RESILIENCE EXAMPLES IN RESERVOIRS, PUMP STATIONS AND PIPELINES – LESSONS LEARNED FROM THE CANTERBURY EARTHQUAKES

Dennis Hunt (CH2M Beca Ltd) and Gavin Hutchison (CH2M Beca Ltd)

ABSTRACT

As a result of the Canterbury earthquakes in 2010/2011, key Christchurch City Council (CCC) water and wastewater infrastructure was severely damaged. This paper looks at important resilience lessons learned for three core components:

- Reservoirs
- Pump Stations
- Pipelines

The paper will look at examples of CCC infrastructure that proved to be resilient during the earthquakes, along with examples of failures and the lessons learned. Particular consideration is given to how these learning's can be incorporated into future designs. A key lesson is that resilience does not need to be an expensive exercise in every case; step changes in resilience can be achieved with appropriate design detailing. Understanding of geotechnical hazards and influence on seismic performance of infrastructure is important. A multi-discipline approach, in conjunction with the asset owner, assists in identifying critical vulnerabilities. This, in turn, allows the design to focus on an appropriate level of resilience, optimising project life cycle value.

KEYWORDS

water, wastewater, infrastructure, reservoirs, pump stations, pipelines, resilience, earthquake repairs

1 INTRODUCTION

CCC's network of reservoirs experienced damage which required remedial works ranging from strengthening to demolition and replacement. The impact of the damage on water supply operations and the associated risks drove key design and construction solutions. The criticality of detailing for a reservoir to reliably provide resilience was demonstrated by the extended time it took to restore reservoirs to service. In the reservoir context, the definition of resilience is a potable water storage facility that has structural integrity, and prevents loss and contamination of all stored water. This definition reflects Importance Level 4 performance (as per AS/NZS 1170.0:2002).

A number of key wastewater pump stations were damaged, requiring full rebuilds. However, these damaged pump stations were still operating three years after the earthquake, albeit with a reduced level of functionality and elevated operational costs, whilst replacements were built. The key considerations for resilient pump station design are discussed to provide the asset owner and designer with an understanding of resilient features employed in the light of the earthquake experience.

Key considerations of pipe design are durability and flexibility of pipe materials to limit structural damage induced by strong ground motion, as well as vulnerability of the system to differential settlement, inducing pipe dips, lateral stretch and translation of pipes. Alternative systems such as pressure sewer and vacuum sewer were adopted during the rebuild, along with traditional gravity sewer systems.

Selection of system type considered anticipated seismic performance of the ground and seismicity to optimise lifecycle value, taking into account of costs of effecting future post-disaster functionality and repair. It is important to design pipelines to the ground conditions and geotechnical hazards, not just to a set of generic civil design standards.

As a result of the Canterbury earthquakes one of the key drivers for the newly-formed Stronger Christchurch Infrastructure Rebuild Team (SCIRT) design team was to design new infrastructure that was resilient. This required designers to balance the resilience of assets in a seismic event against the cost of providing an appropriate level of seismic performance and functionality following an event. SCIRT considered resilience as being an improvement in the seismic performance of new infrastructure and post-disaster network functionality; to improve the speed and ease of the emergency response and recovery following future earthquakes. One lesson that has been applied is the need to undertake assessment of resilience during all stages of design; from feasibility or concept through to detailed design. At the early planning stages, improvement in resilience can be achieved for little or no cost. As the design process progresses, the opportunities reduce and the cost increases; to the point that at detailed design the ability to improve resilience is limited and can increase the cost of the project substantially.

2 RESERVOIRS

2.1 OVERVIEW

As a result of the 22nd February 2011 earthquake several reservoirs in Christchurch were significantly damaged. Apart from one notable exception (the 35ML Huntsbury No. 1 Reservoir) most of these reservoirs were able to be returned to service with temporary repairs within a few weeks. However the June 2011 earthquake damaged the temporary repairs rendering the reservoirs unusable. Permanent repairs had to be put into effect as soon as possible, preferably in advance of the peak summer use period.

2.2 RESILIENCE

Clearly the prime purpose of a reservoir is to reliably store water for the community it serves. The storage becomes critical in the times of emergency events such as fire or earthquake. Resilience of a water supply system relates to all elements including intakes, pump stations, water treatment plants, pipework and reservoirs, and the associated power supplies and control systems to remain functional post-earthquake.

Insofar as the reservoir element of a water supply is concerned, key factors influencing resilience highlighted by the Christchurch earthquakes were;

- Geotechnical conditions below and around the site.
- Pipework connections.
- Strength and ductility of structure connections, particularly wall to roof joints.
- Contamination of contents.
- Ability to isolate the reservoir.
- Monitoring instrumentation.
- The need to consider all the above factors in a co-ordinated manner.

Each of these aspects is considered below following Section 2.3 which summarises example reservoirs.

2.3 RESERVOIR SUMMARY

Table 1 identifies some of the reservoirs in Christchurch which were damaged, the structure type, the nature of the damage, reinstatement period and repairs required.

Table 1: Summary of earthquake related damage to a selection of Christchurch City Council Reservoirs

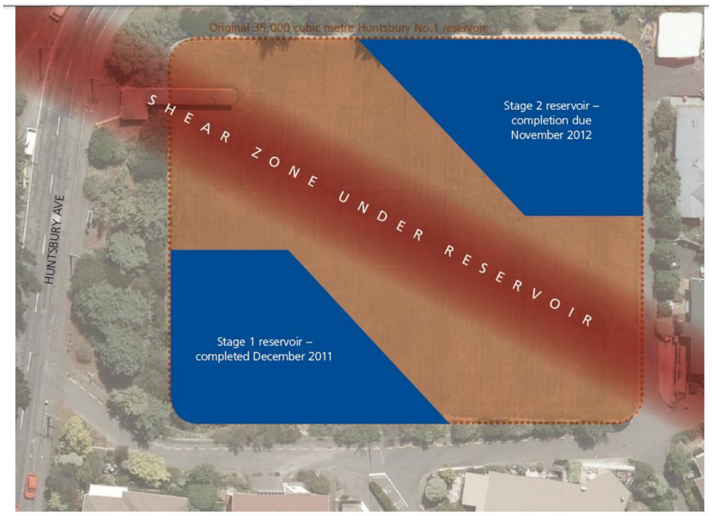
Reservoir/Structure Type	Damage	Period of Time to Repair	Repair Required
Huntsbury No. 1 *Capacity 35ML* *Reinforced Concrete*	▪ Geological shear zone under reservoir moved fracturing floor, walls and roof. ▪ Complete loss of water.	**5 months (Stage 1) 6ML** **17 months (Stage 2) 7ML**	**Partial demolition.** **Reinstatement in two physically separated sections each side of shear zone.** **New Pipework.**
Pump Station	▪ Differential settlement.	**5 months**	**Rebuild on new site.**
McCormacks Bay No. 1 & 2 *Capacity 5ML each* *Post Tensioned Walls*	▪ Roof/Wall connection failure. ▪ Wall/Floor connection failure. ▪ Roof topping cracking. ▪ Floor cracking ▪ All above lead to leakage.	**One reservoir remained operational.** **18 months to repair both reservoirs.**	**Connection upgrades.** **Membrane on roof.** **New floor slabs.** **Crack injection.** **Wall joint sealing.** **Rock fall protection.** **New retaining wall.**
Pump Station	▪ Cracking and spalling, groundwater ingress.	**2 months**	**General remedial**
Upper Balmoral *Capacity 1ML* *Post tensioned walls*	▪ Roof/wall connection and roof support column failure. ▪ Roof topping cracking.	**10 months**	**Connection upgrades.** **Membrane on roof.** **Reinstated central column.**

2.4 GEOTECHNICAL CONDITIONS BELOW AND AROUND THE SITE

A previously unidentified shear zone in the basalt under Huntsbury No 1 reservoir ruptured causing structure dislocation and loss of all stored water. It was estimated by the geologist who reviewed data from inclined boreholes done as part of a site investigation after the June 2011 earthquake that the last movement of this shear zone was in the order of 15,000 years ago.

The reinstatement of the reservoir was limited to sections each side of the shear zone as shown in photographs 1 and 2.

Photograph 1: Huntsbury No. 1 Reservoir Plan



Photograph 2: Huntsbury No. 1 Stage 2 Reservoir – Under Construction

The lesson in the Huntsbury case is the importance of obtaining experienced geological and geotechnical advice on overall site stability when planning a long term structure that has to retain water and be available for post – disaster operation (an Importance Level 4 structure as defined in AS/NZS 1170.0). A conventional subsoil investigation is unlikely to locate a shear zone such as encountered at Huntsbury, so at a minimum a desk - top assessment and a site visit of the area needs to be made by a geologist when planning the physical investigation. For an Importance Level 4 structure in a seismically active area with rock faulting it is the writer's view that inclined boreholes are warranted.

The McCormack's Bay reservoirs are located on a platform cut into a hillside. Rockfall and instability in the cut slope above the reservoirs presented a significant hazard which needed to be mitigated prior to external reservoir remediation works (Refer No. Photograph 3) and localised filled areas on the access road on the downhill side of the site required extensive retaining works and restricted access to the site.

Photograph 3: McCormack's Bay Reservoir Rock Fall Protection

The lessons from McCormack's Bay are the need to inspect and assess the geological conditions, and visualise the likely performance of cut and filled slopes during seismic events.

2.5 PIPEWORK CONNECTIONS

A flanged connection on the 600mm diameter concrete lined steel inlet/outlet pipe failed at the Huntsbury Reservoir. It was a flat plate flange with a single fillet weld to the outside of the pipe.

The lesson here is the importance of weld detailing in such joints. Use of a weld neck flange butt welded to the pipe would have prevented this failure. As a general comment use of restrained joints (i.e. tied or welded together) is preferred.

Readers are also referred to Section 2.8 where pipe/joint flexibility to cope with future movement of the ground is described.

2.6 STRENGTH OF STRUCTURE CONNECTIONS

The most common structure connection failure point was the roof to wall connection on circular reservoirs. At McCormack's Bay these joints consisted of a dowelled connection from the wall up into the roof topping. Several of these dowelled joints failed in shear. A perimeter ring beam was retrofitted to the roof, designed to permit expansion and contraction of the roof and to limit movement during seismic events by bearing onto the side of the wall.

A similar method was used to reinstate the Upper Balmoral reservoir roof/wall connection where the failure mode was roof tee beams punching through the wall (Refer Photograph No's 4, 5 and. 6).

Photograph 4: Upper Balmoral Reservoir Testing Anchorages for Ring Beam

Photograph 5: Upper Balmoral Reservoir Ring Beam Reinforcing

Photograph 6: Upper Balmoral Reservoir Ring Beam and Roof Membrane

In both cases the retrofit design was made difficult due to the lack of sufficiently strong components in the perimeter of the roof and strength limitations in the top of the walls. At Upper Balmoral these difficulties were overcome by core drilling holes into the ends of the radial prestressed tee roof units and epoxy grouting Reidbars into them thereby providing anchorage for the ring beam. However the wall section was not strong enough to resist the horizontal seismic load from the ring beam in bearing against the wall, requiring the transfer of the load to the pilasters and from there via shear to the base of the wall.

At both Upper Balmoral and Clifton the central column roof support failed at the base and had to be reinstated. This included use of carbon fibre reinforced fabric.

The base of wall connection at McCormack's moved causing spalling of concrete on the nib at the external base of the wall. Relatively minor leakage was experienced as a result.

Calculations using 2011 seismic design loadings indicated a lack of shear connection strength at Huntsbury, McCormacks and Upper Balmoral. In each case there had been floor slab damage. The repair method adopted consisted of new reinforced concrete floor slabs (Huntsbury and McCormacks) and a shear connection between the floor slabs and the wall. This connection consisted of Reidbars installed through cored holes in the wall and anchored into a beam above (and connected into) the floor slab within the reservoir (Refer Photograph No's. 7 and 8).

Photograph 7: McCormack's Bay Shear Connection Wall to Floor

Photograph 8: Shear Connection Wall to Floor Slab

The key lessons on connections are to resolve the seismic resistance philosophy for the structure at the outset, and to provide adequate strength and watertightness in the connections. The adequacy of these connections rely on detail and need to be considered at concept design stage to ensure the design of wall, roof and floor slab sections is compatible to accommodate them. The roof to wall detail has the added complexity of providing for temperature expansion and contraction.

2.7 CONTAMINATION OF CONTENTS

Damage at the roof to wall joint and cracking in thin concrete roof toppings increased the risk of contamination. Christchurch water is not normally chlorinated but given the extent of damage in the water supply system the decision was taken to chlorinate.

Due to the difficulty of repairing the extensive cracking in roof toppings, membranes were installed on Upper Balmoral (refer Photograph No. 6) and McCormack's Bay reservoirs. Membranes were also installed on the Huntsbury Stage 1 and 2 reservoirs even though the topping was 200mm thick. In the case of Huntsbury this allowed the roof slabs to be poured in large pours which was critical to achieving the programme. If programme had not been a consideration extensive jointing with PVC waterstops could have been contemplated with the advantage of reduced maintenance.

2.8 ABILITY TO ISOLATE

The loss of all the stored water at Huntsbury graphically demonstrated the need to be able to isolate a reservoir, and maintain integrity of the pipework. Each reservoir has an inlet/outlet pipe branch connected to a common inlet/outlet pipe, installed below the reservoir floor.

An electrically actuated isolation valve has been provided immediately outside the Huntsbury Stage 1 and Stage 2 reservoirs which are closed in the event of an earthquake (via a signal from a seismic sensor device). A manual isolation valve, and a flexible bellows are also provided, and thereafter butt welded PE pipe traverses the shear zone and connects to the common inlet/outlet pipe section.

The philosophy behind the valving arrangement is that it is essential to be able to isolate the reservoir, and that the reservoir pipe stub must connect integrally with the reservoir so that leakage via pipe joints or couplings is eliminated. The seismically actuated valve is provided so that stored water is retained in the event of an earthquake thereby preventing reservoir drainage through broken reticulation pipework. The bellows type coupling in association with the PE pipe provides flexibility between reservoir and pipework and minimises potential loads being transferred to the reservoir pipe stub.

A chamber constructed around the valve pipework spool cantilevered off the main reservoir structure to act structurally with the reservoir.

2.9 MONITORING INSTRUMENTATION

During the February 2011 earthquake, the level sensing device in the Huntsbury reservoir dislodged from its supporting bracket. No level monitoring was available as a result.

The lesson learned is that supports for instrumentation need to be designed to resist seismic loads so that records are available.

2.10 COORDINATED CONSIDERATION OF ALL FACTORS

The lessons learnt from the Christchurch earthquakes have highlighted particularly vulnerable reservoir features and demonstrate that the design of a reservoir required the co-ordinated consideration of several engineering disciplines with the operator's requirements.

Designers of reservoirs need to carefully scope geological as well as geotechnical investigations, and build in strength and resilience in structural and pipework connections. The detailing needs to incorporate these features together with the fundamental watertightness requirements.

3 WASTEWATER PUMP STATIONS

3.1 BACKGROUND

As a result of the earthquakes a number of wastewater pump stations were damaged. Even the most severely damaged pump stations were returned to service within days of the earthquakes. Of those damaged, only two pump stations required a full rebuild. These were, however, two of the main larger pump stations in the city both serving the east of the city. Table 2 below provides an overview of the performance of the large pump stations.

Table 2: Summary of Earthquake-Related Damage at Large CCC Wastewater Pump Stations

Pump Station	Damage	Period of time station remained in service before repair/rebuild	Repair Required
PS36 – Terminal Pump Station *PWWF ~700 l/s* *Constructed 1980* *6 m deep caisson wetwell with no ground improvements and asymmetric configuration*	▪ 350 mm differential settlement and approximately 300 mm total settlement ▪ Breaks in twin AC pressure mains outside structure ▪ Significant wear of impellors from silt in the wastewater ▪ Damaged pump motors from dry well flooding	**3 years**	**Complete Replacement**
PS63 *PWWF = 470 l/s* *Constructed 1983* *Archimedes screw lift station with 6m deep wetwell*	▪ Differential settlement of structure ▪ Archimedes screw continued to operate but with increased maintenance requirements due to bearing lubrication system displaced out of level.	**4 years**	**Complete Replacement**
PS11 – Terminal Pump Station *PWWF = 1440 l/s* *Constructed 2007*	▪ Modern piled design, no significant settlement damage. ▪ Shear failure of discharge pipework immediately outside structure.	**N/A**	**Repairs required to pipe connections**
PS15 – Terminal Pump Station *PWWF = ~500 l/s* *Constructed 1970* *9 m deep circular wetwell*	▪ Damage to inlet pipe with reverse grade. ▪ Settlement around the structure ▪ Shearing of cast iron discharge pipes at the connection to the structure and break of AC pressure main	**4 years**	**Substantial Repair** *Difficult Repair due to manifold design. Required full pump station shut down*
PS1 – Terminal Pump Station *PWWF = 2600* *Constructed 1955*	▪ Wetwell founded on dense sand which did not liquefy but part of superstructure on shallow liquefiable foundations resulted in 115 mm differential settlement.	**Repairs still not completed**	**Replacement of above ground building**

Pump Station	Damage	Period of time station remained in service before repair/rebuild	Repair Required
5 m deep wetwells			

The major issue was that the damaged pump stations now had increased vulnerability to damage. Although the stations were generally returned to service in a matter of days, some stations were now at increased risk from aftershocks until permanent repairs or replacement was undertaken.

3.2 WASTEWATER PUMP STATION PERFORMANCE

Generally the pump stations performed well considering the significance of the events. A number of stations received minor damage allowing the stations to be returned to service in a short time frame. Damage at the pump stations can be broken into the following categories:

- Settlement/flotation;
- Differential movement;
- Increased maintenance (due to increased silt loadings).

Structural damage to the pump stations was generally a result of settlement, with some damage to above ground structures. There were some water pump stations buildings that suffered substantial structural damage requiring a full rebuild. Photograph 9 below shows typical damage to wastewater pump stations from top left, small pump station next to the Avon River, Pump Station 36, Pump Station 15 and pressure main connection PS15.

Photograph 9: Pump Station Earthquake Damage

The most typical and troublesome were failures at the pipe connections to the structures. These failures were caused through ground settlement which also reversed the grade of some gravity inlet pipes. Repairs to the discharge pipes were often impeded due to the depth of the pipe requiring significant temporary works and dewatering.

Although damaged and some requiring a rebuild these stations were returned to service is a short time (days) and have remained in service for a significant amount of time before repairs or rebuilds were complete. Table 2 above shows the duration some of the large pump stations remained in service until full repair or rebuild was undertaken. It is worth noting the performance of PS11 built in 2007 the damage was confined to the outlet pressure main connection.

The main issue the pump stations faced (damaged or not) following the earthquakes was the damage within the upstream network that was allowing inflows of massive volumes of silt and groundwater as a result of liquefaction. Consequently wet wells became inundated with silt and sand and significantly increased maintenance costs at some pump stations from frequent cleaning to remove large volumes of soids. The silt caused excessive pump wear, which reduced pump performance in a network with increased flows due to the damaged network.

Added to the direct impact on the pump stations, the large inflows insufficiently resilient network can severely impact the effective capacity of a pump station. The station may have been designed to Importance Level 3, however if the network has a lower level of resilience, this has a direct impact on the effective performance of the station post-earthquake. When considering resilience we need to consider the system within which the asset functions as a whole.

This does not necessarily mean designing and building a system that won't ever fail, as this often is not a practical option, nor does resilience need to be expensive. The following sections discuss an approach to the design of new pump stations and associated infrastructure. The fact some large pump stations remained in operation for a significant period of time could be viewed as a success in resilience of the infrastructure.

3.3 PUMP STATION RESILIENCE

Learning's taken from the earthquake rebuild and applied to detailed design is discussed below. As discussed the greatest gains in resilience can be achieved prior to the detailed design phase. The feasibility and concept design stages also present the greatest opportunities to provide resilience, and result in significant savings. At detailed design the designer options become limited. There are a number of basic lessons that should be applied to all designs.

Selection of foundations is one of the most important design decisions in the design of a pump station. This will most often have a greater impact on the resilience and the cost of the project/asset. Geotechnical design recommendations for each site are undertaken following analysis of field data from investigations. The analysis assesses the risks and hazards associated with the site. From this a foundation design is selected in weak or liquefiable soils typically in Christchurch these have been:

- Stone columns or high modulus ground improvement, such as deep soil mixing and continuous flight auger (CFA) piles for ground improvement;
- Piles – screw piles or similar;
- Localised excavation and replacement with hardfill.

The new pump stations foundations in Christchurch have typically consisted of localised excavation and replacement with hardfill; a well-compacted relatively permeable granular backfill material will minimise uplift pressures on the underside of the base slab by dissipating pore water pressure. A hardfill block around the pump station improves stability, reducing differential settlement across the pump station and pipe connection. In the event of an earthquake, the pump station will settle if the surrounding ground settles, but differential settlement is less severe. An important point here is the approach is to manage the settlement, not fully mitigate. It is important that the pump stations are symmetrical to minimise the risk of differential settlement (tilting) in an earthquake. Pump Station 63 failure and need to be rebuilt can be partly attributed to the fact the structure was asymmetrical.

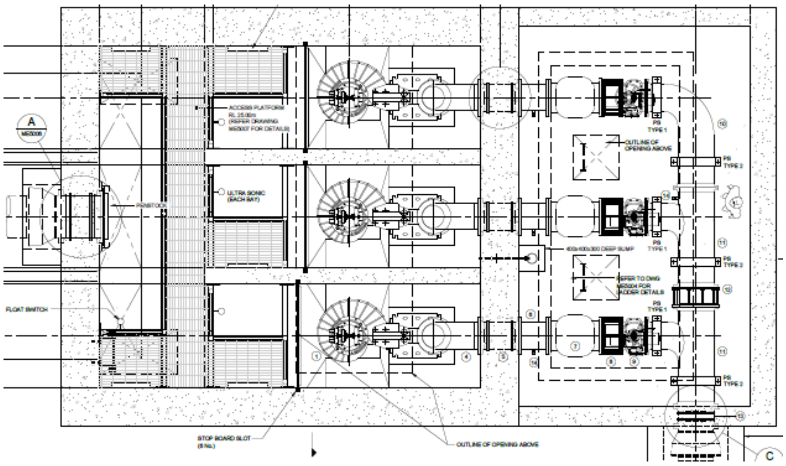
In the case of the larger pump stations piling and CFA's were selected in separate cases. These are both options providing improved resilience for the more critical stations. Final selection is based on existing ground conditions and cost.

Flexible connections at the interface with structures need to be designed to account for the potential settlement that can be expected. This is largely driven by the geotechnical considerations; one commonality between all options for connecting pipes to structures is flexibility. Options include a simple low cost PE pipe connection to a purpose-built fitting such as in Photograph 10 in areas of predicted high settlement. These flexible connections are used extensively on the West Coast of the United States in Earthquake prone areas.

Photograph 10: EBAA Flex-tend Fitting (PS105)

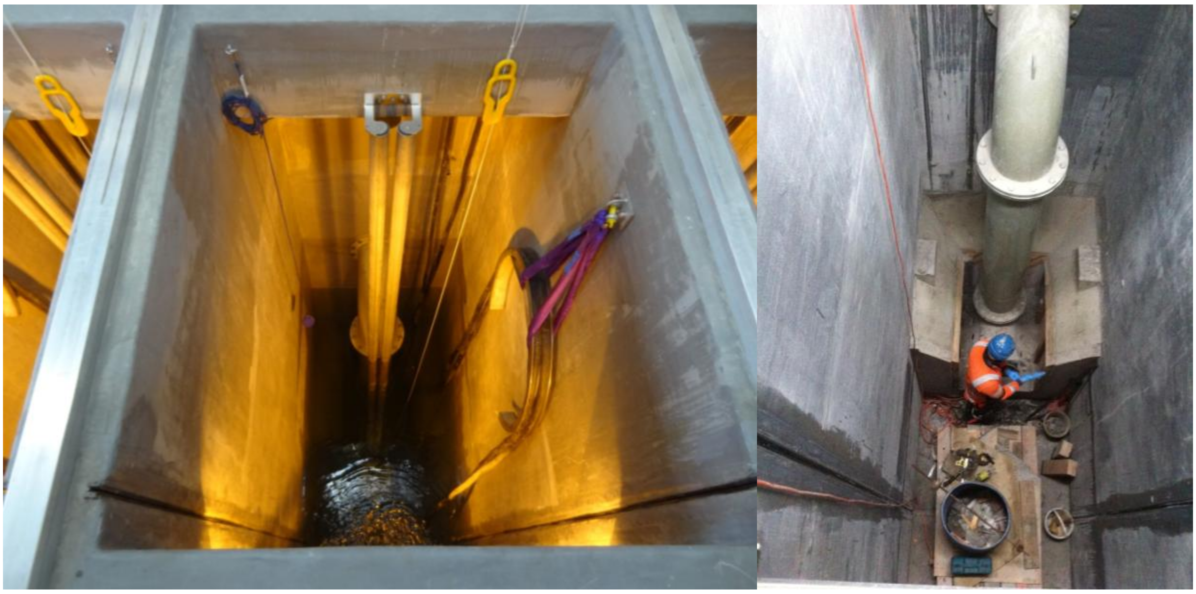
To minimise the risk of pipe failures the large stations (PS105 and PS128) had the discharge manifolds designed within the main structure. This provided a single entry and exit point at the interface of the structure with flexible connection (Refer Figure 1). The external flange was protected to reduce increased loadings during a seismic event by minimising the length of exposed flange and a protective reinforced concrete shroud.

Figure 1: Pump Station Manifold Design within Structure



At the large (critical) pump stations (PS136, PS105 and PS128) pumps were separated into separate pump bays or dual wetwells that can be isolated. This permits maintenance or cleaning to be undertaken within individual pump bays without limiting the station capacity. Access platforms were fitted to facilitate cleaning and inspection of inside the wetwell. Pump bays were hydraulically designed to minimise settlement of silt and build-up of solids. These were all low cost additions but greatly added to the resilience of the stations. Photograph 11 below shows a pump bay under commissioning and construction at PS105

Photograph 11: Single Pump Bay 450 l/s Capacity



4 PIPES

4.1 OVERVIEW

The impact of the earthquake events on the Christchurch pipe network varied across the network. As expected it was heavily influenced by the local ground conditions. However pipe material, design and construction also had an influence. The subject of pipe performance in the earthquake covering all of these parameters and how they can have an influence on the pipe performance is complex. This section provides a high level observations and discussion on the design approach, particularly for large pipe design.

4.2 PIPE PERFORMANCE OBSERVATIONS

4.2.1 GRAVITY

In terms of impact on horizontal infrastructure, the gravity pipe network was impacted the most resulting in loss of service to large numbers of customers, significant overflows to the environment and increased network operational costs. The existing network consisted of various pipe materials. The large trunk mains were reinforced concrete rubber ring jointed (up to 1500mm), some brick barrelled, some uPVC (450mm) and box concrete culvert channels. The main failures of the trunk mains were due to failure of connections and settlement, with pipes in Christchurch laid at shallow grades resulting in pipe dips. Generally the trunk mains faired reasonably well, with only minor joint faults. The Western Trunk main being a 1300 RCRRJ pipe with a PVC liner installed by open trench method in 2009 showed no significant damage. The following are two examples of the more significant failures of trunk mains in the city.

The Northern Relief trunk main suffered significant damage due to liquefiable ground conditions next to the Avon River which experienced large scale lateral spreading. It is acknowledge that there is no practicable design that would have fully mitigated the ground conditions and ground movement experienced. This is an example of the importance that the selection of pipe route/location during the feasibility stage. At the detailed design stages the designers would have had limited means to improve the resilience of the pipe.

The Trunk sewer immediately upstream of PS1 was a 1500mm x 1500mm box culvert. This failed in the 13th June 2011 earthquake resulting in the collapse of Woodham Road (Refer Photograph 12). Prior to the collapse assessment at the pump station had included the survey of the upstream trunk sewer, this indicated a severe dip in the trunk main. On review of the as-built drawings (circa. 1957) the dip had existed since construction and survey indicated the dip had increased by approximately 400mm. It is assumed that all or the majority of the increase occurred in the February earthquake. This highlights the importance of the construction phase, and continued assessment over the lifetime of an asset. In this case a fault in construction had occurred, the trunk main functioned but the resilience had been compromised.

Photograph 12: Woodham Road Trunk Main Failure

4.2.2 REFLECTIONS ON PIPE DESIGN

The designs of large infrastructure, such as pump stations or treatments plants, have design criteria set including importance level as defined in AS/NZS 1170.0:2002. Pump stations are generally designed to IL3, however design of pipe infrastructure has not traditionally had an equivalent. The asset owner and designers need to answer fundamental question about the asset and degree of resilience. An important factor will always be cost, however resilience does not need to come with a high price tag. The approach needs to be pragmatic and provide value for money. You need to sell it! This section discusses some key points in the design of a pipeline following the experience of the earthquake;

- Pipe criticality (upstream and downstream impacts);
- Ground conditions and geotechnical hazards;
- Concept design – Options assessment;
- Pipe materials;
- Appropriate design standards (such as AS/NZS 2566);
- Detailed design;
- Construction;

Pipe Criticality of the pipe should drive the level of resilience that the design and asset owner will consider. This should be undertaken in the scoping phase and determines the minimum seismic performance and post-earthquake functionality. The designer needs to understand the acceptable levels of damage and the importance of the infrastructure element within the wider network.

The designer needs to clearly understand the asset owner’s objectives and the final level of resilience agreed between both parties. The asset owner needs to clearly understand the risks and costs associated with the any design options.

Ground conditions and geotechnical hazards must be understood by the design team. All pipe design teams should have a geotechnical engineer engaged at the beginning of a project. An assessment of overall stability and ground conditions along the proposed alignment will allow the design engineers and asset owner to understand the risks and select the appropriate design option. This is critical information to feed into the concept and detailed designs, and will drive many key design decisions.

Feasibility and Concept design stages offer the greatest opportunity to improve the resilience of an asset and/or system. Once the importance of the pipe has been agreed the designer should optioneer solutions, these need to consider the above steps and overall resilience of the assets and network, post-earthquake functionality and impacts of being out of service and total life cycle costs to optimize the solution.

Pipe design methodology needs to be appropriately selected. When designing for large pipe infrastructure using flexible pipe such as PE or PVC designers normally use AS/NZS 2566.1.1998. This is an accepted standard in the industry for the structural design of buried flexible pipelines. However the designer should understand the background assumptions and calculations in AS/NZS 2566. A common error is designers adopting geotechnical modulus of elasticity which are higher than the recommended moduli provided in AS/NZS 2566 Table 3.2. This could lead to non-conservative design, as the semi empirical equations were developed with the range of specified moduli, not the specific geotechnical properties of the soil. For large pipelines this can lead to a conservative design. In poor ground conditions prone to liquefaction the AS/NZS 2566 does not model the increase in lateral pressures that the pipe will experience. The use of more advance tools design can provide a more 'engineered' design. An example of this is finite element analysis of the pipe under earthquake and non-earthquake scenarios. For large pipelines this could lead to project savings and/or a more robust design. The key message is designers should not take AS/NZS 2566 as a 'cook book' for design; they need to understand the document and the limitations. Consideration should be given to alternative methods such as finite element analysis, particularly on large pipe line design. This can provide optimised design solutions and a better understanding of pipe performance under various conditions.

Pipe materials have been a point of discussion following the earthquakes. Polyethylene pipe (PE100) has been the preferred choice of material for pressure mains as it performed very well in seismic events. However quality control of the material, wields and installation is crucial to provide a resilient asset. Glass reinforced pipe (GRP) pipe has also been used and can be a cost competitive alternative to PE.

Gravity pipe materials such as PVC and RCRRJ performed reasonably well, except in very poor ground conditions. The main failure mechanism was failures of connections, joint displacement and pipe dips. The failure the joints introduced significant amounts of silt and groundwater into the network which impacted the level of service, pump stations and operating costs. In poor ground conditions protection against joint failure should be considered. Fully restrained pipe can be considered, but the selection of system in the concept design which will set the basis for which the pipe materials will be selected. A point to note is that in some ground conditions the pipe material will improve the resilience but is not the only answer.

Detailed design phase should focus on identifying potential failure mechanisms of the asset. These should be ranked on consequence and prioritised. Inputs from the previous stages will guide the designer to the level of resilience; which may mean a design to avoid failure or, alternatively design a failure point that can be readily repaired at low cost. These are sometimes referred to as adding a fuse point, adding a weak point in the asset designed to fail first. As an example, in a zone of lateral spread next to the Avon River, large stormwater pipe outfalls were installed that were fully restrained except at the connections to the manhole structures. These were coupled joints wrapped in a geogrid wrap. In the event of lateral spread this fuse point would effectively fail first, avoiding loads being transferred into the structure which would result in failure of a larger asset. Alternately a more substantial connection would need to be designed at a higher cost. The coupling and geogrid will stop fines entering the pipe, allow the pipe to convey and is in a location selected by the designer and asset owner. This is a good example of resilience that is a low cost engineering solution.

Construction of the design needs to ensure that the design intent is achieved. Monitoring should be undertaken during and post construction to check it complies with the design and specification, but also to verify design assumptions (i.e ground conditions). The PS1 trunk failure is a good example of post construction monitoring, although we don't know the reasons for the settlement, it is highly likely the foundation was inadequate resulting in an asset with a reduced resilience.

5 CONCLUSIONS

This paper has been based on observations immediately following the 2010 -2011 earthquakes and during the recovery and rebuild phases.

The damage sustained by some of the city's reservoirs meant they needed to be taken out of service to facilitate reinstatement or repairs. Lack of strength and resilience of connections was the key reason for failure of the smaller reservoirs, whereas the 35ML Huntsbury Reservoir (1953 vintage) was severely damaged due to geological conditions.

There was widespread damage to the city's wastewater pump stations. Nonetheless even the worst affected were returned to service in a short time, operating for a number of years before being fully repaired or replaced. This indicates there was a good level of resilience in these assets. There were a number of key learnings taken from observations of the key failings in the field and applied to the new pump station rebuilds.

Resilience design needs to begin at the feasibility and concept stages. This is where the best gains in resilience can be made for the least investment. Resilience does not need to come with a high price. Asset owners and designers need to understand that failure of an asset is not a failure in resilience. It is sometimes a more pragmatic approach to not design all features of an asset to be earthquake 'proof'. Inclusion of yield points that are located in easily accessible points and easily repairable is an acceptable resilient feature. Overall this can ultimately provide a network with greater resilience.

The fundamental key lesson for reservoir and pump station design is to select team members with the necessary experience in their discipline, including a design manager and operator's representative with a co-ordinated understanding of all design and operational aspects.

The designers of pipelines need to understand the design standards they use, as well as their limitations. Alternative options for design that can provide a better engineered solution should be considered.

ACKNOWLEDGEMENTS

The inputs of Christchurch City Council staff, Designers and Contractors involved in reservoir and pump station reinstatement and repair after the 2011 earthquakes.

REFERENCES

Hunt, D., Clifton, R,. and Christison, M,. "Huntsbury Reservoir – Post Earthquake Reconstruction", (2013) Ozwater, Perth, Australia

Gibson M.F.L., Newby G 'Design philosophy for improving horizontal infrastructure seismic resilience' (2015) 6th International Conference on Earthquake Geotechnical Engineering, (in print)

Kerr M., 'Resilient Pump Station Design' (2013), NZ Water Conference 2013