LANE COVE TUNNEL COLLAPSE AND SINKHOLE
A FORENSIC REVIEW - 1: THE COLLAPSE

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ABSTRACT

Sinkholes do form as a result of natural processes, such as dissolution in carbonate rock formations, and can also occur as a consequence of made-made activity. The Lane Cove Tunnel sinkhole in Sydney resulted from the collapse of a tunnel intersection being excavated at shallow depth in poor quality Ashfield Shale in November 2005. The collapse endangered the tunneling crew, an apartment block, a major roadway and buried infrastructure. Over the past decade the collapse was subject to a WorkCover investigation, to industrial prosecutions in the NSW Industrial Relations Commission and to civil litigation in the NSW Supreme Court, all of which were informed by a series of expert reports. This paper summarises the factual evidence of the collapse including the project description, the ground conditions and the design of the tunnel support systems. Separate associated papers set out the cause or causes of the collapse, the contractual relationships involved in its design and construction and the legal consequences and outcomes that followed the collapse.

1 INTRODUCTION

1.1 THE COLLAPSE

At approximately 1:40am on Wednesday 2nd November 2005 a section of the roof of the Lane Cove Tunnel collapsed, an unravelling failure ensued and the collapse propagated through a height of 17m to the surface forming a sinkhole that undermined the front section of Kerslake Apartments (Figure 1). The collapse occurred during the final excavation stages of the intersection of a ventilation tunnel and the exit ramp to the Pacific Highway.

Figure 1: Damage to Kerslake Apartments and the surface expression of the sinkhole

By about 2:00am the sinkhole was observed in front of Kerslake Apartments and by 5:30am it had expanded in size to be around 8 to 9 m in diameter. At about 6:00am a secondary collapse occurred involving a number of retaining wall piles and the decision was made to fill the hole with mass concrete to mitigate any further collapse. The spoil from the collapse buried the road header and a Caterpiller loader (Figure 2) but fortunately the 4 members of the tunneling crew retreated from the area without suffering physical injury.

Figure 2: Collapse debris in MC5B ventilation tunnel looking north towards the blocked intersection, 3 /11/2005

The collapse occurred midway through the night shift with only the tunneling crew of four operators present. Understandably, descriptions of the collapse process are sketchy and the most comprehensive version was recorded in a statement made to the WorkCover Inspector by the roadheader operator 5 days after the collapse. The verbatim extract from his statement is as follows:

> *At the time I was just sitting on the roadheader. There was a little bit of rock dropping out of the face we had just cut, which was normal, not much bigger than a cup. It looked like black shale. About that time the leading hand jumped up on the machine as well. The loader driver was mucking out at the time. Then a lot more rock started dropping off the face and seemed to be working its way back towards my position on the header.*
>
> How far was the roadheader back from the face?
> *About 2 to 3 metres from the cutter head. Normally the excavated fill drops onto the apron and is conveyed into a truck. Because of the position we were in there was no room for a truck behind the roadheader. That's why we were using a loader. The cut had been finished about 20 minutes to half an hour before the rockfall. It may have been longer.*
>
> What happened next?
> *John and I must have had the same thought – get off there. I went down the ladder on the machine and got away as far as I could.*

What did you notice about the material that was falling?
I didn't take much notice. At the face it wasn't wet.

What colour rock began the fall?
I don't know what colour rock began the fall. I could see the yellow colour of the dyke in the face of the excavation. They both appeared to fall simultaneously.

Did you hear any unusual noises prior to the rockfall?
No.

1.2 THE RECOVERY

The stabilisation works required approximately 2,750 m^3 of concrete and grout and permanently entombed the roadheader. Subsequently, the Pacific Highway exit ramp was realigned passing immediately south of the collapsed intersection. Spoil from the collapse completely blocked the intersection, as shown in Figure 2, and rendered the intersection inaccessible.

There was structural damage to Kerslake Apartments; to the Longueville Road retaining wall; to Telstra's fibre optic cables copper cables and a co-axial cable; to water mains and pipework; to underground electrical cables; and nearby service businesses. The agreed amount of consequential damage to tenants and owners of Kerslake Apartments; plus economic loss to the affected commercial interests was $21m plus interest, which the Court awarded two-thirds to PB and one-third to PSM.

1.3 THE AFTERMATH

The Lane Cove tunnel collapse provides an opportunity that has become increasingly rare in the field of major infrastructure failures because the lessons that can flow from such failures are increasingly locked within the confidential corporate files of the parties involved. The LCT collapse underwent several stages of investigation and was the subject of litigation between the main participants, the outcome of which was determined by judgement through the Courts (McDougall, 2016). In consequence the entirety of the litgation process, including the documentary evidence, has become available in the public domain and is accessible to the engineering profession as a complete case history covering not only the technical aspects but also the contractual and responsibility issues.

This paper is the first in a set of three companion papers that address the main components of the project itself. It deals with the collapse, the geological conditions, the design of the tunnel support systems and the implementation of the support system as it relates to the collapse and its causation. The second paper describes the post collapse investigations into the causes of the collapse of which there were three separate stages of investigation:

- The first was carried out by Emeritus Professor Ted Brown of Golder Associates as an independent investigation into the causes of the collapse (**the Brown Report**), immediately following the collapse;
- The second investigation was carried out by WorkCover NSW pursuant to Sections 8(1) and 8(2) of the Occupational Health and Safety Act 2000 (**the WorkCover Report**) and took place over a period of about 2 years; and
- The third was a series of investigations carried out by a number of local and international experts in relation to proceedings in the Equity Division of the Supreme Court of NSW as Thiess Pty Ltd and John Holland Pty Ltd-v-Parsons Brinckerhoff Australia Pty Ltd in 2016 (the **Civil Litigation Reports**).

The third paper addresses the aftermath of the collapse which culminated in litigation involving the main participants; the Contractor, the tunnel designer, the Geotechnical Engineer and the Project Verifier. It presents information revealed during court proceedings and the conclusions reached by Justice McDougal in his judgment of 4 March 2016, a decade after the event. It also reviews the impact of legal procedures on those findings and of the contractual conditions on the attribution of parties' contribution to the failure.

1.4 THE AUTHORS

Mr Kotze was retained by WorkCover NSW to assist with its investigations into the collapse and inspected the collapse site initially at 8:00am on 3 November 2005 and subsequently on a number of other occasions. Dr Burman and Mr Kotze were retained by HWL Ebsworth as experts in relation to civil proceedings in the Supreme Court of NSW brought by TJH. Ms Chan was retained as legal counsel by HWL Ebsworth.

2 THE LANE COVE TUNNEL PROJECT

The Lane Cove Tunnel (LCT) is a significant component of the Sydney roadway system connecting the M2 Motorway at North Ryde with the Gore Hill Freeway at Artarmon and bypassing surface traffic in the Lane Cove area. The LCT Project consisted of twin, 3.6km long, 2- and 3-lane tunnels running beneath and just to the north of Epping Road together with tunnels for entry from and exit to the Pacific Highway and associated ventilation tunnels.

In 2003 the NSW RTA awarded Lane Cove Tunnel Company (LCTC) the contract to plan, design, construct, operate and to maintain the Motorway for a period of 33 years with URS Australia in the role of Independent Verifier (IV). LCTC contracted with the Thiess John Holland Joint Venture (TJH) to plan, design, construct and commission the project works including the tunnels. TJH appointed Parsons Brinkerhoff (PB) to provide design consultancy and construction stage services in relation to geotechnical instrumentation, monitoring, tunnel mapping and support. PB, in turn, appointed Coffey Geosciences (Coffey) to carry out geotechnical investigations and to assist in the design of tunnels and retaining walls.

Construction of the LCT commenced in July 2004 and in the early hours of Wednesday (about 1:40 am) 2nd November 2005 a collapse occurred while the intersection of the Marden Street ventilation tunnel (MC5B) and the exit tunnel to the Pacific Highway (MCAA) was being excavated. The general layout of the MC5B/MCAA intersection is shown in Figure 3.

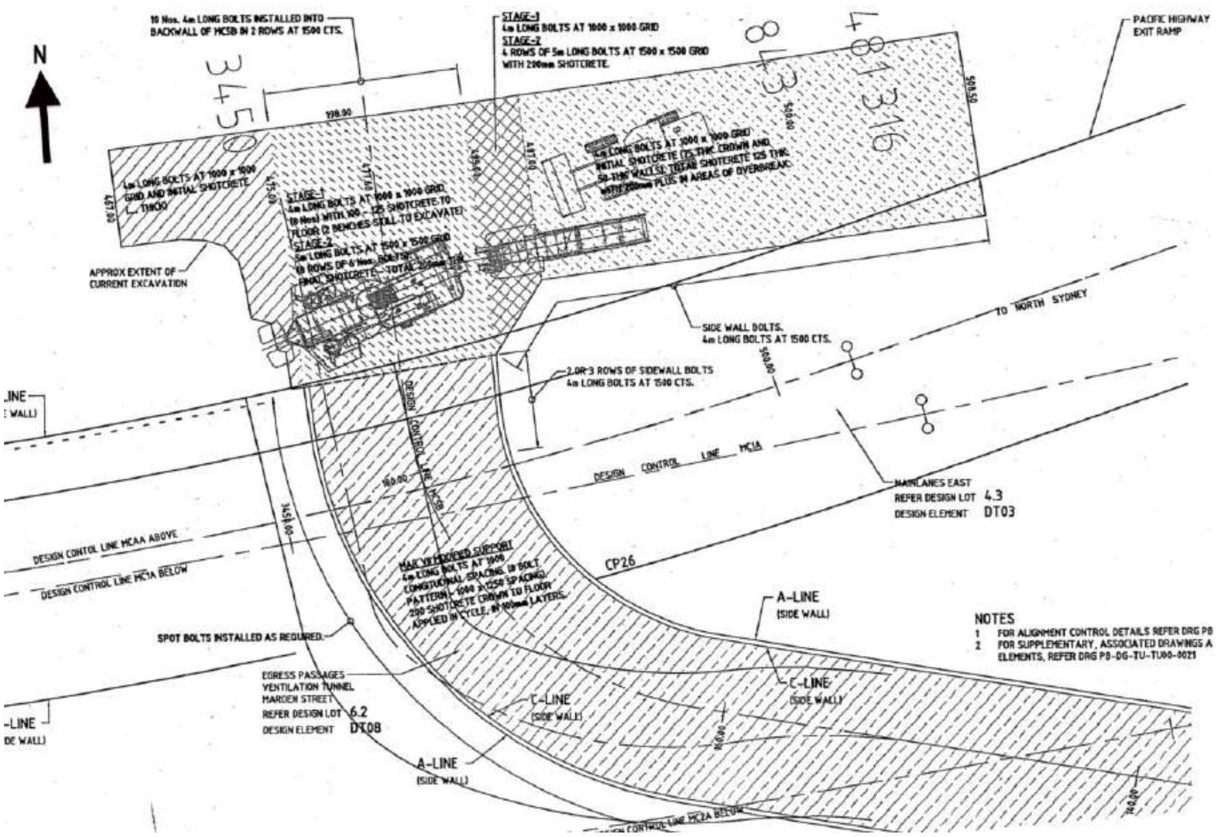

Figure 3: The general layout of the intersection of the Marden St ventilation tunnel (MC5B) and the Pacific Highway exit ramp (MCAA)

3 DESIGN OF THE TUNNEL SUPPORT SYSTEMS

The general geological and geotechnical conditions for the Lane Cove Tunnel Project (LCTP) have been described in some detail by Badelow et al (2005). In essence the LCTP alignment comprises the upper sedimentary formations of the Sydney Basin comprising the Hawkesbury Sandstone and the overlaying Wianamatta Group including the Mittagong Formation and the Ashfield Shale Formation, which occur as a "capping layer" on the elevated ridgeline areas of the alignment at Longueville Road in Lane Cove.

The rock structure within the Ashfield Shale, Mittagong Formation and Hawkesbury Sandstone typically contains defects including subhorizontal bedding planes, cross bedding and laminations, subvertical and lower angled joints, faults, low angled shears and dykes. Typically the bedding planes in the three formations are subhorizontal. The spacing between bedding planes in the Hawkesbury Sandstone is typically 0.5 m to 3 m. The bedding in the Mittagong Formation and Ashfield Shale is more closely spaced. The major joint sets trend NNE-SSW and ESE-WNW with subordinate sets present in shale.

3.1 TUNNEL SUPPORT DESIGN

The LCT Project followed a number of shallow tunnelling projects that had been completed within the Hawkesbury Sandstone and to a lesser extent in the Ashfield Shale. The design of the LCT support schemes was based on precedents from earlier tunnel projects together with empirical and quantified geotechnical models appropriate to the Sydney conditions of essentially flat lying, sedimentary strata. Based on local experience PB determined in principle that roof support would be by rock bolts in combination with fibre reinforced shotcrete (FRS) in areas of competent sandstone and siltstone and in areas where the rock was weathered, faulted or altered, such as the Ashfield Shale, steel sets or other passive support would be provided. The anticipated ground conditions along the alignments of the tunnel network had been assessed by a series of geotechnical investigations prior to and following the commencement of the project.

Precedence was largely encapsulated in a compilation of rock bolt lengths and tunnel spans for about a dozen earlier Sydney tunnels with spans ranging up to 20m. Empirical guidelines for tunnels through jointed rock masses, such as Barton’s Tunnelling Quality Index (Q), were considered. Design calculations for roof sag, rock movements, surface settlements, shear deformations, rock stresses and support loadings were based on two separate numerical methodologies; Voussoir beam analysis and 2D finite element methods or their equivalents. Each of the design methodologies was applied by PB for the relevant geometries and anticipated ground conditions in determining the range of approved tunnel support designs and re-designs (Maconachie et al, 2005).

The design philosophy was not based on providing a single universal design for the worst case condition but rather relied on insitu geological observations and mapping to vary the support locally to address variations in conditions observed in the field and a range of support designed for these various conditions. Implementation of this approach was by way of a ‘support toolbox’ that consisted of a bespoke ground classification system developed specifically for the LCT and a set of compatible support types for various tunnel configurations, tunnel geometries, span width parameters and ground classifications. The support system provided in the toolbox was verified by performing detailed 2-D numerical analyses using PHASE2, PLAXIS and FLAC software. It was intended that ground support would be determined by an observational design approach (ODA).

3.1.1 LCT Ground Classification System

The LCT Ground Classification System was based on the Sydney Rock Classification system (Pells et al, 1998) and past experiences on similar projects. In all there were 9 classes, 5 for Hawkesbury Sandstone and 4 for Ashfield Shale. The LCT Classification System for shale, in which the MC5B/MCAA intersection was located, is set out in Table 1.

The classification system was not elaborated with instructions on its application or how ambiguities were to be treated. It is unclear for example how the classification of shale in one strength class was to be reconciled with the rock defect structure of a different class or how conflicts between Sydney Rock Classes and Q-values were to be resolved.

Table 1: LCT ground classification system for shale ground types

Ground Classification	Strength	Saturated UCS (MPa)	Defects	Typical Sydney Rock Class	Q Value
LCT G6	Medium to High	>7	2 Joint sets plus random (Bedding is one set) Dip of Joints >45º Discontinuity spacing > 0.6m Minor Shear Zones, Faults, Dykes Minor Clay Seams or weak Beds Dry or minor water inflows	Class I - II Shale	>2.2
LCT G7	Low	2 to 15	2 Joint sets plus random (Bedding is one set) Discontinuity spacing > 0.6m Minor Shear Zones, Faults, Dykes Minor Clay or Sandy Beds, seams or Joints Dry or minor water inflows	Class III Shale	>0.2
LCT G8	Very Low	<2	2 Joint sets plus random (Bedding is one set) Discontinuity spacing > 0.02m Minor Shear Zones, Faults, Dykes Minor weak Clayey or Sandy beds, seams or Joints Dry or high water inflows	Class IV Shale	<1
LCT G9	Extremely Low	<1	2 Joint sets plus random (Bedding is one set) Discontinuity spacing < 0.2m Fault Zones with crushed, weathered or broken rock Vertical or sub-vertical; features such as weathered Dykes and associated Clay infill. Significant iron staining. Dry or high water inflows.	Class V Shale	<0.27

Ground classification (GC) using the LCT System was the first step in the procedure for Ground Support Determination (GSD) for any given length of tunnel. The Engineering Geologist was tasked to determine the GC by mapping the rock mass exposed around and above a particular section of tunnel. The second step was to select the support arrangements and excavation procedures from the set of approved designs based on the actual ground conditions assessed by the Engineering Geologist in accordance with the Work Method Statement (WMS) prepared by TJH for that assessment. The suite of original approved designs for the MC5B ventilation tunnel are set out in Table 2 in respect of the associated GC as determined under the LCT Ground Classification System.

Table 2: Approved support designs for the Marden St ventilation tunnel MC5B as of October 2004

LCT Ground Type	Support Type	Advance (mm)	Support	Bolt Spacing (mm) Transverse x Longitudinal	Fibre Reinforced Shotcrete (mm)
LCT-G1,G2 & G3	MAR-VI(B)	1750	6 No. PB300 3000 mm long Spider Plates for G3 only	1500 x 1750	50 Initial 75 Final
LCT G6	MAR VI(B)	1500	6 No. PB300 3000 mm long Spider Plates	1500 x 1500	50 Initial 75 Final
LCT-G4,G5 & G7	MAR-XI(A)	1500	6 No. PB300 3000 mm long Spider Plates	1500 x 1500	50 Initial 75 Final
LCT-G8 & G9	Modified MAR-VIII	1200	200UC46	1200 Steel Set Spacing	75 Initial 200 Final 35 Fire Protection

Tables 1 and 2 comprised the 'support toolbox' that was provided by the tunnel designers to the construction services geotechnical team consisting of a Senior Tunnel Engineer (PB), Senior Rock Mechanics Engineer (PSM) and Engineering Geologist (PSM). This team's responsibilities were closely defined in WMS documents for Geotechnical Mapping & Ground Support Determination and for Geotechnical Monitoring. Individually they were responsible for the functions of mapping and recording geological conditions, implementation of the approved monitoring plans prepared by the tunnel designers, application of the ‘toolbox’ for support determinations, confirmation that construction met the design intent and assessment of design queries at site level. The geotechnical construction team was required by the LCTP procedures and protocols to operate within the strictures of the ‘support toolbox’ provided to them.

The ‘support toolbox’ allowed for the circumstances where the ground conditions encountered did not conform to the expected conditions and if the ground encountered was not sufficiently addressed by the existing support designs then a revised support design could be prepared. There was, however, a detailed and proscriptive process for the preparation and approval of a revised support design involving a series of steps by TJH, PB, URS and RTA including auditing, approval, certification and verification of support re-designs.

3.2 INFERRED GROUND CONDITIONS AND SUPPORT FOR MC5B TUNNEL AND INTERSECTION

The pre-tender geotechnical investigation indicated that the intersection would encounter Class II Ashfield Shale and PB assessed the required support as rock bolts and fibre reinforced shotcrete (FRS). The MC5B tunnel was inferred to start in the Mittagong Formation with about 35m depth of cover and to climb gradually over its approximately 200m length to intersect the MCAA alignment with about 20m cover. From about Ch 70m the MC5B tunnel was predicted to be completely within inferred Class II Ashfield Shale, which had the following properties:

Strength	Medium to high strength, UCS 10MPa to 30MPa
RQD	75% to 95%
Q-value	3 to 5
Permeability	< 5 Lugeon with isolated higher flows at Mittagong/Ashfield contact
LCT Ground Classification (GSC)	LCT-G6

4 GROUND CONDITIONS AND SUPPORT FOR MC5B TUNNEL

Construction of the MC5B tunnel commenced in October 2004 with the sandstone/shale contact in the crown of the tunnel. The tunnel advance continued to mid December 2004 at Ch 67 with LCT-G7 ground classification and support consisting of the 6-bolt pattern at 1500 x1500 spacings plus spot bolts and 125mm FRS (MAR XI(A)).

4.1 MC5A DYKE AND TUNNEL SUPPORT

Ventilation tunnel MC5A was being constructed adjacent to, and more or less concurrently with, the initial advance in MC5B. On 26 October 2004, the Engineering Geologist mapped a dyke in the right (eastern) side, crown and the face of tunnel MC5A at Ch 485.5. The dyke was 0.8m wide and had the consistency of very stiff to hard clay. He reported the ground classification as LCT-G7 and recommended the ground support as MAR-XI. On the following day at Ch 495.5 the Engineering Geologist mapped the dyke as a major defect, aligned slightly oblique to the line of the tunnel with overbreak of 0.2 to 0.4m on either side of the dyke and faulted in the face. He assigned a ground classification as LCT-G9 and recommended support type MAR-VIII. The MAR-VIII support consisted of 200UC46 steel sets at 1200 centres, spot bolts with 75mm primary FRS and 200mm final FRS.

On 27 October 2004 the Senior TJH Project Engineer responsible for permanent civil works at the Marden Street site initiated a Request For Information (RFI). The RFI noted the Engineering Geologist’s recommendation for MAR-VIII passive steel set support and requested advice from PB as to the suitability of an alternative support based on rock bolts with increased FRS thickness because the LCT-G9 ground classification had not been expected and suitable steel sets were not available at that time.

On 29 October 2004 PB responded with a revised support design that consisted of a 200mm thick structural FRS lining to both crown and walls with additional sidewall rock bolts (Support type MAR-VII). This design was to be implemented between Ch 483 and Ch 498 and was considered suitable for Ground Classification types G8 and G9. The basis for this design is not known other than that it was considered as functionally equivalent to, and a structural replacement for, the use of steel sets in the poorest ground conditions. MC5A was, at that time, on a steep decline and ground conditions were expected to improve quickly as the tunnel passed through the Mittagong Formation into the underlying Hawkesbury Sandstone where reduced support would be required as the GC improved.

4.2 MC5B DYKE AND TUNNEL SUPPORT

Excavation of the MC5B tunnel commenced on 20 October 2004, was suspended at Ch 67 in mid December 2004 and resumed in late August 2005. The dyke encountered in MC5B appeared in the RHS sidewall at Ch 69.3 and persisted to Ch 103 where it re-entered the RHS sidewall. Figure 4 shows the multiple dyke occurrences in MC5B as it advanced towards the intersection with MCAA:

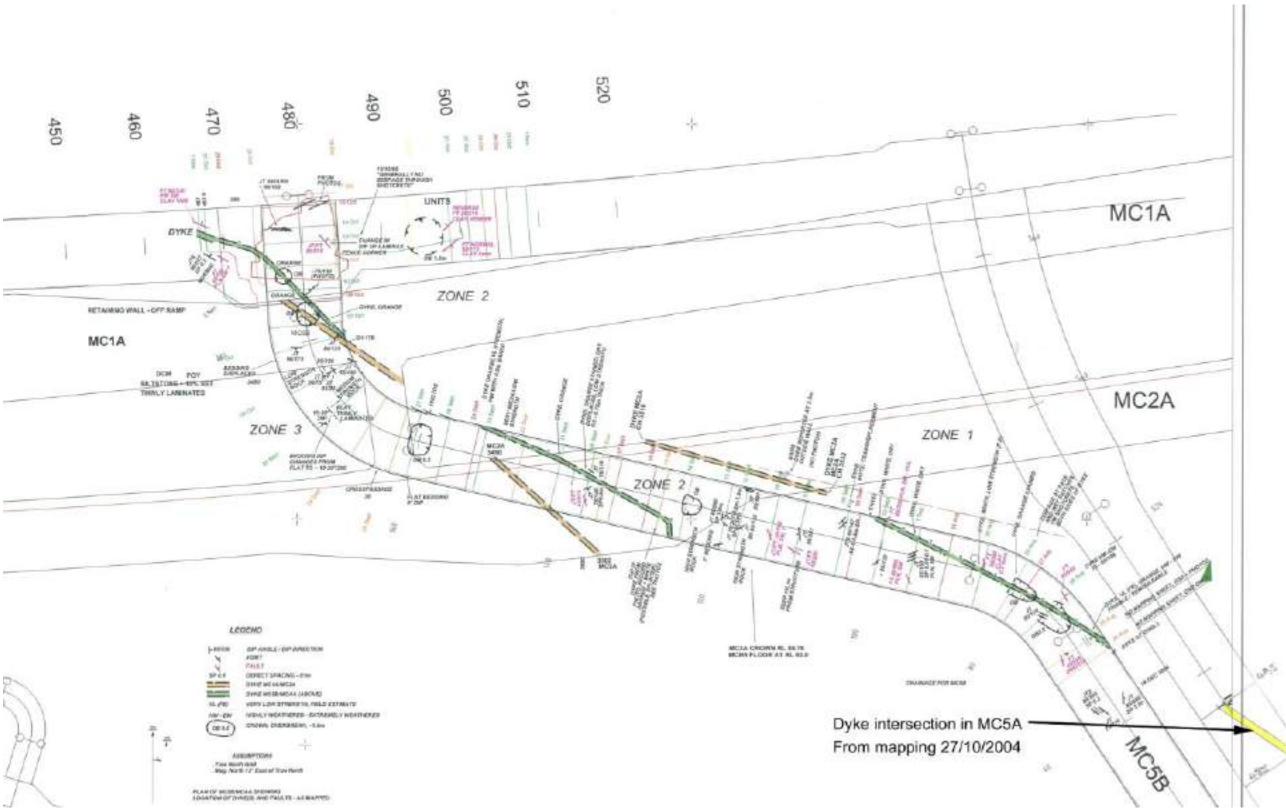

Figure 4: Dyke occurrences in the MC5B tunnel as it progressed towards the MCAA intersection

4.2.1 The MC5B Dyke

The dyke intersected in MC5A is unlikely to be the same dolerite dyke structure that crossed the MC5B/MCAA intersection at a higher elevation and some 130m away to the north-west. The geological mapping indicates the presence of two dykes, or, the bifurcation of a single dyke in a south-easterly direction, from the south-eastern side of the MC5B/MCAA intersection. This interpretation is supported by an historical photograph taken during the construction of the Gore Hill Freeway in 1991 (Ozroads), which shows a single dyke exposed in the central freeway excavation, to the south-east of the red brick Kerslake Apartment building.

It is therefore concluded that the single dyke encountered in the MC5B/MCAA intersection and exposed in the Gore Hill Freeway excavation in 1991 bifurcates to the south-east, as mapped on Figure 4. The dyke encountered by the Engineering Geologist in MC5A in October 2004 and in MC5B between Ch 69.3m and 102m is the northern splay of the bifurcated dyke. The dyke that was encountered in MC5B from Ch 126m towards the MC5B/MCAA intersection is the southern splay of the bifurcated dyke. It is interpreted that the point of bifurcation occurs behind the right-hand or eastern sidewall of the MC5B alignment around Ch 170m. From there the dyke extends in a north-westerly direction through the MC5B/MCAA intersection as a single structure, as mapped on Figure 4.

The southern splay of the bifurcated dyke and its extension through the MC5B/MCAA intersection was encountered, documented and photographed, between MC5B Ch 126m to 152m, 178m to 191m and for approximately 8m into the MCAA downdrive (Figure 4). Photos of the MC5B dyke are included as Figure 5 at Ch 152 and 186, left and right respectively. Typically the dyke margins were planar to irregular, intact sub-vertical boundaries with the host rock being Class III/IV shale showing no sign of significant alteration or weakening.

Figure 5: Dyke exposures in the MC5B tunnel at Ch152 (left) and Ch186 (right)

4.2.2 MC5B tunnel support

Following the dyke encounter in MC5A on 27 October 2004, PB's Principal Tunnel Engineer issued the instruction that when the dyke was exposed in MC5B, as expected from Ch 70 to 100, the structural shotcrete support alternative (MAR VII), with 200mm thick FRS and additional sidewall bolting, was to be implemented. Full face advance was an essential requirement of this support system with the passive shotcrete arch being an important component of the redesigned support type. Ground classifications along MC5B ranged from G7 to G7/G8 and G8 with the latter two classifications in dyke-affected sections as shown in Figure 6.

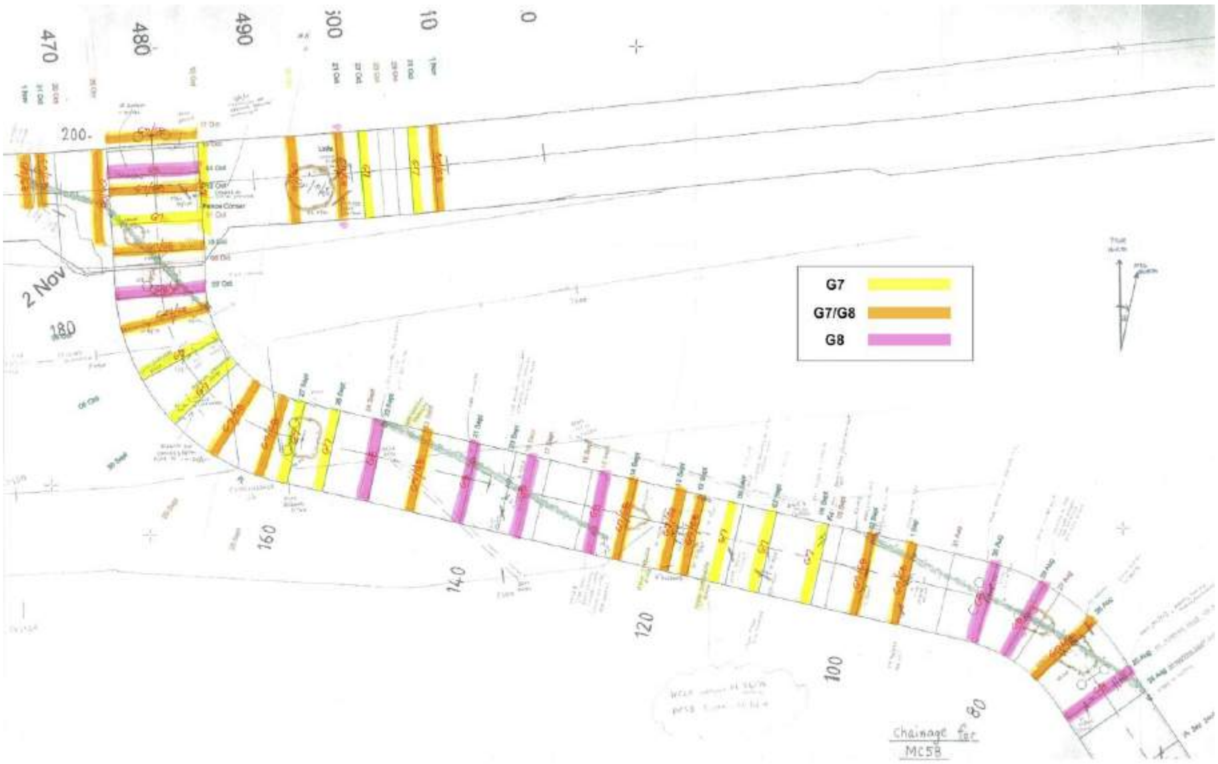

Figure 6: Ground classifications along the MC5B tunnel from the first dyke encounter at Ch 70

In accordance with the revised support instruction the structural shotcrete alternative was installed for the full cut length together with the 'cut one bolt one policy'. This policy arose in response to the fatal rock fall in Sydney's Cross-City Tunnel in August 2004 and mandated one round of pattern bolting must be installed as part of every single excavation sequence irrespective of span and ground conditions. This policy was a safety requirement of construction practice instituted by TJH independent of design requirements so that in contractual terms any failure by TJH to implement its own policy was not a failure to implement the contractual design.

The MARVII structural shotcrete support design, including full face advance, was implemented over the full length of the MC5B tunnel up to the point where it entered the MC5B/MCAA intersection. MC5B was successfully completed in dyke-affected G8 conditions with minor overbreak in the crown. The MARVII support type is shown in Figure 7.

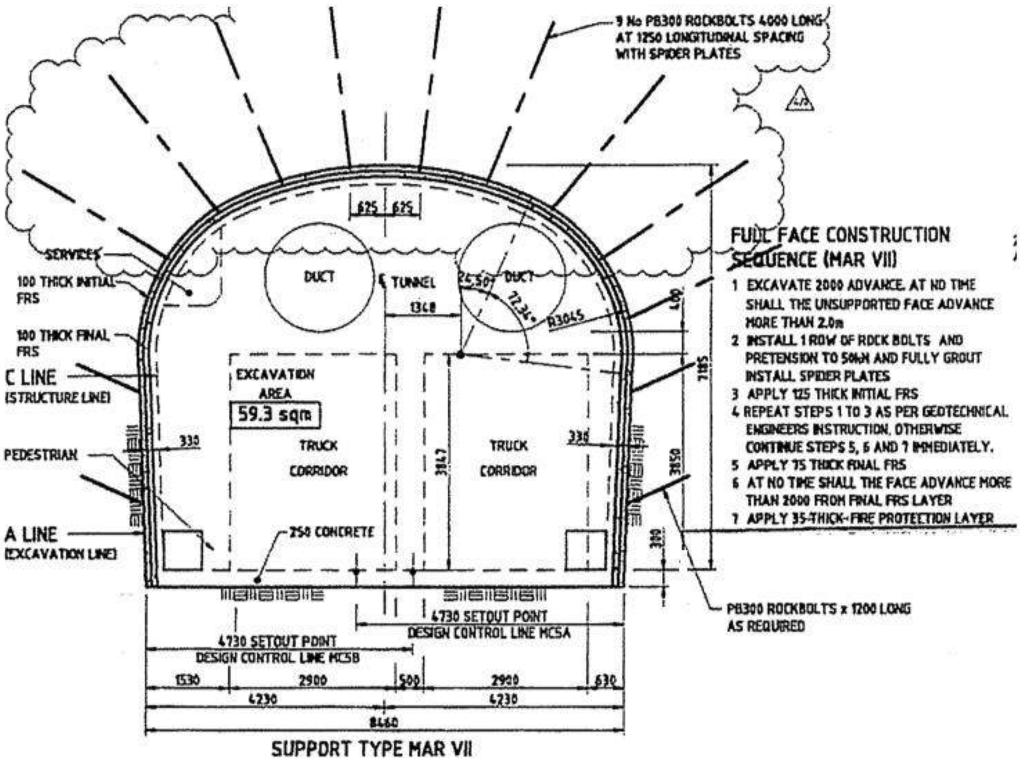

Figure 7: MARVII structural shotcrete and rock bolts support type designed as an alternative to the steel set support in the worst of the LCT ground conditions G8 and G9

5 THE MC5B/MCAA INTERSECTION

The MC5B/MCAA intersection was formed as the 8.5m high ventilation tunnel terminated at its junction with the 6.5m high MCAA exit ramp that was rising at a grade of 7.8% to merge with the northern lanes of the Pacific Highway about 130m to the east.

The construction sequence was that the MC5B tunnel continued through the intersection until it reached the north wall of MCAA on 16 October 2005. The excavation then broke out to the east and was advanced full face as the MCAA up-drive for a distance of almost 40m over a period of a week and a half. The up-drive underlay the Kerslake Apartments and the Longueville Road anchored retaining wall.

On the night shift of Thursday, 27 October 2005, the MCAA downdrive commenced with a 1m full-face, benched breakout to the west that was fully bolted in accordance with the design. The face was mapped and classified by the Engineering Geologist at the start of the Friday morning day shift. The downdrive was then progressed on the Friday day shift and the Sunday night shift as a partial face, benched advance of about 7m and then a further 1.5m on Monday 31 October. This exposed the dyke which was only partly bolted and without the complete floor-to-floor shotcrete arch lining. Because face mapping and ground classifications were carried out only between Monday and Friday the Engineering Geologist was unaware of these conditions and the extent of the advance and support until the start of the day shift on Monday 31 October. The GSC/GSD report for Monday 31 October recorded the downdrive classification as G7/G8 and noted the following in relation to the MCAA downdrive:

- Maintain 1m bolt pattern and shotcrete cycle;
- 200mm shotcrete required to the floor as soon as possible. Understand 1m benching ~ Thursday!!

Figure 8 shows the downdrive breakout on 28 October (left photo) and the partial face advance on 1 November 2005 (right photo) with instability on the left chamfer. The collapse occurred at about 2am on Wednesday, 2nd November 2005.

Figure 8: Downdrive breakout on 28 October (left) and the partial face advance on 1/11/ 2005 (right)

5.1 INTERSECTION SUPPPORT DESIGN

To the best of the authors' knowledge, based on the entirety of the evidence provided to the proceedings by PB, the design for support of the MC5B/MCAA intersection was not an engineered design in the sense that it was not based on detailed calculation of the type undertaken post-collapse to assess its suitability. Similar technology, available in the early 2000s, was apparently not employed in arriving at the design for the intersection. In post-collapse legal proceedings, commenced by TJH, it was contended that a listed series of some 26 documents comprised the support design for the intersection. These documents included design reports, technical memoranda, specifications, work method statements, RFIs, emails, mapping sheets and GSDs and PB drawings. What is conspicuously missing from this collection is the engineering design information for the MC5B/MCAA intersection that was described as 'Design Documentation' in the relevant consultancy contract. None of these documents refers to detailed engineering calculations carried out specifically for the MC5B/MCAA intersection. Subsequently the trial judge confirmed the extent of the design documentation which was without engineering calculations for the intersection.

Contractual requirements aside, it is difficult to conceive that a support design might be prepared for the intersection without carrying out an engineering analysis of the support components and the sequence of excavation. It appears that the support system that had been developed, and shown to be successful, for the dyke affected sections of the MC5B tunnel was continued into the intersection together with an augmented rock bolt pattern but without an engineering analysis. The role of experience and precedence as a factor in engineering design is important. However, they are necessary, but not sufficient, substitutes alone for engineering analysis and design in moving from a successful 2-D tunnel configuration to a more critical 3-D intersection. A series of 2-D analyses had been carried out, on behalf of PB, by Coffey using the PHASE2 finite element code and dealt with individual tunnel sections and the interactions for adjacent tunnels. There was no evidence of a 3-D analysis for the MC5B/MCAA intersection or any of the other LCT intersections in any of the available documentation.

In addition there were no drawings in the 'support toolbox' showing the installation of ground support and the sequencing of excavation for any intersection including the MC5B/MCAA intersection that collapsed.

6 CONCLUSIONS

Collapse of the Lane Cove Tunnel, undermining of a block of residential apartments, damage to roadway structures and underground utilities was a major failure in the construction of transport infrastructure. Fortunately there was no loss of life but there were substantial commercial and financial consequences, including lengthy litigation that is on-going, as a result of the collapse. That said, the collapse occurred during the closing stages of the project, which to the knowledge of the authors had otherwise uneventful.

The project was executed as a design and construction activity with independent over-sight. The collapse occurred as the works progressed towards the poorer geotechnical conditions along the Pacific Highway ridge, marked by weathered shale, geological defects (such as dykes) and reducing cover to the tunnels. It occurred as an L-shaped intersection between the MC5B ventilation tunnel and the MCAA exit ramp tunnel to the Pacific Highway was being converted to a T-intersection. The seeds of the collapse were sown over a weekend during a Friday day shift and a Sunday night shift when supervisory activity was at a reduced level and when there was a change in the method of advance from full-face to partial face excavation as described above in Section 5.

Tunnel support for the project was generally by means of conventional rock bolts and shotcrete, which was modified to rock bolts and thickened structural shotcrete for the MC5B/MCAA intersection. Intersection support was based on a design alternative to the use of steel sets that was introduced to cope with poorer dyke-affected shale conditions about a year earlier in the project. It appears that there were no engineering design calculations carried out for the 3-D intersection and that the support system installed for the intersection was based on continuing the structural shotcrete and rock bolts system, shown to have been successful for the dyke affected sections of the 2-D MC5B ventilation tunnel, across the MC5B/MCAA intersection.

Separate associated papers set out the cause or causes of the collapse, the contractual relationships involved in its design and construction (Burman et al, 2018a), the legal consequences and outcomes that followed the collapse (Burman et al, 2018b) and complete the forensic review of the LCT collapse.

7 ACKNOWLEDGEMENT

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LANE COVE TUNNEL COLLAPSE AND SINKHOLE A FORENSIC REVIEW - 2: POST FAILURE INVESTIGATIONS

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ABSTRACT

The Lane Cove Tunnel sinkhole in Sydney resulted from the collapse of a tunnel intersection being excavated at shallow depth in poor quality Ashfield Shale in November 2005. The collapse endangered the tunneling crew, an apartment block, a major roadway and buried infrastructure. Over the past decade the collapse was subject to a WorkCover investigation, to industrial prosecutions in the NSW Industrial Relations Commission and to civil litigation in the NSW Supreme Court, all of which were informed by a series of expert reports. An earlier first companion paper summarised the factual evidence of the collapse (Burman et al, 2018a). This second paper describes the forensic investigations that were carried out to assess the cause or causes of the collapse immediately after the collapse and subsequently for the purposes of civil litigation between the parties directly involved in the design and construction of the project. A separate third paper explores the contractual relationships involved in its design and construction and the legal consequences and outcomes that followed the collapse (Burman et al, 2018b).

1 INTRODUCTION

When there are failures in major civil infrastructure works two questions arise that need to be answered. The first is why did the failure occur so that the engineering profession can learn whatever the lessons are from the failure and its causation. The second, which follows closely, is who is to blame and who pays to redress the damages resulting from the failure. At its simplest that is the essence of forensic engineering as practiced in the Australian context and to a large extent causation is principally an engineering task whereas prosecution is at the interface between the engineering and the legal fraternities. This paper addresses causation in terms of the post failure engineering investigations.

2 FORENSIC INVESTIGATIONS

There have been 3 sets of investigations into the LCT collapse:

- The first was carried out by Emeritus Professor Ted Brown of Golder Associates as an independent investigation into the causes of the collapse (the **Brown Report**);
- The second investigation was carried out by WorkCover NSW pursuant to Sections 8(1) and 8(2) of the Occupational Health and Safety Act 2000 (the **WorkCover Report**); and
- The third was a series of investigations carried out by a number of local and international experts in relation to proceedings in the Equity Division of the Supreme Court of NSW as Thiess Pty Ltd and John Holland Pty Ltd-v-Parsons Brinckerhoff Australia Pty Ltd in 2016 (the **Civil Litigation Reports**).

2.1 THE BROWN REPORT

Dr Brown commenced his investigation 2 days after the collapse and delivered his report on 14 December 2005 (Golder, 2005), which addressed the considerable volume of material provided to him. His investigation consisted of interviews with some 18 site personnel, surface and underground inspections and consideration of an extensive range of project documentation and records. Dr Brown reached a number of significant conclusions relating to excavation and support processes and practice including that:

i) The design methodology for the LCT tunnels was in accord with the best practice and the designs were generally suitable for their purposes. Although he noted that no specific modelling was carried out of the intersection of the MCAA and MC5B tunnels where the collapse occurred;
ii) The available evidence indicated geotechnical and other risks were identified and mitigated by best practice measures for underground construction;
iii) TJH had appropriate and best practice processes for safe and productive underground construction, which were professionally and effectively implemented; and
iv) The role of the Geologist (PSM) in mapping and preparing Ground Support Determinations (GSDs) and the role of the Senior Tunnel Engineer (PB) in determining support requirements for mapped conditions or in liaising with the PB tunnel design team were executed satisfactorily during construction.

In relation to the collapse itself, and to a range of factors that possibly contributed to the collapse, Dr Brown expressed his concluded views as follows:

v) Groundwater was not present in the rock mass leading up to the collapse and was not a contributory factor to the initial rock falls and the propagation of the collapse;
vi) The shale at the site was not of such poor quality that low rock mass strength alone could be considered as a cause of the collapse;
vii) There was no available evidence to suggest that the dyke, weathered and low strength as it was, was weaker than the host rock mass or the cause of the collapse;
viii) The integrity and continuity of the roof was essential for stability and could be controlled by appropriate support measures. However, if an initial failure took place then progressive failure of the entire roof could occur and occur very rapidly;
ix) At the time of the collapse support had been fully installed in MC5B and the MCAA up-drive. Support in the MC5B down-drive was to Ch 467 with bolts installed and grouted up to 0.5m from the face; shotcrete thickness was unknown;
x) The effective span at intersections was obviously greater than the nominal spans of either of the intersecting excavations. Spans of 17 to 22m in weathered Ashfield Shale were outside the limits of precedent practice in that material; and
xi) The collapse arose from the following combination of factors that had not been present together at anywhere else in the project area:
    - The presence of the dyke
    - Orthogonal, close spaced jointing associated with the dyke
    - The presence of faults oriented so as to form unstable blocks
    - The large effective span and low rock cover
    - Inadequate support in the western side (MCAA downdrive) of the excavation

A decade later the authors are in broad general agreement with the majority of Dr Brown's conclusions but in Section 5.3 of this paper will elaborate and/or present differing views in relation to several factors and to the cause of the collapse based on additional evidence that has subsequently become available through the litigation process.

It is unclear whether Dr Brown knew, or considered relevant, the fact that the MCAA downdrive had been excavated by partial face advance whereas the MCAA updrive (and the intersection support design) had been by full face advance. It is not mentioned in his report although he highlighted the *level of support existing in the western side of the excavation at the time being inadequate to ensure the excavation's stability given the large effective span, the low rock cover, the presence of persistent vertical discontinuity (the dyke) transecting the excavation, and the poor mechanical properties of the overlying rock mass*.

The Brown Report contains a very comprehensive level of detail in respect of the factual data and in particular the geotechnical conditions that contributed to the progressive nature of the LCT collapse mechanism. The Brown Report is publicly available as part of the WorkCover documents (Vol 19, 25.pdf).

2.2 WORKCOVER INVESTIGATION AND PROSECUTION

The WorkCover investigation was carried out by Mr Nathan Hamilton, an inspector employed by WorkCover NSW. The investigation commenced on 3 November 2005, the day after the collapse occurred and continued through to September 2011 when proceedings were commenced in the Industrial Court of NSW against Thiess, John Holland, PB and PSM. Mr Kotze of GHD was engaged by WorkCover to provide expert opinion on matters relevant to the collapse. Mr Kotze carried out surface and underground inspections of the collapse site and subsequently produced a report on the collapse (GHD, 2007). Mr Hamilton's reports were by way of a prosecution brief of evidence (WorkCover, 2007) and statements of uncontested facts (Hamilton, 2007a -2007d) for each of the parties.

2.2.1 The WorkCover Report

The WorkCover report consists of a series of detailed formal interviews with staff members of TJH, PB and PSM and others who were directly connected with or had responsibility for the works that led to the collapse together with a collection of the documentation relevant to the design, construction and investigation of the collapse. There were 44 separate interviewees as follows:

- TJH 25;
- Thiess 3;
- John Holland 4;
- PB 5;
- PSM 3;

- Strata Control 1; and
- Kerslake Residents 3.

The interviews were conducted between December 2005 and September 2007 under Sections 65 and 66 of the OHS Act 2000, which have coercive powers. The WorkCover report contains a large amount of detailed information, in 45 volumes, provided by the interviewees but, being directed principally towards construction safety compliance, it does not reach any conclusions as to the cause of the collapse. It is, however, a useful reference document for factual design and construction information and for factors relating to the collapse. The WorkCover statements of uncontested facts and affidavits address the causes of the collapse and the culpability of the parties (WorkCover, 2007).

2.2.2 The GHD Report

Mr Kotze of GHD was commissioned by WorkCover on the afternoon of the collapse to provide geotechnical services and early the next morning carried out the first of his 3 collapse site inspections. His brief was to provide a report on factual observations and opinions on relevant matters particularly relating to potential danger to the health and safety of persons working in underground construction operations arising out of the collapse.

Initial inspections of the accessible areas at the collapse site revealed issues with the installation and anchoring of rock bolts in the roof of the MC5B tunnel section of the intersection. In 7 of the 10 bolts inspected there was little, and generally no, evidence of grout on the exterior of the plastic sheathed bolts. In several instances the CT end anchors had been overwound during installation to the extent that there was contact between the nut and the plastic sheath, which prevented grout penetrating into the annulus between the sheathed anchor and the shale. These observations raised concerns as to the suitability and effectiveness of CT anchored, fully grouted rock bolts installed into poor quality shale and in particular the use of mechanical as opposed to chemical end anchorage.

In the following months, collapse debris was removed from the MC5B section of the intersection in preparation for the deviated MCAA alignment and a total of 59 remnant bolts and bolt fragments were recovered. Inspection of these bolts revealed the following defects for bolts installed in the MC5B section of the intersection:

- 23 of the 39 bolts with recovered threaded ends had been overwound to the extent that end anchorage could not have been achieved during installation; this represented 60% of those recovered bolts; and
- 6 of those 23 bolts had been so overwound during installation that the plastic sheath was blocked preventing grout penetration into the annulus between bolt and the hole. This represented 25% of recovered threaded ends and suggested the possibility of 1 in 4 bolts installed may have had no grouted bond to the surrounding rock mass.

These statistics, if applicable to the entire intersection, and there is no basis to think otherwise, indicate a serious deficiency in design and construction activities. In addition Mr Kotze identified four conditions that were causative in contributing to the roof collapse and sinkhole (GHD, 2007), viz.;

i) Geological conditions. The roof of the intersection was characterised by low to very low strength (Ground Classes G7 and G8) Ashfield Shale, transected by a 600mm to 700mm wide, subvertical, diagonally through-going, highly weathered, low to very low strength (and low stress) doleritic dyke. The shale is closely fractured by subhorizontal bedding planes and by three intersecting joint sets, one of which is parallel to the dyke. Subvertical joints are locally continuous vertically and spaced as closely as 100mm to 200mm apart. Localised inclined fault planes were also mapped, providing further fragmentation potential;
ii) Inadequacy of tunnel roof support installations. The roof support system as-designed (MAR-VII) relied on the composite effects of a 200mm thickness of structural shotcrete from floor to floor, and arrays of CT rock bolts (Refer Burman et al, 2018a, Figure 4). At the time of the roof collapse, the application of shotcrete in the MC5B down-drive had not achieved full thickness and did not extend from floor to floor (Refer Burman et al, 2018a, Plate 4). Furthermore, as noted above a significant percentage of the CT rock bolts installed, had not achieved end anchorage or a grouted bond in the roof strata;
iii) Large span width of the intersection. The MC5B and MCAA tunnels are approximately 9m wide. The effective span width of their intersection however is up to 22m. In retrospect and given the information as to the inadequacy of down-drive support that emerged during the proceedings the 15m effective span of the MC5B down-drive at the time of the collapse is considered as the relevant and causative effective span;
iv) Proximity to the ground surface. The crown of the intersection was 13.3m below Longueville Road and 17m below the Pacific Highway off-ramp and ground level at the Kerslake Apartments. Furthermore, the tie-back anchors for the piled Longueville Road retaining wall had been drilled and installed to depths in the order of 2m above the crown of the MC5B/MCAA intersection.

The principal lesson from the GHD report is that CT rock bolts should not be used in poor quality rock masses or at least should be used under strict supervision, regular testing of production rock bolts and in an effective quality control regime.

2.2.3 Outcomes from the WorkCover Investigations and Prosecutions

It emerged during WorkCover interviews that THJ tunnelling crews had been having significant difficulties with tensioning and grouting of the CT rock bolts in the MC5B tunnel and in the MCAA/MC5B intersection. They had adopted the procedure that any failed rock bolt was to be replaced with another rock bolt directly beside the failed bolt. It was reported that on occasions the replacement rock bolts also failed to tension.

However, as reported in the decisions of the Industrial Court, the tunnel designers, PB and the geotechnical construction team, PB and PSM, were unaware of difficulties with the installation of CT rock bolts prior to the collapse. There was no documentation to the effect that tensioning or grouting issues with the CT rock bolts had been reported due to a break-down in the NCR system. TJH did not inform PB, and PB did not enquire, whether or not rock bolts installed in the MC5B/MCAA intersection were properly tensioned and properly grouted.

As is generally the case for industrial accidents, each of TJH joint venturers, PB and PSM were charged with offences under sections of the Occupational Health and Safety Act 2000:

- The TJH entities were charged with failure to ensure the safety and the freedom from risks to health of its employees, they pleaded guilty, were convicted and fined;
- PB was charged with failing to advise its employees and its design team about defective rock bolts and to withdraw its employees from the intersection. PB pleaded guilty, was convicted and fined; and
- The PSM entities were separately charged initially with failure to ensure the health, safety and welfare of its own employees and also the safety of nominated non-employees and subsequently with the risk of subsidence and damage of the ground surface resulting in the undermining of the unit block (Kerslake) and the additional risk to which unit block residents were exposed. PSM was convicted on both charges and fined.

The decisions in the WorkCover prosecutions were reached based on the evidence of the four causative conditions identified in the GHD report (Section 2.2.2 above). They relied substantially on the problematic role of rock bolts in the collapse. In doing so, Justice Backman was not exposed to the significance of the redesign of ground support that occurred in October 2004 when the MC5A tunnel intersected the dyke and ground classification G9 was determined (Burman et al, 2018a). The rationale for the redesigned support system (MAR VII support system) was a change from active to passive support through the introduction of structural shotcrete in lieu of steel sets as per the original PB design.

The MARVII support design was intended to provide essentially passive support with reduced reliance of the active component of support from rock bolts. It is the authors' view that, if this knowledge had been in evidence, the weight of the various convictions could well have been directed at strict compliance with the MAR VII support design by TJH rather than for the putative need for a new design and construction sequence. There was no evidence presented to the effect that the intersection support as per MARVII design would have been inadequate, if properly executed.

2.3 THE CIVIL LITIGATION REPORTS

In February 2012 civil proceedings were commenced in the NSW Supreme Court by Thiess Pty Ltd and John Holland Pty Ltd, as 1st and 2nd plaintiffs respectively, against two Parsons Brinkerhoff companies as 1st and 2nd defendants. Subsequently PSM and URS were joined in the proceedings as 3rd and 4th defendants respectively. Each of the parties retained geotechnical experts and a number of expert reports were prepared dealing with a series of issues relating to causation and the contributions of the parties to the collapse. In relation to causation there were several different computer simulations that provided forensic examinations of the design and construction factors that caused, or contributed to, the collapse. They were:

i) An *ABAQUS* simulation carried out by Dr David Beck on behalf of the plaintiffs;
ii) A *FLAC^{3D}* simulation carried out by Dr Mark Diederichs on behalf of PB;
iii) A *3DEC^{3D}* simulation carried out by Prof Dr Giovanni Barla on behalf of PB; and
iv) A modified *FLAC^{3D}* simulation carried out by Dr Brian Burman on behalf of PSM.

The Diederichs and Barla simulations were contained in their respective expert reports that had not been put into evidence at the stage of proceedings when PB settled with TJH. Confidentiality has been claimed on behalf of PB over the content of these reports that may be put into evidence in further legal proceedings arising from the LCT collapse. Consequently neither the Diederichs modelling nor that of Professor Balla can be included in this review of the LCT collapse and further presentation of the tunnel simulations is necessarily restricted to the modelling carried out by Drs Beck and Burman.

The Beck and Burman simulations differed in many respects but shared a range of similar features:

- They were all extensive 3D models that included a substantial length (~100m) of the MC5B tunnel, the MCAA/MC5B intersection, the constructed lengths of the MCAA up-drive and the MCAA down-drive using as-built survey data and surface topography;
- The geological conditions and structures, including the dyke structures, were derived from face/crown mapping, GSDs and the results of pre-construction geotechnical investigations;
- Rock mass parameters were derived from the LCT Classification System, GSI correlations with Hoek-Brown correlations and from published results for Sydney sandstone and shale formations (eg. Pells, 2004); and
- The excavation and installation of tunnel support followed the construction sequence documented in GSDs with typical material properties for rock bolts and specified parameters for fibre reinforced shotcrete (FRS).

2.3.1 The Beck simulation

Dr Beck carried out a detailed simulation of the sequential excavation of the intersection in 2010 on behalf of TJH prior to the commencement of civil proceedings. It, along with the other simulations, did not figure directly in the hearings but nevertheless does provide useful insights into the causation factors for the collapse. The details of Beck's simulation were not subjected to forensic examination by the parties and to that extent its bases and assumptions are limited to what was contained in his report (Beck, 2010).

Dr Beck describes his model as a 3D, strain softening, dilatant explicit finite element using higher order tetrahedral elements with a constitutive rock mass model based on the 'Menetrey and William' failure criterion approximating Hoek-Brown. The strength and deformational parameters were said to have been determined in an iterative calibration process related to observed conditions in the tunnel. In summary the relevant peak parameters were as listed in the following Table 1, which together with disturbance factors for residual parameters were related to disturbance factors for two plastic strain levels:

Table 1: Rock mass parameters adopted for the Beck simulation

Material	Intact UCS [MPa]	Rock mass *E* [GPa]	GSI	Disturbance Factor D at e_p	
				0.5%	3%
Dyke	5	0.53	25	0.3	0.6
Residual Shale	1	0.33	31	0.2	1.0
Shale V/IV	2	0.47	31	0.4	1.0
Dyke/Shale Contact	1	0.24	25	0.4	0.6

For rock bolts and anchor cables the yield and debonding stresses were set at 360MPa and 240MPa respectively. Concrete modulus for walls and piles was 10GPa. FRS parameters including those for aging of the shotcrete or for its post- cracking behaviour were not reported.

Beck simulated the as-built conditions for MC5B and MCAA up-drive construction and two scenarios for the construction of the MCAA down-drive; one that represented the as-designed condition and the other, which approximated the as-built conditions. Based on his description of the simulation, the results of both were generally similar and showed the following behaviour:

i) The first phase, around frame 60 in the model, corresponded to the period as the MC5B was approaching the end wall of the intersection when the MC5B passed near/through the dyke, inducing some local deformation around the dyke in the tunnel as well as the first very minor surface movements. The intersection was stable in the model.

ii) The second phase corresponded to the formation of the intersection by turning out the MCAA up drive in an easterly excavation when surface subsidence increased at approximately 50mm per week in the worst location and at a rate somewhat proportional to the growth in the span. The surface settlement was very localised. At this stage the model suggests that undermining of the weak zone around the dyke by the MC5B tunnel de-stresses the future intersection location, weakening the assembly of material above the future intersection, adversely affecting arching across the intersection and highlighting the importance of a weak zone around the dyke.

The results indicate a relatively small, loosened zone above the intersection, 2-4m deep adjacent the dyke, which in practice, with 4m+ closely spaced bolts and thick fibrecrete would normally have been sufficient to retain the failed material for this stage, if effective. The extent of the localised failed zone occasioned by the breakout for the MCAA up-drive is shown in Figure 1, which is from the Beck modelling results.

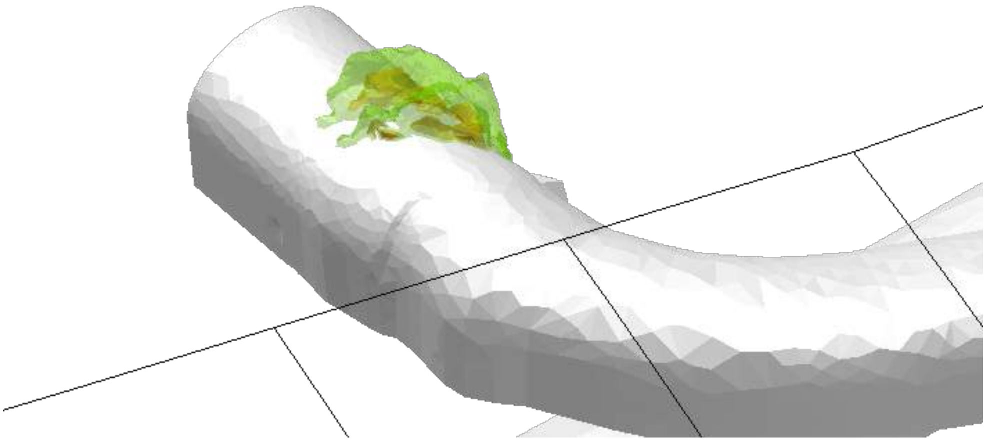

Figure 1: Iso-surface from Beck model showing displacements for up-drive breakout stage with displacements less than 150mm. The curved MC5B tunnel is shown at the end wall intersection with the MCAA ramp tunnel

A particular feature of the Beck model was that yielded elements were deleted when the displacement magnitude exceeded 275mm, which occurred over a very small volume firstly at steps 70-71 as the MCAA up-drive was advanced. The element deletion was done to approximate the effects of material loss from the walls of the tunnel arising from failure.

Dr Beck claimed that models without element deletion showed virtually the same global deformation and timing of the failure through to the ground surface provided that displacements greater than 200mm at surface and exceeding 1m at the tunnel were accepted as indicative of failure. He claimed that, in this case, element deletion slightly changed the interpretation of the development of the failure, better localised the timing of the collapse and resulted in a slightly larger and more realistic area of failure at the surface.

Figure 2 shows the extent of the yielded zone above the MC5B tunnel section of the intersection corresponding to the completion of excavation in the MCAA up-drive predicted by the Beck model with element deletion.

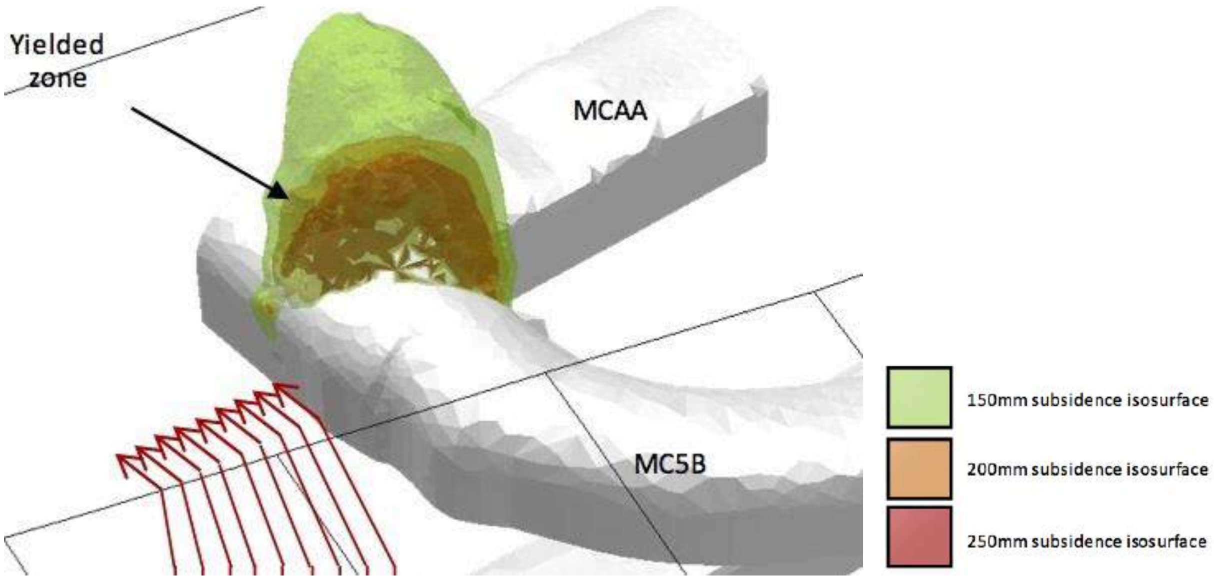

Figure 2: Iso-surface from Beck model showing displacements for the completion of the MCAA up-drive stage

The final phase of deformation corresponded to active failure and started as the MCAA down-drive turned out to the west. The rate of surface movements accelerated rapidly, having reached levels of over 50mm/day in the 3 days prior to collapse. Between 29 October and 2 November, the failed zone grew from a narrow zone local to the dyke surrounds into a thick zone covering much of the intersection and breaking through to the surface over an area closely matching the actual final failed zone. Significant rock mass damage was predicted as the down-drive excavation advanced and anecdotally this appears to correlate with observations as the failure initiated. The final collapsed zone compared favourably with the actual extent of the failure being slightly smaller. Figure 3 shows the extent of the yielded zone above the MC5B tunnel section of the intersection corresponding to the completion of excavation in the MCAA down-drive predicted by the Beck model with element deletion.

Beck's simulation of the as-designed excavation for the MCAA down-drive, while generally similar to the as-built conditions, predicted more extensive damage to the rock and a larger sinkhole. Given that the as-built tunnel support was deficient compared to the as-designed support this result appears anomalous and suggests that the support conditions as modelled may not have been fully realistic particularly in respect of the FRS. The authors observe that the collapse sequence initiated during the MCAA up-drive breakout and developed progressively to encompass the entire area of the intersection but did not include the MCAA down-drive. The simulated collapse sequence is in conflict with the actual collapse sequence, which initiated in the MCAA down-drive, and developed back to the intersection. The reasons for this mismatch are unknown but, in the authors' view, may be associated with the element deletion process that is somewhat arbitrary and subjective by its very nature.

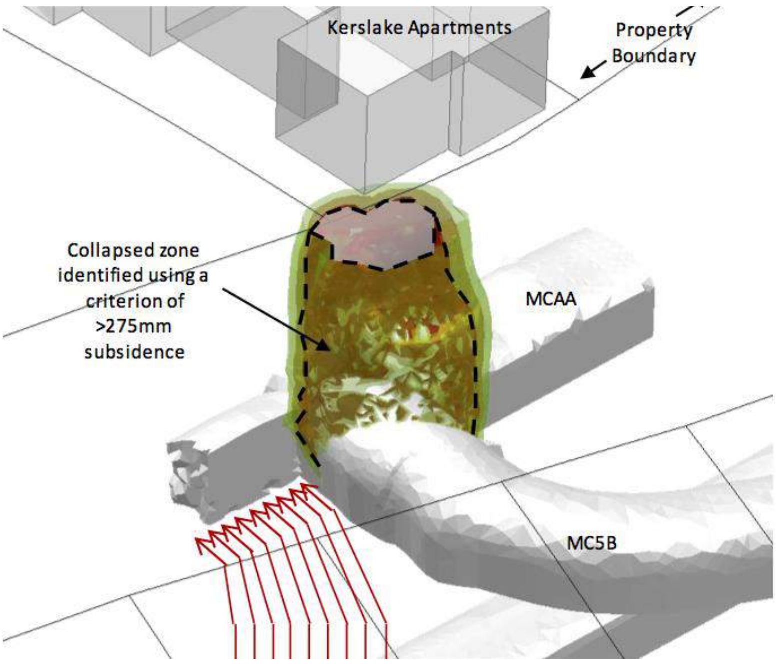

Figure 3: Iso-surface from Beck model showing displacements for the final as-built stage of the MCAA down-drive excavation with element deletion

Regardless, in the authors opinion what Beck comprehensively demonstrated was that if the intersection had been designed by normal engineering methods, including modelling and calculation, the inadequacy of the as-designed support system would have become apparent to the tunnel designers for the geological and geotechnical conditions that Dr Beck assumed. These assumed conditions were only partly revealed and appear to have been overly pessimistic particularly in relation to the properties of the dyke and its surrounds.

2.3.2 The Burman simulation

This simulation was carried out in response to the modelling done on behalf of PB, over which as noted in Section 5.3 above confidentiality has been claimed. It did not form part of the evidence that was put during the proceedings because of rules relating to discovery of evidence in the NSW Supreme Court. However, the results of the simulation are relevant to this forensic review of the collapse and are included for completeness.

The Burman simulation modelled all of the relevant underground and surface structures including the as-built MC5B tunnel from Ch 85 to the intersection at Ch 198, the as-built MCAA up-drive to Ch 502 and as-designed and as-built down-drive to Ch 508.5 and the surface topography including the exit ramp to the Pacific Highway. The Beck and

Burman models were constructed from the same survey dataset. The Burman model was 350m square in plan and over 100m in vertical thickness with a finer meshed zone 70m square centred on the intersection.

The overall geology, as for Beck, was based on geotechnical investigation results and locally on tunnel mapping and GSDs, which were modelled as 4 geological variants. One set was based on GSDs as mapped and the other set with local geology degraded by half a class unit for the LCT Classification System so that for this second variant conditions actually mapped, as say, G7 were modelled as G7/G8; the objective being to examine the effects of overly optimistic field assessments.

For each geological variant there was then two versions of dyke geology; one as mapped during construction (denoted by M) and the second alleged from post-failure geotechnical investigations to have shown poor quality shale conditions associated with the dyke (denoted by P). Dr Beck had assumed that the poorer shale conditions from the ground surface were associated with the dyke. This was at odds with the interpretation of dyke conditions made by Mr Kotze. The 4 geological variants were the 2 as-mapped sets of G7/G8 M and G7/G8 P and the 2 postulated sets G8 M and G8 P, which included the putative halos.

The width of the dyke for the G7/G8 M model was about 0.7m. For the G7/G8 P model within the intersection and at about intersection level the dyke itself was the same width and associated halo zones were about 1.5m and 1.0m respectively to the left and right of the dyke giving a total dyke affected width of about 3.2m. There was no factual evidence that would have supported this width of dyke plus halo (refer Figure 4). The width of the dyke plus halo in G7/G8 P and G8 P models was excessive. The G7/G8 M and G8 M models allowed for a realistic width of the dyke. Figure 12 is a digital representation of the dyke and halo combination for the geological models (G7/G7 P and G8 P) in the FLAC3D analyses and is shown together with the MC5B/MCAA intersection and compared to actual conditions for the dyke.

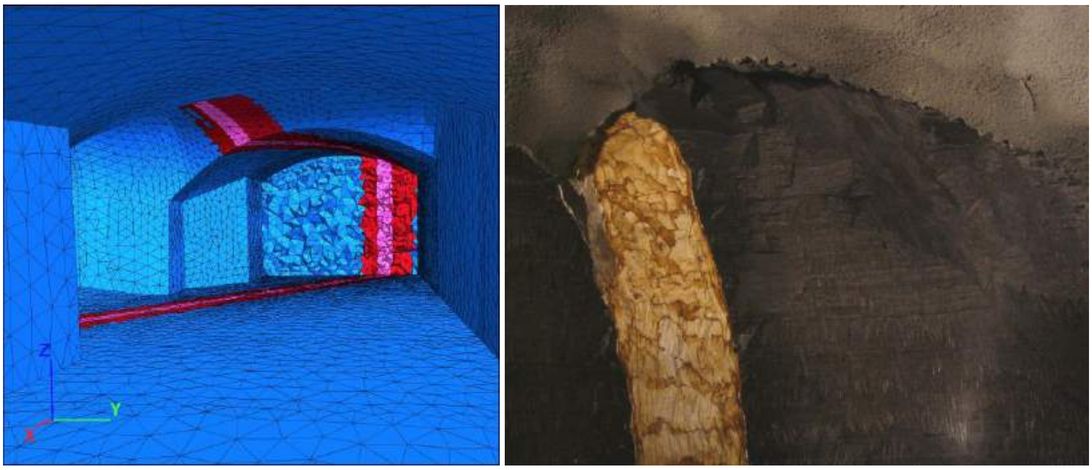

Figure 4: Dyke and alteration halos in the "As Encountered" geological models from the *FLAC^{3D}* analyses G7/G8 P and G8 P showing the view looking at the end wall of the MCAA down-drive (the dyke is shown purple and halo is red). The photo to the right shows the dyke exposed in MCAA down-drive at Ch 467 with no halos and with blocky shale margin to the dyke

Mr Kotze disagreed with Dr Beck's inclusion of a 'halo' effect associated with the dyke on the basis that it was a misinterpretation and overly pessimistic estimation of the intersection geology as it was recorded at the time. The halo hypothesis conflated a sub-metre dyke into a 3.2m wide weakness and in the authors' view the results for the G7/G8 P and the G8 P models should be viewed as unrealistic.

The Burman model can be described as a 3D, strain softening, non-dilatant finite difference continuum with a constitutive rock mass model based on the Hoek-Brown failure criterion. Strength parameters were determined by reference to GSI values assigned to materials classified in accordance with the LCT classification system and with strain softening defined by a range of disturbance factors. Deformational parameters were selected from published data on Sydney Basin formations with calibration based on a dyke-affected section of the MC5B tunnel. The relevant rock mass parameters were as summarised in Table 2.

Table 2: Rock mass parameters used in the Burman simulations

Material	LCT	Intact UCS [MPa]	Rock mass E [GPa]	GSI	Disturbance Factor D Peak	Disturbance Factor D Residual e_p 2%
Dyke		0.3	0.15	80	0	0.2
Shale III	G7	6	1.0	55	0	0.5
	G7/G8	4	0.3	45	0	0.4
Shale IV	G8	2	0.5	40	0	0.3
	G8/G9	1.5	0.75	35	0	0.2
Shale V	G9	1	0.1	30	0	0.1

The model provided a comprehensive set of parameters to model the tunnel support elements that covered the properties of FRS as specified by PB, rock bolts and the rock-FRS interface. FRS properties were defined in terms of tensile and compressive strengths and stiffness, the latter two being time dependent to account for curing of the shotcrete at an average excavation rate of 2m/day for the intersection. There was, however, an issue with the then current version (v5) of *FLAC^3D^* in that the FRS could only be modelled with structural liner elements, which in Version 5 were linearly elastic, and hence effectively had unlimited strengths and were incapable of failing. This particular feature was a characteristic of *FLAC^3D^* v5. Liner elements were rigidly connected within each of the MC5B and MCAA tunnels and were unconnected in the MCAA breakout for up-drive and down-drive to simulate the FRS 'cold joint' between the MC5B and MCAA shotcrete liners.

Rock bolts were modelled as per the PB specification with 25mm diameter bolts in 45mm holes pretensioned to 50kN and yielding at 300kN. Grout had a 28-day strength of 40MPa with strength and modulus both age dependent.

Insitu stresses were assumed to be vertical overburden and horizontal 1.5 times the vertical stress. The excavation cycle was simulated in the following sequence:

- Excavate to the extent consistent with the longitudinal bolt spacing by nulling tunnel elements within that range and solve by cycling to equilibrium;
- Install FRS with grout initially at 6-hour strength, increment aged parameters for previously placed grout. Install rock bolts pretensioned to 50kN and solve by cycling to equilibrium;
- Repeat the excavation and support installation steps.

Two sets of support conditions were considered. The first in accordance with the PB Final Design was As-Designed (AD); the second was As-Built (AB) by TJH and included any and all deviations from the Final Design and also considered 2 variants for the MCAA excavation sequence. The first in accordance with the PB Final Design for the MCAA tunnel was Full Face (FF); the second was As-Excavated (AE) by TJH and included any and all deviations from the Final Design and in particular the use of partial face advance by TJH for the MCAA down-drive. In all variants of geology, construction and excavation there were 16 possible scenarios.

The intersection models were non-linear in respect of limitations on rock strengths, liner-rock interface, rock bolts and consequently were path-dependent. This means that the model results were dependent on the sequencing of events and that the end results were affected, but to an uncertain extent, where actual processes such as liner cracking were not included.

The restriction of the *FLAC*^{3D} v5 liner elements to solely elastic behaviour was judged as unacceptable in any realistic model of the tunnels and the intersection. With the assistance of Dr Gareth Swarbrick of PSM a FISH function was developed that accounted for cracking of the shotcrete lining together with the associated reductions in flexural strength, flexural stiffness and with the redistribution of stresses from cracked liner elements. The liner material properties were based on PB's Shotcrete Specification (Parsons Brinkerhoff, 2005) as listed in Table 3. The introduction of realistic inelastic behaviour of the shotcrete lining by way of cracking and softening of flexural stiffness has a profound effect on tunnel displacements. By way of example the vertical displacements of the crown of the MCAA down-drive for the most pessimistic geological conditions (G8 P) increased from 75mm with an infinitely strong elastic liner to 175mm when liner cracking and softening were introduced. In the authors' view the assumption of an elastic liner that is incapable of cracking and failing is unrealistic and unsatisfactory particularly in the context of a forensic analysis.

Table 3: Specification requirements for strength and stiffness of temporary T-type steel fibre reinforced shotcrete

Parameter	Age	Minimum Requirement
Compressive strength	24 hours	10MPa
	7 days	19MPa
	28 days	26.5MPa
Flexural strength (EFNARC Beam test)	28 days	3.2MPa
Residual flexural strength	28 days	
1mm deflection		1.9MPa
3mm deflection		1.3MPa

The critical condition for cracking of the MC5B liner was the section of the liner near the centre of the MCAA tunnel and at the edge of the MC5B liner exposed when shotcrete forming the MC5B sidewalls was removed for the breakout excavation for both the up-drive and the down-drive. These critical sections were on opposite sides of the MC5B centreline; the up-drive on the eastern side and the down-drive on the western side. The critical condition occurred when shotcrete on the MC5B sidewalls was removed and the first full-face excavation of the MCAA tunnel occurred. The conditions were critical for both the up-drive and the down-drive but more so for the down-drive than for the up-drive. The maximum fibre stresses at the critical sections of the MC5B liner for the up-drive and down-drive are listed in Table 4 for the support system as-designed (AD) by PB and excavated as full face (FF) advances. The elastic liner modelling substantially underestimated the extent of liner cracking for the PB support design at all stages of construction and for the full gamut of geological conditions.

The liner stresses for the *as constructed* (As-Built and As-Excavated, AB AE) models were similar to, but less than, the comparable liner stresses for the corresponding As-Designed Full Face cases, shown in Table 4, except for the G8 P model. Maximum fibre stresses in the MC5B liner under G8 P (AB AE) conditions range from 7.8MPa for the 1st stage up-drive breakout, 12.5MPa for the 1st stage down-drive breakout and 17.8MPa for the final stage down-drive advance prior to the collapse.

Table 4: Maximum tensile stress on the surface of the MC5B shotcrete liner at first breakout excavations for up-drive and down-drive for the support system *as-designed* by PB (As-Designed and Full Face Excavation, AD FF)

Model	**Maximum fibre stress on MC5B liner surface and associated (Syy) component** (MPa)	
	Up-drive liner [firstexcavate_updrive stage]	Down-drive liner [firstexcavate_downdrive stage]
G7/G8 M AD FF	3.4 (2.0)MPa	3.4 (3.5)MPa
G8 M AD FF	4.0 (3.0)MPa	3.9 (3.4)MPa
G7/G8 P AD FF	3.65 (2.2)MPa	6.4 (5.1)MPa
G8 P AD FF	5.5 (4.2)MPa	6.8 (5.3)MPa

Figure 5 shows the extent of cracking in the MC5B liner for the final excavation advance of the MCAA down-drive for *as constructed* (As-Built As-Excavated) conditions for the G8 P model. There were no construction reports, and there was no physical evidence, of extensive cracking of the shotcrete liners having occurred and this can be interpreted as yet further confirmation that the G8 P model was unrealistic in its modelling of the dyke and the postulated halos.

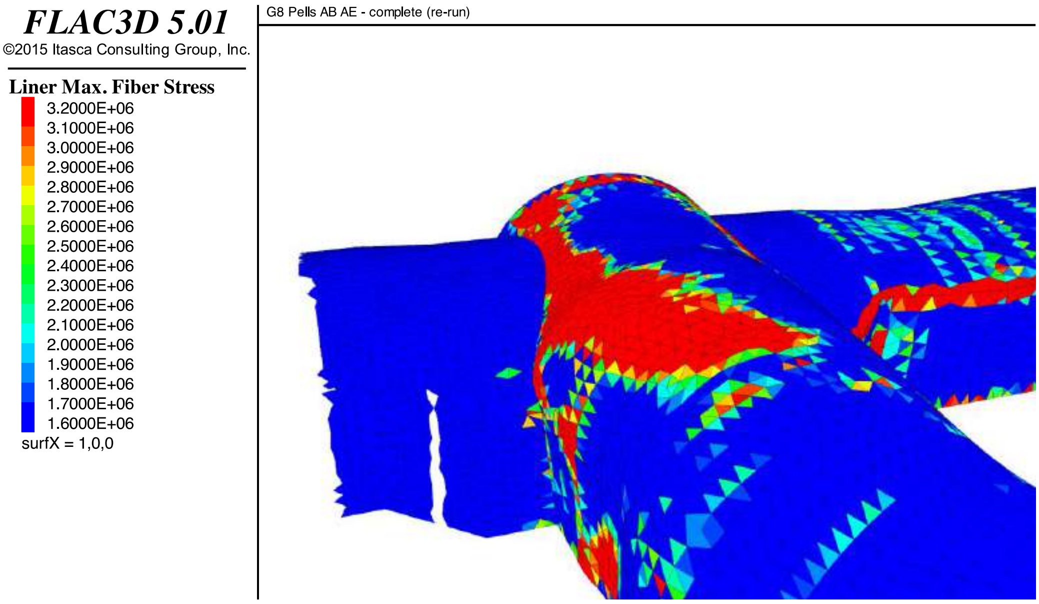

Figure 5: Maximum fibre stresses in MC5B liner for first stage excavation of the MCAA down-drive for G8 P geological conditions. The maximum fibre stress in the down-drive side of the MC5B liner was 17.8MPa. Areas coloured in red exceeded the tensile strength of FRS and would have cracked.

Based on the simulation results there was no evidence of overloading of rock bolts under the assumed *'as constructed'* conditions and for any of the postulated geological conditions. This condition arises because the structural stiffness of the rock bolts was less than that of the structural shotcrete, which therefore acted to provide the major component of the required support.

Maximum liner displacements for the simulation of *as designed* (AD FF) conditions, G8 P geology, and for the complete down-drive excavation were a maximum displacement in the MC5B roof of about 20mm and about 10mm for the down-drive liner. Under the same set of conditions vertical displacements of the rock surface were in excess of 180mm in the roof of the MCAA down-drive along the trace of the dyke and in excess of 40mm in the roof of the MC5B tunnel north of the dyke. The rock displacements exceed the corresponding liner displacements by a substantial margin indicating penetration of the tunnel roof through the liner. Interpenetration of the rock into the tunnel void is physically impossible without failure of the liner but is not precluded within the numerical model. Although the analysis for the final excavation stage was able to execute to completion, the large rock displacements likely indicated the onset of roof failure along the dyke.

For the G8 P *as constructed* (AB AE) conditions and for the fully complete down-drive excavation stage the maximum liner displacement was 30mm in the MC5B roof and the displacement of the down-drive liner was about 10mm. The G8 P *as constructed* simulation (AB AE) did not result in numerical instability or in a failure to complete at the final excavation stage.

The dyke transects the MC5B/MCAA intersection and is aligned more or less diagonally across the MCAA down drive. The dyke was not present in the MCAA up drive. Hence it was to be expected that movements in the crown of the roof would be greater for the down-drive than for the up-drive and if a collapse were to occur then it would likely occur in the down-drive excavation. This is, of course, what actually occurred in the early hours of 2 November 2005.

That said, there were large vertical displacements of the rock surface along the trace of the dyke and in the down-drive. These displacements were up to 180mm vertically and 60mm horizontally along the line of the MCAA tunnel for the worst geological conditions (G8 P) and for *as designed* construction (AD FF) . Although the final excavation stage was completed these relatively large rock displacements are likely indicators of the onset of roof failure along the right (northern) side of the dyke. Corresponding vertical rock displacements for *as constructed* conditions (AB AE) were 13 to16mm for G7/G8 M geological conditions and up to 30mm for G8 M geological conditions.

The development of displacements on the roof of the MC5B and the MCAA tunnels as the excavation sequence proceeded provides a quantified insight into the effect of the dyke on tunnel support conditions as well as implications for the design of support for the intersection. Table 5 presents the simulated horizontal and vertical displacements of the 3 points on the roof of each of the MC5B and the MCAA tunnels where the largest vertical displacements occurred as the excavation proceeded from Ch 186 at the opening of the MC5B tunnel into the intersection and beyond. The zero point for vertical displacement of each of the subject roof points was taken as at the time when the excavation of the intersection commenced at Ch 186 in the MC5B tunnel.

Table 5: Simulated X and Z displacements of points of largest vertical displacement in each of the 3 intersection tunnels due to excavation of the intersection for the worst geological conditions G8P. The shaded sections indicate the extent of excavation for the intersection.

Excavation Stage for the Intersection	X displacement (horizontal east-west; mm)			Z displacement (vertical; mm)		
	MC5B	MCAA Up-drive	MCAA Down-drive	MC5B	MCAA Up-drive	MCAA Down-drive
Start of Excavation of the MC5B tunnel at Ch 186	**0**	**0**	**0**	**0**	**0**	**0**
MC5B_complete at Ch 198	8	**-10**	**23**	-36	**-9**	**-12**
Firstexcavation_updrive at Ch 485	10	-6	24	-38	-21	**-13**
495to501 (updrive complete)	10	-4	24	-42	-27	**-15**
Firstexcavationdowndrive at Ch 475	10	-4	58	-44	-27	-171
Complete at Ch 467	10	-4	62	-48	-30	-180

The results of the Burman simulation under G8P conditions provide the following observations in relation to the performance of the support as designed for the intersection:

i) The largest vertical displacements in the roof of the MC5B tunnel exceed those in the MCAA up-drive by over 50 percent at all stages of excavation of the intersection and since the same support was installed in both the MC5B tunnel and the MCAA up-drive the greater MC5B roof displacement results from the presence of the dyke in MC5B and its absence from the MCAA up-drive;

ii) As the MC5B tunnel advanced into the intersection the points in the roof of the MC5B and MCAA down-drive that were to experience the largest vertical displacements moved horizontally towards the east in an asymmetric fashion (overall movement towards the east) due to the existence of the dyke in the MC5B and MCAA down-drive tunnels and its absence from the MCAA up-drive; and

iii) Breakout and initial excavation for the MCAA down-drive caused substantial increases in the horizontal (~1.5x) and vertical (~11x) displacements in the roof of the down-drive but had negligible additional effect on the MC5B and MCAA up-drive tunnels. There was negligible interaction between the MCAA down-drive excavation and the remainder of the intersection openings. These movements are directly related to the presence of the dyke in the down-drive and its exposure throughout the roof of the down-drive.

These displacement results indicate that the intersection acted substantially as 3 separate tunnel excavations and there was a hierarchical response to excavation related to the geometry of the dyke structure relative to the tunnel axis and with only limited interaction:

- The dyke-free MCAA up-drive showed least roof movements;
- The MC5B tunnel with the dyke aligned across the tunnel axis moved to an intermediate extent; and
- The MCAA down-drive with the dyke tending towards an alignment sub-parallel to the tunnel axis showed greatest roof movements.

For the more realistic G7/G8 geological conditions, the pattern of roof displacements was generally similar, but significantly muted. As noted above it is considered by the authors that the G8P geological conditions are unreasonably conservative in relation to the extent and the properties of the putative 'halo' associated with the dyke structure. The implication for the *as designed* intersection support is that, given the influence of the dyke structure on roof displacements as simulated, the support system should not have been more or less uniform across the intersection. It should have been skewed so that there was a relatively more robust support for the down-drive and with similar support for the up-drive and the MC5B tunnel. The extent to which the intersection support should have been skewed towards the down-drive as compared to the MC5B and the up-drive being determined by the condition of the dyke at those locations.

2.3.3 Summary of conclusions from the simulations

1) There would have been cracking of the MC5B and MCAA down-drive shotcrete liners for all geological conditions apart from the most favourable geological model (G7/G8 M) under *as constructed* conditions and for all geological conditions under *as designed* conditions. There was a contractual requirement for long-term durability (100 year design life) of the support system, which could not be satisfied based on the results of the modeling carried out by Dr Burman. For major civil infrastructure there must necessarily be a margin of safety against adverse effects and against instability. There was no such margin available for PB's Final Design if it had been fully implemented;
2) There was no evidence in the modelling results for conditions that had been postulated as precursors to the collapse including overloading of the unbolted MC5B back wall and the western edge of the intersection, localised overloading of rock bolts in the northwest corner of the down-drive and catastrophic numerical instability at the penultimate stage of the down-drive. The results from modelling did not support any of those mechanisms;
3) There was the potential for horizontal and vertical displacements of the rock surface along the trace of the dyke in the MCAA down-drive that were substantial for the most pessimistic assessment of geological conditions associated with the dyke and were significant for more realistic dyke conditions. Accordingly the support system for the down-drive section should have been more robust than that for the MC5B and up-drive sections of the intersection. For example a hybrid support system in which structural shotcrete with rock bolts was installed in the MC5B tunnel and the MCAA up-drive was paired with the use of steel sets in the MCAA down-drive could have prevented the collapse. Additionally such a support system may have ensured that the complete support system floor-to floor was installed in the down-drive; and
4) Both the Beck and Burman simulations made use of continuum models for which the collapse that actually occurred could not have been predicted without some form of arbitrary intervention in the modelling process which of itself would pre-determine the collapse mode. In that respect the Beck and Burman simulations provide different outcomes; the Beck model indicating collapse generally within the confines of the MC5B area of the intersection and the Burman model indicating critical conditions in the down-drive section. Irrespective of these constraints it is the authors' opinion that some form of engineering calculation and/or modelling should have been included in the design process for the intersection.

3 CONCLUSIONS

Dr Brown in his report used the terms "incident' and "subsidence" to described the LCT collapse possibly suggesting that it was a relatively minor failure in the context of international underground construction activity. Be that as it may, the LCT was a significant event for Australian infrastructure works and was properly treated as such in the post failure investigations, which took place over the decade following the collapse.

Post failure investigations were carried out on behalf of the Contractor immediately after the collapse by an international rock mechanics expert into the causes of the collapse. Safety aspects were the subject of a WorkCover investigation commenced the day after the collapse and continued for several years. A series of expert reports were prepared by some 7 local and overseas geotechnical engineers who between them carried out 4 separate detailed numerical analyses of the collapse, only one of which for various legal reasons was considered as part of the subsequent proceedings in the NSW Supreme Court.

Two of the 4 numerical analyses have been presented in some detail. Interestingly neither of these simulations reproduced the failure as it actually occurred although both would have served useful roles if they had been carried out as part of the design of the intersection. In retrospect it is unrealistic to expect that simulations based on continuum models might have been able to reproduce the type of progressive unraveling failure mechanism that actually occurred. Mr Peck, an expert for TJH, made the prescient observation in his expert report that …*the dyke and its associated blocky rock would need to be included in the model*. To do so properly would have required the use, in part at least, of a discontinuum model and possibly a 3D coupled FEM-DEM simulation (Zhao et al, 2018). That would not have been possible as part of the LCT design process and would be problematic even today. A DEM model was undertaken as part of the post failure simulations but its results are presently embargoed for legal reasons.

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LANE COVE TUNNEL COLLAPSE AND SINKHOLE
A FORENSIC REVIEW - 3: THE LEGAL AFTERMATH

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ABSTRACT

When the roof of the MC5B/MCAA intersection collapsed in the early hours of Wednesday 2 November 2005 and a sinkhole emerged beneath Kerslake Apartments it set in train a process, which culminated in a NSW Supreme Court hearing, that was completed a decade later (Burman et al, 2018a). In the intervening period WorkCover prosecutions were taken against the Contractor, the Tunnel Designers and the Geotechnical Engineer (Burman et al, 2018b). This paper deals with the civil litigation in terms of the resulting judgment and the restricted evidentiary context in which the judgment was reached. It is important that the geotechnical engineering community learns not only the technical lessons from this collapse but also comes to appreciate the commercial and reputational consequences as well as the legal ramifications of such failures.

1 INTRODUCTION

It is a virtual certainty that a failure in a civil infrastructure project will involve litigation. Litigation is uncertain, costly and time consuming and it is also a fact increasingly that more and more such matters are settled prior to the culmination of the court proceedings and that fewer and fewer such matters are determined through judicial proceedings. Where proceedings are settled prior to judgment, settlement almost inevitably involves confidentiality of the proceedings so that the facts of the matter are sealed in corporate files and hence are inaccessible to the engineering profession by way of case histories.

The Lane Cove Tunnel (LCT) proceedings involved 4 parties. Thiess Pty Ltd and John Holland Pty Ltd, known collectively as TJH, were the 1st and 2nd Plaintiffs. Parsons Brinkerhoff Australia Pty (PB) and Brinkerhoff International (Australia) Pty Ltd were 1st and 2nd Defendants, Pells Sullivan and Meynink Ltd (in liq) (PSM) was the 3rd Defendant and URS Australia Pty Ltd (URS) the 4th Defendant; they were respectively Tunnel Designer, Geotechnical Engineer and Independent Verifier. During the proceedings TJH settled their claims against PB and URS and only the dispute between TJH and PSM remained to be decided. Hearings commenced in mid-July 2015 and were completed over a four-week period; the decision was handed down in early March 2016 more than a decade after the date of the collapse.

2 SUPREME COURT JUDGMENT

It is fortunate for this forensic review of the circumstances and the causes of the collapse that the legal proceedings were completed and judgment rendered because the proceedings and the full extent of the evidence are in the public domain and hence available for analysis and publication. Increasingly construction related matters are being settled prior to completion of the legal proceedings and the material contained within the court records remains confidential, publically unavailable and hence inaccessible to the engineering profession. In this instance it has been possible not only to present the majority of the engineering information relevant to the collapse but also to consider the proceedings from a legal perspective. This is important because while we, as geotechnical practitioners, need to learn the technical lessons from failures we also need to learn the consequences on engineering practice of failures to meet contractual, professional and legal standards.

For the avoidance of doubt and for the assurance of those geotechnical practitioners who believe that court judgments are in some way sacrosanct we observe that it is not at all uncommon for such judgments to be scrutinised, analysed and in some cases criticised by members of the legal profession. Indeed that is at the heart of the judicial system as evidenced by the formal processes of appeals, re-trials and appeals to higher jurisdictions. This section of the review has been prepared by our legal co-author (Ms Chan) who is an experienced barrister of some 14 years standing with a practice specialising in construction law. In this section of the review Ms Chan has confined herself to scrutiny and analysis, eschewing criticism as such.

As is often the case in legal proceedings, the decision rested on the balance of several sharply contested issues. The parties had identified some 18 issues in dispute covering design, design departures, contractual, breach, causation, and damage aspects. For the purposes of forensic review the field can be narrowed to 3 principal issues and distinguished from the contested legal issues. They are:

i) Who was responsible for the design of the support system for the intersection;
ii) Was that design properly implemented by TJH; and
iii) Causation.

2.1 BACKGROUND AND EVIDENTIARY CONTEXT TO THE JUDGMENT

During the hearing before Justice R McDougall, the plaintiffs settled their claims against PB and URS. This had ramifications in respect of the defence of the claim of PSM as it had the effect of limiting the geotechnical evidence before McDougall J. This meant that all of the evidence of the geotechnical investigations into the cause of the collapse and in particular the results of Burman's modelling and forensic analysis of the Diederichs modelling were not before his Honour. This was partly because of the settlement of the case between TJH and PB as well as the rules in the Supreme Court in respect of discovery (document disclosure). In short, a party is not obliged to produce documents for the inspection of another party in the absence of a successful application to the court for discovery. These applications typically take place after the exchange of evidence.

Unfortunately, the application of PSM for discovery before the exchange of evidence was unsuccessful (NSWSC, 2015). At the nub of the application for discovery PSM requested documents that would reveal:

i) The ground conditions for which the PB tunnel support design was prepared; and
ii) The tunnel span width for which the PB tunnel support design was prepared.

The documents would also have revealed whether PB had carried out engineering design calculations for the tunnel support at the intersection (Refer Burman et al, 2018b). However, McDougall J was not convinced during the application that the material that PSM had sought on behalf of the Burman simulation had a material bearing upon the facts in issue before the parties. McDougall J held that exceptional circumstances did not exist to warrant a disclosure order. In refusing the application, McDougall J appears to have been comforted by the fact that Dr Diederichs had identified in Appendix C1 of his report, a list of the "documents comprising the Final Design". McDougall J said that *"Final Design may be taken to mean the design as at 5 November 2005, when the tunnel collapse … occurred."* Counsel for PB had conceded at the hearing of the application that the list comprised the totality of the design documents. A reasonable inference from this concession and the refusal to provide PSM with a copy of the detailed calculations is that PB had not undertaken engineering calculations in respect of the tunnel support for the intersection.

The entirety of the engineering calculations in the documentary evidence relating to the LCT design of tunnel support was for the 2-D tunnel sections. There were no engineering calculations for any intersection including for the design of support for the MC5B/MCAA intersection and there were no drawings in the support toolbox for the installation of support in intersections including for the support of the MC5B/MCAA intersection.

Nevertheless as a result of the application, PB agreed to provide the files relevant to the Diederichs simulation. This should have provided PSM's expert with the information required to investigate the cause of the collapse. Unfortunately, as discussed in (Burman et al, 2018b, **Section 5.3.2)**, the files that were provided were corrupted with errors and it was not possible to complete the investigations into causation within the time frame allowed for the service of the evidence of PSM. While Dr Burman was able to conclude his investigations after the expert conclave and before trial when Dr Burman eventually obtained the correct files of Dr Diederichs, the decision of the plaintiffs to settle their claims against PB meant that PSM was effectively precluded from relying upon Dr Burman's last report, the results of which are discussed in the 2nd companion paper (Burman et al, 2018b).

The lateness of Dr Burman's last report was deemed to be prejudicial against TJH in circumstances where TJH was not responsible for the delays in respect of the provision of the files of Dr Diederichs where McDougall J noted that neither Dr Burman nor Mr Kotze carried out any modeling (NSWSC, 2016a). This was unfortunate as it meant that the determination of the issue of causation of the collapse was played out without any evidence from Dr Burman as to whether the tunnel support design for the intersection was fit for purpose, from PB the designer and any evidence from Dr Diederichs or Dr Barla. Instead the issue was determined on the basis of Dr Beck's evidence, which was premised upon a simulation that had been carried out in 2010 and which was not the subject of any forensic examination by any of the experts. The decision to limit the forensic examination by Dr Burman to the simulation of Dr Diederichs had been made during the preparation for the trial when PB, the designers, had been an active party.

It is suggested that it is within this evidentiary context that his Honour's decision for the Plaintiffs against both PB and PSM with responsibility assigned 2/3rd to PB and 1/3rd to PSM should be viewed. Contrary to what was actually occurring on site and the understanding of the professionals on site, McDougall J held that PSM had design responsibilities in respect of the tunnel support for the intersection. Damages were found to be $21m with interest.

2.2 LESSONS TO BE LEARNED

The judgment is instructive and provides some salutary lessons for engineering consultants practicing in the geotechnical field. The authors deal only selectively with the judgment and recommend that the judgment be read in full (McDougall J, 2016). His Honour was critical of a number of matters in the presentation of PSM's case and made particular criticism of the tactical decision by PSM's counsel not to call as witnesses PSM's Engineering Geologist and PSM's Rock Mechanics Engineer. As it turns out, it is unlikely that either could have given any evidence that would have impacted upon the conclusion that the Court reached in respect of any dereliction of duty on the part of PSM. From the authors' point of view, it was the limited nature of the expert evidence before the Court and the terms of the Consultancy Agreement between TJH and PSM that was ultimately determinative of the outcome of the case against PSM.

2.3 THE TJH AND PSM CONSULTANCY AGREEMENT

Under the Consultancy Agreement between PSM and TJH, the Senior Rock Mechanics Engineer's obligations were identified in general terms. Two controversial and, we suggest, mildly ambiguous obligations, were described as:

i) "Analyse tunnel mapping and compare that to conditions described in design reports to ensure that support regimes nominated are **appropriate and efficient**."
ii) "Liaise with the Project designers to facilitate changes to the design to tailor it to conditions experienced based on the results of instrumentation and performance of previously installed support."

The difficulty for PSM arose because after a consideration of the Consultancy Agreement and the Work Method Statement, McDougall J said that the following relevantly comprised the obligations that PSM owed to TJH under the Consultancy Agreement between them:

i) To identify and characterise ground and groundwater conditions during the investigation phase;
ii) To examine the ground as it was encountered during tunneling;
iii) To confirm whether the encountered ground conformed to the expected conditions;
iv) If the encountered ground did so conform, to confirm whether the Ground Support Classification (GSC) and excavation procedures specified should be implemented;
v) To notify the designers and assist in the preparation of a new design, where the ground encountered is not suitably addressed by the existing design (NSWSC, 2016b). The court's finding of the existence of this obligation arose from the Work Method Statement which identified as an element of safe and cost effective tunnel design the following step:

   7. If the ground encountered is not suitably addressed by the existing design, then the designers shall be notified and prepare a new design. It is expected that the geological team will assist the designers with this preparation where required…

vi) To check the installed support in the tunnel for conformity with design;
vii) Where non-conformity was observed, to inform the project manager and witness rework to ensure conformity; and
viii) To work with the designers in the modification of existing designs and preparation of new designs, and in assessing the application and performance of the tunnel support system (NSWSC, 2016c).

McDougall J found that PSM had discharged their obligations in respect of its classification duties (NSWSC, 2016d). However, McDougall J said that PSM did not discharge its obligation identified in paragraph (v) above. It is difficult to see as a matter of practicality what PSM should have done to discharge this obligation in the circumstances of this particular case. Unfortunately, the approach of the Court was that PSM had assumed these obligations under its Consultancy Agreement (as amplified by the Work Method Statement). As there was no evidence that PSM had discharged its obligations, PSM was in breach of the obligation in paragraph (v) above (NSWSC, 2016e).

Presumably McDougall J considered that if this obligation had been discharged then PSM would have ascertained that the tunnel support design was inadequate and this realisation would eventually have led to the installation of steel sets for support. The consequence of this is supposedly that if it had happened then the tunnel would not have collapsed. Issues of timing as to whether this could realistically have happened given the mechanism of failure were not considered. In addition there had been no design prepared by PB for the necessarily complex construction sequence that would have been required to accommodate the up-drive and down-drive breakouts from an MC5B tunnel supported by steel sets rather than by structural shotcrete.

Part of the reason why this conclusion was reached appears to be as a result of confusion in respect of the use of the term 'frozen' in the context of the tunnel support design for the intersection (NSWSC, 2016f). The use of the 'frozen' term arose during the expert conclave and was intended to convey the fact that the tunnel support design for the intersection had been prepared for the worst ground conditions, ie G8 and G9 conditions (Refer Burman et al, 2018a, Section 3.2.2) and that the 200mm structural shotcrete and rock bolt design (MAR-VII) was a suitable alternative to the use of steel sets (Refer Burman et al, 2018a, Section 4.1). This conclusion had been based upon communications of the PB designer in respect of the development of the tunnel support design for the intersection. This meant that even if PSM had incorrectly classified the ground conditions as being better than they in fact were, which was not accepted by his Honour, then this would have had no causative effect. The support design had been intended to cater for the worst possible classification in accordance with the PB design for ground conditions at the tunnel intersection and as a suitable alternative to steel sets.

However, the Court did not construe the submission in this manner. Instead McDougall J interpreted the submission as an assertion that the contractual obligations of PSM had been frozen in respect of the tunnel support design (NSWSC, 2016g). This had not been the intention. Perhaps, if PB had been an active participant in the case then these issues would have been ventilated before the PB witnesses who would have been tested in cross examination. There might then have been direct evidence that the tunnel support design for the intersection had indeed been designed for the worst geological conditions envisaged by PB's support design (G9 conditions, refer Burman et al, 2018a) and this misapprehension might not have occurred.

McDougall J said that PSM had not discharged its obligation to assess the suitability of the tunnel support design in the conditions that were both encountered and predicted to continue. The observational approach to design and PSM's contractual obligations obliged it to continually reassess the adequacy of PB's designs in the conditions actually encountered. McDougall J considered that there was no evidence to suggest that anyone from PSM had undertaken this assessment (NSWSC, 2016h). However, it is unclear what the content of the assessment should have involved and what the causative effect of this failure was. Using the observational method of design, there was no evidence of any failure of the tunnel roof until the failure occurred on 2 November 2005. It is unclear from the judgment whether PSM should have carried out detailed engineering calculations to ascertain whether the tunnel support design was fit for purpose. If that had been the intention of McDougall J then by a process of induction, PSM would also have been obliged to carry out this exercise for the support designs for the entire Lane Cove Tunnel. In reality, this was not the role of PSM. Indeed, it was not even the role of URS, the independent verifier. It is suggested that this finding might have been avoided if the words such as *"based on the results of instrumentation and performance of previously installed support"* had been added after the phrase *"to ensure that support regimes nominated are appropriate and efficient"*. The content of the obligation of PSM would then have been clear.

2.4 INTERSECTION SUPPORT DESIGN - THE REALITY

PB was clearly responsible for tunnel design for the entire project including the MC5B/MCAA intersection. It was contracted to TJH to provide all services required in the design phase and the construction phase of the LCT project. One might reasonably have expected the simple answer to the question of 'who was responsible for the intersection support design' to have been PB. However, McDougall J determined that "all of PB, PSM and TJH were the designers: PB in respect of intersection support design, PSM in respect of GSDs, and TJH in respect of its site instruction" (NSWSC, 2016i).

To a large extent the answer to the question revolves around what was intended and what was done when PB redesigned the support system following the dyke encounter in MC5A on 27 October 2004 (Refer Burman et al, 2018a, Section 3.3.3). When the dyke was encountered in MC5A the GSC went from G7 to G9, which triggered the need for steel set supports in accordance with the set of approved PB support designs. The TJH Senior Project Engineer at the time explained in his WorkCover Interview Statement that "*because the ground condition [G9] was not anticipated by the designers we [TJH] did not have steel sets available and therefore discussed the possibility of using increased shotcrete thickness in lieu of steel sets"*. PB responded with the structural shotcrete support system (MAR-VII) that was a structural equivalent to steel sets and was in effect a replacement or an alternative to steel sets. The authors consider that at that stage the support design for all intents and purposes became 'frozen' in that the revised support design would have been suited to the worst geological conditions envisaged by PB for the entire project.

It was for the reason of structural equivalence that 3 of the 4 experts, including Mr Peck for TJH, maintained their opinion that the MAR-VII structural shotcrete support was designed as an alternative to the steel set support in G8 and G9 ground conditions. There was no evidence as to what, if any, structural calculations underpinned the changed support design but then there was no evidence ever produced to substantiate PB's support designs for shale conditions.

This was a substantial change in PB's approach to support design that was not recognised by the Court for a range of reasons, including an interpretation of contractual obligations.

It proved to be a pivotal point against PSM. It allowed the conclusion that, although PB specifically provided the support design for the MC5B/MCAA intersection on 29 September 2005, PSM had the obligation to have foreseen that PB's support design was inadequate for the actual ground conditions as they were revealed. This was in the face of the following factors to the contrary:

i) PSM was, in the authors' view, entitled to consider that the PB design was structurally equivalent to steel sets and appropriate for G9 ground conditions, the worst combination of geological and other factors conceived by PB for the LCT project;

ii) The design of ground support and the recognition of its potential inadequacy is a complex engineering task requiring resources and design knowledge that were not available to the two field engineering positions that PSM was required to fill under its contractual obligations;
iii) McDougall J concluded that "PSM did perform its [ground] classification duties with appropriate professional skill" and hence by implication PSM's GSCs for the intersection were appropriate; and
iv) PB had its own Senior Tunnel Engineer whose brief explicitly included conformance with design intent as part of its construction phase activities.

However, as discussed above, PSM had entered into a Consultancy Agreement with TJH that contained onerous conditions that were in practice probably beyond its ability to perform and that were further extended by a WMS, drafted by PSM itself. Contractually PSM was required *inter alia* to "ensure that support regimes nominated are appropriate and efficient". PSM had done itself no service by accepting such terms or in allowing itself to assist in rectifying or in clarifying what were clear documentation deficiencies in the PB intersection design.

2.5 IMPLEMENTATION OF INTERSECTION SUPPORT DESIGN

The relevant deficiencies in implementing the intersection support were those associated with the MCAA down-drive; deficiencies in the up-drive were of no consequence. It was PSM's case that the deficiencies were the use of partial face advance plus insufficient rock bolts and shotcrete; TJH's case was that there was adequate bolting and shotcrete for partial face working and that full face advance was not obligatory under the design. His Honour favoured the latter case.

From an engineering perspective, partial face advance of the MCAA down-drive meant that the shotcrete arch could not have been completed as the excavation advanced and support capacity rested solely with the CT rock bolting. This was in circumstances where the role of the structural shotcrete arch was to provide passive support. Partial face advance meant that the structural shotcrete arch was incomplete because the left part of the arch was missing and therefore no passive support whatsoever was provided. At that point it was immaterial whether the passive support was to be provided by structural shotcrete or by steel sets. Its absence for a partial face advance that reached to 8.6m in dyke-affected ground, partially supported by compromised (refer Burman et al, 2018a, Section 5.2.2) CT rock bolts of doubtful capacity in low strength shale, proved decisive and the inevitable roof falls initiated and collapse ensued. It is difficult in the authors' view to reconcile the implementation of the intersection roof support by TJH in the MCAA down-drive with the support that had been installed successfully in the MC5B tunnel and the MCAA up-drive design regardless of contractual and other issues.

2.6 CAUSATION

In his judgment McDougall J concluded *" … there was a causal relationship between PB's breach of its design duties and the collapse. There is no explanation for PB's decision to recommend a support design based on the use of rockbolts and shotcrete only, particularly when its design philosophy required passive support such as steel sets."* To the authors this is a telling instance of where the Court has not been able to be convinced that a passive support system such as steel sets could be replaced by an equivalent passive support system consisting of rock bolts and structural shotcrete; a concept that engineers would find unexceptional.

The authors remain of the view that the collapse was a result of deficiencies in the way PB's support system for the MCAA down-drive was implemented. If the down-drive had been implemented in accord with the design intent it is unlikely that there would have been a collapse. This is because an unraveling collapse will not occur if the roof is adequately supported such that the initiating roof fall cannot itself occur. However, the support design was itself almost certainly deficient in relation to its required 100-year design life due to the prospect of widespread cracking of the structural shotcrete in the MC5B and MC5B/MCAA intersection.

Unfortunately, the various computer simulations were not ventilated during the expert evidence sessions and for the reasons set out above in Section 2.1, the geotechnical evidence was limited. Within this lacuna, his Honour referred to Dr Beck's simulation in his decision on causation and to the extent that McDougall J relied on the results of the Beck simulations, it must be noted that the results of the Beck modelling were not subject to any forensic review and nor were they tested before the Court.

2.7 EFFECTIVE SPAN CONCEPT

At various times PB and others had had recourse to a concept of 'effective span' in seeking to explain causation for the collapse. PB nominated a span figure for the intersection of 15.5m. Dr Brown indicated a figure of 21m noting that measurement of an intersection span was a "matter of opinion" and concluded a span range of 17 to 22m for the intersection at the time of the collapse (Golder, 2005). The effective span concept may be useful as a simplistic indicator of support requirements for 2D excavations but in the authors' view it is of no worth for 3D excavations such as the intersection.

This is demonstrated by observing, as shown in Figure 1, that the effective span for the stable MC5B/MCAA up-drive excavation (L-shaped) was in excess of 25m whereas the T-shaped intersection became unstable for a lesser effective span in the 17 to 22m range. It is also the authors' opinion that reference to 'effective span' of the intersection has no probative value in relation to causation. The reasons are, as Dr Brown noted (Golder 2005), that stability of the intersection is not simply a matter of the span but involves the relevant site specific factors such as geological conditions, installed support, the effectiveness of that support and importantly the type of failure and the mode of its development. Dr Beck also summed up the issue succinctly in the following words (Beck, 2010):

> *Undoubtedly the true cause of any collapse is always an excessive span for the conditions, but the reason why a particular intersection fails when another similarly shaped and sized one does not is always related to strength, structure, stress and strain.*

Effective span is a concept that needs to be treated with caution and, if applied, should be done only in a qualitative context and even then with careful consideration. It would, for example, be dangerously misleading if the simplistic take-out from the LCT collapse was to be that spans of up to 17m to 22m were achievable in poor quality shale with shallow cover.

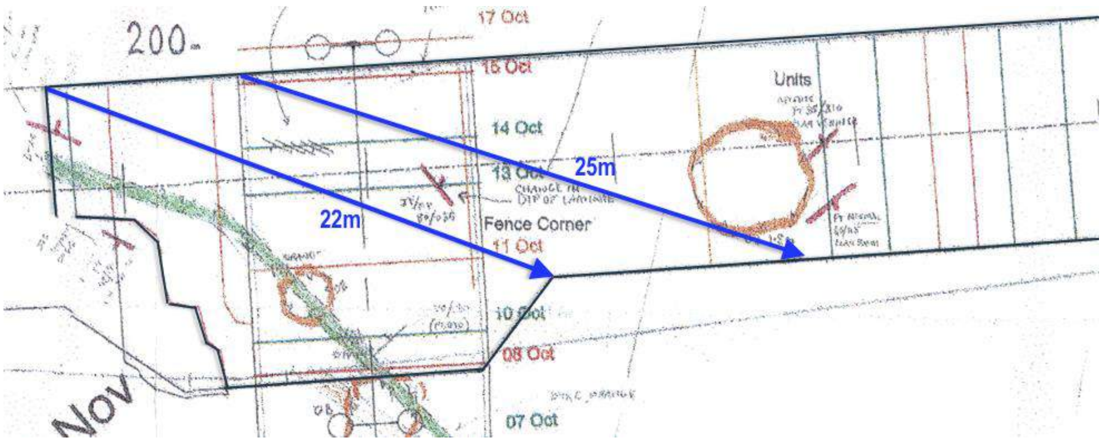

Figure 1. Effective spans of up to 22m for the collapsed T-shaped intersection compared to the 25m+ span for the stable L-shaped intersection formed by the up-drive excavation.

3 CONCLUSIONS

As is often the case with large civil engineering infrastructure projects, the LCT collapse and sinkhole is a failure that should never have occurred. It was resolved finally, a decade later, through a long, involved and costly legal process that apportioned blame according to the established precepts of the justice system; but what lessons has the professional engineering community learned from this failure? It has become increasingly rare to have the opportunity to forensically review infrastructure failures because of the trend for matters to settle rather than to proceed to judgment when the issues become available on the public record.

The LCT project was established under a series of contracts, deeds and undertakings that were intended to define the roles and responsibilities of the various parties at all levels together with international standards for quality assurance. At the operational level there were management plans and sub-plans, communication protocols and work procedures that further defined roles and responsibilities cascading to the individual task levels. No doubt the product of many diligent hours of work by many conscientious contract lawyers and others but, which nevertheless moved His Honour to remark on, if not to complain of, the complexity of the contractual documentation. Such remarks from a senior judge suggest that the contractual process is, at least, somewhat out of control and particularly in relation to the quality systems.

Quality systems evolved from the defence and manufacturing industries and have been part of civil engineering construction for some considerable time. PB was required to operate a documented quality assurance system for the overall control of project management and quality management for its design of the LCT. It was necessarily detailed and proscriptive; it formalised tasks, roles, responsibilities, documentation, communications and processes for both design and construction stages of the project but in a way that appears to have favoured box-ticking and compromised good engineering judgement.

The project management system was a significant factor in the design and implementation of the MC5B/MCAA roof support; it was intended to be. In our view, its contribution to the collapse cannot be easily dismissed. The authors have collectively experienced numerous projects where failures similar to the LCT collapse have occurred in spite of impeccable quality records; indeed some, including the authors, might suggest because of an over-reliance on those systems as they are currently implemented.

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