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BRIDGE OF REMEMBRANCE SEISMIC STRENGTHENING

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SUMMARY

This paper highlights some of the design and construction challenges that were overcome in the seismic strengthening of the Triumphal Arch in Christchurch, a listed category 1 Historic Place. The project involved a complete change to the dynamics of the structure to allow a rocking motion to be induced to dissipate energy during a significant earthquake. The delivery team used a number of simple but effective techniques to overcome the challenges.

INTRODUCTION

As a result of the 2010 and 2011 Christchurch Earthquake sequence, the Triumphal Arch experienced significant structural and non-structure (stone fabric) damage. Built in 1924 as a memorial for soldiers who fought for New Zealand, the Triumphal Arch stands on the east abutment of the Bridge of Remembrance crossing the Avon River. The resulting damage included; differential settlement across the four columns, horizontal displacement of the top half of the structure above the minor arches, cracking and spalling of stonework in a number of locations and damage to the ornate Lions. Structural assessments following the earthquakes deemed the structure to be earthquake prone, meaning strengthening work was required to meet the requirements of the Building Act 2004 and Council Policy of Earthquake Prone Structures and to once again provide safe public passage under the structure.

Figure 1. Location of Triumphal Arch and east elevation of the structure after the earthquake

STRUCTURAL DESCRIPTION

The Triumphal Arch is an integral structure with the Bridge of Remembrance and is a lightly reinforced concrete structure clad with Tasmanian Sand Stone. It was most likely constructed by laying the stone faces and pouring the concrete inside in lifts with the columns and arch elements remaining hollow. It was presumed that the voids were created in the construction of the arch to reduce the volume of concrete and weight of the structure, which was approximately 350 tonnes prior to strengthening. The major arch stands at 14m above ground level with both of the minor arches at 5m above ground level. The width of the structure tapers in from 2.3m at the base to 1.4m at the top of the major arch. The foundations vary across the base of the four columns, where the northern minor column is built into the side of the bridge. The remaining three column foundations are independent of each other with the two southern foundations being shallower. The foundations of the columns penetrate into the ground in the same hexagonal shaped profile of the columns above ground.

DESIGN PHILOSOPHY

The objective of the strengthening works was to increase the resilience of the earthquake prone arch structure to meet requirements of the Building Act 2004 and Council Policy, while at the same time minimising the impact on the heritage fabric of the structure. An importance level of 3 was determined for this structure as defined by NZS 1170.0 due to its significant heritage value and the likelihood of crowds. The designers adopted a design life exceeding 100 years with a 1/2500-year seismic capacity, as the structure is to remain indefinitely through its significance to both people of Christchurch and New Zealand. The strengthening works carried out included the following key elements;

- Installation of deep founded Micro-Piles,
- Construction of two new reinforced concrete pile caps under the structure,
- Construction of reinforced concrete rocking collars at the base of each column,
- Installation of fabricated steel boxes inside the existing column voids,
- Filling the internal voids within the existing columns and arches with concrete,
- Positioning vertical sliding joint assemblies at the centre of the three arch crowns,
- Horizontal Post Tensioning across the three arches,
- Repairs to damaged stonework and the ornate Lions.

The philosophy of incorporating the above elements into the existing structure is to create a structure that rocks in the in-plane direction during an earthquake. Utilising the rocking mechanism aims to not only prevent structural collapse, but also limit significant structural and non-structural damage to the heritage structure during future earthquakes.

DESIGN MODELLING

The strengthening work presented many challenges for the delivery team due to; a technically demanding design, working in the tight confined of the structure and discovering frequent discrepancies in the original as-built drawings. Major design and constructability issues were overcome with the aid of 3D Modeling and 3D Point Cloud Scanning of the structure. The use of these two tools was extremely beneficial in giving the delivery team a clear understanding of what they were constructing in a virtual space and assisting with creating and validating construction methodologies. Construction sequences were generated to convey work methods to operatives onsite and the setting out of multiple elements was aided with the use of these two tools.

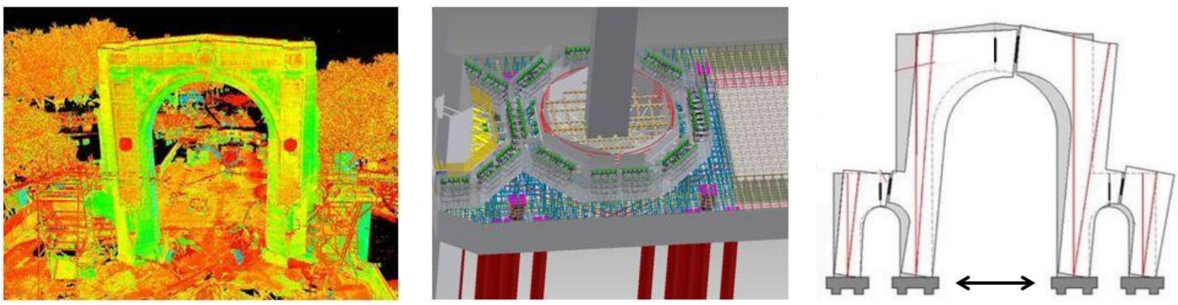

Figure 2. (L-R) External 3D Scan of Structure, 3D Modeling of Rocking Collar and Pile Cap, Example of the in-plane rocking motion induced by ground motion

One of the challenges faced was ensuring that the new steel reinforcing elements would be able to fit into the structure’s internal voids. Utilizing the 3D Point Cloud Scanning enabled precise profiles of both the inside and outside surfaces of the structure to be generated. This eliminated the heath and safety risk of lowering operatives inside the voids to confirm the internal dimensions. Following the scanning, a 3D-Model was produce of the existing structure using the scanned data and the new design elements were positioned in the model. This gave the delivery team the opportunity to verify the design before discovering discrepancies and clashes onsite that would have caused costly delays to the program.

STRUCTURAL MONITORING

Before civil works commenced on the structure a monitoring system was set up to identify any slight movement of the structure and to ensure the safety of operatives working on the site. The system incorporated; computerised survey checks, visual monitoring of existing cracks and identification of newly formed cracks using base line records.

The survey system involved installing monitoring targets on the structure, back-sight targets on existing buildings in the vicinity of the structure and positioning a permanent plinth for a survey total station to be mounted to. The structure was monitored multiple times daily from the plinth and any movement in the structure was able to be determined immediately. Before mobilising onsite a thorough dilapidation photo survey of the whole structure was undertaken. This gave the delivery team the ability to compare suspected new cracks with the state of the structure before construction. To complement the photo record, a number of the large existing cracks on the structure had Tell-Tale crack monitors fixed across them. This was a low cost solution to identify any movement in the existing cracks. Vibration monitoring was also used during the piling operation to ensure the structure was not subjected to damaging vibrations. The German Standard Din4150-3:1991 was adopted for historic buildings and a peak particle velocity of 3mm/s was set in accordance with the Standard. Continuous monitoring was set up to feed alerts to a siren and light on site when vibrations 80% and 100% of the limit were transmitted to the structure.

Figure 3. (L-R) Survey prism, Crack Tell-Tale visual monitor, Vibration monitoring sensor

TEMPORARY WORKS

To enable a large portion of the strengthening works to be undertaken, a number of temporary works items had to be designed, manufactured and installed. Not only were these temporary works needed to carry out the construction work, but they were also required to ensure the safety of the operatives working on and around the structure.

As a result of the damage sustained, the likely action of the main Arch under a seismic event was rocking about the existing crack line at the level of the Lions, midway up the major columns, but it was unlikely to topple as a result. However, uncertainties existed around the integrity of this hinge point, specifically the possibility of degradation of the rocking surfaces under prolonged seismic shaking. This issue was addressed by installing steel circumferential reinforcement clamps above and below the cracks on the major columns to reinforce the hinge point and provide residual capacity to the crack line. The clamps were designed to fit the profile of the columns, then lifted into place and tensioned to provide the required circumferential confinement.

Figure 4. (L-R) Steel Column Clamp, Temporary Access Hatch in side of Column, Scaffolding

A large majority of the enabling works required the localized removal of existing structural concrete. Before any removal could take place, an engineering assessment was required to assess the structure's structural strength following the reduction in cross-sectional area. Where it was deemed that the cross-sectional loss was too much, additional temporary works were installed or staged demolition/rebuild methodologies were implemented. A staged core and grout method was adopted for the multiple large diameter core holes through the base of the columns required for the rocking collar construction. Additional temporary works were required when access holes were cut into the base of the columns to allow operatives access to complete critical work inside the columns. Following the removal of concrete, temporary steel frames with removal cover plates were installed at the base of each column to maintain the global strength of the structure. Another structural strength issue was encountered when removing the concrete baffle walls inside the three arch voids. To maintain the structural integrity of the arches during the baffle wall removal, steel tie plates had to be secured across the top of the voids walls. The steel tie plates were also designed to strengthen the existing concrete walls, acting as formwork, when pouring the new concrete.

Scaffolding was another item that needed careful consideration to enable safe access to the structure over its full height. The delivery team had identified that the top half of the structure had the potential to rock during a moderate earthquake and therefore additional clearance between the structure and scaffold had to be allowed to ensure the structure didn't come into contact with the scaffold. This resulted in harness and lanyard only areas being setup where gaps between the scaffold and structure presented a risk to the workforce. The delivery team saw an opportunity to make a programme saving by erecting the scaffold to the structure whilst work continued on the pile cap construction below. This called for a special lightweight scaffold to be designed with its footings outboard of the pile cap construction area. The design utilized precast concrete blocks to form the footings, with scaffold column towers built off them. To bridge the large spans across the pile cap, scaffold trusses and steel I-beams were incorporated with a standard tube and clip system. An independent lift tower was also constructed to assist with moving materials and equipment up and down the scaffold.

Access to the north end of the structure was extremely difficult due to the proximity to the river. To enable construction work to be undertaken in this area, a temporary access track was installed down along the riverbank. Pre-cast concrete blocks were placed in the river to form a temporary retaining wall, bidim cloth placed over the bank and aggregate with geo-grid was used as infill to construct the track. A temporary dam was also setup around the track to enable the area to be dewatered. Water was pumped out of the dam through a sediment tank before being discharged into the river via the riverbank.

Less significant, but still important designs included; designing lifting eyes on all steel sections, positioning holes in the fabricated steel sections to allow concrete to flow in and out and calculating the allowable heights of concrete pours in the existing columns with considerations made for hydrostatic pressures.

Figure 5. (L-R) Inside of Major Column, Access track and Dam, Site setup during Piling

CONFINED SPACES

In order to carry out aspects of the strengthening work, operatives had to enter the tight confines inside the arches and columns of the structure. Due to the restricted working space in these areas, a comprehensive confined spaces entry procedure was established to control this work. A rescue davit hoist was secured to the top of the columns which provided a secondary means of escape should the entry hatch have ever got blocked or the operative incapacitated. The confined spaces work inside the structure included; reducing the floor level of the columns by 1m using hydro-demolition to accommodate the steel column boxes, setting out the base plates to receive the steel boxes, removal of concrete ledges to allow the steel boxes to fit and a number of concrete removal and coring activities inside the arches.

ENVIRONMENTAL CONTROLS

Working so close to an active waterway, special attention was given to implementing robust environmental controls. It was paramount for the success of the project that no environmentally damaging discharges entered the waterways. Dewatering activities were closely monitored with constant checking of the suspended particles and pH levels of the water. All stockpiles on site were covered to reduce sediment run off and a floating boom was placed in the river around the worksite to capture any sediment discharges. The slurry from concrete coring operations was pumped into holding tanks onsite before being removed from site for disposal. Bunds were setup around all piling plant with plastic sheeting underneath to contain any spillages of bentonite mud. Emergency spill kits were onsite at all times ready to be utilised by trained operatives in the event of a spill, including CO_2 to alter water pH levels.

PERMANENT CONSTRUCTION WORKS

The strengthening works involved a number of different construction operations. In terms of size the project as a whole was relatively small, but what made it challenging for the delivery team was the complexity of the design and tolerances that had to be met. Not only was the structure close to 100 years old, but also the historic significance meant that additional work was required to prevent further damage.

Micro-Piling

To overcome the sandy soils beneath the structure and the potential consequences associated with liquefaction, new piles were installed to replace the existing relatively shallow pad foundations. The initial design called for large diameter reinforced concrete bored piles which the delivery team had concerns about for a number of reasons. These included; the close proximity of the piles to the structure, the potentially damaging vibrations and heavy weight associated with large piling equipment and the limited space on site. The solution was to replace the single large diameter piles with smaller micro-pile clusters. Micro-Piling was a suitable alternative using a smaller rig providing improved maneuverability and a low vibration drilling method. In total, 32 No. 300mmØ micro-piles were drilled to an approximate depth of 26m, with an additional test pile being installed onsite to verify the design capacities. A 15 tonne Casagrande Crawler Drill Rig using a tri-cone drilling head and bentonite drilling mud was used to wash bore the piles. The piles were constructed with steel casings through the top 18m and a 50mmØ high tensile threaded bar was lowered in sections down the drilled hole before grout was pumped in using a tremi line. A large amount of equipment was required for the drilling operation, including the addition of a 50 tonne Crawler Crane used to move materials around the site and feed casings. This made the small site extremely tight and resulted in a number of exclusion zones being set up to manage the safety of operatives.

The finished level for the piles was 1.5m below the existing ground level to tie into the new pile cap. Again due to the nature of the site, the working level could not be lowered and the piles had to be constructed at existing ground level to assist with plant access. Therefore consideration had to be made to extend the pile to existing ground level and once the area was excavated for the pile cap, the tops of the piles were cut off to the required level. The finished piles were capped with a steel top plate secured to the threaded rod and shear studs welded to the steel casing. A number of protection measures were put in place to ensure that the stonework was not damaged during piling. To protect the stonework from stains as a result of hydraulic leaks and bentonite splashes, the structure was wrapped with plastic sheeting and continually washed down. The bases of the columns were also protected from knocks and scrapes by using plywood sheeting placed around the perimeter of the columns.

<u>Pile Cap</u>

The columns of the existing structure were founded independently on separate shallow foundations. To transfer the load from the structure to the new piles, two 900mm thick pile caps were constructed to create a single bearing slab surrounding all columns below the rocking collars. The construction of the main pile cap was not just a straightforward operation. There was risk of ground heave around the existing shallow foundations when excavating for the new pile cap. To mitigate this risk the excavation for the new slab had to be carried out in alternating bays, similar to underpinning, replacing the removed overburden pressures of excavated material with a lightly reinforced 150mm thick blinding slab.

The main pile cap construction involved the placement and tying of 30 tonnes of reinforcing steel consisting of over 250 different tagged items. A large number of DH32 bars were required in the bottom mat in long sections, which proved particularly difficult to install in the excavation. The reinforcing cage was already too congested to allow standard laps, so Ancon threaded couplers were utilised to provide full strength connections to multiple DH32 bars. To connect the pile cap to the structure, over 200 holes were cored horizontally into the base of the columns so that DH25 L-Dowels could be epoxied in. Starter bars for the rocking collar perimeter retaining walls also had to be hung inside the congested steel cage with couplers positioned at the finished concrete height with zero tolerance for misalignment.

Figure 6. (L-R) Pilling Work, Pile Cap reinforcement, Rocking Collar Coring

A smaller pile cap was built at the north end of the structure at river level. The cap connected four piles to the base of the north minor column. The difficulty with constructing the lower pile cap was that the excavation was below the ground water level and required constant de-watering. During excavation, the existing foundation under the north minor column terminated a lot shallower than expected which lead to the foundation requiring additional underpinning to accommodate the design. The main upper level pile cap was connected to the smaller lower level pile cap by coring vertically inside the north minor column and epoxying vertical reinforcing bars in place.

To reduce time and cost associated with installing formwork, permanent Speed Form formwork was used with aggregate placed between the form and excavated bank. Portable conveyor belt units were used to place the aggregate due to there being limited access for standard plant to operate while the steel was being placed. The concrete mix was designed to ensure it could flow evenly through the dense steel cage and the $80m^3$ of concrete was poured in one continuous pour using a pump truck

A number of safety measures were implemented due to the size of the excavation including; benching the sides of the exaction, erecting fencing around the exaction perimeter and installing stairs to enable easy access to the work area.

<u>Rocking Collars</u>

The Rocking Collars consisted of creating a reinforced rocking plane on top of the new pile cap. This involved installing 64mmØ high tensile steel bars horizontally in holes cored through the column foundations and passing through the new fabricated steel boxes inside the structure. One of the big challenges involved with the project was the initial set out for the construction of the Rocking Collars. All the elements involved with creating the rocking mechanism depended on the precise construction of the rocking collars. It was critical for the success of the project that the set out was perfect to avoid costly delays caused by rework.

The set out was completed utilizing both the 3D scanning and 3D modeling. The biggest issue was trying to transfer setout points for the steel boxes inside the columns to the exterior of the structure to allow accurate coring for the high tensile bars, which passed through these steel boxes. Following the 3D scanning, measurements from discrete targets were obtained to allow the setout of the steel box base plates that were grouted in place to assist with positioning the fabricated steel boxes. The correct level inside the columns was achieved by transferring levels through the lower access hatches using a dumpy level. The distances between the centers of the plates were confirmed using lasers, string lines and tape measures. Timber templates of the rocking collar profile, with the core hole locations marked, were constructed and set at the correct level and orientation around the base of the columns using measurements from the 3D model. Small pilot holes were cored through the base of the columns and string lines setup to confirm that the timber templates aligned with the base plates inside the columns.

The coring for the rocking collar bars was undertaken before laying the steel in the pile cap excavation due to the location of the holes above the finished pile cap. Had the pile cap been constructed first, there would not have been enough room to position the coring rig. The holes were cored through the structure using an electric high frequency coring machine fixed to a bespoke fabricated steel jig. The length of the cored holes varied with the longest being 3m and it was critical that the core barrel remained level throughout the core. Two different sized holes were cored, where 100mmØ holes were cored for the bars running through the steel boxes and 80mmØ holes for the rest of the cores. Steel templates of the base of the steel boxes were fabricated and installed on the grouted base plates to check that the cored holes lined up with the holes in the steel boxes yet to be installed. The bars were inserted through the steel boxes and resin epoxied in place with a compressed air powered application gun. The holes were cored and bars positioned in a number of stages in each column to ensure that the structures integrity was not compromised. Horizontal cuts 300mm deep and 10mm wide were made around the perimeter of each column at the rocking plane level, then replaced with two layers of polythene sheeting and grouted. This was to create a de-bonding cut to allow the rocking mechanism to be engaged in future earthquakes.

Fabricated Steel Box Sections

The structure's hollow columns were strengthened by installing fabricated steel box sections and followed by filling the remaining voids with concrete. The first major challenge associated with installing the steel boxes was how to get the boxes inside the columns and having the confidence they would fit. The minor columns had one 6m long fabricated steel box per column that could be easily lowered into position from the top of the column. The major columns had a more complicated design, where two 7m long steel channels sections would be lowered into place, followed by lowering a 13m long steel box inside the two channels. The lifting procedure was modeled in virtual space using the 3D model and this allowed the delivery team to try a number of methodologies and select the most suitable one. The final methodology saw the steel channel sections being rotated inside the column as they were being lowered to account for the changing void profile.

The second major challenge was how to lift the sections into the columns. A 100 tonne mobile crane was mobilized onsite to perform the lifting operation. The steel sections varied in size with the smallest channel section weighing 2.5 tonnes to the major column steel box weighing 10 tonnes. To ensure the ground had the capacity to take the cane outrigger loads, a Plate Bearing Test was undertaken prior to the lifting operation. The size of the crane provided enough lifting capacity at the required lifting radius and lift height. A remote quick release hook was used to release the lifting hook from the steel channels once in place inside the columns, as operative access to the top of the channels was not possible.

To give the delivery team confidence that the steel box could be lowered successfully inside the two channels and to identify any issues, a test lift was performed at the manufacturer's yard. The two channels were stood up and temporarily supported while the main box was lowered inside. The objective was to ensure that the shear studs on the outside of the main box did not get hung up on the shear studs on the inside of the channels. The test lift was a success and the only improvement was to weld additional guide fins on the inside of the two channels to assist with sliding the studs between each other.

Sliding Joints

A fundamental element of the rocking mechanism was the construction of a vertical sliding joint detail at the crown of each of the three arches to allow the in-plane rocking. This involved cutting the arches in half vertically and installing a stainless steel plate sliding assembly. The construction of the sliding joints also involved the horizontal post-tensioning of the arches through the sliding plates. The post tensioning system provided the strength across the arches, while still allowing the sliding plates to function in earthquakes. A bespoke post- tensioning system was detailed to allow future controlled de-stressing or load testing of strands following a significant earthquake.

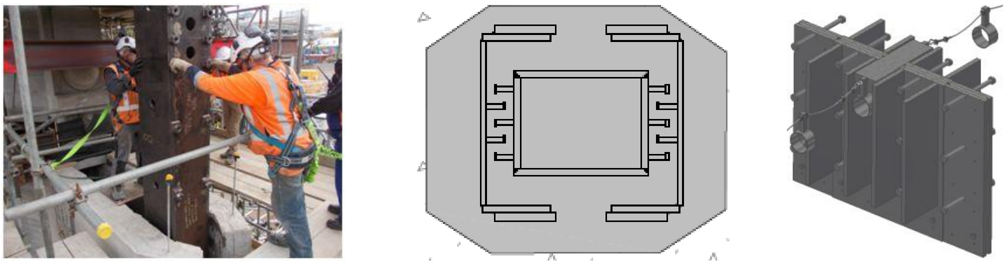

Figure 7. (L-R) Positioning Steel Box, Steel Box configuration, Sliding Joint section view

The sliding plate assembles were installed in two stages to reduce the need for temporary propping of the arches during installation. Stage one involved installing the main sliding plate, post tensioning elements and pouring concrete into the arch cavity. Once the concrete had reached its 28-day strength, the arch was post-tensioned. Construction of stage two could begin after post tensioning by removing the stonework around the center of the arch. With stones removed, the concrete walls of the arch were removed to expose the stage one plate. The second stage of the plate was connected to the first stage and new concrete poured. The stones were replaced and a new mortar joint was cut into the stones at the joint position.

<u>Stone Repair</u>

A major portion of the works involved repairing the damaged Tasmanian Sand Stone cladding on the structure. In order to retain as much of the original stonework as possible, damaged stonework was repaired in-situ. In many places the stonework was too badly damaged to fix, in which case stones were removed and replaced with new stones. The original Quarry was located in Tasmania, Australia, and matching stone was shipped to New Zealand. To preserve the ornate Lions that sit on top of the minor arches, the delivery team removed them before any other work began. This gave the Stonemasons an opportunity to fix the damage off-site. The Lions were replaced at the completion of the strengthening work.

Not only did stonework have to be fixed, a large number of stones had to be removed to enable the civil works to be undertaken. In this case, the Stonemasons took extra care in ensuring the original stones were not damage during their removal. A water lubricated cutting bar saw was used to remove the stones at the mortar joints and detach them from the concrete behind. Stones could not just be removed locally where access was needed to the structure, they had to be removed back to a mortar joint to preserve the original fabric detail. In this case it meant that more stone was removed than ideally liked. All the stones that were taken off the structure were logged and stored onsite in locked cages to reduce the chance of misplacing individual stones if they were taken off site.

Stones were replaced on the structure using stainless steel threaded rods epoxied into the stone and structure. New mortar joints were pointed and the remaining gaps filled with grout. The new stones were roughly cut to size and then position in place on the structure in the same way as the original stones were replaced. Once in place, the face of the stone was cut to the correct profile and when an area had all the stones replaced, the face of the new stones were levigated to achieve a smooth transition between the new and old stones.

Figure 8. (L-R) Stonemason working, Limited space inside Major Arch, Concrete Test Pour

CONCRETE DESIGN

In a city where most of the historic buildings have been demolished, it is paramount that the remaining structures retain their historic value. A very important consideration for the Triumphal Arch project was that the historic fabric was not disfigured by efflorescence. Given the importance and complex nature of the project, a delivery mechanism based upon communication and teamwork was adopted. The concrete supply contract was awarded to

Firth Industries based upon their concrete mix design skills and track record with other heritage projects for which efflorescence prevention was also extremely important. Early engagement meant the concrete supplier, delivery team, and engineer could all provide input into determining the appropriate repair strategy.

Efflorescence on the outside stonework had to be avoided at all costs. Investigations of the internal concrete structure revealed multiple cracks in the concrete, which could result in water/concrete leakage and therefore the potential for surface efflorescence on the stonework. These cracks needed to be sealed. Suggestions where provide by the concrete supplier as to the most appropriate product. The challenge for the delivery team was applying the waterproofing agent over the larger cracks in a very confined place. A mix design was also formulated which limited bleed and contained an admixture designed to minimize the potential for efflorescence. Efflorescence control is however not solely a mix design issue. Advice was provided on the management of water to prevent rain entering the voids during construction. The inside of the structure remained free of water intrusion by building temporary roofs over the arch tops that could be easily removed during the works.

The congested nature within the arch columns meant that a Self-Compacting Concrete (SCC) mix was required. However the only practical method of introducing the concrete was the inside of the fabricated steel boxes. Designer, Delivery Team, and Concrete Supplier identified where holes could be provide in the boxes to allow the flow of SCC to fill the gaps between the steel boxes and the existing structure. The number of allowable holes was limited so a mix was formulated with high spread, 10mm maximum aggregate size, however still having low bleed. A mix trial was undertaken on site to demonstrate the capability of the mix. A scale timber form of the inside of the column with steel boxes was built onsite and the trial mix pumped in. The trial proved very successful and the concrete filled all the voids.

When pouring concrete within an existing structure, an evaluation of the wet concrete pressures on the structures stability need to be considered. A process of staged construction was adopted to prevent potential blowout of the existing stonework. Particularly challenging being the arches where temporary steel tie plates were required to support the structure. Self-Compacting mixes which are used outside the usually controlled environments of a precast factory need to be robust. On site construction can be typified by unexpected delays either due to inner city traffic or construction hick ups. The SCC solution designed for the columns was designed to have an extended workability life. However for the arch cavities the mix needed to contain both steel fibers and achieve high early strength to allow post tensioning of the reinforcing strands. A mix was developed which targeted high early strength. Temperature probes supplied by Firth Industries were used to monitor the maturity of the concrete to assist in confirming that adequate concrete strength had developed to allow stressing.

The limited space in the upper arch presented more challenges for the concrete supplier. The proximity of the strengthening steel elements to the existing concrete face would make it likely that the fibers would prevent concrete filling the small gap. Dialogue with the Engineer reveled that in this location the fibers could be excluded, meaning that a revised mix which could fill all voids was used. This consultative and proactive resolution of issues typified the approach taken for this project.

CONCLUSION

This project represented a good example of a construction team working towards a common goal of delivering a successful project. Many challenges were presented to the delivery team throughout this unique project, but by thinking outside the box, using state of the art technology and engaging specialist suppliers a high quality finished product was presented to the people of Christchurch.