

DESIGN GROUNDWATER LEVELS

Gareth Swarbrick¹ and Strath Clarke²

*¹Senior Principal Engineer, ²Senior Principal Engineer
Pells Sullivan Meynink, Sydney, Australia*

ABSTRACT

Successful design of structures located within or below the groundwater table requires assessment of the variation of groundwater levels over the design life. Currently the methods used for prediction of future water levels are poorly developed usually relying on extrapolation of a limited period of on-site monitoring or reliance on other historical records that may bear little relevance to the site.

Adoption of overly conservative water levels can have a very significant impact on design and construction costs. Therefore, an accurate assessment is required to achieve a design that achieves the optimal balance between risk and cost.

This paper discusses the pitfalls associated with these methods and presents some examples of failure to select appropriate design groundwater levels. Other methods are discussed than may be employed to provide an alternate and potentially more accurate assessment of design water levels so that the risk of adopting poor design levels may be reduced in the future.

1 INTRODUCTION

Structural design considers many load cases, including groundwater pressures. Groundwater pressures can be a significant loading component particularly if the structure is design to prevent water ingress, resulting in the development of hydrostatic pressures (i.e. a 'tanked' system). In addition, water pressure reduce the inter-particle soil stresses (i.e. effective stress) and thus the strength of granular soils, such as sand. This can result in increased soil loads applied to structures.

While many load cases can to some degree be considered to be relatively constant (e.g. dead loads), groundwater levels are known to rise and fall due to climate, topography, geologic factors, as well as man-made influences such as drainage measures or failure of water mains. These variations give rise to considerable uncertainty in the way design groundwater pressures are defined, estimated and applied.

Often the methods used for estimating design pressures are poorly developed, and may rely on extrapolation of a limited period of on-site monitoring or reliance on other historical records that may bear little relevance to the site. In addition, some of the standards developed to specify how design loads are to be determined are unclear.

This paper discusses the pitfalls associated with these methods and presents some examples of failure to select appropriate design groundwater levels.

2 LEGISLATIVE REQUIREMENTS

2.1 DESIGN STANDARDS

Most of the current international standards, including Australian standards, adopt a limit state design (LSD) format. In principle this methodology adopts the following design approach (AS1170.1:2002):

Limiting states are defined as "states beyond which the structure no longer satisfies the design criteria". The principal limit states are:

- Serviceability Limit State (SLS): states associated with specified service requirements (e.g. deformation).
- Ultimate Limit State (ULS): states associated with structural stability.

An appropriate level of reliability is required for each limit state. Standards based on limit state formats implicitly define the appropriate levels of reliability by the prescriptions contained within them.

Partial factors are applied to loads (actions), material characteristic values, resistances or all three. Partial factors are defined by EuroCode EN 1990 as factors "...which take[s] account of the possibility of unfavourable deviations of the action values from the representative". In Australian standards these are generally referred to as load factors when

applied to loads, material factors when applied to material characteristic values and resistance factors when applied to a resistance or reaction.

To satisfy the ULS, the factored actions are compared to the factored resistance where the resistance may be derived from factored material characteristic values, reactions derived from testing or measurement or a combination of the two. The factored actions must not exceed the factored resistance.

The partial factors for actions are multiplied by the characteristic action (or the effect of the characteristic action).

In Australian LSD standards the characteristic resistance (or the effect of the characteristic resistance) is multiplied by a strength reduction factor. Some codes replace the strength reduction factor with its reciprocal, which is divided into the characteristic resistance.

Australian Standards which consider groundwater loads include:

- Australian/New Zealand Standard AS/NZS1170.1:2002 *Structural Design Actions - Part 1: Permanent, Imposed and Other Actions*.
- Australian Standards AS3735:2001 *Concrete Structures for Retaining Liquids*.
- Australian Standards AS4678:2002 *Earth-retaining Structures*.
- Australian Standard AS5100.1:2004 *Bridge Design - Part 1: Scope and General Principles*.
- Australian Standard AS5100.2:2004 *Bridge Design - Part 2: Design Loads*.
- Australian Standard AS5100.3:2004 *Bridge Design - Part 3: Foundations and Soil-supporting Structures*.
- Australian National Committee On Large Dams (ANCOLD) *Guidelines on design criteria for concrete gravity dams*, September 2013.

There are also commentaries available for many of the above standards.

Relevant international standards include:

- European Standard EN1990:2002 *Eurocode: Basis of Structural Design*.
- European Standard EN1991-1-1:2002 *Eurocode 1: Actions on Structures - Part 1-1: General Actions – Densities, Self Weight, Imposed Loads for Buildings*.
- European Standard EN1992-1-1:2005 *Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules and Rules for Buildings*.
- European Standard EN1997-1:2005 *Eurocode 7: Geotechnical Design - Part 1: General Rules*.
- International Organization for Standardization ISO2394:1998 *General Principles on Reliability for Structures*.

2.2 GROUNDWATER DESIGN PRESSURES

Groundwater pressures can be present in different ways:

- Positive pore pressures associated with soil and rock below the phreatic surface. Generally this condition is assumed to be 'saturated' with no entrapped air.
- Negative pore pressures associated with soil and rock above the phreatic surface. Generally this condition is considered to be 'unsaturated' although the soil may still be at or close to saturation immediately above the phreatic surface; a region known as the capillary fringe.
- Steady state where pressures do not change over time under either a no-flow or a steady flow condition. Under a no flow condition pressures everywhere are hydrostatic and determined by the depth and density of water. Steady flow pore pressures are not hydrostatic but decrease in the direction of flow.
- Transient where pressures change over time.
- Dynamic pressure such as wave action, tidal variation – Generally these loads are much smaller within a porous media and are reflected in transient pore pressures.

Groundwater pressures should always be thought of as a continuum under positive and negative conditions. The phreatic surface or water table is simply the location where pressures equal atmospheric. As we generally use atmospheric pressure as a reference value pressures below the phreatic surface are termed 'positive' while those above are 'negative'. A good demonstration of this continuum is evaporation whereby water flows upwards to the surface in response to a reduction in negative pore pressure. This flow can only occur because the network of water is continuous.

In Australia the majority of design situations relate to permanent, static, saturated groundwater pressures. Sometimes there is consideration for changes in phreatic surface levels due to transient conditions.

Negative pore pressures are generally not considered because:

- They are more difficult to measure and model than positive pressures.

- They can fluctuate significantly (reduced to zero after rainfall or increase to large values under evaporation).
- It is generally conservative to ignore them.

Table 1 presents a summary of positive groundwater pressure design parameters recommended in selected standards. The key components within these standards are:

Groundwater Pressure Definition a definition of how the load or action from groundwater pressure is to be derived

Partial Factor the load factor that should be applied to this derived load or action

Table 1: Summary of partial groundwater factors in selected standards

Standard	Groundwater Pressure Definition	Partial Factor	Comments
AS1170.1 (2002)	<i>The action resulting from hydrostatic pressure of water acting on surfaces below ground level shall be the value assuming the water level is at ground level or, where information is available, the ground water level with an annual probability of exceedance of 1 in 50.</i>	1.2 for controlled design water level 1.5 where levels cannot be readily controlled or limited	The commentary to the standard acknowledges that this could lead to very conservative or uneconomical solutions and permits the designer to consider alternative methods
AS5100 Bridge Code	<i>Variation in water levels shall be taken into account by using design levels based on a return period of 1000 years for the ultimate limit state or 100 years for the serviceability limit state.</i>	1.0	The 5100 commentary states that "Design methods and factors in AS 4678 are not compatible with AS 5100"
AS4678 Earth Retaining Structures	<i>The estimate of the height of the water table shall represent the worst credible location during the life of the structure</i>	1.0	In lieu of requirements in AS 1170.1
AS3735 Liquid Retaining Structures	<i>Normally, concrete structures for retaining liquids are designed on the basis of the liquid level being at the top of the wall</i>	1.0	This code is primarily concerned with serviceability limit states
ANCOLD Concrete Gravity Dams	<i>Usually several water load cases will need to be considered, corresponding to Normal, Unusual and Extreme Water Levels. The water levels may range from Dam Empty to Full Storage Level (FSL) and to Probable Maximum Flood (PMF) level.</i>	1.05 for dead loads 1.0 for 'unusual' loads 1.0 for 'extreme' loads	
EN1997 Geotechnical Design Code	<i>Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, the design ground-water table should be taken as the maximum possible level, which may be the ground surface</i>	1.35 for permanent unfavourable 1.5 for variable unfavourable	Favourable loads have a factor of 1.0 for permanent and 0 for variable

There are several factors in common amongst these standards:

- There is no single definition of the derivation of groundwater levels.
- There is a range of partial factors across and sometimes within codes.
- Partial factors are generally lower for more severe groundwater cases.
- There is some allowance for variation in partial factors based on alternate methods.

Importantly the Australian loading code provides some clarity in its definition of groundwater levels as equating to conditions that represent an annual probability of exceedance of 1 in 50. The method or methods by which this is to be determined are not discussed in the standard. Further discussion of this requirement follows.

2.3 ENVIRONMENTAL APPROVAL

Structures built within and beneath the groundwater are commonly designed to be either drained or tanked structures, or sometimes a combination.

During construction, temporary dewatering is required so that work may take place under dry conditions. Temporary dewatering generally requires a water extraction licence that applies to short-term dewatering activities. In NSW licenses are required for extraction over 3 M/L this being the basic landowner extraction right for most areas not within water sharing plan as per the NSW Water Act 1912. Temporary construction licences in NSW require data on current groundwater levels, the likely effect on groundwater due to pumping and written authorisation for the disposal of the extracted groundwater. License approvals are typically for 1 year.

Drained designs may result in permanent impacts on groundwater potentially intercepting water from shared groundwater resources such as aquifers. Consequently, drained designs normally require environmental approval. In NSW the Office of Water (NoW) is responsible for approving or otherwise this type of design in accordance with the NSW Water Management Act (2000). The policy is set out in the NoW document *NSW Aquifer Interference Policy* dated September 2012.

Aquifer interference activities are defined within the NSW Water Management Act (2000) as:

- The penetration of an aquifer,
- The interference with water in an aquifer,
- The obstruction of the flow of water in an aquifer,
- The taking of water from an aquifer in the course of carrying out mining or any other activity prescribed by the regulations, and
- The disposal of water taken from an aquifer in the course of carrying out mining or any other activity prescribed by the regulations.

Any permanent intake of water above 3 ML/year requires a water license and in NSW such licenses are perpetual. For many aquifers, such as the Botany Sands Aquifer in Sydney, NoW considers groundwater extraction to currently be oversubscribed due to the number of approved licences and therefore approval of new licences is virtually impossible. Licenses can, however, be traded.

Where interception, collection or diversion of groundwater is not permitted submerged structures must be designed to prevent the ingress of water. Such structures are commonly termed 'tanked' and must be designed to accommodate the full potential groundwater pressure.

If drainage is permitted then pressures on structures can be discounted or even ignored reducing the cost of the structural elements. However, drained designs may require extensive drainage infrastructure, including pipework, collection tanks, a treatment plant, and pumps to discharge the collected water. This can mean significantly higher maintenance costs associated with collection and running costs over the life of the structure.

Tanked designs typically have higher costs for structural members but little to no on-going costs associated with groundwater.

3 ESTIMATION OF GROUNDWATER LEVELS

Estimation of groundwater levels is the realm of hydrogeologists. However, the importance of pore pressure in geotechnical design necessitates knowledge by geotechnical engineers of groundwater conditions including distribution, depth to the water table and other factors such as direction and rate of flow. Consequently geotechnical engineers have developed some affinity with groundwater within the local setting upon which their design is based.

Current practice is such that hydrogeologists, geotechnical engineers, geologists and other professions can be asked to provide estimates of groundwater pressures. Importantly many interpret the location of the groundwater table as being synonymous with groundwater pressure. However, this relationship is only true when there is no groundwater flow within the system being analysed – a condition that is rare, or near impossible. Even when there appears to be good lateral confinement the simple mechanisms of recharge and evaporation both result in groundwater flow and therefore a disconnect occurs between groundwater tables and groundwater pressure.

In many situations an assumption of steady state conditions with a static water table can suffice as a default design position from which to derive design groundwater pressures. Furthermore groundwater flow in many cases results in a reduction in groundwater pressure ensuring that this assumption provides a conservative approach.

This approach is conservative wherever there is a situation of net inflow into a void such as an excavation, mine pit or drained tunnel. In these cases the phreatic surface at the excavation boundary does not represent groundwater pressures further away from the boundary. A linear extrapolation assuming hydrostatic conditions in particular will produce an underestimate of the pressures away from the excavation.

Where static groundwater conditions are considered appropriate for design the groundwater level is often based on one or more of the following methods:

- Historic data from nearby public wells
- Estimated from topography and neighbouring water bodies
- Site specific monitoring wells

More complex estimates of groundwater pressure distributions may be derived from:

- A combination of historic data and wells on site including point measurements at varying depths.
- Groundwater modelling in 1, 2, 3 or 4 dimensions (the 4th dimension represents transient conditions).

In most practical applications groundwater levels are estimated from historic data or from wells installed on site. Adverse conditions (usually high water tables) are commonly extrapolated from these initial estimates by consideration of rainfall, topography and the influence of nearby constant water sources, as discussed further below.

3.1 HISTORIC DATA

Most states and territories in Australia have their groundwater resources available on-line. These include:

- Bureau of Meteorology - <http://www.bom.gov.au/water/groundwater/explorer/map.shtml>
- Geoscience Australia - <http://www.ga.gov.au/cedda/maps/96>
- National Research Infrastructure for Australia (NCRIS) - <http://groundwater.anu.edu.au/>
- NSW - NoW Pinneena Database (<http://www.water.nsw.gov.au>) or <http://allwaterdata.water.nsw.gov.au/water.stm>
- SA – WaterConnect (<https://www.waterconnect.sa.gov.au/>)
- QLD – QLD Govt <https://data.qld.gov.au/dataset/groundwater-database-queensland>
- WA – Department of Water <http://wir.water.wa.gov.au/SitePages/SiteExplorer.aspx>
- Vic – VVG <http://maps.cerdi.com.au/vvg.php?agreement=Agree+and+Continue>
- Tas – Groundwater Information Access Portal <http://wrt.tas.gov.au/groundwater-info/>
- NT – DLRM <http://www.lrm.nt.gov.au/spatial-lrm/nrmapsnt>

The NSW *allwaterdata* site only came on-line this year. Prior to this the NSW Office of Water (NoW) produced an annual DVD of all data called PINNENA.

Figure 1 shows a map depicting all of the 90,000 registered bores in NSW. About 370 of these sites are telemetered and available as real-time data. A further 3000 are manually monitored and periodically added to the database. The vast remainder are extraction bores for use on-site with a further few hundred being part of a coal mining observation network.

NSW legislation ensures that property owners have a right to extract water on their own property. This is occasionally limited or prevented through the application of an embargo as is the case for the Botany Sands Aquifer in Sydney. The embargo was applied in recognition that the quantity of water abstraction, if it continued increasing, would place undue stress on the aquifer and impact on existing users.

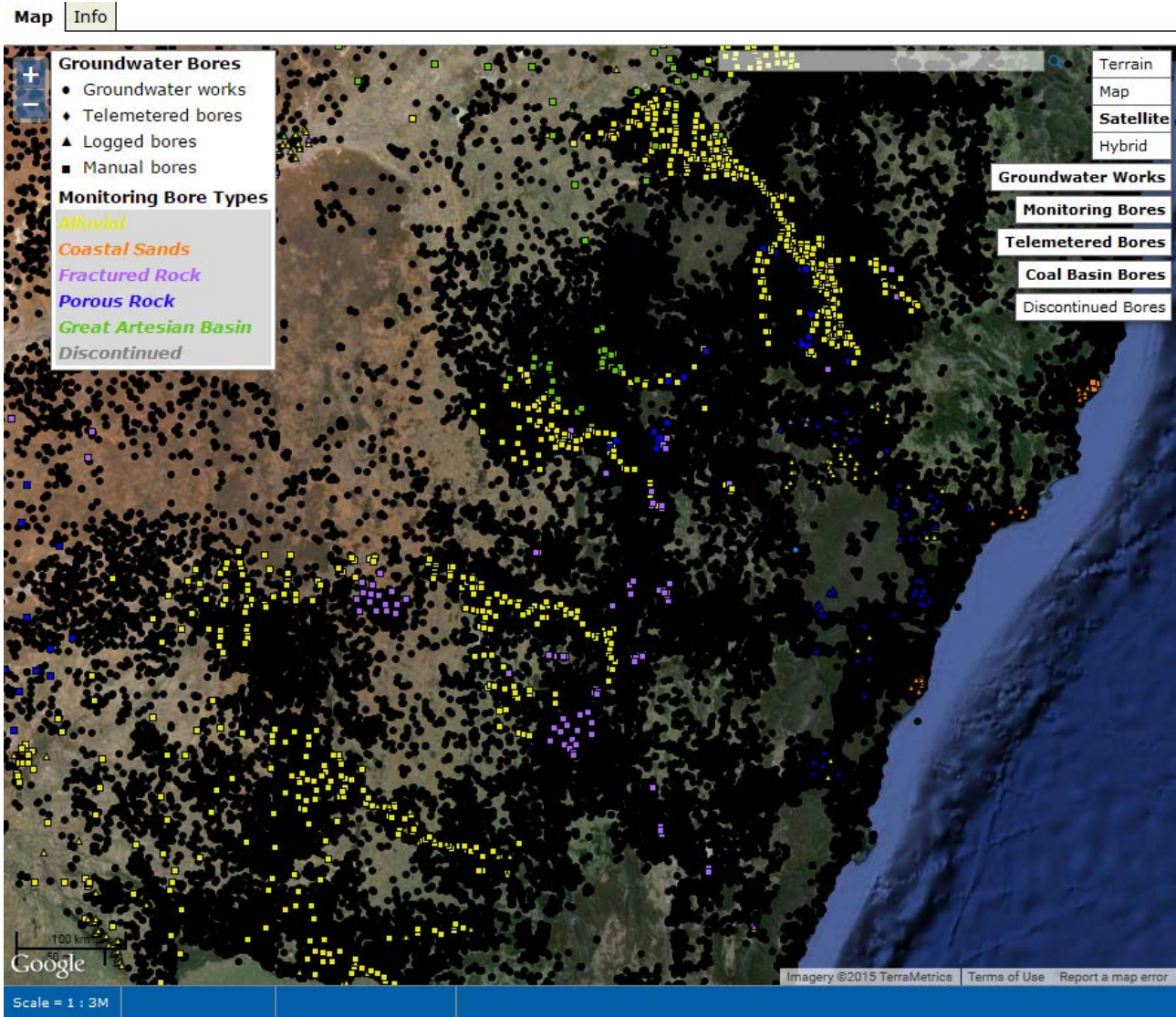


Figure 1 – Location of groundwater monitoring wells within NSW

NSW Office of Water

HYPLOT V133 Output 16/03/2015

Bore Water Level below Measuring Point (Metres) for site GW075020

Plot Start 00:00_01/01/1998

1998-2015

Plot End 00:00_01/01/2015

— Logger Data - Hole 1 Pipe 1 Slotted Interval 24.500 - 27.500 m

— Manually Read Data - Hole 1 Pipe 1 Slotted Interval 24.500 - 27.500 m



Figure 2 – Historical bore water levels obtained from the NSW Office of Water

An example of groundwater level data is shown in Figure 2. The logged data is recorded at least daily and sometimes hourly. Accompany this data are details of the well installation including construction date, location, installation details (collar RL, base RL, screen interval, driller, etc.), initial water level and drillers log.

While data quantity and access are generally excellent the data quality is often not assured. From Figure 2 it is evident that:

- The available records (15 years) are relatively short compared to rainfall records.
- Data can be somewhat erratic; the sharp rise in 2014 for example being incongruent with the rest of the data
- There is a disconnect of 4.5 m between manual readings (red line on far left) and that recorded by a data logger

The disconnect between manual and data logger readings is often the result of conversion between the depth to water reading (manual) and water pressure (data logger). This particular site also exhibits a discrepancy between on-line data and the PINENNA database due to differences in the collar RLs adopted in each database. Unfortunately there are many instances of data error in this and other public databases and reliance upon these values alone is generally not recommended.

Potential errors in the data can be highlighted and minimised by comparing a number of sources. An example of plots for several wells few kilometres apart within Sydney's Botany Sands Aquifer is shown in Figure 3. Here the behaviour of well GW075020 exhibits a different trend when compared to other bores.

Figure 3 shows groundwater levels for well GW07025 from two sources; the original PINNENA database (postfix 'orig') and the new on-line database (postfix 'new'). It is evident from Figure 3 that there are periods in the record where the two data sources do not agree. By inspection it would appear that the new on-line data is the more consistent dataset based on the unlikely instantaneous jumps in the PINNENA data. However there is no definitive way of determining which record (if any) is more reliable.

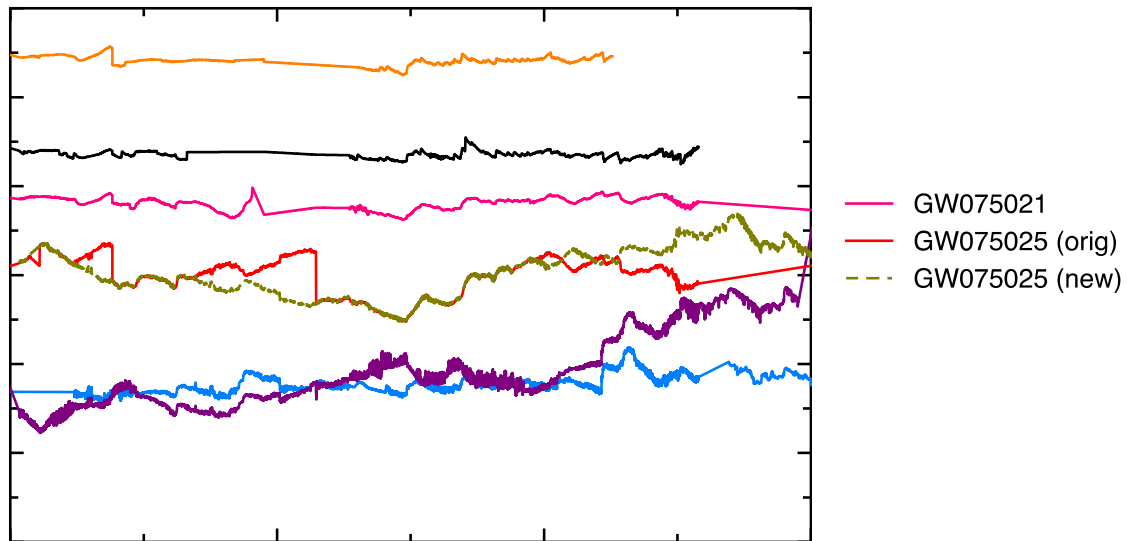


Figure 3 – Comparison of multiple groundwater monitoring wells within the Botany Sands Aquifer

3.2 ON-SITE MONITORING

The installation and measurement of site specific monitoring wells is generally well understood and standard practice for many medium to large projects. For larger sites several wells are often constructed. Manual monitoring may be undertaken on a weekly or fortnightly basis, though use of data loggers is becoming much more common, with logging intervals of as little as a few minutes. Monitoring is typically undertaken for as long as possible.

On-site wells can provide information on:

- Variation of water levels across the site and over time.
- The magnitude and direction of the hydraulic gradient (when three or more wells have been installed).
- The rate and magnitude of response to rainfall.
- Hydraulic conductivity (permeability) when used as part of a pump or injection test.
- Water quality (when installed to appropriate standards).

Common deficiencies for on-site wells include:

- The relatively short period over which readings can be taken (typically less than a few months)
- Use of long screened lengths, which indicate an average pore pressure over its depth thereby dampening the response to rainfall and recharge. The main reason wells are slotted over long lengths is to facilitate purging for environmental testing.
- Poor detailing such that surface water inflows are permitted to enter the well, resulting in excessively high water levels. This is a common problem in flush-finished wells (e.g. located in roads).
- Wells are sealed at the surface such that the air in the top of the well becomes pressurised and restricts groundwater level fluctuations.
- Damage to data loggers from water ingress or sometimes corrosion.

Figure 4 shows data from a typical site monitoring campaign. This site has used three wells and monitored for nearly a year. Note the initial equilibration period of up to a few months before levels stabilise.

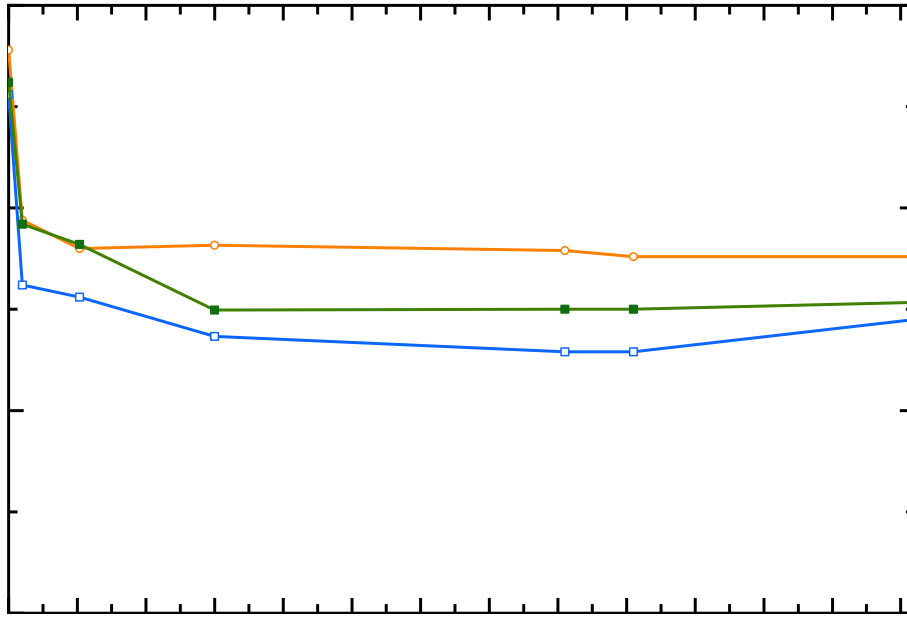


Figure 4 – Example of site groundwater monitoring data

3.3 GROUNDWATER MODELLING

Groundwater modelling is commonly performed using numerical codes that equilibrate pore pressure within a network of finite elements or finite difference nodes. This method can be performed over one to three dimensions, include essentially any type of source, sink or boundary condition and produce steady state or transient predictions within the model space.

Reliable groundwater models need to model a region significantly larger than the study area to overcome boundary effects. They should also be calibrated to site-specific data. If such data is limited or of poor quality the groundwater model will also be poor and may provide worse estimates than historical or site specific measurements.

Even when data is plentiful other data gaps (e.g. depth to bedrock) or differences in scale may still limit the ability to reduce calibration errors to acceptable limits.

For example, in mining applications a ‘good’ data fit for a groundwater model is generally taken to be around ± 20 m of head or ± 200 kPa. This level of error is comparatively small when considering the model extent is often 4 to 5 times the size of the mining activity being modelled. However, this error in comparison to the magnitude of pore pressure in a particular slope only, say, 50 m high may be completely unacceptable.

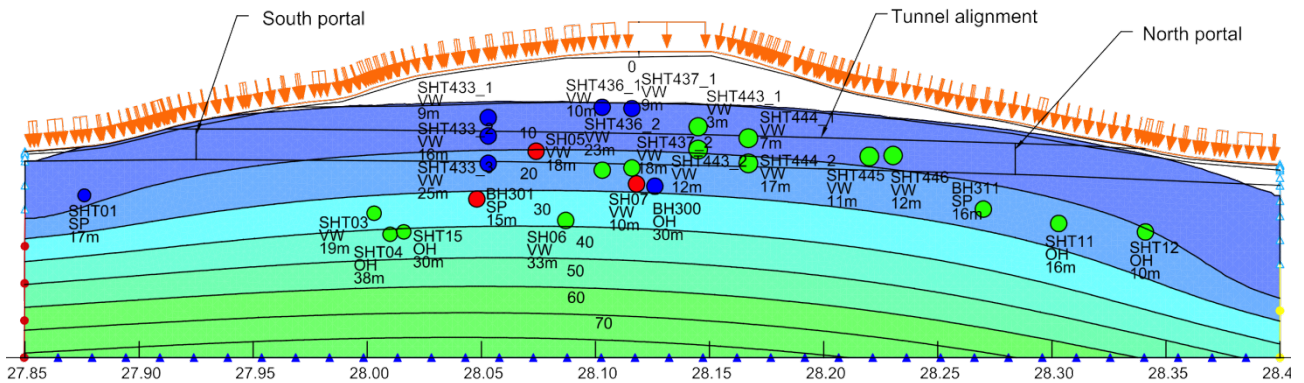


Figure 5 – Two-dimensional groundwater model of St Helena tunnel

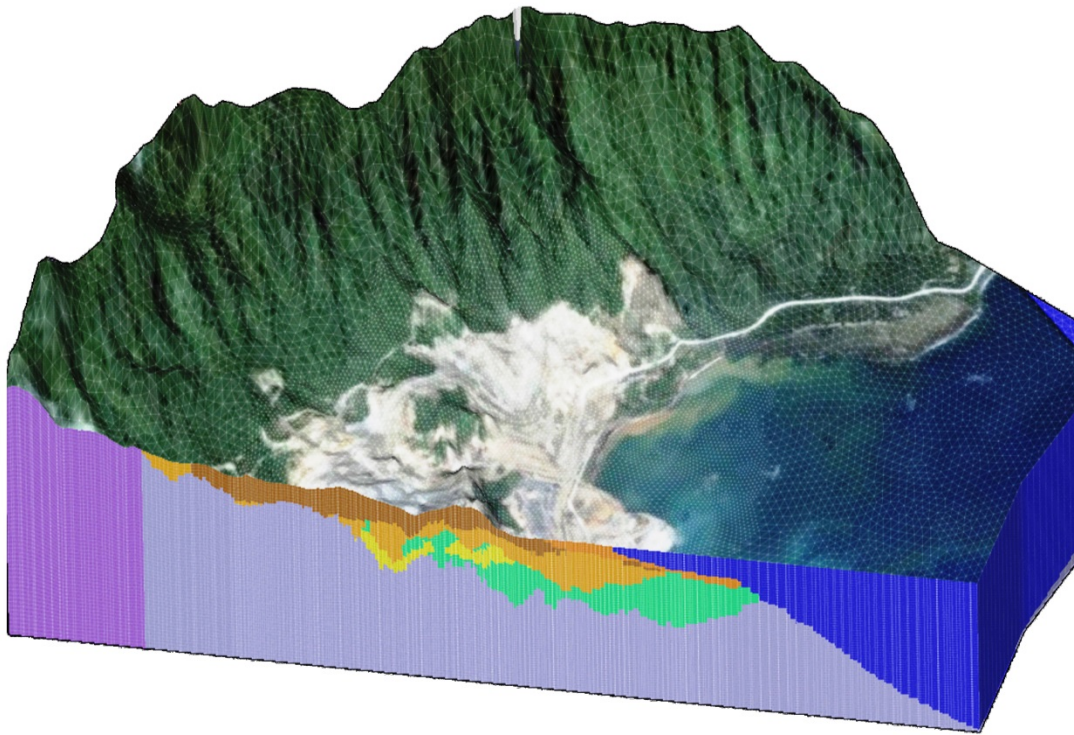


Figure 6 – Three-dimensional groundwater model of Lihir Gold Mine

3.4 EXTRAPOLATION OF GROUNDWATER LEVELS

All estimates for groundwater pressures require some form of extrapolation. Historical data must be extrapolated spatially to the site under assessment. On-site monitoring data, groundwater modelling data and most likely historical data will require temporal extrapolation to estimate variation in pressures over the design life.

Extrapolation temporally relies upon the duration of records. In NSW the earliest available records in the public database are from 1990. Therefore this record may not capture variations over a 50-year design life. This limitation directly affects extrapolation from historical or site monitoring and indirectly effects groundwater modelling, which are typically calibrated to historic or site data.

Historic data will usually involve some form of spatial extrapolation. An example of spatial extrapolation is shown in Figure 7 whereby triangulation has been used to estimate the water level at a site situated in between three sites; Well 1, Well 2 and Well 3. In this case the extrapolation slightly exceeds all well data as the point of extrapolation lies just outside and up-gradient of the location of wells 1, 2 and 3.

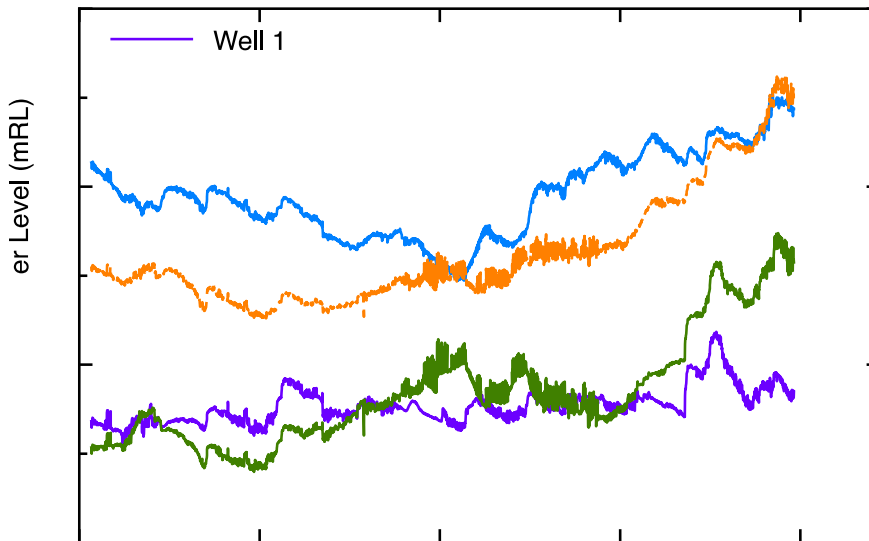


Figure 7 – Example of historical well data extrapolation

In this example the data suggest a significant variation in water levels over time and this is reflected in the estimated water level on site. Despite this variation the hydraulic gradient based on triangulation does not vary significantly over time. This observation is common to many sites and can be used to extrapolate historic levels when data from only one or two wells is available. In this case it may be possible to estimate the hydraulic gradient by other means or from a limited subset of data when additional well data is available.

Extrapolation by these methods is well suited to sites where conditions at monitored wells and the site are similar. Factors that may limit the effectiveness of these methods include differences in:

- Presence or absence of aquifers or aquitards.
- Bedrock topography.
- Surface topography.
- Local sources and sinks such as lakes, rivers, streams, detention basins and even proximity to the ocean.
- Changes rates of recharge due to geology or urban development.

In cases where there is doubt about the effectiveness of extrapolation then the common fall-back is to assume the water table is at the surface. If this approach is adopted then there should be some allowance for a reduction in load factor to reflect this extreme position. A logical load factor in this case would be 1.0.

4 CASE STUDIES

4.1 GENERAL

The case studies presented here are based on real sites. However site location and other details have been kept confidential. The authors consider that the lessons learnt from these sites would be applicable to other site with similar conditions and challenges.

4.2 COMMERCIAL SITE

This site comprises a medium size commercial development with 3 basement levels for car parking. The site is situated within the Botany Sands; a large aquifer system situated north of Botany Bay extending to Centennial Park just south of the Sydney CBD.

The design was partial tanking below a groundwater level primarily based on site measurements of water levels. Three wells were installed prior to construction and monitored for about 1 year recording data similar to the example shown in Figure 4. After this period of monitoring a design water level was recommended which was about 1 m above the maximum recorded groundwater levels. The basement below this design level was tanked while the structure above was designed assuming dry conditions.

About 5 years after construction groundwater levels rose at the site exceeding the design levels by 2.5 m. Water entered the basement at the join between the tanked and non-tanked sections causing disruption to users and causing concern for

the structural integrity of the tanked walls and slab. The load on the slab at the full height of groundwater was estimated to be about twice the design value.

The exact cause of the discrepancy between design and actual groundwater levels is unknown. However, examination of available data indicates that:

- The period during which groundwater monitoring was undertaken on site was relatively dry compared to long term rainfall records
- Groundwater levels within the Botany Sands Aquifer can vary by 5 m or more as demonstrated in Figure 3.

Currently remedial measures are under development to maintain building amenity and serviceability under revised design groundwater levels. Design levels have been based on a number of considerations including 3D groundwater modelling, extrapolation of historic groundwater levels and analysis of rainfall data. Further discussion on rainfall analysis tools is provided in Section 5.

4.3 COASTAL DEVELOPMENT

The attraction of water views and water-based leisure activities ensures that residential development is concentrated close to the edge of water bodies, such as rivers, harbours and beaches. Residences located at the water’s edge are highly valued. Other structures such as wharfs and export terminals are required to be built close to the water’s edge to facilitate ship movements. Groundwater conditions at these locations can appear to be relatively simple, with the primary variable being tidal. The following case studies highlight selected design aspects of groundwater levels associated with tidal conditions.

A development in Sydney experienced intermittent groundwater inflows that were associated with particularly high tide levels. In these instances the inflow rate increased by about 600% compared to the normal inflow rate, and resulted in some minor flooding. The groundwater in these instances was predominantly seawater that meant that the structure was exposed to more adverse durability conditions, requiring extensive repairs.

In investigating and sealing the source of the inflows around the building it was important to be able to predict the occurrence of the high tide levels associated with the inflows. Figure 8 below shows a comparison between tide levels published in tide charts against measured levels. Tidal predictions are regularly wrong by up to ±0.25 m, which was a significant error in this instance. This error is due to variations in atmospheric pressure, and to a lesser extent wind direction.

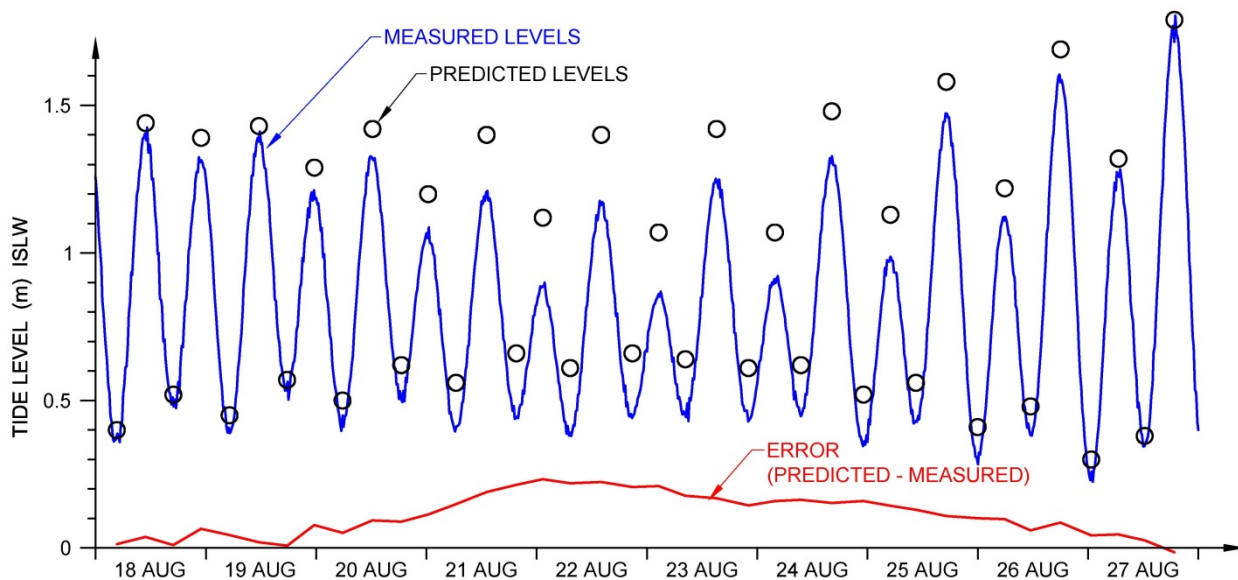


Figure 8 – Comparison of predicted and measured tide levels in Sydney Harbour

Figure 9 below shows measured groundwater levels on a site comprising reclaimed land in Port Curtis, Gladstone. Also shown is the tidal variation in the adjacent harbour. This monitoring was undertaken to assess groundwater conditions for a temporary excavation in the reclaimed ground in order to build a reclaim tunnel beneath a future coal stockpile.

The monitoring indicated that although the tidal range was ± 2 m, the damping due to groundwater flow through the dredged sand fill reduced the groundwater tidal range to only ± 0.25 m at a distance of 20 m from the shoreline, and negligible variation beyond about 75 m inland. This meant that the design groundwater level at the excavation was 2 m lower than the high tide level in the adjacent harbour.

The observed damping was also used to back-analyse the overall hydraulic conductivity of the dredged sand fill using a transient groundwater model, and permitted assessment of a reliable estimate of groundwater inflows into the temporary excavation.

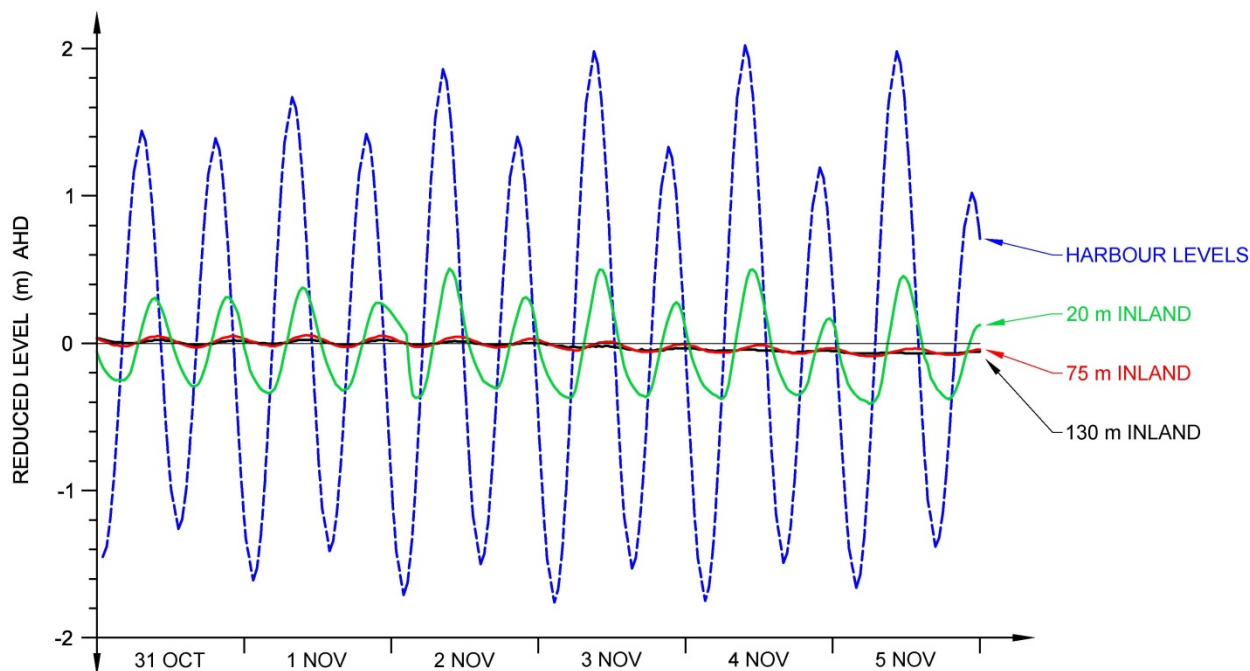


Figure 9 – Measured groundwater levels in reclaimed land, Port Curtis, Gladstone.

Design of a large wharf structure in Darwin required consideration of a maximum tidal range of 7.8 m. The structure comprised a 22 m high steel sheet pile wall, rock back fill, and tie-back anchors supporting the top of the wall. The structure was designed to be free-draining, so that there was no significant net water pressure acting on the wall. Drain holes were provided in the sheet piles to achieve this. The very high tidal range corresponds to a maximum rate of water level rise (and fall) of almost 2 m/hr, which meant that the required flow rates through the drain holes needed to be large to maintain the groundwater level inside the wharf at the same level as the harbour. The large water level variations in the backfill, and high flow rates through it, also meant that deterioration and flushing action was a potential cause of material loss and settlement.

4.4 DEEP TUNNEL

Groundwater pressures on a tunnel constructed at 60 m depth resulted in very high loads applied to the tunnel lining. Simple approaches based on adoption of partial factors derived from some of the design codes would have led to extremely conservative loads, corresponding to a water level well in excess of that possible in reality (i.e. more than 10 m above the ground surface).

A detailed design review of the tunnel design was undertaken, which initially involved review of the various national and international codes, including ISO2394:1998 *General Principles on Reliability for Structures*. Historical river level records over a period of 40 years were obtained, in addition to published flood maps. Figure 10 shows normal river levels and those corresponding to a 100 year ARI flood.

As noted in Table 1, several codes acknowledge that the design level may be represented by the maximum value possible over the design life of the structure. For this tunnel this is associated with flooding of the overlying river. As the tunnel portals are located within the flood plain, the tunnel would be inundated and filled with water when the flood defences were breached. Therefore for flood levels above the height of the flood defences the net water pressure on the flooded tunnel lining would be zero because the water pressure would be essentially the same on both sides of the liner.

A design phreatic surface level of RL 7 m was proposed. This level is:

- 5 m above the level of the flood defences around one of the portals (i.e. allows for future raising of the flood protection).
- Equivalent to a partial factor of 1.05 on the groundwater pressures applied to the tunnel invert from a 100 year ARI flood.
- 12 m above the measured phreatic surface at the end of construction.

A reliability analysis was also undertaken to check that the proposed level achieved the required degree of reliability as defined by ISO 2394 for structures where the consequences of failure is “great”. This analysis was possible because the structural adequacy of the tunnel lining was almost entirely dependent on two factors; the resistance afforded by the strength of the concrete used for the lining, and the loads from groundwater pressures. The statistical distribution of concrete strength was well established based on extensive quality control test records.

The groundwater loads were characterised using two approaches. The first involved adoption of the 100 year ARI flood level as the 95 percentile value. The second used a concept of maximum credible values, which was based on the level of the flood defences.

The reliability analysis concluded that the partial factor of 1.05 was acceptable.

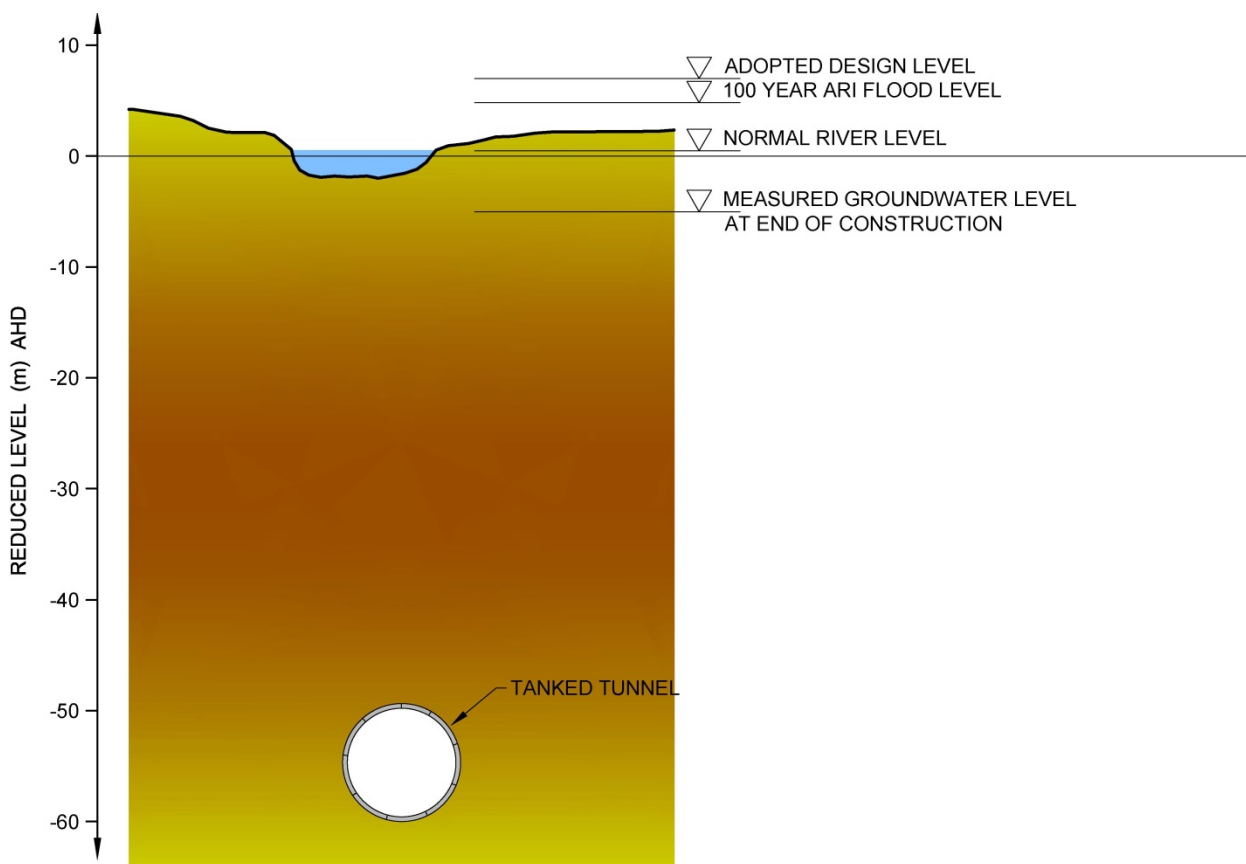


Figure 10 – Groundwater levels in comparison to the design level for a deep tunnel

5 RAINFALL ANALYSIS TOOLS

5.1 RAINFALL RESIDUAL MASS ANALYSIS

A key to understanding groundwater conditions at a site is to assess whether measurements were taken during periods of elevated or reduced rainfall. A simple tool for assessing this condition is a plot of the rainfall residual mass.

The residual mass analysis is undertaken as follows:

1. Determine the average daily rainfall for the entire rainfall record.
2. Assume that the daily loss in moisture due to evaporation and other means is equal to the average daily rainfall.

3. Calculate the rainfall residual mass cumulative water balance based on measured daily rainfall less the average daily moisture loss.

The result is a plot that highlights periods where rainfall exceeds moisture loss and vice versa. Residual mass analyses for Sydney Observatory, Sydney Airport and Randwick Racecourse are shown in Figure 11.

The residual mass plots in Figure 11 highlights the variation of rainfall patterns over periods comparable to design life. The rainfall record for Sydney Observatory is about 160 years and the residual mass plot shows the variation during this time is substantially greater than that captured by the groundwater level records. Major drying and wetting periods are evident such as the Federation and Millennium droughts.

The residual mass plots for Sydney Airport and Randwick Racecourse in Figure 11 are substantially shorter than for Sydney Observatory. However, the trends are similar for all three records.

The variation in residual mass highlights the difficulty in using site or even historical measurements of groundwater levels for predicting design life groundwater pressures, these being of the order of 50 years. It can be a useful tool for determining the relative variation at a site over the design life and where in the cycle the current groundwater measurements lie.

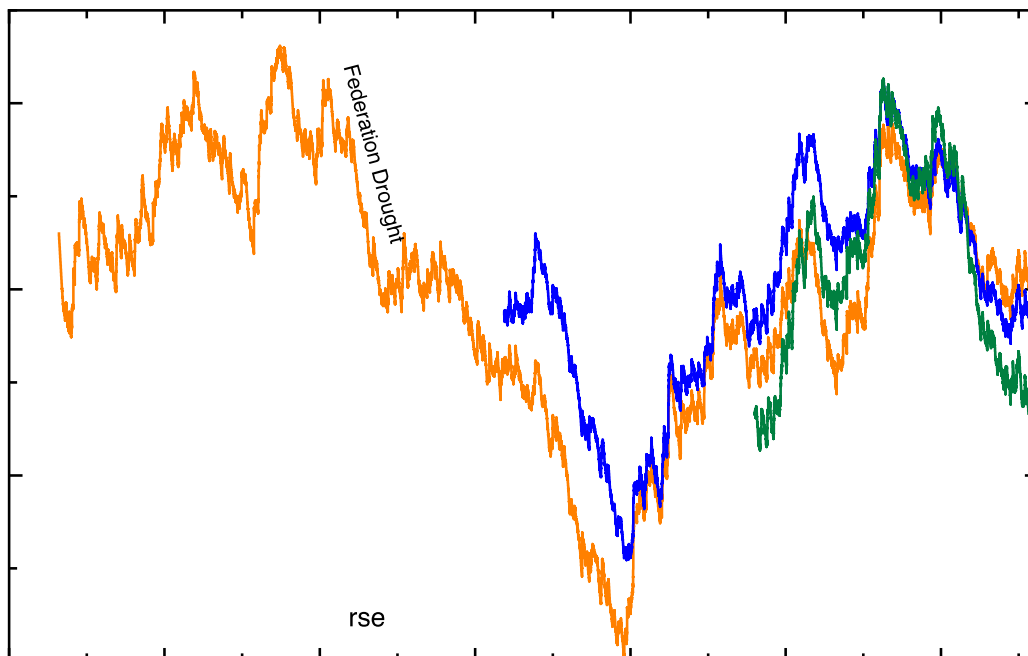


Figure 11 – Residual mass analysis for selected Sydney rainfall sites

5.2 RAINFALL EXCEEDANCE PROBABILITY CHART

Residual mass provides an overview of relative moisture condition but not necessarily the moisture condition of an individual year. This can be somewhat redressed by creating a plot of rainfall probability of exceedance (PoE). This is usually calculated on an annual basis using the following steps:

1. Calculate the annual rainfall totals for each year in the rainfall record
2. Sort and rank each year in ascending order
3. Calculate the PoE for each year based on the rank divided by the total number of years plus one.

The PoE plots for Sydney Observatory, Sydney Airport and Randwick Racecourse rainfall are shown in Figure 12. Although these are usually calculated on an annual basis the period over which the years are based can be modified to coincide with site date. This allows comparison of site measurement periods with PoE.

With this type of analysis the designer can:

- Assess the PoE of dataset periods for a range of water related datasets including rainfall, groundwater levels, river heights, flood levels etc.
- Extrapolate between datasets by scaling from the rainfall probability distribution

Extrapolation between datasets is illustrated by way of the following example, adopting the Sydney Observatory data in Figure 12 as the relevant rainfall record.

Assume that groundwater measurements for two different years are say 20 mRL and 22 mRL with annual rainfalls of 980 mm and 1450 mm respectively. From Figure 12 these rainfalls correspond to PoEs of 75% and 25% respectively.

Interpolation suggests that the average groundwater level (50% PoE) is 21 mRL with a corresponding average annual rainfall total of 1210 mm. Assuming a design life of 50 years the design rainfall (a 2% PoE) from Figure 12 is about 2050 mm.

The groundwater level at 2% PoE can be found from the 50% PoE condition (groundwater at 21 mRL and average rainfall 1210 mm for Sydney Observatory) and the 75% PoE condition (groundwater at 22 mRL and average rainfall 1450 mm for Sydney Observatory) by linear extrapolation to the design rainfall of 2050 mm. The resulting estimate is a groundwater level of 24.5 mRL.

This example is one of many ways that PoE charts can be used to estimate design water levels. In many cases estimates using this approach are somewhat conservative because it ignores other effects that occur during periods of high rainfall including increased runoff proportion and increased groundwater flow. However, there are many other factors that influence groundwater levels so this method is only a guide. As a minimum, it may inform the designer as to the severity of temporal variation at the site and justify the need for further analysis and monitoring.

For the commercial site case study described above the year over which site groundwater level measurements were taken corresponded to an annual exceedance probability of around 80%. Therefore these readings represented a relatively dry year and may not adequately capture average conditions let alone adverse conditions over a particular design life.

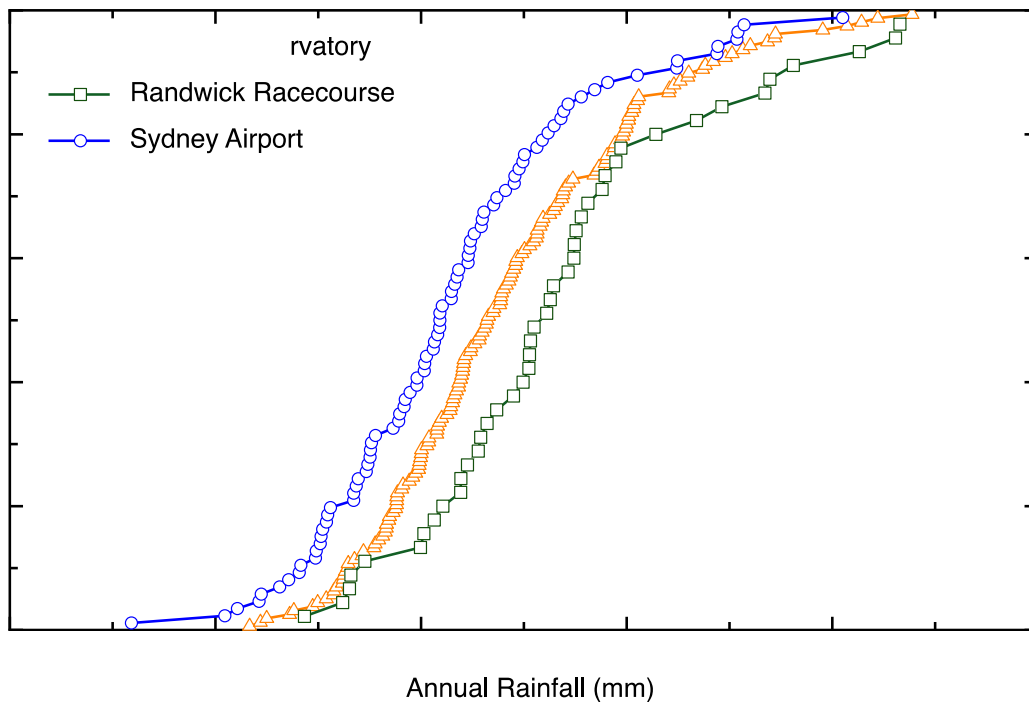


Figure 12 – Rainfall exceedance probability chart

6 CONTINGENCY MEASURES

6.1 LEAKAGE OF TANKED STRUCTURES

Tanked structures have two main characteristics:

- Watertight (i.e. no flow into the structure).
- Subject to hydrostatic loads (e.g. uplift and lateral pressures).

In addition to reduced maintenance costs, structures are sometimes tanked to:

- Limit impacts of adjacent groundwater levels.

- Improve internal amenity (i.e. dry conditions).

Review of notionally watertight structures suggests that all water-retaining structures leak to some degree. Even highly engineered machines such as submarines leak through the various penetrations through their hulls (i.e. seals, joints and gaskets), with water collected in the bilge. Where the magnitude of leakage is very low inflows may not be apparent, either because of evaporation or because the inflows remain unseen.

In the design of tanked structures an acceptable leakage rate needs to be defined, for example based on consideration of the above three reasons. Where there is a possibility of inflows occurring, appropriate internal drainage measures should be provided to ensure that even minor flows do not adversely affect the serviceability of the structure.

Impacts on durability of the structure also require consideration based on the quality of the water that could potentially enter the structure.

6.2 GROUNDWATER FUSES

In the groundwater context the term ‘fuse’ is used to describe a design feature that prevents a structure from becoming overloaded by excessive groundwater pressures, in the same way that the electrical safety device of the same name protects an electrical circuit from becoming overloaded.

Due to the inherent uncertainties in predicting groundwater levels, and the significant cost in designing for conservatively high levels (e.g. ground surface), it is sometimes prudent to include a drainage detail that prevents groundwater pressures from exceeding a predetermined limit. Details may be as simple as drainage outlets in the wall of a tanked basement, or relief wells with valves set at a particular pressure.

The purpose of such a fuse is to avoid excessively large water pressures from causing damage to the structure. The water permitted to enter the building (i.e. to relieve the pressures) may be collected and discharged in a controlled manner, or else allowed to enter the structure in an essentially uncontrolled manner, potentially causing it to flood. The latter option may result in substantial damage to plant, vehicles, and other stored items and should therefore only be contemplated where the risk is acceptable in comparison to the cost of providing a structure capable of resisting greater groundwater loads.

The level of the flood defences provided around the tunnel portal described in Section 4.4 above is another example of a fuse.

7 CONCLUSIONS

Knowledge of groundwater levels is an important component of the design of structures located within or below the groundwater table. Current codes and standards require consideration of adverse groundwater pressures over the entire design life, commonly 50 years or more. These pressures must be used in conjunction with a load factor that reflects the confidence at which such pressures were derived.

Currently there appears to be a disconnect between the derivation of groundwater pressures for design and their method of application. In many cases proper consideration of design life has not been undertaken and the confidence and applicability of the groundwater pressure estimate that has been provided is unknown.

While historic data provides the most relevant groundwater levels for design the record durations are typically too short from which to derive design values. The rainfall residual mass and probability of exceedance analyses can provide some insight as to the relevance of available data for design purposes and a measure of the variation of groundwater levels over the design life.

The uncertainty that surrounds the derivation of groundwater pressures highlights the need to:

- Apply multiple methods.
- Consider multiple sites where possible.
- Recognise that the length of record is sometimes of greater importance than the type of record.
- Consider the relevance of prescribed load factors compared to the means by which groundwater levels have been estimated. For example, a load factor of 1.0 may be suitable if the design water table has been taken as the surface.
- Assess the relevance or otherwise of data using tools like the residual mass or probability of exceedance value.
- Consider the use of contingency methods to address risk in design.

The authors would like to acknowledge the assistance of their colleagues and clients in the preparation of this paper. We would like to especially thank Dr Noel Merrick for his insights in the assessment of climate and groundwater data.

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