1 INTRODUCTION

Construction has begun on the $85 million Stage Three of the Rouse Hill Infrastructure Project for new housing areas in Sydney, Australia. Stage Three will provide water, sewerage and drainage infrastructure for 10,000 new homes. These homes will be connected to dual water supply systems – drinking water and recycled water from the Rouse Hill Recycled Water Plant. When built, the homes will use recycled water for garden watering and washing of cars – activities that consume almost 25 per cent of Sydney's total residential drinking water – as well as toilet flushing, park and golf course irrigation, and industry in the area.

John Holland Pty Ltd has been awarded the contract to undertake design and construction of Stage Three, incorporating more than 40 kilometres of water and sewer mains, seven stormwater detention basins, one water and one sewage pumping station, and one reservoir for two new areas, known as Second Pond's Creek Area and Balmoral Road Release Area.

Tierney Opus were engaged by John Holland Pty Ltd to undertake the structural design of the 4 ML operating capacity elevated "wine glass shaped" steel reservoir which will contain 5.4 ML of stored water. Construction of the reservoir commenced during July 2005 at Kellyville, and, when completed it will be a dominant feature on the landscape standing some 37 metres high, eclipsing that of the existing adjacent smaller 2ML recycled water reservoir. Total mass of the above ground structure including water will be some 5700 tonnes. The reservoir when constructed will supply drinking water at the rate of 660 litres per second to the new housing areas.

The reservoir structure comprises an 8 metre diameter stem of 13 metres height, then a 7.5 metre high enlarging truncated cone section followed by a 23 metre diameter cylindrical water compartment of 13 metres height with a steel plate roof. (Fig.1)

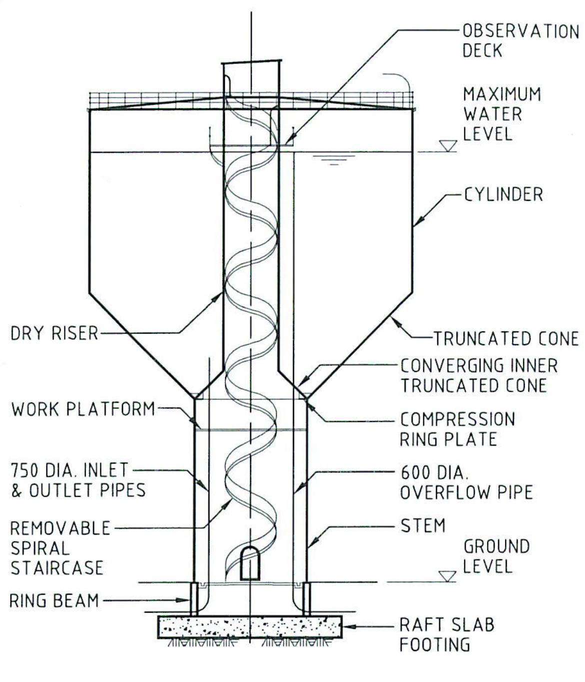

Fig. 1 Reservoir Elevation

The structure is supported on a 13 metre wide octagonal reinforced concrete raft slab footing of thickness 1.5 metres, founded on class IV shale at a depth of 3.9 metres. A 450mm thick reinforced concrete circular ring beam above the raft slab allows for the mild steel cement lined 750mm diameter inlet and outlet mains and the 600mm diameter overflow pipe to enter the reservoir stem below ground level but above the raft slab. This arrangement eliminates the need to construct pipe trenches under the raft slab, which could compromise the available bearing capacity of the shale and which would have to be accompanied by large sized penetrations through the raft slab.

At the top of the stem, a 60mm thick by 600mm wide steel compression ring plate is provided on which the truncated cone section is attached. Also at this location, an inner converging truncated cone is attached which forms the lower section of a 4 metre diameter dry riser, which passes up through the water compartment and protrudes 3.5 metres above the roof of the reservoir. Steel plate for the structure varies in thickness from 28mm for the stem, 60mm for the compression ring plate, 36mm to 20mm for the truncated cone, 16mm to 8mm for the cylindrical section, 20mm for the dry riser shaft and 6mm for the roof.

Access is provided by way of a spiral stairway within the stem and dry riser to the work platform near the top of the stem, to the observation platform which is above and within the water compartment, and to the roof.

The structure was analysed using a commercial computer programme that includes buckling analysis. In addition to wind and hydraulic actions, the structure was also required to be designed to resist earthquake actions.

2 OWNER'S NEEDS

Because the project was a design and construct contract, the owner – Sydney Water via Rouse Hill 3 Infrastructure – specified minimum requirements to meet its operating requirements in both a Technical Specification and a Needs Specification. For the reservoir structure, those requirements included the basic shape and size of the structure together with ancillary items such as hatches with covers, monorail beams, davit and spiral staircase.

The Needs Specification covered road access, structure, foundations, pipelines, access to and within the reservoir structure, ventilation, operation, controls, instrumentation, power requirements, corrosion protection, chlorination system and safety.

3 TECHNICAL SPECIFICATION

The technical specification listed the relevant standards which had to be complied with, in the design of the reservoir. In the absence of an Australian Standard, the American Water Works Association Standard for Welded Steel Tanks for Water Storage – AWWA D100 – was designated as the design code for the reservoir. However, the design loads specified in clause 3.1 of AWWA D100 were superseded and replaced by specific clauses relating to AS/NZS 1170 and AS1657. Alternative standards that could also be used for the design of parts of the shell were BS 2654, API 620 and API 650.

Other steelwork such as roof rafters, monorails, stairs and platforms were designed in accordance with AS 4100 Steel Structures.

It was stipulated that the design of the steel reservoir "shall be based on elastic allowable working stress appropriate for the grade of steel and a welded joint efficiency of 85%." Also for strake plates (i.e. the reservoir walls) less than or equal to 20 mm thick AS/NZS 3678 Grade 250 steel was to be used and for thicker plates AS/NZS 3678 Grade L0 250 steel was to be used. The Grade L0 250 steel was specified because of its superior notch ductility properties.

4 GEOTECHNICAL SITE CONDITIONS

The initial geotechnical information provided indicated that below 2.0m depth, pad footings could be founded on low strength shale Class IV or better with an allowable bearing pressure of 1.0 MPa. A further site investigation bore hole drilled approximately at the centre of the reservoir indicated that very low strength shale could extend to a depth of 3.35m. The level of the top of the raft slab and hence the final founding depth of 3.9m was governed by the size of the 750mm diameter segmental 90 degree pipe bend located above the raft slab footing.

5 SEISMIC ANALYSIS

The owner's Technical Specification included the following clause to replace the original AWWA D100 clause.

"AS/NZS 1170.4 – Earthquake Loads, for earthquakes loads. The reservoir shall be designed as a Type III structure using the ground acceleration applicable to its geographical location multiplied by the probability factor k_p of 1.0, as in Appendix D of AS/NZS 1170.0. The method used in API 650 could be used".

From this specified probability factor of 1.0 it can be deduced from Table D1 of AS/NZS 1170.0 that the owner considered earthquake loads had an annual probability of exceedance of 1 in 500. Hence from Tables B1.2b and B1.2a of the BCA 2004 the Importance Level would be 3 – buildings or structures that are designed to contain a large number of people. Usually a large reservoir would be considered as a structure essential to post-disaster recovery with an Importance Level of 4 and therefore have a k_p value of 1.25. Type III structures had an Importance Factor I of 1.25 in the original version of AS/NZS 1170.4, but since the publication of AS/NZS 1170.0 all structure types now have an Importance Factor of 1.0 and a variable probability factor. The probability factor has a similar influence on the magnitude of the design earthquake force as the original Importance Factor.

However in this case, acknowledging that the specification had been written by the owner who had design engineering experience, the specified k_p value was adopted in the initial design. Nevertheless, the independent verifier would not accept the owner's k_p value and so the design was finalised using a k_p of 1.25.

Initially, a modal analysis of the reservoir, considering it to be an inverted pendulum, was contemplated. Because of the importance of the structure and the fact that it was not a type of structure frequently designed in Australia, advice was sought from one of the company's earthquake engineering specialists in Opus' Wellington N.Z. office. The advice received was that because of the uncertainty in the relationship between the hydrodynamic forces of the stored water and the dynamic forces on the inverted pendulum-type structure during an earthquake event, a static analysis of the tank would be an adequate analysis model. Thus a simple static analysis was adopted in accordance with AS/NZS 1170.4.

In determining the earthquake base shear of the reservoir the structural response factor $R_t = 2.1$ for an "Inverted pendulum-type structure" from AS/NZS 1170.4 was used.

6 DESIGN

The design shear actions and overturning moments derived from the seismic analysis were used directly for the design of the reinforced concrete raft foundation. These ultimate strength design values were converted into working stress design values for the design of the steel shell. Design of the external cylinder walls could have been undertaken manually in that the major action was hoop tension together with vertical compression from the roof loads. Hydrodynamic action on the wall was accounted for by the AWWA D100 design graphs for convective and impulsive actions. The values are a function of tank diameter and water depth.

For the truncated cone section of the reservoir, the shell plates are subjected to hoop tension and compression due to water pressure and the supported mass of water, as well as the compression load at the base of the cylinder section. Analysis of the truncated cone section could not be done readily by simple hand calculation methods. Accordingly a computer model of the whole structure containing 8600 elements was generated. A frame structures computer analysis programme was chosen for the analysis and design rather than a finite element analysis. The reason being that the programme not only determined the forces in the plates but it also undertook a buckling analysis of the whole tank. The shell plate thicknesses were varied in a trial and error approach to achieve a load factor of 2 against buckling. According to the available literature this was the first time the method had been used on this type of structure. The independent verifier used a finite element analysis to check the design.

In order to resist the compressive forces at the base of the truncated cone, a large steel compression ring beam, 600 mm wide x 60 mm thick, was incorporated into the structure.

For the design of the stem, holding down bolts and raft slab foundation, P-Delta effects were considered. Unlike a normal building where the majority of mass comprises the structure, the major mass component of the reservoir is the water which is a non rigid body. Consequently sloshing of the stored water had also to be included in the P-Delta effects.

In addition to the specified protective coat system and the cathodic protection system, a nominal corrosion allowance was also provided for in the detailed plate thicknesses.

7 STAIRWAYS AND HOISTS

Access was provided within the reservoir to a work platform at the top of the stem, to an observation platform at the top of the storage vessel and to the roof by way of a central spiral stairway.

The spiral stairway was required to fit within the 4.0m diameter dry riser with clearance to the riser wall for vertical service conduits. A 1.6m diameter enclosed central hoist shaft was also required within the stairway. The design of the stairway was complicated by the requirement that it had to be removable, that only 2 columns were permitted on the inside of the stairway and that only 3 columns were permitted on the outside up to the level of the work platform. The diameter of the stairway just permitted each flight of stairs, with either 17 or 18 risers and a landing, to be accommodated within a 180 degree arc.

Within the water storage compartment, access was provided from the observation platform down to the bottom of the truncated cone by means of a Fibre Reinforced Plastic spiral stairway.

Three concentric circular monorail crane beams above the water storage compartment were suspended from the roof rafters. Other monorail crane beams were provided between the central hoist shaft and the water compartment and above the hatch in the work platform. All monorail cranes were designed for a 500kg SWL. Both the central hoist and davit crane at the edge of the roof were also designed for a 500 kg load.

8 CONCLUSIONS

(a) "Because of the uncertainty in the relationship between the hydrodynamic forces of the stored water and the dynamic forces on the inverted pendulum-type structure during an earthquake event, a static analysis of the tank would be an adequate analysis model." i.e. Keep the analysis simple.

(b) A shell type structure can be analysed and designed using a computer programme with a buckling analysis module. A trial and error approach is required to vary the shell thickness until a satisfactory load factor against buckling is found.

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REFERENCES

API 620 — American Petroleum Institute, "Recommended Rules for Design and Construction of Large Welded, Low Pressure Storage Tanks".

API 650 — American Petroleum Institute, "Standard for Welded Steel Tanks for Oil Storage".

AS/NZS 1170.0 — Standards Australia/Standards New Zealand, "Structural design actions. Part 0: General principles".

AS 1170.4 — Standards Australia, "Minimum design loads on structures (known as the SAA Loading Code) Part 4: Earthquake loads".

AS/NZS 3678 — Standards Australia/Standards New Zealand, "Structural Steel – Hot rolled plates, floorplates and slabs".

AS 4100 — Standards Australia, "Steel structures".

AWWA D100 — American Water Works Association, "Welded Steel Tanks for Water Storage".

BCA 2004 — Australian Building Codes Board, "Building Code of Australia".

BS 2564	British Standards Institution, "Specification for vertical steel welded storage tanks with butt-welded shells for the petroleum industry".