test specimens

© ASCE

This is a preview. Click here to purchase the full publication.

Table 1 - Test details

Type	Surface Roughness	Embedment Depth (mm)	Concrete Cube Strength in Biscuits on Day of Testing (N/mm^2)	Concrete Cube Strength in Core on Day of Testing (N/mm^2)	Dowel Bar Diameter (mm)
A	Smooth	20	35	35	3
B	Very smooth	20	35	35	3
C	Rough	20	35	35	3
D	Smooth	20	35	10	3
E	Smooth	20	10	10	3
F	Smooth	15	35	35	3
G	Smooth	25	35	35	3
H	Smooth	20	35	35	6

After curing for 24 hours the specimens were inverted and the upper half of the cage supported on spacers to achieve an embedment of 20 mm, and positioned in a second timber mould and concreted to form Biscuit 2.

After curing the concrete in Biscuit 2 for 24 hours the two sets of moulds were removed and specimens rotated into the vertical position. Shuttering was then attached to the outside faces and 50mm from the base of each specimen, and the core was cast. The shuttering was removed after 24 hours and specimens air cured for a further four days. It was assumed that this method of preparation would correspond with the 'smooth' surface roughness category in EC2.

Four cubes were also cast from each batch of concrete and cured and conditioned in the same way as the concrete in the test specimen. The target 28-day strength of the concrete used for both biscuits and the core was 50 N/mm^2. After a total of seven days the specimen was painted white to help monitor crack development during testing.

The other specimens (types B-H) were prepared in a similar fashion except that the surface roughness of the concrete on the inner face of the biscuits, embedment depth of the lattice in the biscuits, the strength of the concrete used for the biscuits and core and the diameter of the diagonal lattice bar (dowel) were varied as summarised in Table 1. Five tests were conducted for each specimen type.

In the case of the type "B" specimen, a 3mm thick neoprene sheet was attached to the inner face of each biscuit prior to casting the core, which was designed to produce a finish corresponding to the "very smooth" category in EC2. The "rough" interface finish in type "C" specimens was achieved by painting the inner face of the biscuits with a retarder and pressure washing in order to remove cement particles prior to casting the core.

Figure 5 shows the test set-up. The push-out tests were carried out generally in accordance with the recommendations in Annex B of Eurocode 4:Part 1-1 (BSI, 2009). Load was applied to the core in 2 kN increments until failure. The deflection at the top and bottom of the core was measured using dial gauges. The lateral displacement of specimens was also measured by means of DEMEC studs but the

© ASCE

This is a preview. Click here to purchase the full publication.

results are not discussed in the paper. The results presented are the average of the five tests for each specimen type, and one standard deviation is also shown.

Figure 5 – Test set-up in the laboratory at UCL

RESULTS AND DISCUSSION

Effect of interface surface roughness

Figure 6 shows the effect of interface surface roughness on failure load. It can be seen that the failure load increases with increasing surface roughness.

The design values were determined via equation (1) assuming the c and μ values shown in Table 2, f_{ctd} = 1.29 N/mm^2, σ_n = 0, A_i = 48,000 mm^2 [= 480 mm (lever arm) x 100 (width)] and f_{yd} = 217 N/mm^2. The area of reinforcement crossing the reinforcement and inclined at 39° to the slip surface, A_{s1} = 28.3 mm^2 (= $4\times\pi 3^2/4$). Similarly, the area of reinforcement crossing the reinforcement and inclined at 141° to the slip surface, A_{s2} = 28.3 mm^2. The design shear resistance of reinforcement inclined at 39° assuming the surface is 'smooth' is given by:

© ASCE

This is a preview. Click here to purchase the full publication.

ρf_{yd} (μ sin α + cos α) = (28.3/48,000) × 217 (0.6 × sin 39° + cos 39°) = 0.148 N/mm^2
and for the reinforcement inclined at 141° is given by:
ρf_{yd} (μ sin α + cos α) = (28.3/48,000) × 217 (0.6×sin 141° + 0) = 0.048 N/mm^2

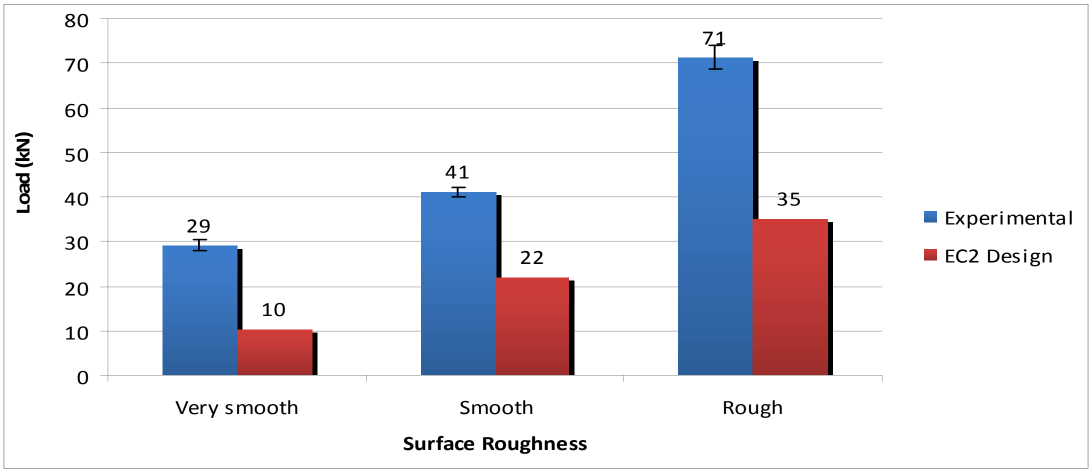

Figure 6 – Effect of surface roughness on failure load

Table 2 – c and μ values for various surface roughness categories (BSI, 2010)

Surface Roughness	c	μ
Very smooth	0.025	0.5
Smooth	0.2	0.6
Rough	0.4	0.7

The results show that EC2 theoretical failure loads are much less than the experimental values for the 'very smooth', 'smooth' and 'rough' surfaces. The EC2 estimate for 'very smooth' conditions appears to be the most conservative.

Effect of concrete strength

Figure 7 shows the effect of concrete strength on the experimental and EC2 design failure loads. It can be seen that the EC2 design failure loads are significantly less than the experimental values. The experimental failure loads suggest that the concrete strength in the biscuits is of more importance than the core strength, which may be related to the lattice girder pull-out mechanism.

Effect of embedment depth

Figure 8 shows the effect of embedment depth on experimental and EC2 design failure loads. It can be seen that the EC2 design calculations are not affected by the embedment depth of the lattice, although BS EN 13747: 2005 (BSI, 2005) requires a minimum of 10mm cover to the inside face of the lattice chord. This latter requirement is expressed relative to the base of the lattice triangle, and not the apex. However, the results indicate that the design values are significantly less than the

© ASCE

This is a preview. Click here to purchase the full publication.

experimental failure loads for the range of embedment investigated, and thus the 10mm cover requirement appears to be satisfactory. This indicates that EC2 provides satisfactory estimates of the interface shear strength of twinwalls where the lattice embedment is sufficient.

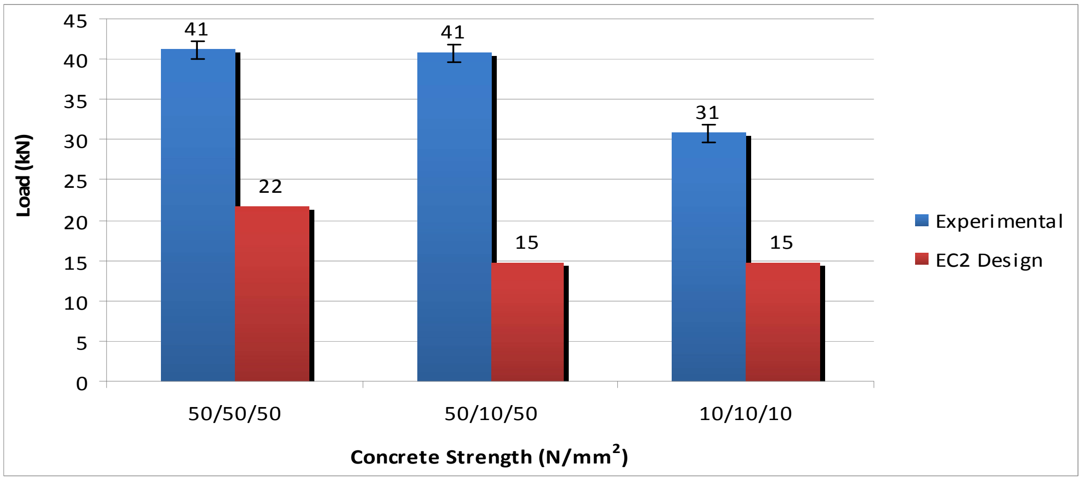

Figure 7 – Effect of concrete strength on failure load

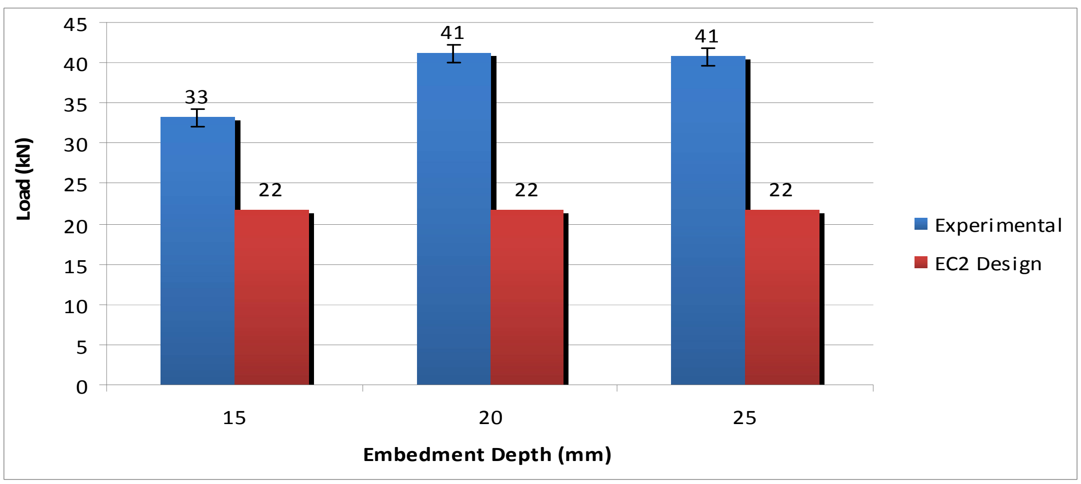

Figure 8 – Effect of embedment depth on failure Load

Effect of dowel bar diameter

Figure 9 shows the effect of dowel bar diameter on failure load. It can be seen that the failure load increases with increasing dowel bar diameter. The EC2 values were again determined via equation (1) assuming c = 0.20 and $\mu = 0.6$, $f_{ctd} = 1.29$ N/mm^2, $\sigma_n = 0$, $A_i = 48{,}000$ mm^2 [= 480 mm (lever arm) x 100 (width)] and $f_{yd} = 217$ N/mm^2. In the case of the specimens with 3mm dowel bars A_{s1} ($\alpha = 39°$) = A_{s2} ($\alpha = 141°$) = 28.3 mm^2 (= $4\times\pi 3^2/4$) and for specimens with 6mm dowel bars A_{s1} ($\alpha = 39°$) = A_{s2} ($\alpha = 141°$) = 113.1 mm^2 (= $4\times\pi 6^2/4$).

In figure 9 it can be seen that the experimental failure loads are greater than the EC2 design loads.

© ASCE

This is a preview. Click here to purchase the full publication.