DESIGN AND PERFORMANCE OF 11350 CU.M. RECTANGULAR JUBILEE RESERVOIR IN SRI LANKA

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Synopsis

This paper deals with the design and first filling performance of a very large rectangular concrete reservoir constructed in Sri Lanka. Among other things, it describes how an in- plane deep beam mode of load transfer was used in plate elements; this mode of structural action was necessitated in the non ideal structural form that resulted from client requirements. The difficulties that arose in trying to use British codes of practice in Sri Lanka, with respect to materials and environmental aspects, are also pointed out, as are the ways those difficulties were overcome. Finally, two distinct patterns of fine cracks that were noticed on first filling are described and explained, to serve as a feedback to design theory from construction experience.

1.0 Introduction

This paper describes the design and first filling performance aspects of a rectangular reservoir constructed in Sri Lanka during the period 1992-1993. The design and construction was carried out by Josef Riepl Bau-AG, for whom the second author worked as a design engineer. The client was the National Water Supply and Drainage Board, Sri Lanka. Engineering Science Ltd., for whom the first author worked as a checking engineer, were the consultants looking after the client's interests.

Three features of significance are highlighted in this paper. First, the client's brief resulted in a structural form that was not ideally suited to the reservoir volume and height; this necessitated some unusual features in the load transfer system. Second, the fact that Sri Lanka is a tropical developing country caused some difficulties in using British codes of practice for design. Finally, the first filling of the reservoir revealed some very interesting and systematic fine crack patterns. Measurements of crack spacing enabled comparisons to be made with design assumptions and calculations. All these cracks however healed during storage following first filling.

2.0 Client's Brief and Structural Form

The client required an above ground distribution reservoir capable of storing 11,325 cu.m. of potable water. In addition, the top and bottom water levels were to be identical to those of an existing adjacent circular reservoir (of much smaller capacity) with a water depth of 9.14 m. However, in order to optimise the use of the site, the client also required that the reservoir be rectangular, as opposed to circular, in plan form.

This of course posed some immediate problems, in that retaining a 9.14 m depth of water by a wall element acting as a vertical cantilever would cause a very large moment at the base of the wall, even if propped by membrane forces in the roof. In general, rectangular reservoirs are not used for water depths in excess of around 6 to 7 metres1,2. Sometimes, a sloping base can be adopted to reduce the wall height2. However, the most convenient structural form for a 9 m depth of water would have been a tank with a circular plan form, preferably prestressed, where the lateral thrust of the water could be accommodated in membrane hoop action, as opposed to bending action.

Since a circular plan form was precluded by the brief, a rectangular plan form had to be adopted. Figs. 1 and 2 show plan views of the roof and base respectively. Since the length was just over 50 m, it was decided to use an expansion joint, in order to accommodate expansion, excessive shrinkange and any differential settlement. This joint was continuous through the roof, walls and base, and thus bisected the reservoir. Therefore it caused some problems in designing the reservoir wall as a propped cantilever, since

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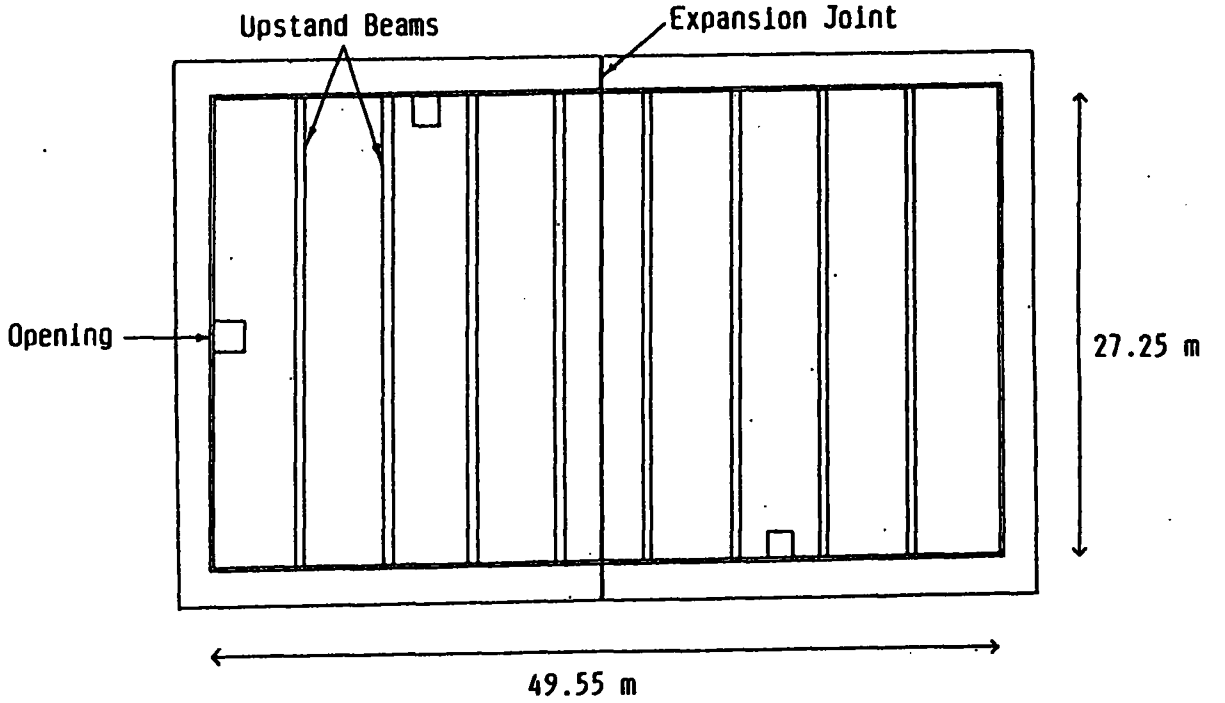

Figure1: Plan View of Reservoir Roof

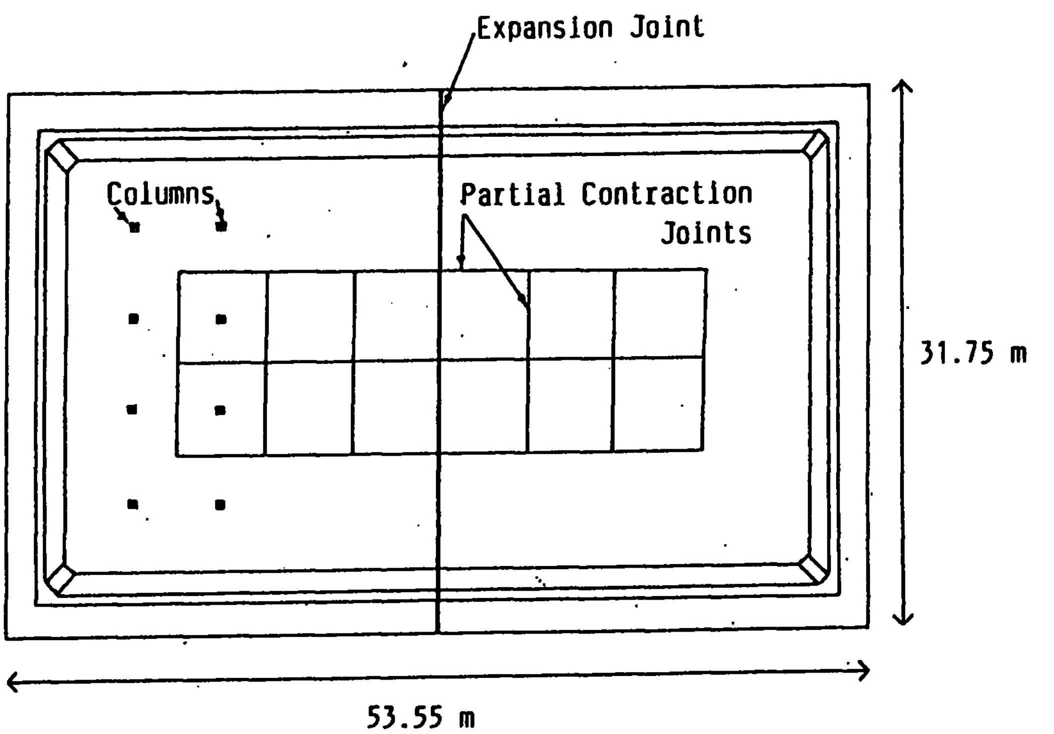

Figure 2: Plan View of Reservoir Base

membrane forces in the roof could not be transferred across the joint in the long plan direction. Furthermore, the large horizontal forces on the walls would cause the base to slide as well. The ways in which these problems were overcome are described later.

The roof arrangement was very simple, with beams spanning in the short plan direction and carrying continuous one way slabs, which in turn spanned in the long plan direction; the slab panels adjacent to the movement joint were cantilevered (see Fig. 1). Upstand beams were used, in order to reduce the wall height, given that a freeboard was required above the operational water level as well. The roof sloped slightly in the short plan direction for rain water drainage; hence, the upstand beams did not interfere with drainage, as they too spanned in the short plan direction.

The walls were of uniform thickness of 500 mm in the upper part, but tapered to a thickness of 1800 mm at the base (see Fig. 3). The walls were monolithic with the wall base (of 900 mm thickness), which also carried the first interior lines of columns (see Figs. 2 and 3). The areas corresponding to each column in the central part of the base were separated from each other and the wall base by partial contraction joints (see Fig. 2). This central part was only 200 mm thick, except for 1600 mm x 1600 mm areas directly under the columns which were thickened to 500 mm (see Fig. 3). The base was also bisected by the expansion joint.

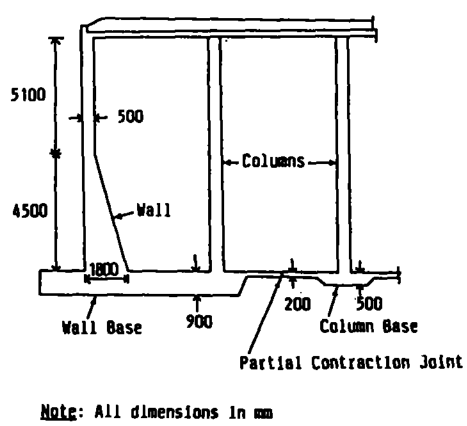

Figure 3: Part Cross Sectional Elevation Through Reservoir

3.0 British Codes and Sri Lankan Practice

Being a small Commonwealth country, Sri Lanka still uses British codes of practice for structural design. However, two major problems were encountered in carrying out this design to BS 81103 and BS 80074. The first had to do with the grade of concrete to be specified and the second with the heat of hydration temperature rise values to be assumed in the design.

4.0 Grade of concrete

BS 80074 specifies the use of grade 35A concrete for water retaining structures. This specification is linked to the exposure condition being defined as "severe", as per BS 81103, which in fact requires a concrete grade of 40. However, a grade termed 35A is permitted, provided the minimum cement content (i.e. 325 kg/cu.m.) and maximum free water/cement ratio (i.e. 0.55) corresponding to grade 40 concrete in BS 8110 is maintained4.

It must be pointed out that the achievement of high strengths for comparatively low cement contents and high water/cement ratios is a phenomenon of rather recent origin, due primarily to changes in cement manufacturing processes5. Developing countries such as Sri Lanka have not kept up with such changes; neither are their batching practices fine tuned to achieve high strengths. In this context, when durability is specified for by grade, as in BS 8110 and BS 8007, the high grades so specified are not easily achievable. If they were to be achieved, uneconomical quantities of cement would have to be used, which in turn would also cause heat of hydration and shrinkage problems.

The solution to this problem is to concentrate on achieving the minimum cement content and maximum water/cement ratio in the concrete, and to specify a grade that would result from those mix proportions in the specific country concerned. This is because durability is related more to mix proportions; specifying durability by strength is done for convenience6. A study relating mix proportions to strengths achievable in Sri Lankan batching plants has in fact been published by the first author7.

It was decided to use grade 25 concrete for the reservoir; this was justified by the above study7. In addition, a minimum cement content of 350 kg/cu.m. and a maximum water/cement ratio of 0.45 were also specified. These mix proportion limits are more stringent than the BS 8110 limits; in spite of this, it was not possible to achieve concrete strengths very much in excess of grade 25.

The point is however, that modifications had to be made to BS 8110 and BS 8007 before using them in Sri Lanka. Such modifications will be necessary when transposing codes across national boundaries, at least where material and environmental aspects are concerned. The Irish Republic has also made such modifications to BS 8110 to suit their own concrete production[8].

5.0 Heat of hydration temperature rise values

The heat of hydration temperature rise (T_1) values to be used for design are given in Appendix A of BS 8007[4]. However, those values are relevant for a mean daily temperature of 15°C and a concrete placing temperature of 20°C. The operating temperatures in Sri Lanka are around 15°C higher than these values. Furthermore, there were no published data available regarding heat of hydration temperature rise values for Sri Lanka, or for other countries with similar climates.

In the ambient temperature range 0°C to 15°C, a very approximate rule is that a change in ambient temperature of 1oC causes a change in the heat of hydration temperature rise (i.e. the difference between the heat of hydration peak and ambient temperature) of around 0.5°C[9]. Although experimental results are not available for ambient temperatures in excess of 15°C, the above approximate rule was extrapolated to a mean daily temperature of 30°C, resulting in around 8°C being added to the T_1 values in BS 8007, where appropriate.

The above requirement imposed by the consultant caused some contractual dispute as well, especially because it necessitated fairly large amounts of steel in the massive wall sections. The reservoir was for the most part constructed according to the continuous construction option in BS 8007. Hence, where the walls were concerned, the increase of 8°C was not made for the T_1 values. Instead, it was decided to construct the walls of the reservoir in vertical segments, leaving 1 m gaps in between them. Although the horizontal steel was not lapped within these gaps, this sequence of construction was nevertheless expected to reduce the crack widths due to thermal effects.

6.0 Design, Detailing and Construction Aspects

We shall now describe some of the more interesting features of the reservoir, with respect to design, detailing and construction aspects.

7.0 Membrane action in roof and walls

As indicated before, since the wall height was very large, the resulting vertical cantilever had to be propped at roof level, in order to reduce the bending moments at the base of the wall. This presented no difficulty in the short plan direction, as the propping was achieved by membrane tensile forces within the slab that balanced out each other in a self reacting fashion.

However, it was not possible to do this in the long plan direction, since membrane tensile forces could not be transmitted across the expansion joint. Hence, propping of the short wall by the roof was achieved by causing the entire roof slab on one side of the expansion joint to act as a deep beam10 in its plane. This deep beam therefore carried a uniformly distributed load at its "lower" edge and was supported on the parts of the long walls to one side of the expansion joint (see Fig. 4). The flexural compressive stresses were too small to cause any out of plane buckling of the slab. In any case, the slab was also stiffened by the upstand beams and the flexural compressive stresses were reduced by the membrane tension in the short plan direction (see Fig. 4).

The support reactions, which acted along the top edges of the long plan walls, were transmitted to the ground by these walls acting as in-plane vertical cantilevers, again in deep beam action (see Fig. 4). Thus, in-plane deep beam action was utilised to resolve the conflicting requirements of propping the walls at roof level, while having a movement joint across the long plan direction.

8.0 Resistance against sliding

Where the overall stability of the reservoir was concerned, resistance against sliding was the main consideration, since the lateral thrust of the water pressure in the long plan direction could not be carried within the base slab in a self reacting manner, once again because no tensile forces could be transferred across the expansion joint. Where the short plan direction was concerned of course such tensile forces could be transferred from one wall to another.

However, both in the long plan and short plan directions, it was initially attempted to resist the lateral water pressure by friction under the wall base alone, so that full contraction joints could be used between the wall base and the central part of the base (see Fig. 2). Calculations revealed however that the area of the wall base was not sufficient do develop enough frictional resistance. Hence, some of the steel

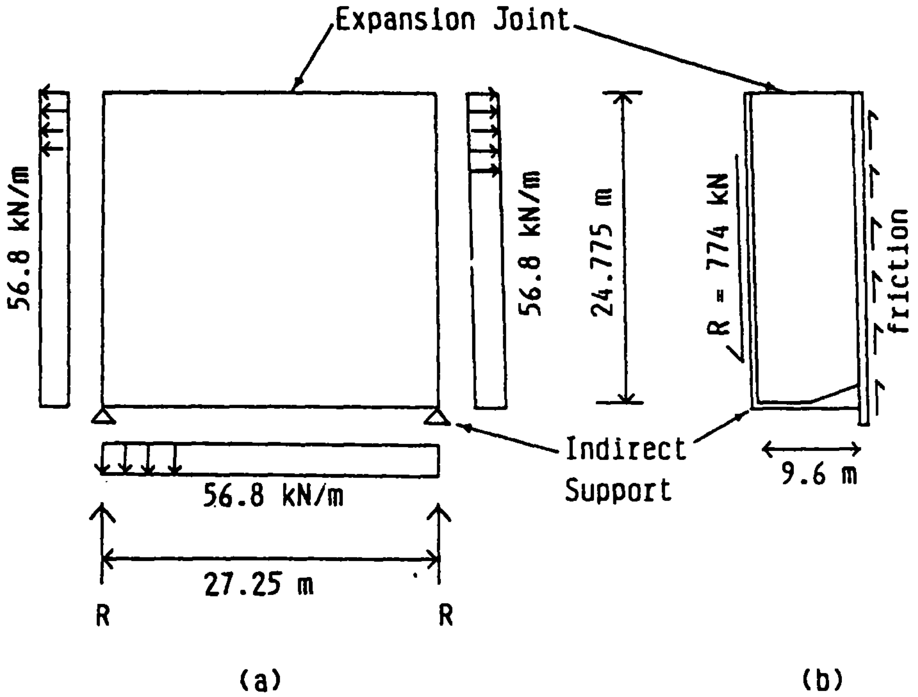

Figure 4: Membrane Action in Roof and Long Walls Shown in

(a) Part plan of roof and

(b) Part Sectional elevation of long wall

in the wall base had to be continued into the central part of the base, and the proposed full contraction joints converted to partial contraction joints.

9.0 Design of wall and wall base

The rotation at the wall-base junction was determined using a soil-structure interaction computer programme. Since the value for the modulus of subgrade reaction could not be accurately relied on, the design was carried out using two values, one representing good soil (i.e. 40 MN/m^3) and the other representing weak soil (i.e. 20 MN/m^3). The fixed end moment at the wall base was then modified using a compatibility condition on the rotation of the wall at its base.

The wall plus wall base component was also checked for overturning, since the wall base was separated from the rest of the tank base by a partial contraction joint.

10.0 Detailing

Deformed type 2 steel bars of diameters ranging from 8 mm to 25 mm were used in the structure. The bar spacing was maintained in all cases at 170 mm, or where not possible, halved to 85 mm, in order to facilitate steel fixing.

At the roof expansion joint, mild steel dowels of 6 mm diameter with a semicircular kink in the middle were embedded on both sides of the joint. It was assumed that the kink (which was placed within the joint), would permit movement. At the same time, the dowels were capable of transmitting the propping force for the vertical cantilever walls across the joint, in case rotational settlement would cause large openings at the central expansion joint.

11.0 Construction

Although it was originally intended to cast the base slab in chequerboard style segments, this was changed to casting adjacent segments in sequence, with a minimum of 1 week's duration between pours, in order to allow a significant part of the thermal contractions and shrinkage to take place. The chequerboard style results in faster construction, but the segments that are filled in will be subject to restraint from all four directions11.

The walls were cast in vertical panels of lengths varying from 5.75 m to 6.185 m. The four corner panels were of 3.25 m length in two mutually perpendicular directions. All these panels were cast with gaps of 1 m between them, as described before. Each vertical panel was concreted to its full height in a single continuous operation. The durations allowed

Figure 5: Inclined Cracks in Long Panel and Horizontal Cracks in Gap Panels

between the casting of the longer panels and their corresponding 1 m gap panels ranged from 1 week to 15 weeks.

12.0 Cracks First Filling

The first filling of the reservoir was carried out almost a year after the walls were concreted. The reservoir was filled in steps of 1 m depth, in the course of around a week. On some days, filling was done both in the morning and afternoon. Two distinct crack patterns were evident soon after first filling, as seen in Fig. 5. .

13.0 Cracks in long panels

Most of the long panels had two inclined cracks, which can be characterised by the general pattern shown in Fig. 6. These cracks would be due to the restraint exerted on the wall contractions by the heavy base. Such restraint is accompanied by bond creep, which tends to concentrate the cracks at isolated locations, as opposed to distributing them. Furthermore, such restraint also causes warping of the wall, which explains the slight inclination of the cracks[12].

It is instructive to note that most of the cracks have originated at around 2.6 m from the free ends. A length of 2.4 m is considered to be capable of accommodating free movement of concrete4. By this logic, a free panel of 4.8 m or less should not display this type of cracking. This was indeed the case in the corner panels, which had free lengths of only 3.25 m in two mutually perpendicular directions.

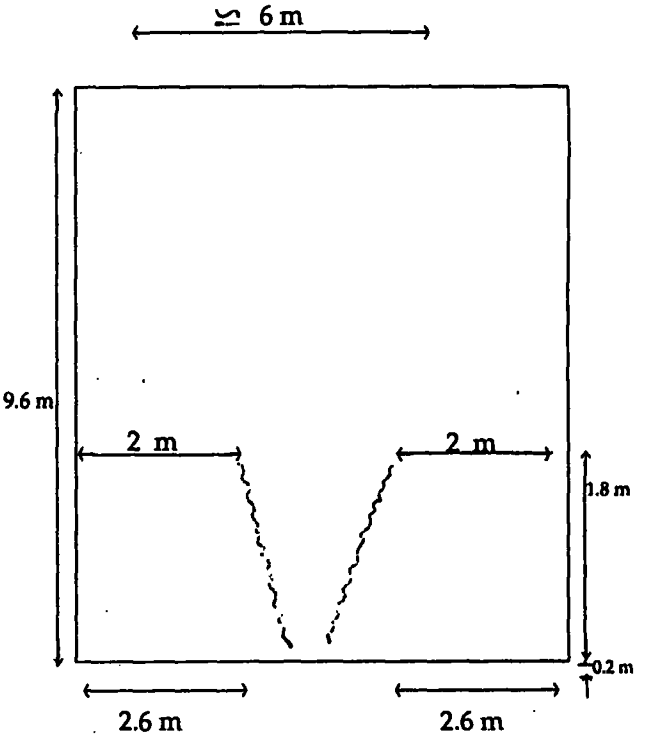

Figure 6: Characteristic Crack Pattern in Long Panel

14.0 Cracks in gap panels

Most of the short, 1 m gap panels had a series of horizontal cracks. A typical set of cracks is shown in Fig. 7. The distance from the wall base to the first of these horizontal cracks had a mean value of 1.69 m, while the average spacings of these cracks, below and above the 4.5 m height from wall base were 1.41 m and 1.07 m respectively. No horizontal cracks were visible above 7.4 m from the tank base, i.e. 2.2 m below the top of the wall. This confirms again that a distance of around 2.4 m can accommodate free movement of concrete.

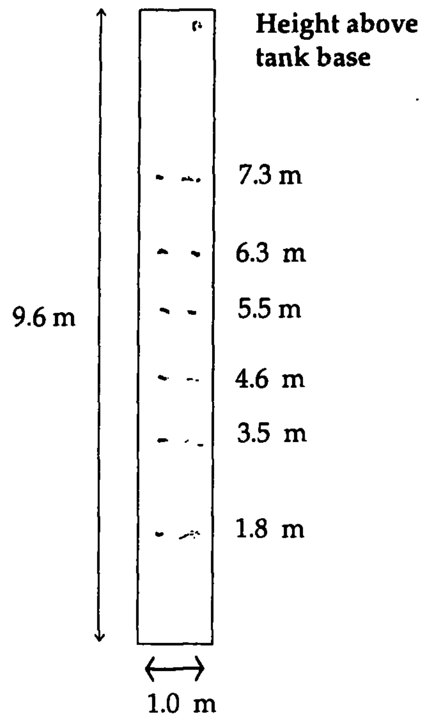

Figure 7: Typical Crack Pattern in gap Panel

The cracking in these panels would have been due to the significant restraint exercised by the long panels on either side of it. Vertical cracks were absent in these panels, because the length over which horizontal contraction took place was only 1 m. However, since the vertical contraction (over 9.6 m) would also have been restrained by the long panels in shear, horizontal cracking had taken place.

The cracking on the external face would be governed by the surface zone of 250 mm. The vertical reinforcement in the bottom 4.5 m height of wall was T16 @ 170 mm and in the upper 5.1 m height of wall T20 @ 170 mm. This yields expected maximum crack spacings of 1.13 m in the bottom section and 0.90 m in the top section, as per BS 80074. This can be compared with the observed mean values of 1.41 m and 1.07 m respectively.

The bond creep effect mentioned before may be responsible for the measured spacings being higher than the calculated ones, especially in the very thick lower portions of the walls. In fact, the observed mean spacing of the first crack from the wall base was 1.69 m, which is considerably in excess of 1.13 m.

The temperature difference for contraction strains (due both to hydration and ambient variations) assumed for the wall design was 53°C. The restraint factor for alternate bay construction in the gap panels was 0.474, for a panel length of 1 m and height of 9.6 m (Figure A.3(d) in ref. 4). This gave[4] crack width values of 0.28 mm and 0.23 mm in the bottom and top sections respectively, based on the theoretical crack spacing values. If the measured mean spacings are used, the crack widths would increase to 0.35 mm and 0.27 mm. It must be appreciated that these values are higher than the 0.20 mm that is permitted. This is because no vertical restraint was expected in the wall and thermal steel provided corresponding only to (2/3)p[4]. However, the sequence of construction which involved 1 m gap panels did in fact cause vertical restraint in those panels.

When the above crack widths were measured 5 days after filling had been completed, they were found to be not greater than 0.2 mm. Since some healing would have taken place prior to measurement, the original crack widths would have been larger than 0.2 mm but not larger than 0.5 mm, as self healing will not take place in cracks exceeding around 0.5 mm width[13]. These observations tend to confirm the crack width calculations above.

15.0 Crack sealing

Apart from a few of the cracks near the bases of the longer panels being injected with epoxy resin from the outside, no further treatment of cracks was necessary, as they all healed autogenously.

16.0 Closure

This paper has shown how a client requirement that resulted in a non ideal structural form could still be accommodated by mobilising the slightly unconventional structural action of in- plane deep beam behaviour in plate elements. It has also highlighted the difficulties of transposing codes of practice across national boundaries, with respect to their provisions for materials and environmental considerations. Finally, it has described two very definite patterns of cracking that appeared at first filling (and healed subsequently), and explained those patterns in terms of the currently accepted design theory regarding early age thermal cracking.

17.0 Acknowledgements

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18.0 References

1. Studies on design of ground reservoirs and elevated tanks, Dept. of Civil Engineering, University of Moratuwa, Sri Lanka, April 1989, 223 pp.

2. Walmsley, J.: 'Service reservoir design, with particular reference to economics and the 'simply supported wall' concept', The Structural Engineer, Vol. 59A, No. 9, September 1981, pp. 285-291.

3. BS 8110: 1985: Structural use of concrete, London, British Standards Institution, 1985.

4. BS 8007: 1987: Design of concrete structures for retaining aqueous liquids, London, British Standards Institution, 1987.

5. Changes in cement properties and their effects on concrete, Report of a Concrete Society Working Party, London, Concrete Society, September 1984.

6. Dias, W.P.S.: 'Specifying for concrete durability: Part I - A critical review', Engineer, Sri Lanka, December 1991, pp. 51-63.

7. Dias, W.P.S.: 'Specifying for concrete durability: Part II - The Sri Lankan context', Engineer, Sri Lanka, 1992, pp. 4-24.

8. IS 326: 1988: Code of practice for the structural use of concrete, Dublin, National Standards Authority of Ireland, 1899.

9. Harrison, T.A.: Early age temperature rises in concrete sections with reference to BS 5337: 1976, Interim Technical Note 5, Slough, Cement and Concrete Association, 1978.

10. DAfStb-Heft 240: Hilfsmittel zur Berechnung der Schnittgroßen und Formanderungen von Stahlbetontragwerken, Berlin, Wilhelm Ernst & Sohn KG, 1976.

11. Deacon, R.C.: Concrete ground floors: their design, construction and finish, 2nd ed., Slough, Cement and Concrete Association, 1975.

12. Anchor, R.D., Hill, A.W. and Hughes, B.P.: Handbook on BS 5337:1976, Slough, Cement and Concrete Association, 1979.

13. Perkins, P.H.: Repair, protection and waterproofing of concrete structures, London, Elsevier Applied Science Publishers, 1986.