

# ***Controlling high risk blasting in an urban environment The Airport Link Story***

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## **Abstract**

This paper tells the story of using controlled surface and underground blasting methods to excavate hard rock in three areas on Australia's largest road infrastructure project; the Airportlink Project in Brisbane; worth a total value of AUD \$4.8 billion. The project used blasting methods including design, modelling, drilling and loading of the explosives that had never been used before in Australia, some of these were probably unique in the world.

Firstly, the blasting contractor was asked to find a solution to excavate 30,000 cubic metres of hard massive volcanic tuff from the site of a critical tunnel portal, right next to a major arterial road, heritage listed buildings, a church and a three story office building. This Cut and Cover area was known as "The Plenum" as it is the area where multiple tunnels converge, as well as being the area where the ventilation ducts would channel the exhaust gasses away to be filtered in large ventilation buildings. The rock right throughout this area was hard massive volcanic tuff which in some cases exceeded 200 MPa in Uniaxial Compressive Strength.

During excavation of the Plenum, the principal contractors realised that a wedge of rock (approximately 2,250 cubic metres) had become sterilised by construction operations. The rock wedge consisted of a hard rock wall (Brisbane Tuff) with concrete piles embedded within one metre behind where the new face was to be created. Access to the area was limited so all explosives and equipment, including the 20 tonne drill rig, had to be lifted in by crane. The wall left from blasting the rock wedge had to be a stable presplit as this was where the northern section of tunnel would breakthrough into the Plenum.

Finally, the blasting contractor used underground development blasting techniques to remove the rock and breakthrough into the open construction site. This was a challenging operation as the concrete structure in the Plenum, including concrete walls, and a suspended concrete floor, had been built up against the rock wall left behind from blasting the rock wedge.

Blasting on site commenced in April 2010 and was completed by September 2011. The environmental results for vibration and overpressure were well under the imposed limits for all blasting onsite, and the blasting project was completed without incident or injury.

## Introduction

Brisbane's Airport Link Project is Australia's biggest road infrastructure project and has a total estimated value of AUD \$4.8 billion. The 6.7 kilometre underground toll road is expected to be completed by mid 2012, and will connect Brisbane's central business district(CBD) and the Clem Jones Tunnel (CLEM7) to the East-West Arterial Road, which leads directly to the Brisbane Airport (BrisConnections, 2010). It will be the first major motorway to link Brisbane city to the northern suburbs and the airport precinct, allowing motorists to avoid up to 18 sets of traffic lights.

One of the major excavations in the project was a Cut and Cover area was known as "The Plenum". This is an area where multiple tunnels converge, as well as being the area where the ventilation ducts channel the exhaust gasses away to be filtered in large ventilation buildings. The rock right throughout this area was hard massive volcanic tuff which in some cases exceeded 200 MPa. Excavation of the Plenum area initially proceeded using mechanical methods to break the rock (hydraulic rock hammers). However, about half way through excavation it became apparent that the productivity of this method was falling below that required to keep the excavation on schedule. As this excavation was on the critical path for the project, the principal contractor sought the blasting contractor's advice to come up with a solution to excavate the remaining 30,000 cubic metres of hard rock right next to a major arterial road, heritage listed buildings, a church and a three story office building.

This close proximity blasting in the middle of a busy construction site had to be strictly controlled as the rock to be removed was located next to a tunnel portal that had to remain operational 24hours a day. There were strict vibration limits applying to the site infrastructure, and the adjacent privately owned and heritage listed structures. The risks associated with blasting in this environment had to be managed, while complying with the site's strict environmental limits and reducing the effects of blasting operations on nearby residents. Excavation of the plenum was accelerated and completed by 12 controlled blasts using bulk emulsion and electronic detonators over 4 months.

During excavation of the Plenum, the principal contractor realised that a wedge of rock (approximately 2,250 cubic metres) had become sterilised by construction operations and could no longer be reached with conventional equipment. This wedge of rock, which the site engineers dubbed "The Floodwall", was between 2m and 6m thick, 15m high and 35m wide. Constructed on top and behind the rock mass was a 2m thick concrete roof slab which would become the roof of one section of tunnel. It was on top of this slab that a Flood prevention structure had been constructed to withhold any water ingress from a flood prone brook which was only 50 meters away. Removal of the wedge was further complicated by a series of concrete piles embedded within the rock at a distance of one metre behind where the new face had to be created. These embedded piles would form the support structure for the tunnel which would be constructed in this area, and could not be damaged by during excavation of the wedge. The wedge of rock was also on the critical path and could not be delayed.

The principal contractor sought novel and unique ideas to remove the wedge of rock under the floodwall. Although blasting was originally considered an undesirable option, other options were gradually found to be infeasible due to cost, safety and practical reasons. Hence, the principal contractor asked the blasting contractor to come up with a blast design to remove the rock.

Blasting in this area required a very unique solution as access was limited and any equipment required, including all explosives and the 20 tonne drilling rig, had to be brought in by crane. Due to the site constraints and with a 25 metre high concrete pumping tower only 11metres directly in front of the 15 metre high face, there was no opportunity to trial or test the blast design in practice. It had to be done right the first time.

The floodwall blast was successfully executed, leaving behind a smooth presplit that would form the final breakthrough of the northern tunnel into the open construction site. This tunnel would then link the northern suburbs to the tunnel system that directed traffic towards the CBD and Airport precincts. The blasting contractor had successfully used modified underground development blasting techniques with bulk emulsion with electronic detonators to remove the rock from between the four rows of concrete piles and under a cast-in-situ concrete roof slab.

Once all the rock beneath the roof slab had been blasted, the final breakthrough from the underground tunnel section into the Plenum could be completed. This was a challenging operation as concrete walls and a suspended concrete floor had been built up against the rock wall left behind by the floodwall blast. There were now over 100 workers in the Plenum working to complete the structure by the deadline, so a controlled blast had to be executed without damaging any of the structure or disrupting the construction works.

## **The Plenum**

Blasting for excavation of the Plenum was a “relatively” conventional affair for a busy inner city construction site. However, the restrictions imposed by the adjacent arterial road, and the need for other work onsite to remain unimpeded by blasting called for some unconventional thinking to ensure the blasts were as large, and therefore as infrequent, as possible. This included the use of decked charges of bulk emulsion, initiated electronically, to maximise the volume of each blast while minimising the area occupied. Computer aided design was used to match the uneven profile left by the unsuccessful mechanical excavation methods with the final, complex excavation shape.

Conventional blasting methods on busy construction sites usually involve firing many small blasts daily. However, on this site this method was simply not feasible. With each blast requiring road closures, shutting down the worksite, and evacuating the adjacent three story office building, the daily disruption to workers and neighbours would have been considerable and costly. The contractors worked together to develop a unique method to fire large blasts weekly instead of a small blast every day, to thereby reduce the overall number of blasts required to complete the project. The innovative method involved loading blastholes with up to three individual bulk explosive decks with each charge firing separately to control vibration levels. A maximum instantaneous charge weight (MIC) of 4kg was derived from measurements taken from blasting in Brisbane Tuff nearby to the Kedron site. With this low charge weight, decked loading was required to achieve the energy distribution necessary to effectively break the rock. Special techniques had to be developed to ensure the relatively small MIC of 4kg per deck could be controlled while loading with bulk emulsion through a 25mm hose.

An initial trial blast was designed to test the blasting parameters and confirm the blasting methodology would effectively break the rock while still containing the blast within the site. A pattern of 76mm holes on a 1.7m x 1.7m square pattern was drilled to a depth of 6 meters. Each hole was loaded with two 4 kg decks of bulk emulsion explosive, with an inert deck of 2.4 meters separating the charges. The stemming height of 2.2 meters (28-30 hole diameters) was used across the blast to give a Scaled Depth of Burial of  $1.6\text{m/kg}^{1/3}$ . The blast was then covered with 3 layers of blasting mats anchored by 4 tonne concrete barricades. With a powder factor of  $0.46\text{kg/m}^3$  this was believed to be a conservative design.

As there would be no free faces to blast to, a 5 metre deep sump was hammered out of the rock at one end of the blast. This sump provided some relief for the blast and allowed the designer to direct the energy in this direction to limit the heave of the blast.

The explosive chosen was a chemically sensitized bulk emulsion with a bulk strength of 165% relative to ANFO. This high energy, water resistant pumped emulsion blend gave the blast designer the flexibility and energy required to break the hard massive volcanic tuff, while being significantly cheaper than packaged emulsion explosives typically used in this type of application.

The bulk explosive was delivered by a conventional Mobile Manufacturing Unit (MMU). The product is manufactured on the MMU and pumped into blastholes on demand. Due to the requirement to maintain low charge weights to manage vibration, the blasting contractor developed a method to deliver charge weights to a precision of 1 kg. The delivery flexibility and energy of this product allowed the blasting contractor to work within the principal contractor's strict production schedules and significantly improve the productivity of blasting compared to methods using packaged explosive.

Even though the initial design was thought to be quite conservative, the result produced excessive heave and movement for this environment. Figure 1 displays screen captures of the blast video at critical points during the blast. With over 3 metres of heave through the body of the shot, the three layers of blast mats only just contained the blast. There was also a significant amount of oversize material from the 2.2m stemming zone in the muckpile. It was clear to the designers that this rock reacted quite dynamically to the explosive energy, and a revised method of containing the blast while achieving the required fragmentation was needed.



Figure 1: Screen captures of the trial blast video.

The blast designer decided to reduce the stemming height to improve fragmentation in the collar region. Calculations suggested that decreasing the stemming height to 1.8m would allow the stemming zone to be sufficiently fragmented. However this would decrease the Scaled Depth of Burial to  $1.37\text{kg/m}^{1/3}$  which could result in unacceptable flyrock and ejection. Therefore, extra cover material was required. This was achieved by importing 3mm quarry fines ("crusher dust") and designing a cover system combining blast mats, concrete barriers and the crusher dust to provide good flyrock control while still allowing relatively large blasts to be covered.

The cover design comprised 1.8m of stemming in the blasthole followed by 300mm of crusher dust to cover and protect the initiation system, then 1 layer of blast mats followed by 700mm of crusher dust over the top of the blast mats. The extra meter of burden increased the Scaled Depth of Burial to a safe range of  $2.00\text{kg/m}^{1/3}$  and was considered to be more than acceptable to contain the remaining blasts. Figure 2 shows the revised Scaled Depth of Burial calculations.

Scaled Depth of Burial Calculator		Trial Blast	Reduced Stemming	With Extra Burden
Depth (Top Deck length)	(m)	2.9	2.9	2.9
Stemming	(m)	2.2	1.8	2.8
Blasthole diameter	(mm)	76	76	76
Explosives density	(g/cc)	1.2	1.2	1.2
Explosives Mass per m of blasthole	(kg/m)	5.44	5.44	5.44
W - Charge weight	(kg)	4.00	4.00	4.00
D - distance from surface to center of creater charge	(m)	2.6	2.2	3.2
SD - Scaled Depth of Burial		1.62	1.37	2.00

Figure 2: Revised Scaled Depth of Burial calculations.

The initiation of the large, decked blasts was only made possible by using a specialised electronic detonator. This detonator has an armoured shell to provide high resistance to dynamic shock desensitization that can be experienced in decked loading applications in wet, hard rock with high confinement. This detonator provided the designer with accurate, flexible and reliable sequencing and enabled the shotfirer to have 2 way communications with the detonators as the blasts were being covered and up until the moment of firing. By utilising a fully programmable detonator with delays between 0 and 15,000 milliseconds, the blast designer could maintain precise control over the maximum instantaneous charge, and the designed firing sequence.

Blasting in the Plenum required novel timing techniques to reduce adverse affects on the public and the surrounding worksite. The engineer designed a firing sequence intended to generate high dominant vibration frequencies. With many of the blasts recording a frequency of over 100Hz, the displacements measured at the sensitive receivers were less than 0.05mm, thus eliminating the possibility of damage to the surrounding structures.

Twelve blasts were fired in the plenum area to complete this part of the project. All shots were successfully fired and excavated on time with no delays, with vibration and overpressure readings well under the compliance limits.

## The Floodwall

“The Floodwall” was a wedge of Brisbane Tuff 15 metres high, 35 metres wide, and ranging in thickness from 6 metres at the base to 2 metres at the top. “The Floodwall” was so named because a steel flood protection structure was built close to the top of the wedge to prevent inundation of the site by water from a flood prone brook about 50 metres away. The wedge of rock had to be removed urgently as construction in this area was on a critical path and could not be delayed.

The flood protection structure sat on a 2m thick concrete slab that would form the roof of the future tunnel. Four rows of concrete piles were embedded within the rock, beneath the slab, starting about 3 metres behind the existing 15 metre high face. These piles would eventually support the slab once the tunnel was excavated, and hence it was critical that they not be damaged during the excavation process. Until then, the wedge of rock remained because the construction method required it to be left behind to support the concrete roof slab. By the time the construction had reached the point where the wedge could be removed, other construction activities immediately in front of the face had proceeded to such a point that rock breakers could not safely reach the top of the face, and it would have been unsafe for other work to continue in the excavation while hammers were used.

Figure 3 illustrates the front face of the wedge. The flood protection structure is the red steel structure at the top of the photo. Note in the photo that the face of the wedge has been fully bolted,

and other construction works are proceeding immediately in front, including the erection of a concrete pumping tower, 25 metres high, and only 11 metres in front of the 15 metre high face. Access to the top of the wedge was limited to pedestrian access only. All equipment required, including all explosives and the 20 tonne drilling rig, had to be brought in by crane.



Figure 3: Rock Wedge to be removed.

The three primary concerns of the principal when deciding on the method to remove the wedge were:

1. The safety of the community and protection of the principal's reputation in relation to undertaking a potentially high risk operation;
2. The protection of the existing infrastructure in front of and adjacent to the wedge; and
3. The integrity of the in situ precast concrete piles embedded within the rock mass, just one metre behind the intended back row of blast holes.

A number of alternative blasting methods were considered and discussed by the project leaders under the advice of industry experts and blasting engineers. Any blast design had to take into consideration all of the primary goals. No other structure could be damaged, including the concrete piles within the rock mass, and all rock had to fall within the designated drop zone. The drop zone extended 11 metres out from the toe of the face, to the 25 metre high concrete pumping tower.

An accurate three dimensional model was prepared to ensure all possible outcomes could be reviewed. The face of the wedge was surveyed using a combination of MDL and Trimble Laser equipment. All existing structures were included in the model. These included the existing roof slabs and flood protection structure, as well as all structures in front of and below the wedge and the location of the concrete piles embedded within the rock. For safety reasons during construction activities, the face of the rock wedge had previously been fully rock-bolted; this provided multiple paths of opportunity for explosive energy and gasses to escape. The rock bolting plans were reviewed and the location, angle, and bearing of each bolt was surveyed and introduced into the blast model.

With the roof slab anchored to the top of the rock mass, and the rock wedge extending only two metres from the underside of this slab, the drill bench was inaccessible to drill rig and blast crew. A creative plan was required to address this problem. To give the drill and crew access to the rock wedge, a sacrificial steel reinforced concrete block was constructed on top of the rock wedge, up to the same height as the roof slab.

This concrete block was added to the computer model and work began on the conceptual blast design. The engineers had to visualise how the rock mass was required to flow out from its current position and into the drop zone. Three rows of blast holes were required. The front row would be used to thrust the toe of the face out into the drop zone. The middle row would be required to fragment the body of the shot so it could be easily handled by the onsite equipment. The back row would split the wedge away from the embedded concrete piles within the rock mass. This would ensure that the energy from the blast would not propagate past the split line and damage the concrete piles. The loading design methodology is outlined in Figure 4.

Figure 4: Cross section, blast hole loading design and calculations.

The design loading of the second row was spilt into five decks over the 15 metre bench height. The decks were split into a 5 kg bottom deck with offset 2.5 kg and 1.7 kg decks for the remaining four. The bottom deck ensured good fragmentation and breakout at the toe. The four offset decks ensured even explosives distribution throughout the wedge, without exceeding the maximum charge weight allowed to manage vibration. Great care was taken to ensure drill holes were not going to intersect with the rock bolts, and later during the design process, the placement of each individual charge included measuring the proximity of charges to the rock bolts, to reduce the risk of energy escaping via the bolt holes. (Prior to blasting, the tension on all rock bolts was released and each hole was fully grouted to assist in containing the blast energy).

ensure the presplit cracked cleanly along the required line. A light powder factor of 0.56 kg/m<sup>2</sup> was used on the 0.3 m spaced row, with only every second hole loaded. The uncharged hole between each loaded hole helped to ensure the energy would be sufficient to crack along the split line without damaging the concrete piles or roof slab.

The decks were sequenced with the bottom decks firing first. The intention was to have the bottom of the wedge thrust out as far from the face as was allowable. This created a void for the rock broken by the upper decks to drop into. This method of firing was needed to help control the movement of the rock mass, and ensure it fell within the designated drop zone. The sequence of initiation between the decks was designed to ensure the rock flowed down and forward into the drop zone. By sequencing the holes so that the bottom and second decks in the front holes fired with at least 55 ms/m relief between them, followed by a further 75 ms/m relief to the bottom deck in the second row, a 45 degree downward angle of initiation was created.

Several conceptual blast designs were modelled using the blasting contractor's proprietary software. This software allowed the blast designer to run multiple iterations quickly and consider the fragmentation and blast induced vibration of each design, as well as determine how the rock would move and flow into the drop zone. Examples of these simulations are shown in Figure 5.

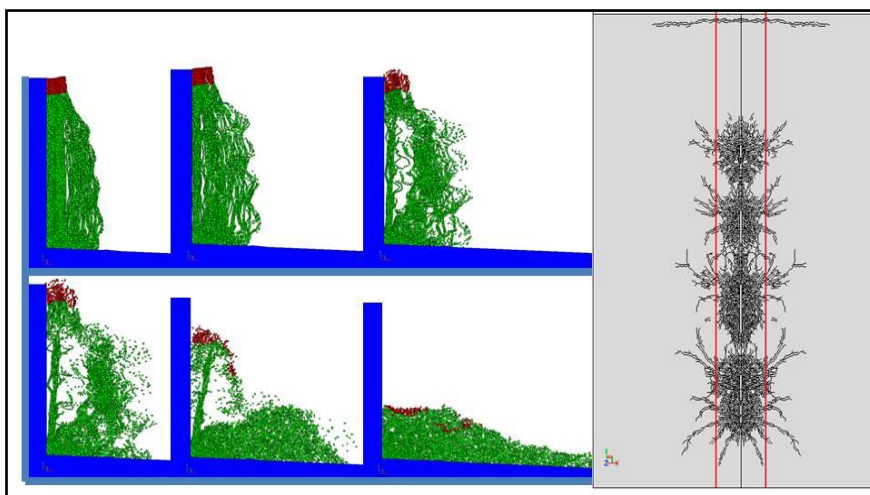


Figure 5: Cast profile and Damage envelope as modelled using a numerical code (Minchinton and Lynch, 1996).

A 55mm x 300mm packaged detonator sensitive emulsion with a bulk strength of 183% was selected as the primary explosive for this blast because it is specifically designed for small diameter construction blasting applications. A centre traced 26mm emulsion based presplit product was chosen for use in the presplit row. The detonator sensitive emulsion explosive is internally traced with 10g/m detonating cord that ensures fast and complete detonation. The small diameter, high velocity of detonation, and low decoupled energy of the presplit product minimises blast damage to the walls leaving behind a smooth profile with minimal overbreak.

## Preparations for blasting

The work method included drilling through the sacrificial concrete constructed as a drilling platform, and it was foreseen that the reinforcing mesh in the concrete could be a major problem to the drilling. The blast designer ensured that the crew had total control over the location of reinforcing mesh and

the drill holes. 100mm PVC collar pipes were used to form the collars of the holes through the concrete, as shown in Figure 6.

This was achieved by placing the 100mm collar tubes throughout the reinforcing mesh prior to the concrete being poured. The principal contractor's surveyors were given the drill plan, and one by one they made sure that each collar pipe was in the correct location and at the correct angle and bearing. Prior to the concrete pour, the location, angle, and bearing of each collar pipe, was crosschecked by both the blast designer and the shotfirer.



Figure 6: Collar pipes and reinforcement.

The work area for the drill was only 3.5m wide and it was bound by a 15m drop off on one side. Each end was effectively blocked from anything other than pedestrian access. This meant that all equipment had to be brought in by crane; this included the explosives deliveries and the 20 tonne drill rig, which can be seen in Figure 7.

Detailed drilling logs were kept by the driller, and these logs were used to map the varying strata within the wedge. Each of the 211 drill holes was boretracked so the face burden could be analysed and loading adjusted accordingly.

As each and every deck length was known in advance, the blast designer ordered custom made 57 mm internal diameter cardboard tubes to the exact lengths required for each deck. These were packed with the 55 mm packaged explosive and electronic detonators, and lowered into position. This ensured the exact amount of decoupled explosive was precisely where the designer wanted it to be.



Figure 7: Lifting in the drill rig.

As the modelling had shown there was potential for the blast to cast material over 50 metres into the excavation, secondary methods of controlling the rock movement were required. Blast mats were hung over the face, as shown in Figure 8, to ensure any ejection from the face or from the rock bolt holes was contained. Heavy chain was used to secure the blast mats together and to anchor the mats to the roof slab. The curtain of mats was suspended so the lower edge was held away from the face, to allow the rock to flow out from the face without risk of the mats being damaged and torn down from the roof slab.



Figure 8: Blast mat cover being installed.

Due to the height of the face, the sheer volume of rock would still have the potential to extend past the required drop zone. The base of the concrete pumping tower and the concrete infrastructure was

packed with hay bales to assist in absorbing any impact. A sacrificial barrier of decommissioned shipping containers was then placed along the edge of the drop zone to protect the infrastructure from the rubble of the blast.

On the morning of the blast, every detonator, wire, and connection was cross checked separately by three highly experienced shotfirers. There would be no second chance with this shot so every detail had to be perfect. The blast was fired on time at 10 am on the 25<sup>th</sup> September 2011, under the management of a blast controller, twelve blast sentries, four traffic controllers and four police officers. Figure 9 shows the result with no damage to the infrastructure in front of the blast. Several days later the embedded concrete piles were exposed and were fully intact with no damage. This is shown in Figure 10.



Figure 9: Post blast; no damage to site infrastructure.



Figure 10: Embedded concrete piles remain intact.

## The Breakthrough

The face of the wall left from the floodwall blast was a flat consistent presplit; this would now allow the northern section of tunnel to breakthrough into the open construction site. This was a challenging operation as the concrete structure in the Plenum was now advanced to the point that concrete walls and a suspended concrete floor, had been built up against the rock wall left behind by the floodwall blast. There were over 100 workers in the Plenum working to complete the structure by the deadline, so a controlled blast was needed to avoid damaging any of the existing structure or disrupting the progress of the construction works.

The blasting contractor had successfully used modified underground development blasting techniques with bulk emulsion with electronic detonators to remove the rock from between the four rows of concrete piles and under a cast-in-situ concrete roof slab. As the shotfirer had refined the development blasting design, to effectively remove the rock from under the roof slab of the tunnel, a proven heading design was available for use on the breakthrough. The heading design consisted of four 102mm reamers and 45 mm production holes on a 0.9 metre x 1.0 metre rectangular pattern. The Shotfirer and Engineer marked up this pattern onto the underground face, and the location of each intended collar was surveyed.

The site survey staff laser profiled the Plenum face, concrete walls, and suspended floor under construction, as well as the underground faces. The survey data from both the plenum and underground faces was imported into the designer's blast design software. An accurate 3D model of the area was created and the drilling, loading, and timing plans were created. It was determined that a consistent stand off of 900mm from the toe of each hole to the outside face would be required. This was critical as rock from the blast had to only just breakthrough or the blasting contractor would risk projecting rock across the site, potentially injuring workers and public as well as damaging the structure under construction and privately owned residences.

A drill plan with each hole depth was created and given to the driller. Each hole was checked after it had been drilled, and the depth angle and bearing was recorded and compared against the design. Any over drilled holes were backfilled with stemming plugs and any out of spec holes were fully grouted and redrilled. Once drilling was completed the loading and timing plans were adjusted to suit the as-drilled face and loading commenced. Figure 11 displays the drilling in operation on the first breakthrough blast, with the shotfirer in the background carrying out the quality control.

The holes were primed using a 25gm booster and an electronic detonator. These boosters have been specifically designed to provide reliable initiation in conjunction with an underground bulk emulsion explosive. The bulk explosive chosen was a chemically sensitised explosive specifically designed for use in civil tunnelling and underground construction applications.

The electronic detonator chosen was specifically designed for use in development tunnelling. The software associated with this system allowed the blast designer to assign unique delay numbers with individual detonator offsets that ensured single charge firing, without the risk of cap scatter that can be found in non-electric long series delay detonators. The system provides two way communication allowing the shotfirer to check the functionality of each detonator prior to firing and significantly reduce the risk of misfires.

It was planned to fire one heading to test the blast design to allow changes to be made prior to firing the other two headings. Lane 3 was chosen to be the first heading to be fired, as while it was the closest to the public road, it was directly adjacent to a 12 metre high rock wall, and was the most contained of the three headings. The outside face of the blast had to be covered to reduce the risk of flyrock being projected across the site and into the public areas if the breakthrough didn't perform as planned.



Figure 11: Drilling of the first breakthrough blast.

Blast mats were once again hung over the face to ensure any ejection from the face was contained, as illustrated in Figure 12. One layer of mats were hung from the roof slab and secured only at the top. These mats lay against the face to form the first layer of protection. A second layer of mats was secured at the top and bottom. These mats were pulled out from the wall at the bottom and secured to a row of concrete barriers. This was to allow the rock to flow out from the face without risk of the mats being damaged and torn down from the roof slab. Heavy chain was again used to secure the blast mats together and to anchor the mats to the roof slab.



Figure 12: Blast mats in place for the first breakthrough.

With the first heading loaded and the flyrock protection in place the site was secured and blast clearance commenced. The blast was successfully fired on the 15th February 2011, with the majority of the rock pulled into the underground drive as planned, and only approximately 15% of the rock volume flowing out into the Plenum structure. The blast mat cover design worked perfectly with all rock being contained. Figure 13 displays the shotfirer inspecting the blast from within the tunnel, fresh air and sunlight is flowing in.



Figure 13: Inspecting the blast, a successful breakthrough.

The first breakthrough blast was so successful that the design wasn't changed at all for the next two headings. The Principal contractor was so confident in the design that they asked the blasting contractor to fire the other two headings in the same blasting event just 4 days later. As shown in Figure 14, more structures were being built in front of the face every day, so time was critical.

The Lane 2 & 3 headings were fired on the morning of the 19th February 2011. The result was a carbon copy of the first blast with all rock being contained and minimal rock flowing out onto the Plenum.



Figure 14: Site prior to blasting Lanes 1 & 2.

## Risk Management

Detailed risk assessments and work method statements were compiled for each section of this project to fully understand and address the unique blasting requirements. A cross section of the workforce had input into the process, including the site engineers, blast designers, public liaison and traffic management personnel, as well as the shotfirers on the project and an independent design auditor. The risk assessments were mapped into the primary contractor's safety management system to ensure all risks had been addressed and a project specific blasting management plan was created for the project.

Apart from protection of people and third party property, the integrity of the embedded concrete piles and existing roof slab structure were the highest priority concerns. It was essential the blast designer and project leaders understood how blasting activities would affect the integrity of the concrete piles and existing roof slab, and what preventative measures that should be incorporated into the blast design.

Australian standards and legislation were developed to ensure that blast-induced vibration levels are maintained at or below levels for human comfort. They often limit the permissible levels of vibration to well below those capable of causing structural or even cosmetic damage to structures. It is well accepted that steel reinforced concrete structures can withstand vibration levels several orders of magnitude higher than those applied for human comfort. Safe vibration levels for engineered structures are therefore higher than commonly quoted environmental conditions applied to residential areas for quarry and mine blasting.

It is also accepted that both frequency and amplitude of vibration affect the probability of damage. For the same reasons that low frequency vibration, similar to earthquakes, are damaging and accompanied by low limits, high frequency vibrations that occur from construction activities at much closer distances result in lower strains and correspondingly reduced chances of damage.

Section J of the Australian Standard AS2187.2 2006 recommends maximum peak particle velocities for different types of structures and for different project durations based on damage rather than human comfort as described in the earlier versions of the same document. The Standard defines cosmetic damage as the formation of hairline cracks on drywall surfaces, the growth of existing cracks in plaster or drywall surfaces or the formation of hairline cracks in the mortar joints of brick/concrete constructions. Minor damage is defined as the formation of cracks or loosening and falling of plaster or drywall surfaces, or cracks through brick/concrete blocks. The same standard proposes limits for ground vibration for control of damage to structures. The limits are shown in Table 1 and suggest a value of 100mm/s for structures of reinforced concrete construction.

Table 1 – Recommended ground vibration limits for control of damage (AS2187.2-2006)

<i>Category</i>	<i>Type of Blasting Operations</i>	<i>Peak component particle velocity (mm/s)</i>
Structures or architectural elements that include masonry, plaster and plasterboard in their construction	All blasting	Frequency dependent damage limit criteria as in Table J4.4.2.1
Unoccupied structures of reinforced concrete or steel construction	All blasting	100mm/s maximum unless agreement is reached with the owner that a higher limit may apply
Service elements, such as pipelines, powerlines and cables	All blasting	Limit to be determined by structural design methodology

Tart, Oriard and Plump (1980) observed the effects of blasting during a concrete demolition programme and correlated these with measured levels of particle motion (acceleration, velocity and strain). The measurement range covered instances where no damage was detected through to cases where concrete was heavily damaged and thrown from the structure. Their findings are summarised in Table 2 and show levels of vibration which are an order of magnitude higher than levels commonly stated for a similar effect.

Table 2 – Summary of vibration level associated with various effects (Tart, Oriard, and Plump,1980)

<i>Effect</i>	<i>Strain</i>	<i>Particle Velocity (mm/s)</i>
Theoretical static failure in tension	140μ	500
Spalling of freshly set grout	700 μ	2,500
Spalling of loose/weathered surface skin	1300 μ	5,000
Cracks develop extending from blastholes	2400 μ	9,500
Mass concrete blown out	3800 μ	15,000

During the 1970's, the Tennessee Valley Authority embarked on a construction program necessitating blasting very near to concrete. Because of the cost implications, a re-write of the specifications was ordered based on specific conditions, field trials and full scale blasting activities. Blasting was completed at distances ranging from tens of metres through to less than 1 metre. Oriard (2002) reported on the new criteria.

The study derived a relationship between the allowable level of vibration for mass concrete as a function of the concrete age, ranging between 0 hours and 10 days or more. Oriard reported that for aged concrete, an acceptable level of vibration is 400mm/s for blasting at distances of between 15 and 50 metres. At nearer distances, the acceptable vibration increases to 500mm/s and at further distances, reduces to 300mm/s.

The assessment by the project leaders and an independent design auditor, indicated a wide variation in accepted criteria, however it was appropriately noted that in almost all cases, the levels that have been applied for specific construction activities have not resulted in any damage. It was agreed by the project leaders that the commonly imposed limits and standards reflected to a greater degree the conservatism in the approach, rather than what may be an appropriate level to prevent damage. It was further noted that where vibration has continually increased to the level where damage has occurred, the levels are in excess of 500mm/s.

The blasting contractor's technical personnel worked closely with the principal contractor and an independent design consultant during the design phase of the project to research the vibration transmission characteristics of the ground, and the potential travel distance of flyrock. Vibration was estimated by analysing all of the data from previous blasting operations performed on site, as shown in Figure 15.

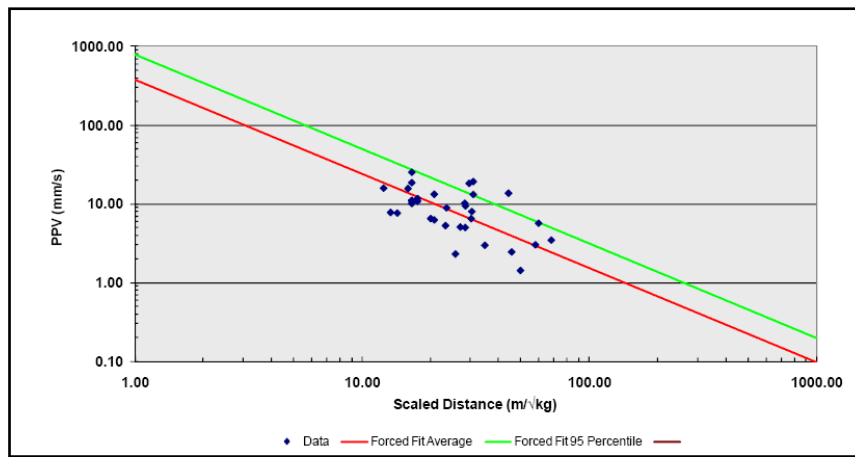


Figure 15: Site data regression by independent design consultant (Heilig, 2010).

A relationship was developed between the level of vibration, explosive quantity and distance between the blast holes and the sensitive receivers. Based on this relationship it was determined that an MIC of between 4 kg and 5 kg would not be expected to impact on the integrity of the site infrastructure. This charge weight would allow the blast designer to effectively break the rock, without damaging the embedded concrete piles, roof slab and Floodwall structure. A summary of these vibration predictions are outlined in Table 3.

Table 3: Vibration prediction by independent design consultant (Heilig, 2010).

<i>Distance from final blast hole (loaded with 26mm decoupled pre-split product) (mm)</i>	<i>Predicted level of vibration (mm/s)</i>
750	620
1000	490
1250	400
1500	340
1750	300
2000	260

The blast designer and Shotfirer worked together to develop a unique method of loading and timing each blast. In some cases deck loading of blastholes with up to five explosive decks was required to control vibration levels.

The innovative timing sequences used across the project was only made possible by using an electronic blasting system. The use of an electronic detonator provided accurate, flexible and reliable sequencing and also ensured the user had two way communications with the detonators from prior to deployment right up until the moment of firing.

## Results

The project was an excellent example of the requirements of large scale construction blasting, and involved:

- Managing the risks of using explosives in a large, busy urban construction project;
- Firing large blasts to reduce disruption to other activities on the site and surrounding roads;
- 3D computer blast design using specialised software programs, to meet precise excavation tolerances; and
- Precise loading of explosives, to manage vibration cost-effectively;

By the end of the project, even though the principal contractor had not originally planned to blast, the site had used:

- 21000 kg of open cut bulk emulsion explosives;
- 83000 kg of underground bulk emulsion explosives;
- 30000 kg of packaged explosives; and
- 39400 electronic detonators.

The blasting component of the project lasted 18 months and a total of 250 blasts. The environmental results for vibration and overpressure were well under the imposed limits, and the embedded precast concrete piles remained undamaged.

One of the Principal Contractor's Senior Project Engineers said "Controlled blasting was the solution on this project, allowing (us) to improve the productivity of (our) rockbreakers and excavate this critical site while protecting the sensitive nature of the site surrounds," (Alcon, 2011)

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