DESIGN AND CONSTRUCTION DECISIONS FOR MAJOR IMPOUNDING STRUCTURES 50 METRES FROM ACTIVE FAULT

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SUMMARY

This paper describes the design and construction decisions taken to achieve a safe and rational design for two large off-river storage lákes built in close proximity to an active fault which is capable of generating earthquakes with magnitudes reaching 8.25. The background situation is presented, the design philosophy discussed and the seismically important structural details of embankments, intake towers, buried galleries and pipework are described.

INTRODUCTION

The present run of river water supply for the Wellington Region is turbid in high rainfall. The nearly completed Te Marua Water Storage Project will provide 3.2 Mm^3 of clear off-river storage in two lakes up to 16 m deep covering 30.6 ha. The only suitable site for the lakes on the hydraulic grade line was situated on alluvial terraces traversed by an active fault. This site is at Te Marua some 40 kilometres northeast of Wellington, the capital of New Zealand.

The Wellington Fault which crosses the site is classified "Class I active," having an estimated interval of 500-1500 years between movements and the potential to generate up to magnitude 8.25 earthquakes. The last movement occurred some 900 years ago. This seismic setting combined with the permeable alluvial foundations presented significant problems for design of safe and serviceable lakes.

REGIONAL GEOLOGY AND SEISMICITY

New Zealand straddles the boundary between the Indian and Pacific tectonic plates and has moderate to high seismicity. In the Wellington region, northeast trending active faults form part of the tectonic belt marking the boundary between the two plates. These faults have influenced local geology.

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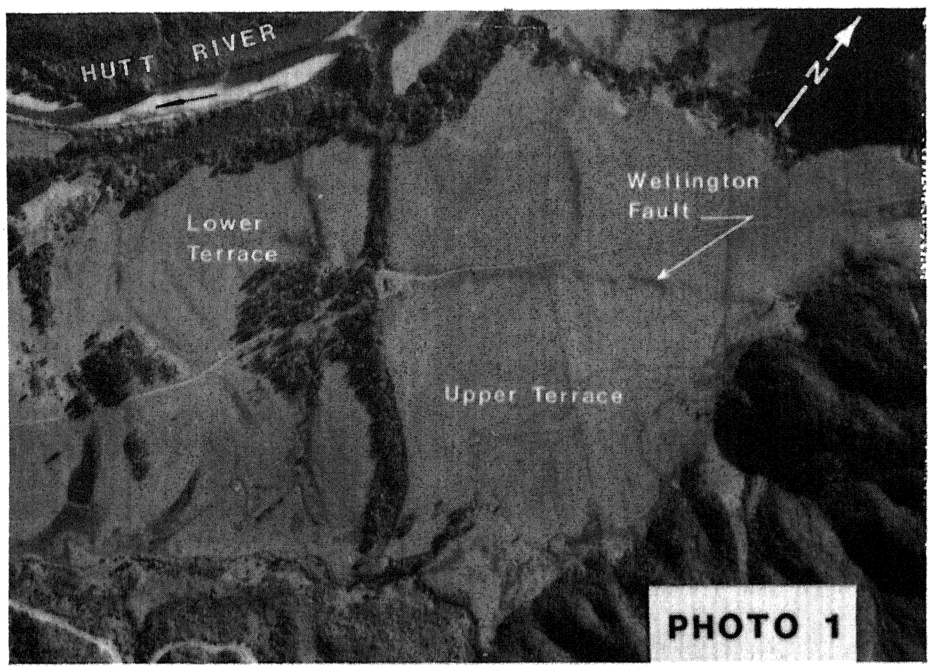
The site geology comprises remnant alluvial terraces of gravels and boulders which overlie Mesozoic greywacke rock and are covered by a thin cover of loessal soil. The Hutt River adjacent to the site was diverted along the Wellington fault line depression during Pliocene era faulting. Glacial activity fed solifluxion debris into the river which subsequently downcut through the gravels in interglacial periods leaving remnant terraces.

The Wellington region is one of the more active seismic regions in New Zealand. The last major earthquake was in 1855 involving large movement on the West Wairarapa fault which parallels the Wellington fault. The Wellington fault which crosses the site is dextral transcurrent with an average strain rate of 7.5 mm/yr.



SITE FEATURES AND SCHEME ARRANGEMENT

The site is confined between the Hutt River and high ground to the southeast on two gently sloping terraces with an elevation difference of 12 m. Photo 1 shows the predevelopment site on which the Wellington fault trace is clear. The 30 m and 15 to 20 m horizontal displacements of the upper and lower terraces are evident. South of the fault these terraces comprised up to 32 m depth of well graded gravels, cobbles and boulders. A 1 m thick loess layer covered the upper level terrace but had been eroded from the lower terrace. Zones of fine grained alluvium and colluvium existed at the hillslope margins and soft lacustrine deposits were found at depth in the same area caused by infilling of older river channels which skirted the high ground.

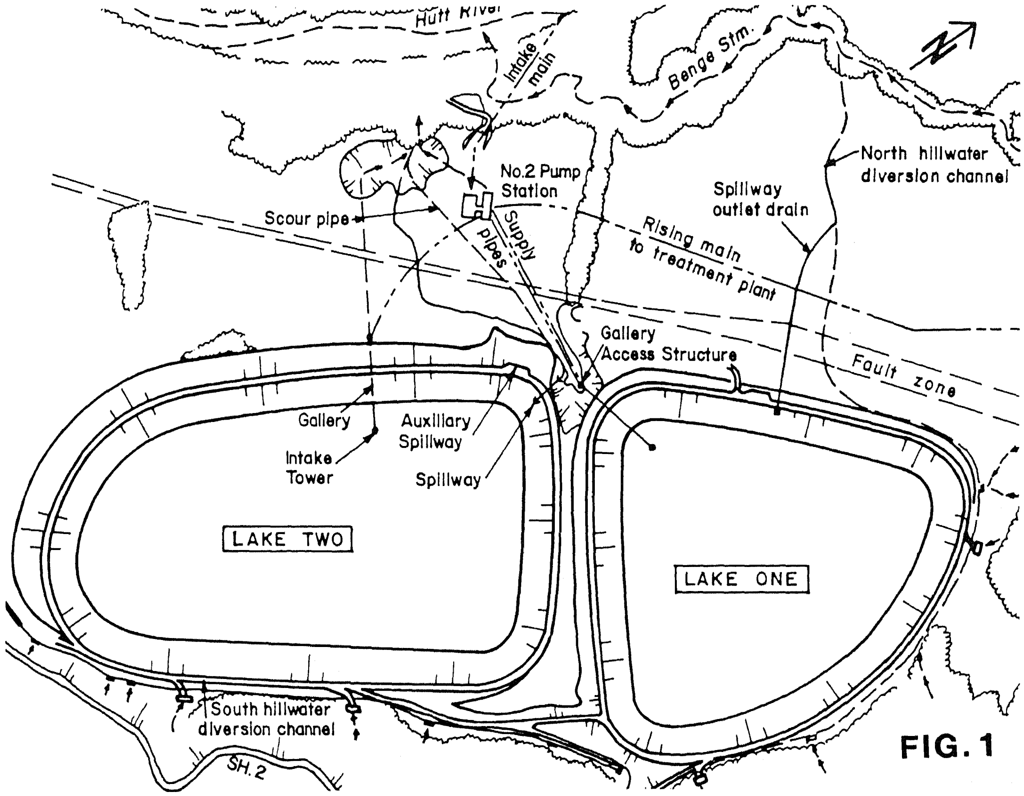


FIG. 1

The layout adopted to meet the client's objectives of adequate storage at minimum cost, interception and diversion of all extraneous flow, prevention of excessive leakage, and avoidance of excessive loss or damage caused by earthquake, is shown in Fig.1. Both lakes are located just clear of the fault and abut the high ground to the south. Lake One is sited on the upper terrace principally in cut and Lake Two on the lower terrace involving both cut and fill. Spillways are provided and each lake has an Intake Tower in the lake and a Gallery under the embankment through which all connecting pipework is taken (refer Fig.2).

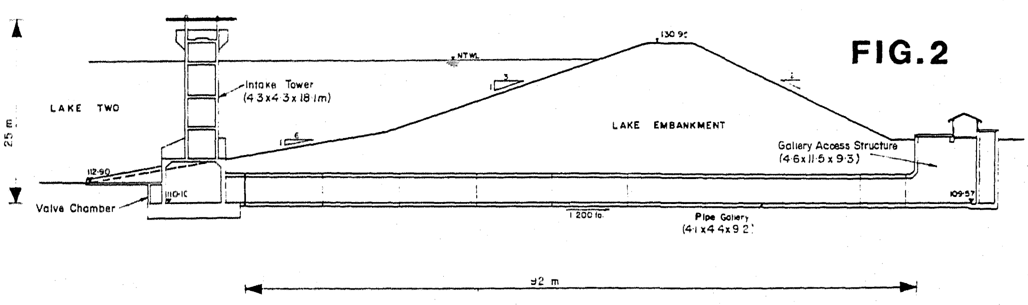


FIG.2

INVESTIGATIONS

Investigations included study of disturbance in the fault zone, close examination of site materials and trial construction. Substantial cost savings were achievable if local materials could be used to line the lakes instead of manufactured linings as originally envisaged.

Trenches 4-5 m deep dug across the fault trace on the upper terrace were logged by engineering geologists including staff of the New Zealand Geological Survey. An approximately 30 m wide zone of disturbed gravels was observed. Based on this observation and prediction that future movement of the fault will be on or close to the existing fault trace, all significant construction was required to be located 50 m beyond the centre of the trace. A 10 m deep trench dug across the fault during construction enabled further mapping and continuous recording of strains at the fault by government scientists.

Materials investigations included extensive use of shafts, pits and boreholes. A large part of laboratory testing comprised gradings tests on natural and blended materials to assess the scope for zoning materials in a 'zero-waste' earthmoving operation. A large shear box (450 x 450 x 660 mm) and similarly sized permeameter were developed to test natural gravels to 150 mm size. Field testing included small scale screening trials, trial compaction and construction of a small trial 'lake'.

DESIGN PHILOSOPHY

Under static conditions the lakes and ancillary structures were to perform safely with prevention of excessive leakage and avoidance of infiltration by groundwater presenting the major design tasks. Shape was important for water circulation reasons and appearance significant because the lakes are included in a regional park. The seismic design philosophy adopted has three elements. Firstly, for earthquakes with return periods matching the expected useful life of the scheme (75 years nominal), all structures should remain undamaged. Secondly, for earthquakes with a 10% probability of exceedence in 50 years (viz., average return period 475 years) all structures should remain safe and only suffer small repairable damage. Thirdly, for larger earthquakes, significant damage and displacement would be accepted, but the damage and consequences should be restricted and reinstatement facilitated.

Earthquake generated ground displacements are difficult to predict and it would be prohibitively costly if not impossible to design buried structures to withstand large displacements. As demonstrated in the 1971 San Francisco earthquake, joint design can improve the chances of buried structures surviving without collapse, and the approach taken provides for articulation in the Galleries and at the interface with the Towers. Similarly, pipework which has to cross the fault has a degree of joint articulation within the disturbance zone but relies on emergency overflow from the Gallery Access Structure if pipes rupture at the fault.

Estimates of earthquake generated ground motions were based on the work of Matuschka (Ref.1) who assessed seismic hazard in New Zealand using a probabilistic approach. The model involves the location and geometry of the sources of seismic activity, reference curves relating earthquake numbers/magnitudes per year within the siesmic source zones and attenuation equations relating amplitudes of ground motion to earthquake magnitude and source-site distance. The maximum horizontal ground motions with 10% probability in a 50 year period were estimated to be; acceleration 0.7 g, velocity 79 cm/s and displacement 33 cm.

DESIGN COMMENTARY AND DETAILS

Embankments

The embankments are essentially homogeneous structures built of terrace gravels or excavated into hillslopes, blanketed by a thin lining of low permeability materials and separated from the lining by a drainage layer which intercepts underseepage and leakage (Refer Fig's 3 & 4). On-site materials have been processed to provide rip rap, drainage zones, filters and lining materials in an operation involving little waste. The drainage layer prevents a seepage gradient through the embankment thereby preserving a high degree of stability. However, stability was checked assuming the lining ruptured by an earthquake and the drainage layer being ineffective. A downstream toe drain is provided for fill embankments to promote a low phreatic profile under these conditions.

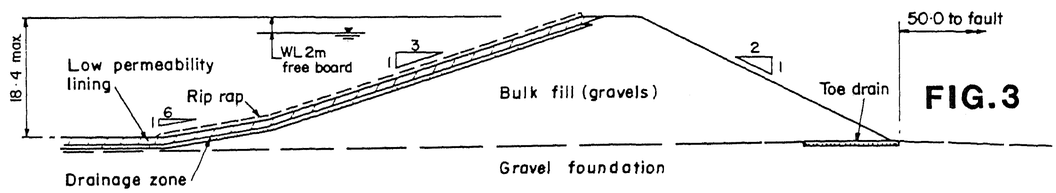


FIG.3

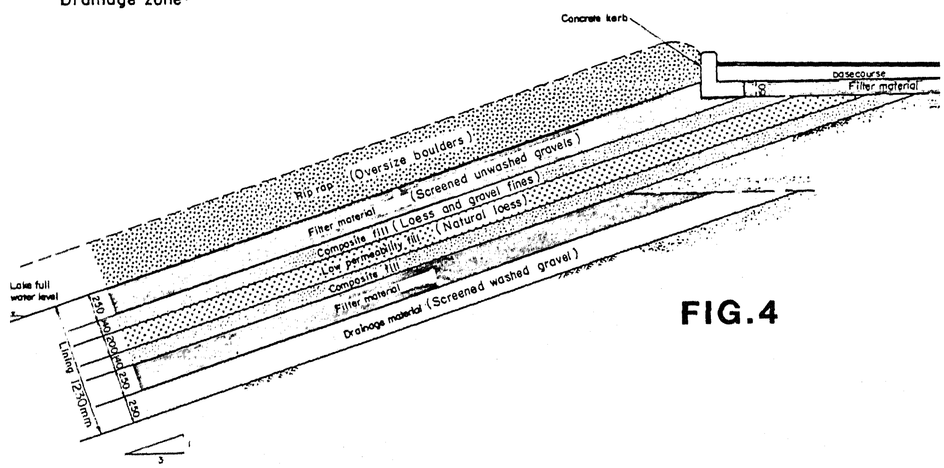


FIG.4

Under conditions of controlled drainage, the high shear strength of the compacted gravels (ϕ' up to 63^o) allows steep slopes even under high seismic accelerations. Internal slopes were governed by practical construction requirements and avoidance of lining failure under conservative rapid drawdown assumptions. Upper slopes are 1 on 3 to limit difficulties in placing thin surfacing layers and lower slopes are 1 on 6 to suit the transition of machinery between the steep slopes and flat lake floor. The downstream slope was made as steep as possible using the simplified methods of Makdisi and Seed (Refs. 2, 3) to check seismic stability. Liquefaction was judged not to be a risk because of the dense nature of the natural gravels and controlled fill.

Yield acceleration studies showed yield potential only for shallow slip surfaces near the crest, with maximum crest displacements of 30 mm calculated assuming no strength loss. Only a small strength loss is anticipated but even with much reduced fill strengths, estimated crest displacements are still only 100 to 200 mm.

Freeboard and Flood Protection

Seismic wave generation was checked using the work of Das and Weigel (Ref.4). A wave height less than 1 m is likely to be generated by movement of the lake boundary and seiching is unlikely to exceed a few centimetres because the natural period of each lake is large compared with the dominant period of earthquake shaking. Freeboard has been set at 2 m.

A semicircular ogee spillway in each lake will cope with water from a 1,000 year return period storm plus continuous inflow from the river should intake pumps not switch off. Allowance has also been made for hillwater overspill even though this conditon should never arise. To provide extra security and promote controlled damage in any untoward seismic behaviour, auxiliary spillways 20 m wide set 1 m below crest are provided linking with the Hutt River via low velocity, overland waterways.

Structures

The Intake Tower/Valve Chamber, the buried Gallery and the downstream Access Structure for each lake as shown in Fig.2, have been designed to perform as follows:

(i) response to be fully elastic under ground accelerations up to 0.4g predicted for earthquakes with an average return period of 75 years,

(ii) yield permitted in some selected zones but system to remain fully functional under the maximum design horizontal acceleration of 0.7 g and repairs able to be made without disruption to operations,

(iii) structures not to collapse totally under peak horizontal accelerations of 1g as could occur during a maximum credible earthquake.

A base shear coefficient of 0.5 g was derived for the Intake Towers using the approach of Berrill et al (Refs. 5, 6). Energy is absorbed by rocking about the Valve Chamber foundation slabs: therefore plastic hinging at the Tower/Chamber interface is prevented. The additional hydrodynamic pressures on the Towers from earthquake shaking calculated from the added mass concept, create forces almost equal to those from structural inertia. Damping was assumed to be small and ignored.

A special repairable joint as indicated on Fig.5 was designed for the Valve Chamber/Gallery intersection. The joint comprises a "floating" reinforced concrete link slab less than half the thickness of the adjoining walls, cast against an expanding cork filler. A pour-in polyurethane elastomer separates the elements at the outside face and expanded polystyrene fills the gap at the inside face. The joint allows rocking deformation of up to 30 mm under 0.7 g but will partially fail at higher accelerations. Repairs would be made from the inside face.

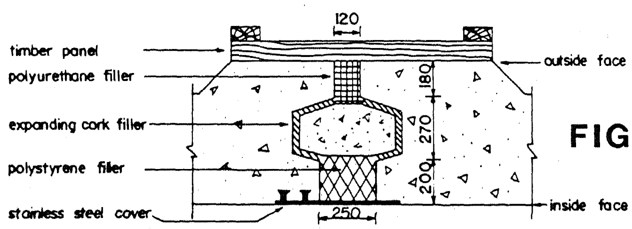


FIG.5

The rectangular Galleries have been provided with semi-articulated control joints at 10 m spacing to enable movement with the ground. They have been designed for at-rest pressures with no allowance for higher pressures resulting from transverse ground displacement. External water pressures have been allowed for to accommodate rupture of the lake lining. Vertical pressures have been limited by providing a less rigid fill zone above each gallery with earth pressure cells confirming performance.

Gallery joints are shear keyed and dowelled and have external waterstops with a special detail employed on the roof to promote intimate contact between the waterstop surface and fresh concrete (Refer Fig.6). Substantial timber panels are provided over the joints to prevent ingress of soils after large earthquake displacements and to facilitate repairs.

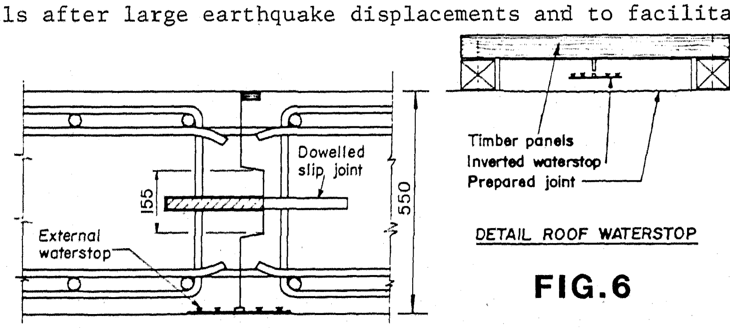


FIG.6

Pipelines

Pressure pipelines are concrete lined steel with spigot and socket joints welded internally and externally. Potential rupture of pipes within the Galleries is accepted and the downstream Access Structure has a wet well for emergency discharge. Drains beyond the Access Structure carrying seepage and scour flows across the fault are rubber ring jointed concrete pipes oversized to 1,200 mm diameter to allow access for inspection and repair. Regular discharges can be pumped from the wet well if the drains have to be excavated for repairs following severe fault movement.

Articulated pressure pipe joints within the fault zone are made up with register free bolted couplings and tie bars spaced uniformly around the pipe providing 50% of the area of steel in the pipe barrel. Thirty millimeters clearance is maintained between each pipe and tie bars are made up on rocking washers to hand-tight condition.

Valve operation features emergency air driven actuators powered by high pressure gas bottles so that valves in the Chamber may be opened or closed from ground level with the Towers and Chambers flooded.

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