APPENDIX A6

Refinements to project configuration

A6.1 INTRODUCTION

Chapter 5 of the Draft EIS described in detail the scope of the proposed expansion of Olympic Dam, and chapters 9 through to 23 presented the assessment of the likely environmental, social and economic impacts and benefits arising from the construction, operation and closure of the expansion project. As noted in the Draft EIS, the expansion project is currently in what is termed for BHP Billiton internal purposes the Selection Phase, and therefore will continue to be refined through the Definition Phase prior to construction and operation.

Either as a natural part of a progressing development project, or as a result of a response by BHP Billiton to submissions received on the Draft EIS, some changes to the project configuration are proposed, although not so as to materially alter the character of the project as originally proposed. Section 1.4 of the Supplementary EIS discussed the proposed changes and presented the implications of each. This appendix provides further information on the two main changes, namely:

- the change to the proposed installation method for the outfall pipe associated with the desalination plant, with a tunnelling method now proposed rather than the trenching method described and assessed in the Draft EIS
- the introduction of a second access road into Olympic Dam to provide a dedicated access to the mine site from the proposed Hiltaba Village. The introduction of this new road has also presented the opportunity to duplicate or relocate some of the proposed facilities on the Special Mining Lease to take advantage of shorter travelling distances.

Sections A6.2 and A6.3 discuss each of these changes.

Further to these project configuration changes, the outcomes of several additional studies are provided in the Supplementary EIS. These additional studies were primarily undertaken in response to specific issues raised in the submissions on the Draft EIS. One additional study however was undertaken on a broader scale and not in direct response to an issue raised. This study was the assessment of a slowed market demand for Olympic Dam products or other economic factors that may result in a corresponding extension to the construction phase. The outcomes of this assessment are presented in Section A6.4.

A6.2 PROPOSED DESALINATION PLANT OUTFALL PIPELINE INSTALLATION METHOD

A6.2.1 CONCEPT OUTLINED IN THE DRAFT EIS

Section 5.7.4 of the Draft EIS proposed that the outfall pipe would be either buried for its full length, or buried in the land-based sections and laid on the seabed in the deeper waters. The method of burial was proposed to be either wheel or dredge trenching, potentially in combination with blasting where the strength of the underlying rock necessitated, followed by backfilling. The potential impacts of this installation method, including the environmental effects of marine blasting and sediment plume modelling were provided in Section 16.6.11 of the Draft EIS.

A6.2.2 PROPOSED REVISION

It is now proposed that the outfall pipe for the proposed Point Lowly desalination plant would be installed by a tunnelling, rather than a trenching, method. The intake pipe would be installed by a trenching method as described and assessed in the Draft EIS. Recognising that the desalination plant would not be required for about 5 years after the expansion project commenced, the final tunnelling method would be selected at the appropriate time in the future and after further geotechnical investigations. However, prior to making the commitment to tunnel the outfall pipe, BHP Billiton along with the Halcrow Group Limited investigated the feasibility of various tunnelling installation methods (see Attachment A for the Halcrow Group Limited report). The key outcomes were:

- From the initial site visit and observations of the geology of the surround area and the site, it is considered that the tunnel would be driven through horizontally bedded quartzite and high strength sandstone (e.g. in the range of 200MPa to 300Mpa). The Rock Quality Designation (RQD) value used in the designation of rock quality could be in the range 25-100, which indicates sound rock. However, the level of fracturing and permeability of the rock would need to be established before design as this will affect the types and methods of support and excavation of the tunnel.
- For the design, a cover of competent rock above the tunnel of at least 10 m has been assumed. This may increase depending on the outcome of the geotechnical investigations and the final tunnelling technique chosen.
- With a maximum depth to the sea bed of about 25 m, and a 10 m cover to the tunnel crown, there is a risk of high water pressures within tunneling zones and potentially water ingress into the tunnel during construction, particularly if the rock has hydraulic pathways.
- The vertical alignment of the tunnel has still to be determined. However, an alignment that involves tunnelling from a deep launch shaft, 'up hill' to the diffuser end of the tunnel, is preferred because this enables water that may have entered the tunnel to drain away from the cutting face to a sump in the launch shaft where the water can be pumped a short distance to the surface.

- Three installation methods were assessed:
  - Tunnel boring machine (TBM) – with three types of machines investigated
  - Drill and blast or a mechanical excavation used with temporary then permanent support
  - Pipe jacking/micro tunnelling machine and lining.
- While all three methods may be feasible, the use of a TBM was preferred as this method does not require marine blasting, was considered to provide the quickest construction time and would have the least risk to safely completing the tunnelling operation.
- Due to the tunnel being driven below the sea bed and the unknown nature of discontinuities that may provide hydraulic pathways and therefore the possible risk of water inflows, the least risk option of the three TBM types assessed is a pressurised TBM with fibre reinforced concrete segments (slurry or Earth Pressure Balance machine). These machines provide continuous support to the tunnel face by balancing the inside earth and water pressure against the thrust pressure of the machine, enabling the anticipated presence of water to be contained and controlled at all times.
- Should this installation method be chosen, the site set up and excavation of the shaft from which the TBM would be launched would take about three months. The advance rate for a 2.8 m diameter tunnel would vary depending on the substrate (estimated range to be 0.6 m/hr to 1.2 m/hr), and with a contingency of maintenance time built in, the estimated construction time for the outfall tunnel is between six and eight months. Over this time, about 53,000 tonnes of spoil would be excavated from the launch shaft and tunnel.
- The launch shaft arrangement would have a minimum diameter of 7 m, would be about 87 m deep and may be excavated using a drill and blast method and lined temporarily with shotcrete with spot bolting where required. The permanent lining would depend on the required final arrangements.
- A laydown area of approximately 4 ha would be required for the tunnel and shaft construction operations, and depending on the segment manufacturer, a further 0.6 ha may be required for segment storage. These areas would all be located within the footprint of the desalination plant as described and assessed in the Draft EIS.
- Marine risers would connect the tunnel to the outfall pipe diffusers (most likely to be four rosette style diffusers). It is anticipated that four risers would be required, and each riser would be GRP lined and offset from the line of the tunnel to allow flexibility in the construction program and to increase the safety of making the connection. Each riser would take about one month to install and this would occur at the same time that the tunnel was being excavated.
- The risers would be drilled, with no marine blasting required. Each riser would be constructed by first installing a 3 m diameter vertical steel casing to rock head (this may require pile driving and if so this activity would take no more than two days). The rock inside the steel casing and beyond the toe is then excavated by drill until the required depth at invert of the tunnel level is achieved. The excavated material is contained within the casing and pumped to a nearby barge for removal. Once the excavation is complete a 2 m diameter GRP liner is lowered into the excavation and grouted into position. The GRP liner is then sealed ready for a dry and safe connection to be made to the tunnel.
- It is anticipated that the marine risers would be approximately 15 m long, which allows for the 10 m of cover and for the depth of the tunnel and a sump below.
- The installation of the risers would require a stable working platform from which to carry out heavy lifts. Ancillary requirements would be:
  - marine service barge
  - tug support
  - service boat and helicopter access to platform
  - drill rig
  - shore side storage of about 0.8 ha, with at least 50 m of water frontage for loading, vehicle turning, office space and fabrication.

A6.2.3 IMPLICATIONS

The implications of the tunnelling installation method for the outfall pipe are provided in Table A6.1.

Table A6.1 Implications of tunnelling the outfall pipe rather than trenching as assessed in the Draft EIS

Issue	Implications	Section of Draft EIS	Residual impact rating[1]
Soils	The laydown areas at the desalination plant (about 4 ha) and shore (about 0.6 ha) would disturb soils and erosion control measures to manage sedimentation resulting from stormwater flows over these disturbed areas would be required. These however are standard engineering practice and measures beyond that provided in the Draft EIS would not be required. The nature of the soils in this area being strongly dissected stony tablelands complex of the Tent Hill land system, and review of the coastal acid sulphate soils risk mapping, indicate that there is no potential for disturbance to acid sulphate soils in this area.	10.3, 10.5	No change
Surface water	Management of surface water flows from the laydown areas would be required as noted for soils above. However, this too would require standard engineering practices and there would be no effect to surface water beyond that presented in the Draft EIS.	11.5	No change
Groundwater	There is the potential for some groundwater ingress into the tunnel during excavation and prior to the tunnel being lined. The vertical alignment of the tunnel would be such to ensure that this water moved away from the excavation head back along the tunnel and into the launch shaft. Water collected from the sump in this shaft would re-used at the tunnelling head and/or pumped to the surface, held in temporary ponds for storage where it would be treated to within applicable compliance limits before being ultimately discharged to the gulf.	12.6	Low
Greenhouse gas and air quality	There would be no effect to greenhouse gas and air quality beyond that presented in the Draft EIS.	13.2, 13.3	No change
Noise and vibration	While the tunnelling installation method would generate some marine and land-based noise, this would be similar to, if not lower than, that generated via the trenching method described and assessed in the Draft EIS. It is possible however that the tunnelling method would increase land-based vibrations beyond that described in the Draft EIS (with marine vibrations and potential concussion effects from tunnelling being less than that assessed for blasting). For the purpose of impact assessment, the nearest residence to the tunnel alignment shown on Figure A6.1 is located 43 m horizontally, and 52 m vertically above the proposed alignment (total separation of 67 m). The Point Lowly lighthouse structure would be 75 m horizontally and 50 m vertically (total separation of 91 m). The calculated vibration at these sensitive receivers is: • nearest residence would be less than 1.3 mm/s (with 95% confidence) and 0.15 mm/s (mean) • lighthouse structure would be 0.8 mm/s (95% confidence) and 0.09 mm/s (mean). Vibration generally falls into one of two categories: • Human exposure-related • Building (structural) damage-related. There are no legislative criteria for either in Australia. Australian Standard AS2670.2 provides guidance on acceptable levels of human exposure to ensure 'human comfort' (corresponding to a low probability of reaction), arriving at the following general criteria (noting that 0.15 mm/s is the level of human detection): • Residences (night) – 0.2 mm/s • Residences (day) – 0.3 mm/s to 0.6 mm/s. A British standard (BS 7385) is generally used as the basis for assessment of the potential for building (or cosmetic) damage: • Historic buildings – 2 mm/s • Residential buildings – 5 mm/s. These levels are considered conservative, the blasting report in Appendix O of the Draft EIS quoted 5 mm/s and 10 mm/s respectively, and maintains that these are adequate to prevent structural or cosmetic damage to buildings. The analysis shows that the predicted vibration levels would be below all stated guidelines for residences and buildings, and may in fact be below the level of human perception at the nearest residence. Nevertheless, prior notice of tunnelling to nearby residence would occur and monitoring of vibration levels would be undertaken prior to and during tunnelling activities.	14.5	Low

Table A6.1 Implications of tunnelling the outfall pipe rather than trenching as assessed in the Draft EIS (cont'd)

Issue	Implications	Section of Draft EIS	Residual impact rating[1]
Fauna and flora	Land disturbance at the site of the desalination plant was assessed in the Draft EIS. The additional 0.8 ha of land required for the near shore facilities would not impact nationally, state or regionally significant flora or fauna species or communities.	15.5	Low
Marine environment	The change in installation method from trenching to tunnelling is a positive response by BHP Billiton to address concerns raised about marine blasting and potential impacts on marine ecosystems. Tunnelling would also reduce sediment loads created during the installation of the pipe and risers over a trenching method.	16.6	Lower than previously assessed
Aboriginal cultural heritage	There would be no effect to items or places of Aboriginal heritage beyond that presented in the Draft EIS.	17.5	No change
Non-Aboriginal heritage	There would be no effect to items or places of Non-Aboriginal heritage beyond that presented in the Draft EIS.	18.5	No change
Social environment	The relevant social issue for the changed pipeline installation method is the generation of traffic resulting from transport of the spoil recovered from the tunnel. As discussed in the Draft EIS, the construction of the desalination plant and associated pipelines would occur some years after open pit mining had commenced. As such, a local re-use option for the spoil would be investigated at the appropriate time in the future. For the purpose of the Supplementary EIS, the disposal of up to 53,000 tonnes of spoil would require between 16 and 27 trucks per day to be added to the existing road network. The number of vehicles (mostly B-double trucks) per day depends on the tunnel advance rate, with a faster advance rate (1.2 m/h) generating spoil more quickly and therefore more vehicles, but over a shorter timeframe (estimated to be about three months). The slower advance rate (0.6 m/h) would require less vehicles but over a longer period (estimated to be about 6 months). Including a provision for maintenance to the tunnelling equipment, the total construction time for the launch shaft and tunnel is estimated at eight months. The assessment of increased traffic volumes, taking into account the 27 vehicles per day and thus the upper end of the predicted range, found that no road closures would be required and the level of service along major roads between the desalination plant and Olympic Dam would not change from the current level of service (see Attachment B for details). Also, the existing intersection on Port Bonython Road and the Lincoln Highway would continue to operate at the same level of service (LoS 'B'). The minor increase in traffic volumes at this intersection indicates that the average delay for vehicles undertaking a right turn from Port Bonython Road onto the Lincoln Highway would increase from 12.2 seconds to 14.2 seconds during peak movements. Therefore, while local re-use of the material would be investigated, the worst-case proposal to truck all of the spoil back to Olympic Dam would increase road movements but this would have a negligible effect on the existing road network.	19.5	Negligible
Visual amenity	The installation of the marine risers would necessitate a floating platform to be located about 600–800 m offshore for about four months.	20.5	Low
Health and safety	There would be no effect on health and safety beyond that presented in the Draft EIS.	22.6	No change

[1] Residual impact categorisation as per the criteria used throughout the Draft EIS (refer Section 1.6.2 of the Draft EIS for details).

Figure A6.1 Revised outfall pipe tunnelling alignment

A6.3 NEW MINE ACCESS ROAD AND ON-SITE FACILITIES

A6.3.1 CONCEPT OUTLINED IN THE DRAFT EIS

Section 5.9.4 and 19.5.6 of the Draft EIS discussed the proposed new access road from the northern intersection of the heavy vehicle bypass and Olympic Way to a new main gate at Olympic Dam (see Figure A6.2). The new western access road would be two lanes in both directions, separated by a median strip, and would therefore provide for the safe movement of traffic to the existing and expanded metallurgical processing section of the operation. The western access road would be a private road, although it would be open to the public for access to the Olympic Dam main gate.

At the time of publishing the Draft EIS, it was envisaged that the workforce accommodated at Hiltaba Village would be bussed to Olympic Dam along Andamooka Road, the heavy vehicle bypass, Olympic Way and the new western access road.

A6.3.2 PROPOSED REVISION

A second entry gate and eastern access road providing a direct link between Hiltaba Village and Olympic Dam is now proposed. Figure A6.2 shows the location of the newly proposed eastern access road, linking Hiltaba Village to the open pit mining area of the expanded operation. This road, together with a second entry gate as shown on Figure A6.2, would provide a second point of access to Olympic Dam. It is proposed that the mining-related workforce is bussed to Olympic Dam along the eastern access road, while the processing-related and administrative workforce is bussed along Andamooka Road and the western access road as described in the Draft EIS.

The implications of this change are an increased traffic flow through the staggered T intersection of Axehead Road – the heavy vehicle bypass – and Andamooka Road. Table A6.2 shows the predicted average annual daily traffic (AADT) for light vehicles and buses through this intersection until a constant traffic flow is reached.

Table A6.2 Mine workforce traffic profile through the staggered T intersection

	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9
Light Vehicle	104	208	312	416	520	650	650	650	650
Bus	5	10	14	19	24	30	30	30	30
Total AADT	109	218	326	435	544	680	680	680	680

The AADT for heavy vehicles (i.e. B-doubles) transporting mining equipment along the heavy vehicle bypass and turning right into Andamooka Road to the eastern access road is predicted to be:

- Year 1: 11
- Year 2: 32
- Year 3: 39
- Year 4: 44
- Years 5 to 9: 48.

The addition of the eastern access road also creates the opportunity to provide additional on-site facilities, or relocate proposed facilities, to improve access to these facilities and reduce on-site travel times. The relevant facilities are shown in Figure A6.2 and include the on-site desalination plant, an additional mine maintenance area, and an additional laydown area for mining equipment.

A6.3.3 IMPLICATIONS

The implications of the new mine access road and on-site facilities are provided in Table A6.3.

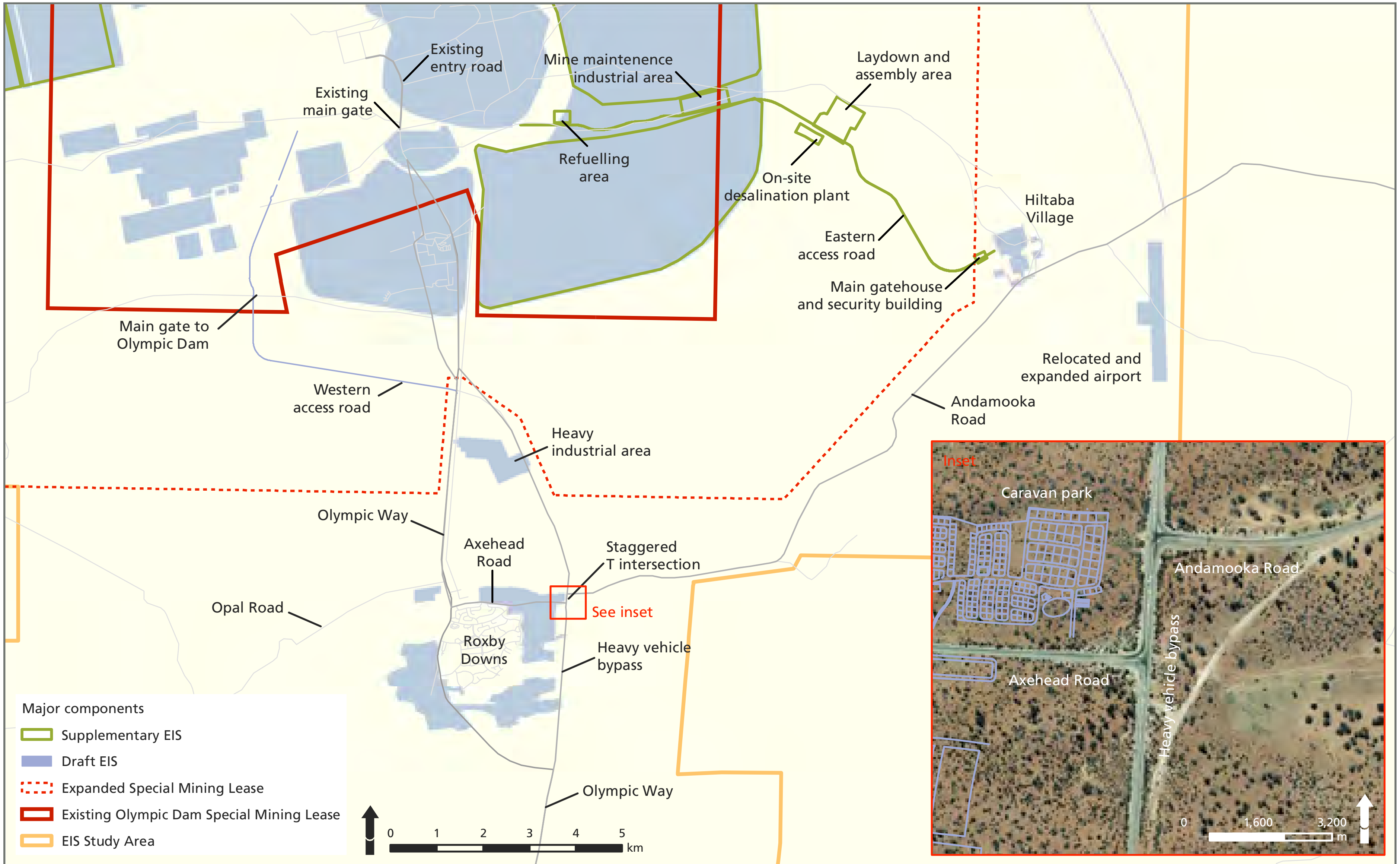


Figure A6.2 Proposed mine access roads and staggered T instersection

Table A6.3 Implications of the new eastern access road and on-site facilities compared with that assessed in the Draft EIS

Issue	Implications	Section of Draft EIS	Residual impact rating[1]
Soils	While disturbance to soils over an increased area of approximately 130 ha would occur, there would not be an increase to the residual impact rating for soil erosion, soil contamination or the potential disturbance to fossils to that presented in the Draft EIS.	10.5	No change
Surface water	While there would be an increase in hardstand area of approximately 130 ha, and therefore the potential for increased stormwater runoff velocities from these areas, there would not be an increase to the residual impact rating for changes to drainage patterns, infiltration of stormwater to groundwater, or changes to surface water quality to that presented in the Draft EIS.	11.5	No change
Groundwater	There would be no effect to groundwater beyond that presented in the Draft EIS.	12.6	No change
Greenhouse gas and air quality	There would be no effect to greenhouse gas and air quality beyond that presented in the Draft EIS.	13.2, 13.3	No change
Noise and vibration	The eastern access road would be constructed in the early stages of the expanded project, likely to be concurrent with the construction of the first stage of Hiltaba Village. As such, noise generated from the construction of the new road would not impact residence at Hiltaba Village. In terms of the operational phase, the most significant source of noise would be from the additional mine maintenance and industrial area located adjacent to the new road and close to the rock storage facility (see Figure A6.2). Noise modelling of anticipated activities from this facility, unmitigated, predict that under worse-case meteorological conditions (i.e. winds blowing towards sensitive receptors and/or temperature inversions) noise levels at the most northern residential areas of Roxby Downs and at Hiltaba Village would increase by 1 dB and 7 dB respectively above that reported in the Draft EIS (see Attachment C for details). These levels are within compliance limits for Roxby Downs, but exceed night-time compliance limits for Hiltaba Village. As such, management measures would be applied to ensure compliance. Noise modelling has shown that compliance with night-time noise limits can be achieved by avoiding the use and/or testing of haul truck air horns whilst at the facility, or if this activity is necessary, by enclosing an area with acoustic shielding for the use and/or testing of the air horns.	14.5	Low
Fauna and flora	The new road and additional facilities would require the clearing of an additional 130 ha of vegetation. The vegetation communities to be cleared are widespread throughout the Olympic Dam region and have not been found to support flora or fauna species listed under Commonwealth or State legislation. In particular, the affected vegetation communities would be the Acacia shrubland and Scleroleana spp. low shrubland, almost 145,000 ha and more than 65,000 ha of these communities occur within the assessed EIS study area, respectively.	15.5	Low
Aboriginal cultural heritage	While disturbance beyond that described in the Draft EIS would be required to accommodate the newly proposed road and additional facilities, this clearing would occur within the expanded Special Mining Lease. The Olympic Dam Agreement signed between BHP Billiton and the Aboriginal groups with a native title interest in this area provides an agreed procedure for managing potential impacts associated with these newly proposed activities.	17.5	Low
Non-Aboriginal heritage	There would be no effect to items or places of Non-Aboriginal heritage beyond that presented in the Draft EIS.	18.5	No change

Table A6.3 Implications of the new eastern access road and on-site facilities compared with that assessed in the Draft EIS

Issue	Implications	Section of Draft EIS	Residual impact rating[1]
Social environment	Potential social impacts are essentially associated with traffic. The Draft EIS assessed all traffic entering Olympic Dam via the western access road, whereas traffic volumes will now be split between the western and eastern access roads. This has the effect of reducing traffic numbers along Olympic Way and the western access road, but increasing traffic numbers along Axehead Road and Andamooka Road (see Attachment D for details). The main implication of this traffic increase would be a reduction in the level of service and safe operating capacity of the staggered 'T' intersection of Axehead Road, the heavy vehicle bypass and Andamooka Road (see Figure A6.2). The level of service for the Axehead Road / heavy vehicle bypass intersection during the peak of the construction phase would reduce from a level A to a level B, whereas the heavy vehicle bypass intersection during peak construction traffic would reduce from a level C to a level D (noting that LoS is a measure of delay for an intersection and a LoS of 'D' is considered within acceptable limits although a LoS of 'C' is preferred and more comfortable for drivers). The operating capacity of any given intersection can be measured by the 'degree of saturation' (DoS) of turning movements, with a DoS lower than 0.85 typically being an intersection operating within a safe capacity (i.e. operating at less than 85% of its capacity). Based on traffic volume predictions for the expanded operation, the DoS for the staggered 'T' intersection at times of peak traffic flows would increase from 0.1 to 0.2. As such, the proposed traffic volumes would operate well within the design capacity of the intersection. Having said that, neither the intersection nor Andamooka Road is an approved network route for the safe movement of Restricted Access Vehicles (RAV's) such as B-doubles, Double and Triple road trains. As such, BHP Billiton would collaborate with the South Australian Department of Transport, Energy and Infrastructure (DTEI) to develop an appropriate strategy for the intersection and Andamooka Road to allow the movement of RAV's to the proposed eastern access gate.	19.5	Moderate
Visual amenity	There would be no effect to visual amenity beyond that presented in the Draft EIS.	20.5	No change
Health and safety	The mine maintenance industrial area would be a designated radiation work area, with workers in this facility declared as radiation workers and therefore subject to the Olympic Dam radiological protection program as described in the Draft EIS (e.g. routine monitoring and a requirement that at the beginning of shift workers change into work clothes, and at the end of shift they shower and change into street clothes). As a designated radiation area, any material leaving the area to go off-site would require radiation clearances. The area would be designed for ease of cleanup, including wash down facilities. It is expected that radiation doses to full time workers in the area would be similar to the metallurgical plant workers (i.e. up to 3 mSv/y). Further to radiological issues, this project configuration change would see a controlled interaction of the mining fleet and busses transporting the workforce from Hiltaba Village to the mine site. This interaction would be managed via grade separated roads (e.g. underpasses) and active traffic management controls (e.g. signalised crossing points for at-grade intersections). The management of potential rock fall from dumping at the RSF outer face would be managed via standard engineering controls (e.g. catch banks, fences).	22.6	Low

[1] Residual impact categorisation as per the criteria used throughout the Draft EIS (refer Section 1.6.2 of the Draft EIS for details)

A6.4 ADDITIONS TO THE DRAFT EIS

The Supplementary EIS presents the outcomes of many studies undertaken in addition to those provided in the Draft EIS. Most of these additional studies were undertaken to address a specific issue raised in a submission/s. One exception is a study undertaken to address the status of the expansion of Olympic Dam being in the Selection Phase, and more specifically to provide an additional assessment for possible changes to schedule.

The proposed expansion is BHP Billiton's response to predicted global increases in the demand for copper, uranium oxide, gold and silver. However, changes in economic factors at any time over the long life of the project may result in changes to the construction stages detailed in Chapter 5 of the Draft EIS. This could allow Olympic Dam to increase its output to meet growing demand. On the other hand, should the global demand increase at a slower rate than projected, or as a result of other economic factors, BHP Billiton may slow its ramp up in metal production accordingly. However, it is important to note that the scale and longevity of the project would continue to realise significant economic and social benefits even if a slowing in demand occurred.

The main outcomes of the additional assessment against each of the impact assessment chapters presented in the Draft EIS are presented in Table A6.4.

Table A6.4 Implications of the extended construction phase compared with that assessed in the Draft EIS

Issue	Implications	Section of Draft EIS	Residual impact rating[1]
Soils	There would be no effect to soils beyond that presented in the Draft EIS.	10.5	No change
Surface water	There would be no effect to surface water beyond that presented in the Draft EIS.	11.5	No change
Groundwater	There would be no effect to groundwater beyond that presented in the Draft EIS.	12.6	No change
Greenhouse gas and air quality	The generation of greenhouse gases and air emissions would occur at a slower rate than that assessed within the Draft EIS, however these would ultimately reach the same levels. As such, the effects would not be beyond those presented in the Draft EIS.	13.2, 13.3	No change
Noise and vibration	The time taken to construct any particular project component would not change from that described in the Draft EIS and as such noise levels associated with construction activities would be as per those assessed. The Draft EIS noted that the landing facility west of Port Augusta would accommodate about 280 vessels over a seven year period with intermittent use during the 40 year project period. In the event of a slower ramp up in metal production, the facility would have fewer vessels during that time with the total number of vessels extended over a longer period. As such, the noise levels predicted from activities associated with berthing and unloading a vessel would not change, but they would occur less frequently and over a longer time period. If the project schedule was to change and the plan for use of the landing facility, access corridor and pre-assembly yard was going to be different to that outlined in the Draft EIS, the company would seek to discuss this with those members of the community who may be impacted.	14.5	Moderate
Fauna and flora	There would be no effect to fauna and flora beyond that presented in the Draft EIS.	15.5	No change
Marine environment	There would be no effect to the marine environment beyond that presented in the Draft EIS.	16.6	No change
Aboriginal cultural heritage	There would be no effect to items or places of Aboriginal cultural heritage beyond that presented in the Draft EIS.	17.5	No change
Non-Aboriginal heritage	There would be no effect to items or places of Non-Aboriginal heritage beyond that presented in the Draft EIS.	18.5	No change
Social environment	An extended construction phase would have the implication of ‘stretching’ the timeframe over which impacts would be felt and benefits would be realised. The categorisation of impacts and benefits however would not change from that presented in the Draft EIS.	19.5	No change
Visual amenity	There would be no effect to visual amenity beyond that presented in the Draft EIS.	20.5	No change
Economic assessment	An economic sensitivity analysis has been undertaken which shows significant economic and social benefits of this project even in the event of an extended construction period. The key outcomes of the range analysis are provided in Table A6.5. Overall, the proposed Olympic Dam expansion, whether undertaken on the schedule provided in the Draft EIS or in the event of a slowed ramp up before reaching full operating capacity, is a very large investment project that would have a significant impact on the national economy. The results shown in Table A6.5 compare the benefits that would be expected to occur as a result of the proposed Olympic Dam expansion in Australia as a whole, South Australia, key South Australian Statistical Divisions (regions) and the Northern Territory against a few different ramp up scenarios and from the 2008 terms used for the Draft EIS updated to 2010 terms.	21.4	No change
Health and safety	The Draft EIS noted that the construction phase is the period when workers, tasks and circumstances are typically new, and the safety risks are correspondingly higher than in the operational phase. It is possible that an extended construction timeframe for mining and processing at Olympic Dam would result in a greater number of individuals being exposed to these conditions (presuming that staff turnover remains at predicted levels). However, an extended construction period would reduce the intensity and overall interactions, thus reducing the risk of incident. Also, the systematic approach to health and safety embedded in the culture of BHP Billiton suggests that safety throughout an extended construction phase would remain a focus of management attention and the potential impacts as described in the Draft EIS would remain unchanged.	22.6	No change

[1] Residual impact categorisation as per the criteria used throughout the Draft EIS (refer Section 1.6.2 of the Draft EIS for details).

Table A6.5 Results of economic sensitivity analysis

Economic measure	Draft EIS case ($2008 terms)	Draft EIS case ($2010 terms)	Extended construction scenario
Gross Domestic Product ($NPV_{7\%,\ Year\ 0–Year\ 30}$)[2]			
Australia[5]	$18,721m	$18,399m	$13,768m
Gross State Product ($NPV_{7\%,\ Year\ 0–Year\ 30}$)[3]			
South Australia	$45,701m	$48,397m	$34,192m
Northern Territory	$936m	$915m	$607m
Gross Regional Product ($NPV_{7\%,\ Year\ 0–Year\ 30}$)[2]			
Northern Statistical Division	$22,627m	$22,048m	$14,904m
Adelaide Statistical Division	$24,223m	$27,577m	$20,259m
Consumption / economic welfare (NPV)			
Australia	$21,754m	$23,088m	$16,625m
South Australia	$19,822m	$21,346m	$15,157m
Northern Territory	$1,088m	$1,000m	$674m
Government revenues (NPV)			
Australian Government[3]	$2,599m	$2,780m	$1,949m
South Australian Government[4]	$3,422m	$3,515m	$2,460m
Northern Territory Government	$47m	$46m	$30m

1 Assumes that processing of 20 Mtpa is delayed by three years and processing at full operating capacity at 72 Mtpa is delayed by five years to that presented in the Draft EIS.
2 All NPV 7% calculations are taken over a Year 0–Year 30 period, which includes the two construction phases and the full operational phase, discounted at a conservative real social discount rate of 7%.
3 This includes all GST collected (including from South Australia), company tax, income tax, and excise taxes.
4 This includes payroll, other local taxes, and royalties but excludes GST revenue collected in South Australia.

ATTACHMENT A

Seawater desalination plant – Tunnels feasibility

Halcrow Group Australia

Olympic Dam Expansion – Seawater Desalination Plant

Tunnels Feasibility

June 2009

Halcrow Group Limited

Halcrow
Level 22, 68 Pitt Street, Sydney, NSW 2000 Australia
PO Box R1573 Royal Exchange, NSW 1225
Tel +61 2 9250 9900 Fax +61 2 9241 2228
www.halcrow.com/australasia

Contents

1 Executive Summary

The purpose of this report is to identify and specify the most appropriate tunnelling method that could be employed for the sub sea intake and discharge pipelines for the proposed Olympic Dam Desalination Plant Project. The risks, logistics and programme durations for three tunnelling options were identified.

Various lengths of tunnel were considered ranging from 550m to 2800m, to be driven through quartzite and sandstone with an initial estimated strength of 200 to 300mpa, 10m below the seabed with a potential 3bar water pressure. However, the permeability of the rock and nature of the discontinuities is unknown at this stage. Also unknown is the variability of the geology along the routes.

Based upon the above and the discussions within this document it is suggested that a pressurised tunnelling machine is used for this project. The main reasons for this are;

- Construction times
- Flood risk
- Connection to reception shaft
- Smoothness of lining
- Previous experience and machine availability
- No effect on lighthouse at Pt Lowly

Specifications are presented for three tunnelling options. Based on these specifications, BHPB has chosen to tunnel a 2800m pipeline from the desalination plant to the outfall (diffuser). This option provides the least construction risk and shortest construction time and minimises environmental impacts.

2 Introduction

Following submission of the Draft EIS, concerns were raised over the potential environmental impacts of the trenching construction method proposed for the intake and outfall pipelines. As a consequence, alternative methods of construction were investigated. Halcrow Group Limited were contracted to assess the viability of tunnelling the intake and outfall pipelines. Preferred tunnelling methods and designs were investigated.

Extensive studies have determined the most probable intake, outfall and desalination plant locations (Figure 1), which will require;

- An area for the proposed plant site location approximating 400m×600m on ground at elevation of approximately +21m

- An intake pipeline consisting of a 550m gravity feed pipe from the intake structure to the pump station and a 700m pipeline from the pump station to the desalination plant.

- An outfall pipeline consisting of 2,200 m gravity pipeline from the desalination plant to the shore and an 800 m gravity pipeline to the diffuser (including a 200 m diffuser).

The gravity intake pipe is expected to be approximately 2.4 m diameter, and the outfall pipe is expected to be approximately 2.1 m diameter.

The purpose of this report is to assess the feasibility of various methods of tunnelling that could be employed for the intake pipeline from sea to pump station and for the outfall pipeline, and to identify the associated risks, logistics and duration of the options considered.

Figure 1: Proposed tunnel alignments, and desalination plant site at Point Lowly. (update to show footprint of proposed desalination plant, pipelines and pump station)

3 Description of Tunnel Options

Three tunnel routes have been considered (Table 1);

- the intake pipeline from sea to pump station
- an outfall pipeline from the diffuser to the shore (option 1)
- and an outfall pipeline from the diffuser to the desalination plant (option 2)

Option 1 requires a pipeline buried in a trench connecting the tunnel shaft at the shore to the desalination plant. This option does not impact on the system hydraulics.

The horizontal alignment of the tunnels has been assumed to be straight. The vertical alignment has still to be determined, however, an alignment that involves tunnelling up hill is preferred in tunnelling, as this enables water to drain away from the face to a sump where it can be pumped a short distance to the surface.

For the purposes of this study the internal diameter of the tunnel has been taken as 2.8m for constructability, whereas the Draft EIS states diameters of 2.1m and 2.4m. However, for the purpose of this report the size has been standardised as 2.8m assuming an allowance for improved hydraulics of the system and alignment with known costs.

For the design a cover of competent rock of 2 to 3 diameters, approximately 10m, has been assumed. This may increase depending upon the outcome of the geotechnical investigations and the final tunnelling technique chosen.

Table 1: Details of tunnelling options

	Length (m)	***Max Cover (m)***	***Internal Diameter(m)***	***Internal Area(m^2)***
Intake	550	10	2.8	6.2
Discharge Option 1- Tunnel from diffuser to shoreline , with trench and buried pipe on land.	600	10	2.8	6.2
Discharge Option 2 – Tunnel from diffuser to desalination plant	2800	10	2.8	6.2

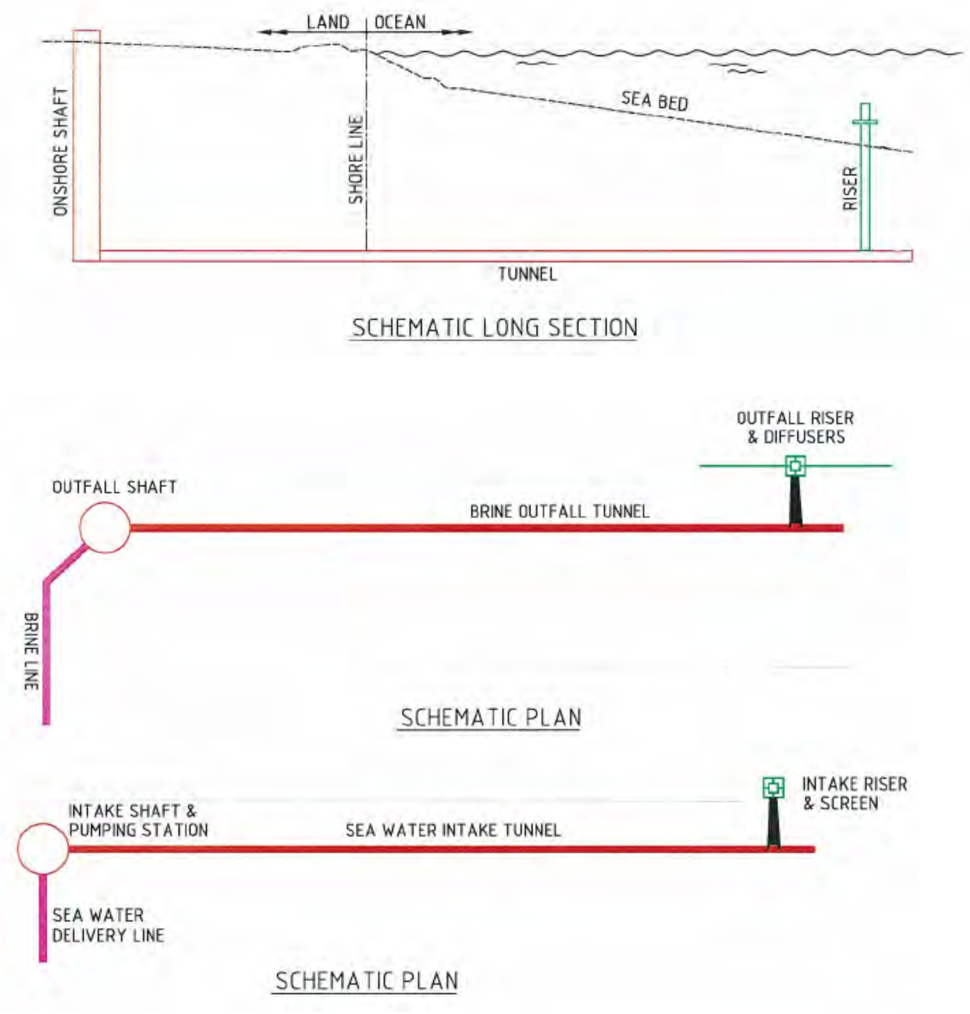


Figure 2: Proposed outfall (a and b) and intake (c) pipeline plans

4 Geology of tunnels

4.1 Geology

Based on a site visit and initial inspection of the geology of the surround area and the site it is considered that the tunnels are to be driven through horizontally bedded quartzite and high strength sandstone. It is thought, at the depth that the tunnels will be constructed, that the jointing will be tight.

Depth probes along the intake line offshore showed sediment thicknesses from 0.8m to more than 4m. Along the proposed outlet line, offshore sediment thickness ranges from 0 to 1.3m.

Before detailed design, a detailed geotechnical investigation will be required to establish the depth of the sediment on the sea bed. In addition, the type, composition and bedding of the rock where the tunnelling will occur, will need to be established.

Photo 1: Quartzite outcrop near proposed intake

Photo 2: Quartzite outcrop

4.2 Engineering parameters for the rock

From the initial site visit and observation of the site, the strength of these rocks could easily be in the range of 200MPa to 300Mpa. At this stage of the project there is no observed or recorded data for this region. The Rock Quality Designation (RQD) value used in the designation of rock quality could be in the range 25-100 which indicates sound rock.

Generally, at this early stage it is felt that the various tunnels will be driven through strong rock. However, the level of fracturing and permeability of the rock will need to be established before design as this will effect the types and methods of support and excavation of the tunnel. The next stages of the geological and geotechnical design and investigation will provide greater definition of the rock mass conditions and the location of faults, in the area of the tunnels.

4.3 Groundwater

The maximum depth to the sea bed has been given as 25m and a cover to the tunnel crown of 10m. Therefore, there is a risk of high water pressures within tunneling zones and maybe high flows during construction if the rock has hydraulic pathways.

5 Design Standards for the tunnels

To determine the hydraulic performance of tunnels a hydraulic roughness coefficient is used to model the surface of the tunnel. Depending upon the material used for the tunnel lining this “k” value can range from 3mm for a fair insitu concrete finish, to 60 to 600mm for an unlined rock tunnel. The impact of this coefficient on the hydraulics of the system also depends upon the cross sectional area of the tunnel and hence the velocity of the flow within it.

The shape of the completed tunnel will vary depending on the method of construction. A mechanically excavated tunnel with a segmental lining would have a circular profile, while a drill and blast tunnel is likely to have a horse shoe shaped profile.

If a tunnel is constructed using a Tunnel Boring Machine (TBM) the profile of the tunnel will be circular.

Photo 3 : Desalination Plant Tunnel showing the tunnel segments and temporary services

The design life for the tunnel is assumed to be 100 years based on the specifications for other similar projects.

6 Tunnel Construction Method

There are three main methods of constructing a tunnel, these are:

- Tunnel Boring Machine (TBM)
- Drill and Blast or a mechanical excavation used with temporary then permanent support
- Pipe Jacking / micro tunnelling machine and lining

Drill and Blast methods were considered to be less safe in comparison to TBM methods and Pipe Jacking was considered unsuitable for the location and tunnel type required (Appendix 1). Horizontal Directional Drilling (HDD) has not been considered here. Installation via tunnel boring machines is considered at this stage to be the most feasible method.

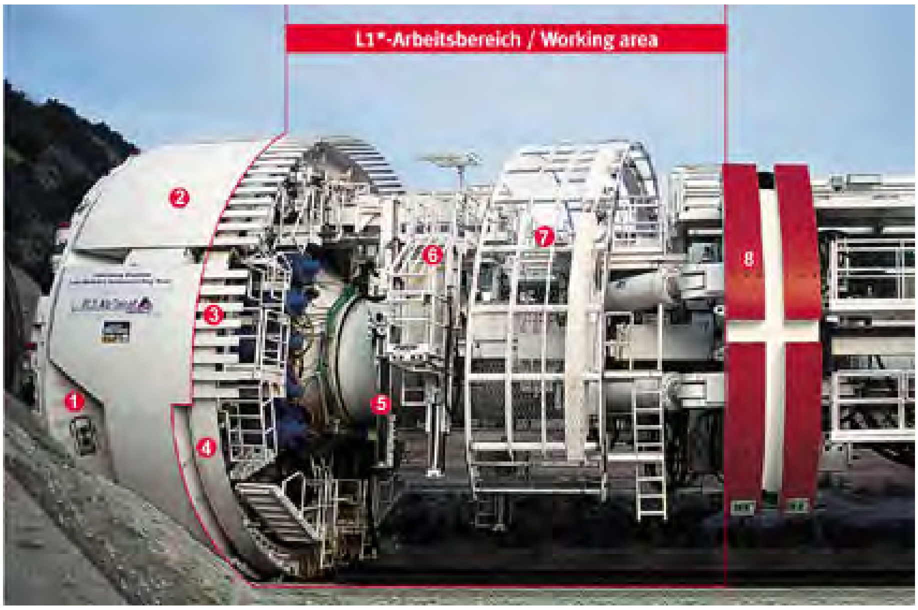
6.1 Tunnel Boring Machines

The following types of machines for mechanical excavation in hard rock have been considered with respect to the anticipated ground conditions;

- Open, hard rock, full face, rotary TBM
- Single shield hard rock TBM
- Slurry / Earth Pressure Balancing Machine

In the following sections these methods are briefly described by summarising the principal features of each type of machine and their mode of operation. The descriptions concentrate on the features that may affect performance and operation for the key conditions anticipated for the tunnel routes as indicated by the geology.

6.1.1 *Open Hard Rock TBM*



1. Cutterhead
2. Gripper shield
3. Finger shield
4. Ring erector
5. Anchor drill
6. Work cage with safety roof
7. Wire mesh erector
8. Gripper plates

Photo 4 : Open hard rock TBM

The open, hard rock, full face, rotary TBM consists of a full face rotary cutting head with cutting disks, drive unit, muck conveyor and grippers.

To advance the TBM the grippers push out into the rock and then provide resistance to the thrust from the cutter head moving forwards. As the working area of the TBM is open, the excavated area immediately behind the cutter head is supported temporarily using a variety of methods that include, steel arches, precast concrete, rock bolts or timber lagging.

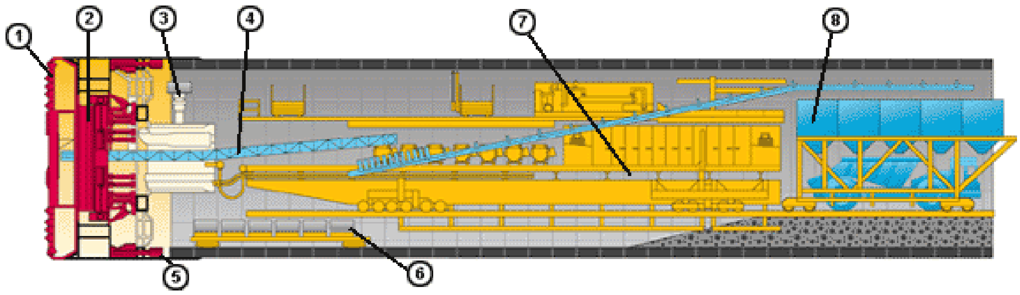
Open TBMs should, therefore, only be employed where thorough ground investigations indicate that minimal early support is expected and the final lining can be cast in situ at a later time, usually when the TBM has completed excavation. With this method early support is difficult to install, but should it not be installed immediately behind the cutterhead there are few other opportunities until the rear of the backup train is reached. This presents some risk to the operation and personnel.

Open TBMs can also be faced with difficulties where high water inflows are anticipated, unless well designed pre-grouting equipment is incorporated. Grouting equipment can interfere with the temporary support erection required behind the cutter head, therefore careful thought and design of the machinery is required.

Due to their dependence on gripper reaction for thrust, open TBMs can often be difficult to advance through zones of weak, faulted or weathered rock, as the rock can not withstand the pressure exerted by the grippers.

Tunnelling performance of a Gripper TBM very much depends on the time required for rock stabilization measures behind the cutter head.

6.1.2 Single Shield Hard Rock TBM



1. cutting head	4. conveyor belt	7. backup system
2. drive	5. tunnelling jacks	8. silo car
3. erector	6. lining segment supply	

Photo 5 : Single Shield TBM

The single shield hard rock TBM has a cutting wheel fitted with hard rock discs which rotate on the working face and notch into it. The notching effect causes sections of rock to break off. Buckets, located behind the discs, transport the rock behind the cutting wheel. Conveyor equipment then transports the excavated material out of the tunnel.

The single shield TBM, prevents ravelled rock in weak or blocky or highly fractured ground from falling into the tunnel and therefore also provides protection to the miners working on the TBM. The single shield TBM is therefore suited to ground conditions with short stand-up times for the rock. The single shield TBM employs shove rams to provide forward propulsion for the TBM and the reaction for the cutting head. These rams thrust off a segmental lining. As the single shield TBM does not have grippers it is therefore used in conjunction with a segmental tunnel lining.

Further considerations include conditions where ravelling of closely jointed rock may occur. In these conditions it is necessary to build the segments inside the tail skin of the TBM. As the single shield TBM has no grippers, excavation can not take place at the same time as erecting the segmental lining. It requires the machine to stop forward progress while a ring is being erected.

6.1.3 Slurry or Earth Pressure Balance Machines

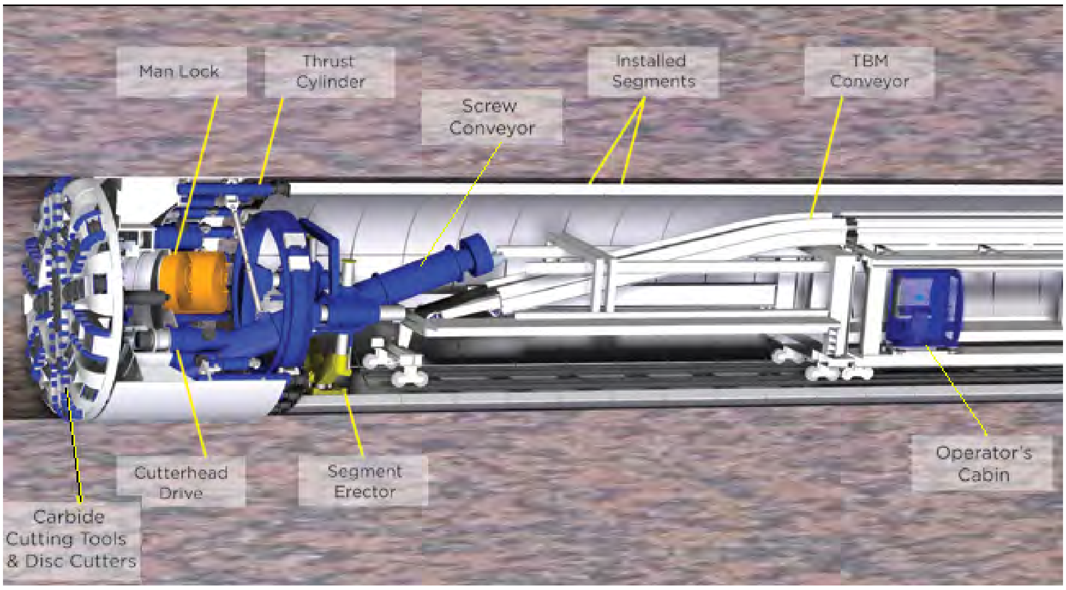


Photo 6 : EPB TBM

Single shield TBM's come in two different types of mode of operation, EPB and Slurry.

Earth Pressure Balance Mode - This excavation mode provides continuous support to the tunnel face by balancing the inside earth and water pressure against the thrust pressure of the machine. The ground excavated by the cuttinghead is mixed and accumulated under pressure in the cuttinghead chamber, and is then extracted by a Screw Conveyor and removed by conveyer to the surface. The pressure in the cuttinghead chamber is controlled by balancing the rate of advance of the machine and the rate of extraction of the excavated material by the Screw Conveyor.
Manual or automatic operation of the EPB system is possible through integrated PLC and computer-control systems.

Slurry Pressure Balance Mode – The Slurry excavation mode provides continuous support to the tunnel face by balancing earth and water pressure in the in-situ soil with a pressurized bentonite slurry. This excavation mode provides continuous support to the tunnel face by balancing the inside earth and water pressure against the slurry pressure of the machine. The ground excavated by the cuttinghead is mixed and accumulated under pressure in the cuttinghead chamber, and is then extracted by a Screw Conveyor and pumped to the surface for removal. The pressure in the cuttinghead chamber is controlled by balancing the rate of advance of the machine and the rate of extraction of the excavated material by the Screw Conveyor.

6.2 Selection of most appropriate TBM

Before any final decision can be made on the type of TBM most suitable to this project, more detailed information is required on the rock that will be encountered. The type of rock to be driven through will determine the head configuration and the type of machine. As such the type of rock, permeability and its abrasiveness will need to be determined as these have different impacts on the various designs of TBM. In addition, the variability of the ground along the route will need to be established to determine the worst case conditions.

Due to the tunnel being driven below the sea bed, the unknown nature of the discontinuities, and the possible risk of high water inflows, the least risk option is to use a pressurised TBM with fibre reinforced concrete segments (slurry or Earth Pressure Balance machine). These machines enable the anticipated presence of water to be contained and controlled at all times.

Fibre reinforced concrete segment have the advantage of ease of manufacture, as they do not contain reinforcement cages and they are also very durable. However, care is required during their manufacture as the concrete quality required is high.

TBM construction will also provide the quickest construction time of the longer tunnel lengths, would have the least risk to the tunnelling operation, and studies into the impact of blasting on nearby structures would not be required.

The equipment and risks associated with TBM methodology are provided in Table 2.

Table 2: Equipment and risks associated with TBM method

TBM Equipment	TBM risks
TBM, back up and train	Breakdown of plant: TBM, back up or train
Power Supply	Blockage of slurry or grout pipes
Ventilation	Squeezing ground (v unlikely in this case)
Crane and access	Settlement monitoring
Segment manufacture	Supply of segments and materials
Surface muck handling	Connection to rising shaft/diffuser / screen

7 Construction duration

In order to establish average advance rates which can reasonably be adopted for planning, experience of past projects has been considered. This is considered to be a more realistic approach than making theoretical assessments of boreability. Case histories suggest that average rates of progress are invariably controlled by factors other than the ability of a TBM to cut rock, or a drilling jumbo to drill a face of charge holes, for example. It is the proper identification of these factors, and the assessment of the risk, that is considered to be the most realistic approach to the planning of the tunnels on this project.

Average TBM advance rates depend on several key factors as follows:

1. Rock Structure: depending on the rock structure encountered by the machine, the advance rate will vary. The cutterhead is designed for an average rock type, based on the anticipated geology. If bad ground is encountered, with highly weathered material, the advance rate will generally be slower than if medium strength rock is being bored and for which the machine was most likely designed. Similarly, stronger rock will probably reduce the advance rate in this situation.

2. Rock Strength and Abrasivity: Penetration rates of a TBM decrease as the rock get harder (higher confined compressive strength). Highly abrasive rocks will reduce overall progress rates by not only reducing the penetration rate, but by requiring more frequent stoppages for cutter changes.

3. Flexibility of Machine Design: If a TBM is properly designed for the ground conditions to be encountered, it will of course perform better than if it is not. The ‘correct’ design is very dependent on the level of site investigation carried out, as proper knowledge of the geology and rock structure is fundamental.

4. Personnel: The TBM crew and backup staff must have the appropriate experience and skills to achieve good advance rates and to maintain the machine in good working order over several years. If this is not the case, much time can be wasted as crew and staff learn ‘on the job’.

5. Ancillary Equipment: Properly planned back up systems and transport to and from the TBM, will improve advance rates as materials such as precast segments, grout and other materials are brought quickly to the face, and muck is taken out equally efficiently. No matter how good the transport system is, it is common to have a TBM standing idle waiting for muck cars in order to start boring. This is especially true in the case of long drives. This is a reason why an increased number of modern TBMs have a conveyor or slurry mucking system.

Assuming these, and other key factors , are assessed properly and / or are well catered for, the rate of advance of a TBM can be impressive from a consideration of case histories where no major problems were encountered. On the other hand, extremely poor progress has been recorded on projects where some things did not go according to plan, or were simply not anticipated or fully assessed for risk, with appropriate mitigation measures taken in the first instance.

For the information available on the geology of this project at the moment, it is considered a reasonable assumption for the average time to advance on a 2.8m diameter TBM by 1.2m, ie one ring width, would normally be approximately 60 minutes. (It is emphasised that this is an average figure within a range of approximately 30 to 90 minutes). However in this instance an allowance of 120 minutes has been made to take into account the short length of the tunnel, crew learning curve and the strength of the rock. The calculation for the overall average advance per week on the basis of the above figure, assuming 2, 8 hour shifts working 5 day weeks is therefore

*8*2*5*4.2 = 336 metres/ month*

In addition to this figure the time taken to set up the TBM before the drive starts needs to be included. This will involve setting up the launch chamber and constructing the TBM "train". A reasonable assumption of 4 weeks should be allowed for this.

Times for construction of the three tunnelling options are presented in Table 3.

Table 3: Construction times for TBM options

	Length of tunnel (m)	Site set up/ shaft construction (months)	TBM construction Time (months)	Total Construction time (months)
Intake	550	3.0	1.6	4.6
Discharge Option 1- Tunnel from shoreline to diffuser only, with trench and buried pipe on land.	600	3.0	1.8	4.8
Discharge Option 2 – Tunnel from plant to diffuser	2800	3.0	8.3	11.3

8 Construction Costs for Tunnels

TBM tunnel construction has expensive start up costs which make the method less cost effective than drill and blast construction in lengths under 1000m. In tunnels longer than 1000m TBM construction becomes more cost effective as start / mobilisation costs are spread and production rates increase.

For the TBM construction it is assumed that the TBM will not be retrieved from the tunnel as it cannot be easily removed from the marine environment. Works required to create a shaft or cofferdam sufficient to recover the TBM, would likely involve blasting in the marine environment and significant marine resources which would counter the cost savings of TBM retrieval. Following detailed design it may be possible that a TBM can be purchased that can be dismantled within the tunnel and all sections other than the shield be removed via the tunnel.

9 Launch and Reception Arrangements

9.1 Launch Shaft

The launch shaft arrangement will need to be of sufficient size that a TBM can be launched and serviced. It is suggested that a minimum diameter of 7m is considered at this stage. However, when the final arrangement for the tunnelling method and machine has been determined then the diameter may need to be increased or a back shunt used.

For the purposes of this report, it is assumed that the shaft will be driven through competent rock that will be self supporting. The shaft could therefore be excavated using a drill and blast method and temporarily lined with shotcrete with spot bolting where required. The permanent lining will depend on the required final arrangements.

An estimate depth of 45m is required for the shafts close to the shore. This is based on the assumption that the tunnels will terminate where water depth to seabed is 20m, depth to seabed from the invert of the tunnel is 15m and the shaft is constructed on land where the ground height is 10m above sea level. For Discharge Option 2 where the shaft may be constructed on higher ground a depth allowance of 87m should be made to allow for the higher ground and the extra depth to achieve a 1% fall for drainage during construction.

Photo 9 : Installation of TBM in launch shaft

9.2 Reception shaft

The recovery of a TBM can be accomplished either via a shaft, driven up into a trench in the sea bed, or dismantled within the tunnel.

In this instance assuming hard rock, a sub sea shaft could be constructed using a vertical drilling machine to drive a shaft into the bedrock and provide the shaft with a steel liner. The limitation with this method is the size, which is 6m ID. However, with these limitations known, a TBM machine can be configured, so it can be dismantled within this space. However, as with the marine work discussed later, the cost and time taken to drill such a shaft significantly reduces any benefit of retrieving the TBM. It is estimated that once the equipment has been mobilised it could take 2 to 3 months to install a shaft for TBM retrieval.

Driving a trench into the seabed would require blasting in the marine environment. This is discounted in this report, as the idea of tunnelling is to avoid blasting in the marine environment.

9.3 Tunnelling site lay down / working areas

To carry out any tunnelling operation there is a requirement for an above ground site working area to service the tunnelling operation. Assuming administrative buildings and offices are already accounted for this space allows for equipment and materials to be stored and work to be carried out on material elements before lowering into the confined tunnel environment. To enable efficient working in the tunnel environment it is essential to have a properly set out working area above ground to minimise movements and crane delays into the tunnel. Space is required for ventilation equipment and other tunnelling support services.

For TBM operations a significant area is required for the storage of the lining segments. Tunnelling operations will be carried out 24 hours a day so adequate segments must be stored to allow for un-interrupted operation. Depending upon the concrete specification, segment design and the manufacturer's facilities, the segments could also be stored on site while the concrete reaches the allowable strength before they are used (eg 28 days). Hence this could require storage facilities for over 28 days worth of segments at full tunnel production rates. The storage would also require sufficient space to allow access to the correct segments and to allow delivery and unloading of recently manufactured segments. From previous projects it would be reasonable to suggest a lay down area of approximately 40,000m2 is required for the tunnel and shaft construction operations. Depending upon segment manufacture a further 6,000m2 should be allowed for segment storage.

10 Marine works

10.1 Marine Risers

Marine risers are required to connect the tunnels to the seabed to make the hydraulic connection. For the purpose of this report it is assumed that the marine risers will be similar in design to those used in the Gold Coast Desalination Plant, Sydney Desalination plant and those proposed at both Adelaide and Victorian Desalination plants.

It is envisaged that each tunnel will have one riser which will be topped with either an intake screen or brine diffuser. Each riser will be GRP lined and offset from the line of the tunnel to allow flexibility in construction programme and safety in making the connection.

Drilling is the preferred method of construction as it is more accurate than blasting, which is the only other viable alternative, considering the strength of rock that will be encountered. Blasting is not considered, as the idea of tunnelling is to avoid blasting in the marine environment.

Each riser will be constructed by first installing a 3m diameter vertical steel casing to rock head. The rock inside the steel casing and beyond the toe is then excavated by drill until the required depth at invert of the tunnel level. Due to the strength of the rock a specialist drill will be used which is supported by the casing. The excavated material is contained within the casing and pumped to a nearby barge for removal. Once the excavation is complete a 2m diameter GRP liner is lowered into the excavation and grouted into position. The GRP liner is then sealed ready for a dry and safe connection to be made to the tunnel. See Appendix for details of general arrangement for connection to tunnel and the installation steps.

It is anticipated that the marine risers will be approximately 15m long. This allows for the 10m of cover to the tunnel and for the depth of the tunnel with a sump below.

Photo 10 : Construction sub sea shaft using a vertical drill rig

10.2 Marine Equipment Required

To carry out the marine works some high cost equipment is required to overcome working in 25m depth of water with high tidal currents and provide a stable working platform from which to carry out heavy lifts. The platform must also be serviced and the materials required for construction transported to it.

- Marine working platform capable of carrying a large heavy lift crawler crane
- Marine service barge
- Tug Support
- Service boat and Helicopter access to platform

- Drill Rig
- Shore side storage and working area with crane.

All these items will require mobilisation with an associated cost. There is availability of suitable working platforms in Australia at present, but the drill rig may need mobilisation from Europe.

In addition to the equipment a suitable lay down area is required for the marine construction elements. This area will need to be located in a port facility and be used for storage of the main construction items and assembly of the intake structure, marine risers and diffuser structure before their transport to site. Ideally the lay down facility should be located as close to the work site as possible to minimise transfer times. It should also have good road access and be able to facilitate lifting of large loads by crane. The minimum marine lay down area required would be approximately 8000m2, with at least 50m of water frontage for loading, and allow for heavy lift cranage and vehicle turning, office space and fabrication.

10.3 Marine construction risk

Risk associated with the construction of the marine risers is in two parts. Firstly programme risk which holds a significant cost implication due to the equipment involved. Programme risk stems from the weather and sea conditions that can be expected in the area which prevent work being carried out. In the area of this study a reasonable assumption would be that 25% of the working time would be lost due to swell and high winds preventing works.

Safety risks in the marine environment can be considered to be higher but this is countered by a greater awareness of the risk. The risks associated with the work that differ from a similar land based operation come from the risk of working over water. These are addressed by correct personal safety equipment, ensuring a safe route to and from the work site and addressing the risk of falling from the platform while working.

Making the connection between the riser and the tunnel involves a risk of flooding. However this is mitigated by ensuring that the riser is sealed and watertight and that test holes are provided before excavation of the connection starts to check for water paths.

Risks:

- Heavy lifting and general construction risks with machinery
- Working over water
- Flooding of the tunnel

10.4 Marine Construction Programme

It is anticipated that 8 months will be required for the marine works, with a total of 4 months of on site activity constructing both of the risers. These 4 months include a 1 month allocation for lost time due to weather. The additional 4 months allows for mobilisation and de mobilisation on site. If equipment is to be mobilised from outside Australia then more time may be required.

It is recognised that the cuttlefish breeding season only allows a 7 month construction window between October and May (construction excluded between 1 May and end of September) so the construction programme will need to be carefully managed. However this marine work would have minimal impact on the environment and is a distance from the areas identified for breeding so may be considered exempt.

If the construction programme exceeds the duration of the non-breeding period, construction would be delayed until after the breeding season, resulting in costs significantly increasing to allow for longer rental times and demobilisation/ remobilisation of equipment. The barge would need to be kept as it would be fitted for the role and if it was released then it would be utilised elsewhere in the world.

11 Previous Tunnel failures

11.1 Tunnel failures

Once constructed, tunnels generally do not fail, in that they do not collapse. Some tunnelling techniques contain a risk of excavation instability during construction before support is installed, but this is managed in the design and by the contractor.

A tunnel failure in its fitness for purpose is related to the hydraulic capacity of tunnels. This is considered in the design stage and this takes into account any fouling of the tunnel surface that may occur. This is generally managed by operational and maintenance procedures.

11.2 TBM Tunnels

There are no known failures of TBM tunnels, either during the construction phase or the operational phase. This is due to the excavation being fully supported during construction, and the segmental linings being adequately designed to support the tunnel excavation. The design of the tunnel linings also takes into account any requirement for durability and waterproofing.

Delays may occur in the tunnel construction programme due to unexpected ground conditions or mechanical breakdown, but this does not constitute a failure.

Appendix

Alternative tunnelling construction methods

Pipe jacking

Pipe jacking is a technique of installing underground pipes using hydraulic jacks to push specially designed pipes through the ground behind a tunnel shield from the thrust pit. After driving a length of pipe through the ground a new pipe is placed behind it in the pit and the process is repeated.

A lubricant may be used on the outside of the pipe to reduce friction along its length. For long lengths of pipe intermediate jacking stations can be incorporated to move the total length of the pipe along in sections, thus reducing the jacking forces required at the jacking pit.

There are a full range of machines that can be used at the head depending to the material to be tunnelled through. However at the diameters of tunnel that are being considered in this report the use of pipejacking equipment is limited in its capability and availability. In addition Discharge Option 2 maybe to long for this method

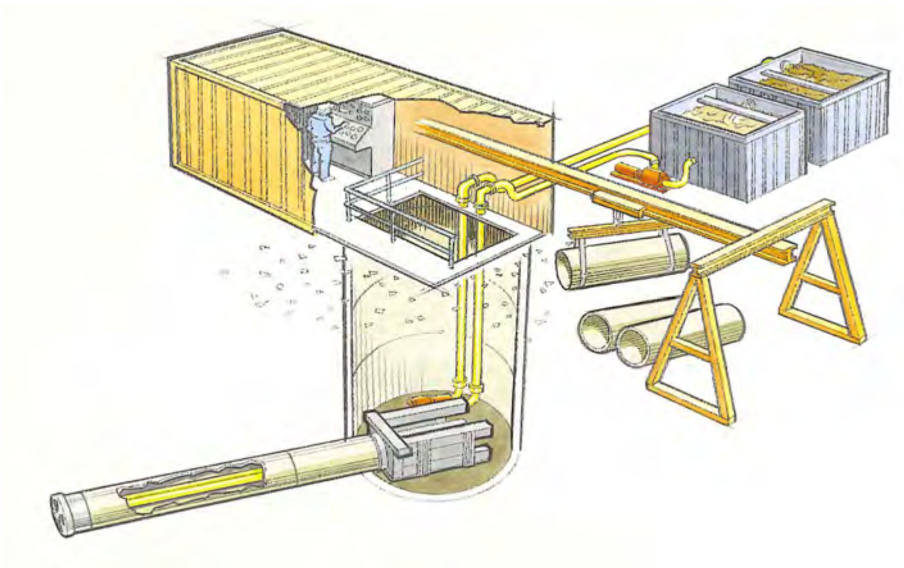


Figure 2 : Pipe Jacking arrangement

Drill and Blast or a Mechanical excavation

Drill and Blast or mechanical excavation tunnelling techniques use either of the two methods to excavate limited advances of a tunnel before installing support. Once the excavated section has been supported the next section of the tunnel is excavated. The support can either be temporary while waiting to the tunnel completion or the permanent support.

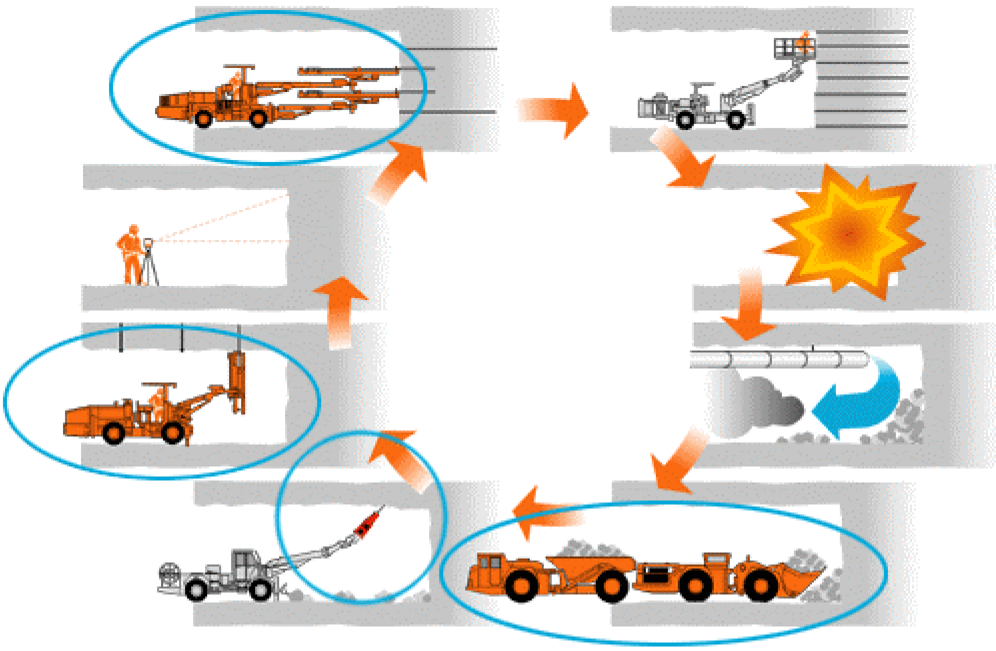


Figure 3 : Drill and Blast operation

A Drill and Blast excavation cycle typically consists of the following activities:

- Probing and pre-injection grouting (the latter only if required)
- Drilling the face
- Charging the holes, detonation and ventilation
- Scaling and mucking
- Installing temporary support as required

- Construction of permanent drainage, waterproofing and concrete lining
- Repeat of process

In the following sections these activities are discussed in relation to the geotechnical programme and risk constraints anticipated for these small diameter sub sea tunnels.

A particular constraint on the use of drill and blast methods can be any limitation on the permissible vibrations from blasting. These may be of particular significance where blasting near to shafts or at shallow cover in the vicinity of sensitive installations such as Pt Lowly lighthouse. These limits and appropriate mitigation measures, are discussed first with particular reference to this project.

Non explosive methods are available in the form of expansive cement that is used in place of explosives. However the use of these materials in a project such as this is limited due to programme constraints and the effectiveness on a large scale.

Where weaker or heavily fractured rock is encountered, mechanical excavation by a roadheader can be used instead of blasting. The roadheader is generally restricted to excavation of rock with unconfined compressive strength (UCS) values less than approximately 75 to 100MPa. This is a result of the self-weight of the machine and stiffness of boom limiting its ability to provide the necessary reaction for the cutting head. Hence, in this instance, mechanical excavation does not seem practical because of the rock strength.

Vibration Limits and Mitigation Measures

For sub sea tunnels such as these, consideration of vibration limits and mitigation measures due to blasting will need to be consider due to the sensitive nature of the wild life in the overlying sea bed and the presence of the lighthouse.

The following are general key considerations relevant to the use of explosives. It is assumed that detailed Blast Risk Studies and onshore trials and verification will be undertaken as required, prior to construction commencing:

I. Blasting times and warning signals should be coordinated and agreed with local authorities,

II. Non-explosive methods may be used for the sections close to shafts

III. Blast doors should be constructed at tunnel portals, for short tunnels

IV. Blast designs should be such that peak particle velocities (PPV) do not exceed agreed limits (typical 25mm/sec); vibration monitoring stations should be set up at critical locations

Pre-injection Grouting

With drill and blast construction there is no protection offered to the workers, therefore the heading stability is of the utmost importance. As such, probe drilling is carried out and criteria set for inflow rates. At the Hvalfordur sub sea tunnel, the criteria of 5 litres/minute was set and if this amount was exceeded then fan grouting was carried out. Additional, items such as, penetration rate, water colour, temperature, pressure, salinity can also be monitored during the probing.

Probing, and drilling for the pre-injection when required, is usually carried out using a drilling "jumbo". Depending on the preferred machine configuration, this may allow two or three holes to be drilled simultaneously. The machine used also depends upon the size of the tunnel.

Photo 7 : Work on the Face and Construction Ventilation

For maximum efficiency the pre-injection itself should be carried out using a multipoint injection rig. Mircrocement grouts with hydration control additives to control the spread of the grout are generally found to be the most efficient material. Chemical grouts should be avoided as far as possible for environmental reasons, but may be required in certain conditions such as some fault zones.

Shotcrete

Shotcrete is an accepted form of structural support in tunnels both in temporary and permanent support. Concrete is sprayed against a surface and rapidly gains strength and becomes a structural element to provide support.

For this project it can be anticipated that shotcrete may be used for initial ground support if poor ground is encountered. If so it would be applied in layers to a total thickness varying between 50 and 150mm, as close to the face as possible. Shotcrete will also be required for immediate support if loosening or ravelling ground is encountered. It may be used in conjunction with rock bolts in fresh, but closely jointed rock conditions.

Depending upon the results of geological investigations the tunnel excavation may intercept areas with high water flows and /or pressures. The presence of water adversely affects the application and performance of shotcrete by reducing its adhesion to the rock. Significant water sources must generally be detected and treated ahead of the tunnel face by forward probing and grouting. Therefore as far as possible, major water inflows should have been dealt with before the rock is exposed in the tunnel. However, if running water is present the water source has to be channelled and controlled prior to shotcreting. This can be significant factor in affecting advance rates.

There are two types of shotcrete, dry-mix where the components are mixed dry and water is introduced at the nozzle and wet-mix where water is added as the components are mixed as for conventional concrete. In recent times, wet-mix shotcrete has been favoured. Hydration control development has enabled shotcrete to be mixed well in advance of it being required. In long tunnels, such as those on this project, this allows the shotcrete to be batched and taken into the tunnel at a convenient time and stored until such time as it is needed. The wet-mix process has the advantage of greater control on the mix proportions and reduces cement dust build-up in the tunnel.

The use of this technique is dependant upon the size of the tunnel and the size of the equipment required.

Photo 8 : Shotcreting as the Permanent Support for the tunnel

Shotcrete can be used as the permanent support in the tunnel. As such there will be two applications of the material, the first being the initial support, of about 50mm to 200mm of fibre reinforced shotcrete and the second or final lining having a thickness of about 150mm to 300mm, depending on the design, of structural strength concrete and with a troweled finished. Application must be carried out by a skilled operator to ensure the quality.

.

Rock Bolts

Bolts are applied to provide support to the rock either in spots (single) or patterns (multiple). There are two types of bolt that appear to be appropriate for the ground conditions of this project:

- Fully grouted rock bolts

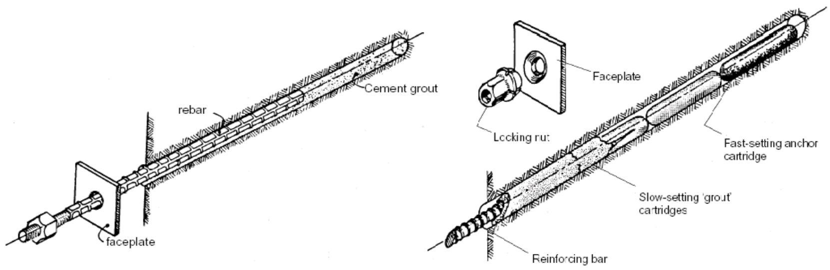


Figure 4 : Grouted Dowel and Rock Bolt

- Friction anchored rock bolts

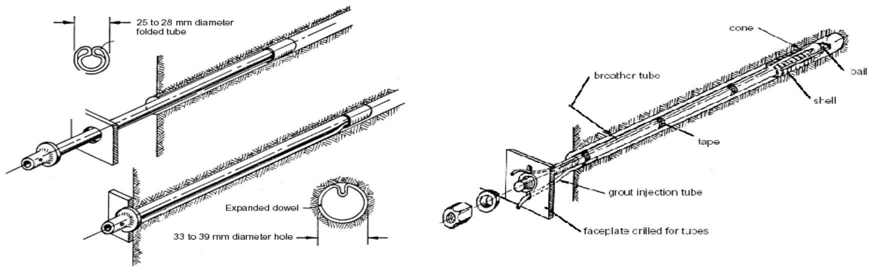


Figure 5 : Swellex Dowel and Mechanically anchored rock bolt

For routine spot and pattern bolting in a moderately jointed rock mass the fully grouted rock bolts appears to be the most appropriate choice for ground support. However if an area where the rock mass has high permeability is encountered, a friction anchored bolt may be considered to be more suitable.

Friction anchored bolts include Split Sets by Ingersoll Rand or Swellex produced by Atlas Copco. The latter can be considered as being more flexible because it has a wider range of application, in terms of ground conditions and is not too dependent on the 'as drilled' hole diameter.

The Swellex bolt would be preferred where immediate rock bolt support is required. The advantage of Swellex is that it can be installed very quickly and provides immediate support along the full bolt length. This type of bolt is more expensive than grouted rebar.

The installation rate for bolting will vary with the type of equipment used.. However, bolts are often simply installed by hand, in holes drilled by a drilling jumbo.

The above discussion assumes that the rockbolts are to be considered to be temporary support, ie effective until the permanent concrete load bearing lining is constructed. If bolts are to be considered as permanent support then proper consideration should be given to long term durability appropriate to a design life of 100 years. For these, double corrosion protection should be provided and for simple rock bolts, this is most efficiently provided by galvanising and epoxy coating the bolt which is then fully cement grouted. A number of proprietary systems are available. In addition, bolt design could be oversized to allow a sacrificial layer of steel.

Primary Support Arrangements for Drill & Blast

The initial primary support design would be made using the support recommendations of the rock mass classification system, such as Q-system or Geomechanics classification RMR. In Barton's Q-system, 38 support categories are given depending on the rock tunnelling quality index (Q) and the equivalent ratio. A design chart is produced which gives details of support design using rockbolts and fibre reinforced shotcrete . Various thickness of fibre reinforced shotcrete, bolt spacing and bolt length depending on the Q value and equivalent dimension are then chosen.

For the purpose of this study it has been assumed that the rock will be class I in Bieniawski's Geomechanics Classification RMR System. Therefore a primary excavation and support rate of 3m could be achieved using spot bolts. This is due to the expected rock structure of the quatzite.

Drainage, Waterproofing and Lining Construction

It is not know at this stage if the final lining will require water proofing, or if a certain leakage could be accepted. This decision is driven by ground contamination concerns or future dewatering requirements. This will depend upon the client requirements and specification. The material surrounding the tunnel will be water bearing but the water pressure will be at a similar value to the water in the tunnels so any leakage will be minimal and is not a concern in relation to the system hydraulics.

For a drill and blast tunnel the activities starting with the provisions of drainage, waterproofing and concrete lining can usually only commence once a section of tunnel has been fully excavated. The tunnel surface profile, with its rough surface and overbroken areas, must generally first be backfilled to create a relatively smooth surface. This is generally done using shotcrete. When a suitable surface has been prepared, a drainage layer, in the form of an appropriate geotextile, is pinned to the smoothing shotcrete surface, using steel nails with large PVC washers. Finally, a PVC waterproofing membrane (typically 2mm in thickness) is erected by heat welding the membrane to the PVC washers, after which the mass concrete lining is ready for casting as the inside surface.

Alternatively a GRP liner can be installed and backfilled with grout.

The use of a waterproofing membrane in the manner described above will generally produce a permanent lining of a very high quality with little cracking. This is generally considered to be due to the low frictional restraint between the lining and the rock.

It should be possible for the linings on this project to be constructed in mass concrete if the drill and blast method is adopted. This avoids consideration of corrosion protection of reinforcement.

Discussion and Summary of Drill and Blast method of tunnel construction

As with all tunnelling sites, their success is very much dependent on have the logistics properly planned and designed. The selection of equipment is very important and proper choice will lead to a safe and healthy working environment, to maximise progress and minimise cost. Flexibility in probing and pre-injection operations and in drilling pattern and blasting design, will help to cope with the various groundwater and rock conditions. For rock support, adopting modern techniques, such as robotically applied steel fibre reinforced shotcrete, together with proper guidelines for various support requirements for all kind of ground conditions, will allow a prompt and effective response to varying ground conditions.

The disadvantage of this method of tunnelling is mainly down to the exposed nature of the face and safety risks that this leads to. The nature of any disconuities and the permeability of the rock needs to be determined in order to establish if this exposed method of tunnelling could be employed. In addition, the geology of the routes will need to be determined to establish the continuity of the bed rock. Unforeseen, soft ground would be difficult to cope with, with an exposed face. The effect of blasting will also need to be determined, to establish if the blasting process will open up plains of weakness in the rock and therefore lead to unacceptable levels of water ingress.

The equipment size and availability for the drilling of the shot holes and grouting also needs to be considered as this will have a bearing on the size of the tunnel excavation. South Australia has a history of mining and it is accepted that there will be significant local experience available in drill and blast tunnel construction from the mining industry.

Drill and Blast Advance Rates

A typical drill and blast cycle is made up of the following activities:

1. Drilling charge holes
2. Charging the face and blasting
3. Ventilation before re-entry
4. Mucking
5. Scaling
6. Installing rock support
7. Delays for various reasons (typically 20% of the total available time).

For this project, systematic probing ahead of the face will be required. In a drill and blast tunnel probing would normally be carried out up to 20m ahead of a face using the drilling jumbo. Two probe holes might be expected per week on average. However, these would normally only impact by a small number of hours on the week’s production and can therefore reasonably be assumed to be included under the allowance for ‘delays’.

Depending on ground conditions encountered at the face, the time it takes for the typical cycle will vary. For example, if dense support is required it will clearly take longer than if no support is required. To determine the rate of advance for this project, a typical drill and blast cycle can be considered with average support requirements (support type Class I in five support system , typically bolts and shotcrete) as set out below.

For a typical 7 day working week in a drill and blast tunnel, the advance rate is thus:

*3x7x*24.2= 176m/month/face*

	Length (m)	Site set up and shaft construction (months)	Drill & Blast construction Time (months)	Total construction time for Stage 1 (months)	Construction of Final Lining	Total construction time (months)
Intake	550	2.0	3.1	5.1	1.6	6.7
Discharge Option 1- Tunnel from shoreline to diffuser only, with trench and buried pipe on land.	600	2.0	3.7	5.7	1.8	7.5
Discharge Option 2 – Tunnel from plant to diffuser	2800	2.0	15.9	17.9	7.9	25.8

Table 3 : Comparison of Construction times for the Drill and blast option and TBM segmental lining.

Failure of Drill and Blast Tunnels

Failure of drill and blast tunnels does not occur in the operational stage as the permanent lining is designed to take the loads imposed upon it and accounts for the requirements for waterproofing and durability.

During construction there is a risk of partial collapse of the face and rock fall in the unsupported sections which have been recently excavated. This risk is limited to the time these sections are exposed between their excavation and when the correct temporary support is installed. These risks are managed at all times with the rock being assessed and the amount and type of excavation being evaluated and the appropriate support being installed. Generally any rock fall or instability is a result of a lack of appreciation of any change in the properties of the rock being excavated or incorrect installation of the temporary support that is specified in the design.

ATTACHMENT B

Desalination plant: Tunnelling waste transfer – Traffic impact assessment

BHP Billiton

Attachment B - Desalination Plant: Tunnelling Waste Transfer

Traffic Impact Assessment

BHP Billiton

Attachment B - Desalination Plant: Tunnelling Waste Transfer

Traffic Impact Assessment

November 2010

Arup
Arup Pty Ltd ABN 18 000 966 165

Arup
Level 17, 1 Nicholson Street
Melbourne, VIC 3000, Australia
Tel +61 3 9668 5500 Fax +61 3 9663 1546
www.arup.com

Job number 085200-10

Contents

Appendices

1 Introduction

Arup was engaged by BHP Billiton (BHPB) to undertake a Traffic Impact Assessment (TIA) on the effects of constructing and operating a desalination plant at Port Bonython, South Australia. This includes an assessment of the operation of Port Bonython Road, the Lincoln Highway, Stuart Highway and Olympic Way as a result of the additional traffic. The desalination plant is part of the development associated with the proposed Olympic Dam Expansion (ODX).

The proposal for the desalination plant follows the public exhibition of the Environmental Impact Statement (EIS) for the expansion of Olympic Dam in late 2008. During 2009, a number of responses from various bodies and the public were received in relation to the EIS including in relation to the proposed overland routes of the infall and outfall pipe lines associated with the Desalination Plant at Port Bonython.

Submissions received raised environmental concerns with the proposed overland routes for both pipelines compared to other desalination plants around Australia. The supplementary EIS is proposing to tunnel the outfall pipe line whilst retaining the infall pipeline as an overland route. As a result of the required tunnelling the tunnelling waste has been proposed to be transported back to Olympic Dam for disposal. This disposal of tunnelling waste will generate truck movements travelling to and from Olympic Dam. Concurrent with the proposed desalination plant is the proposal by Stuart Petroleum to construct a fuel storage facility at Port Bonython (Port Bonython Fuels).

This report reviews the forecast traffic demands as documented in the EIS for ODX as well as the estimated impact on the road network as a result of the BHPB Desalination Plant and Port Bonython Fuels projects. The results of this report should be read in conjunction with the supplementary EIS.

A site location map is shown in Figure 1.

Figure 1 Site location of Desalination plant at Port Bonython, South Australia

2 Existing Conditions

A review of existing conditions along Port Bonython Road, Lincoln Highway, Stuart Highway and Olympic Way is described within this section.

2.1 Port Bonython Road

Port Bonython Road connects the community of Point Lowly to the Lincoln Highway over a distance of approximately 24 kilometres. It is an undulating two lane two-way single carriageway road with unsealed shoulders. The current carriageway width is approximately 7 metres. The Santos Port Bonython Fractionation Plant is located 500 metres west of Point Lowly. Between the Lincoln Highway and immediately west of the Fractionation Plant the speed limit is 110 km/hr which reduces to 80km/hr for the remaining road length to Point Lowly. The road operates well below capacity, including during peak times.

The traffic volumes along Port Bonython Road are considered to be relatively constant in recent years given the road provides connectivity to the small residential community of Point Lowly; the Santos Point Lowly Fractionation plant and Clean Seas Kingfish Aquaculture Farm only. The most recent available traffic count (assumed to be reflective of traffic volumes along the road) was undertaken in 2001 by the Department for Transport, Energy and Infrastructure (DTEI), as noted in the Port Bonython Fuels, Traffic Impact Assessment report dated 21st May 2009[1]. The Average Annual Daily Traffic noted at that time was 400 vehicles with 10% commercial vehicles. The report also notes that (since that date) the "only increase would be due to the Clean Seas factory which would increase traffic by 20 to 30 vehicles per day".

For the purposes of this assessment it has been conservatively assumed that traffic along Port Bonython Road has grown at nominal 1% per annum since the 2001 count. Accordingly, the 2010 two-way volumes along the road i.e. after nine years of growth are estimated as 437 vehicles / day (including 44 commercial vehicles).

2.2 Lincoln Highway

The Lincoln Highway is a two-lane two way road along the vast majority of its length between the cities of Port Augusta and Port Lincoln, a distance of approximately 200 kilometres. It comprises mainly a two lane two-way single carriageway with unsealed shoulders and localised widening in the vicinity of intersections. The current carriageway width is approximately 7 metres and the road operates below capacity, including during peak times.

Whilst no intersection traffic volume counts at Port Bonython Road / Lincoln Highway were available, there was a DTEI count for the Lincoln Highway north of the site i.e. 1.6 km south of the Eyre Highway intersection. The volume count recorded traffic across practically all of the year 2006. The Average Annual Daily Traffic (AADT) for the road was 1,700 vehicles per day (v.p.d) including 16% commercial vehicles i.e. 272 vehicles. The five day average daily traffic for 2006 was 1,769 vehicles and the seven day average daily traffic was 1,722 vehicles.

The TIA report prepared for Port Bonython Fuels (May 2009) indicates that traffic volumes on the Lincoln Highway are approximately 1,800 v.p.d., however, the year of the survey is not specified. Assuming, the traffic volume refers to 2009, then the growth rate since 2006, when weekday volumes were 1,769 vehicles per day, equates to a 0.6% per annum. Although seemingly low, it is considered that this is a reasonable estimate of traffic growth given that the land use in the area has not changed significantly in recent years.

Having regard to the above, for the purpose of this assessment the 2010 volumes along the Lincoln Highway are estimated at 1,810 v.p.d. (including 280 heavy vehicles).

[1] Port Bonython Fuel Storage and Processing facility, Traffic Impact Assessment", by QED Pty. Ltd. and dated 21st May 2009

2.3 Stuart Highway

The Stuart Highway is a two-lane two way road that connects Darwin and Port Augusta. Each lane is between 3.4 and 3.5 metres wide and there are unsealed shoulders of varying width. Localised widening is generally provided in the vicinity of intersections. The speed limit along the road is 110 km/hr except through built up areas such as Port Augusta.

Traffic volumes along the Stuart Highway were obtained from DTEI for the calendar year 2006 (see Table 1) and assessed as part of the 2008 Olympic Dam EIS.

Table 1 Stuart Highway and Olympic Dam traffic volumes for 2006

Road (Location)	Survey Period	Direction	Traffic Volume		Type (%)		
			5 day	7 day	Car*	Bus /LC	HV*
Stuart Highway (northwest of Yorkeys Crossing, north of Port Augusta)	Jan 1 – Dec 31, 2006	Two-way	803	784	75	5	20
		Northbound	401	400	-	-	-
		Southbound	402	385	-	-	-

**Note: Car includes 'cars towing' (i.e. class 1 and 2 vehicles), Bus/LC is Bus and 2 Axle Trucks, HV is Heavy Vehicles*

2.3.1 2008 Traffic Surveys

In addition to the 2006 volumes, as a means of identifying existing traffic conditions along the route for input into the EIS, traffic counts were undertaken in July 2008 at six locations between Adelaide and Olympic Dam. The results of the counts along Stuart Highway are shown in Table 2. Traffic volumes at the Stuart Highway location have been modified based on a Weekly Seasonal Factor (WSF) to achieve AADT volumes.

Table 2 Seasonally adjusted daily traffic volumes for Stuart Highway 2008

Location	Survey Period	WSF[2]	Direction	Cars/ Car Towing	Bus/2 Axle Trucks	Heavy Vehicles Volume	Total AADT
1.1km Northwest of Yorkeys Crossing	14-18 July 2008	0.80 (Week 29)	Two-way	606 (71%)	69 (8%)	180 (21%)	855 (100%)
			Northbound	283 (68%)	38 (9%)	96 (23%)	417 (100%)
			Southbound	324 (74%)	31 (7%)	83 (19%)	438 (100%)

[2] *Weekly Seasonal Factor*

2.4 Olympic Way

Olympic Way travels between the Stuart Highway at Pimba and Olympic Dam, approximately 77 kilometres. The road is a two-way two lane road with 3.5 metre lanes and unsealed shoulders approximately 1.75 metres wide. The speed limit along the road is 110 km / hour.

Traffic volumes along Olympic Way were obtained from DTEI for the calendar year 2006 (see Table 1) and assessed as part of the 2008 Olympic Dam EIS.

Table 3 Olympic Dam traffic volumes for 2006

Road (Location)	Survey Period	Direction	Traffic Volume		Type (%)		
			5 day	7 day	Car*	Bus /LC	HV*
Olympic Way, northeast of Woomera	Aug 21 – Aug 27, 2006	Two-way	547	484	74	5	21
		Northbound	262	229	-	-	-
		Southbound	285	256	-	-	-

**Note: Car includes 'cars towing' (i.e. class 1 and 2 vehicles), Bus/LC is Bus and 2 Axle Trucks, HV is Heavy Vehicles*

2.4.1 2008 Traffic Surveys

In addition to the 2006 volumes, as a means of identifying existing traffic conditions along the route for input into the EIS, traffic counts were undertaken in July 2008 at six locations between Adelaide and Olympic Dam. The results of the counts along Olympic Way are shown in Table 4 .

Table 4 Seasonally adjusted daily traffic volumes for Olympic Way 2008

Location	Survey Period	WSF[3]	Direction	Cars/ Car Towing	Bus/2 Axle Trucks	Heavy Vehicles Volume	Total AADT
24.2km Northeast of Woomera	14-18 July 2008	n/a	Two-way	458 (73%)	49 (8%)	116 (19%)	623 (100%)
			Northbound	228 (74%)	26 (8%)	56 (18%)	310 (100%)
			Southbound	230 (74%)	23 (7%)	60 (19%)	313 (100%)

[3] *Weekly Seasonal Factor*

3 Proposed BHPB Desalination Plant

As part of the works associated with the Olympic Dam expansion, BHP Billiton is proposing to construct a Desalination Plant at Point Lowly, South Australia commencing in 2014. The works comprise:

- The desalination plant;
- an 87 metre deep shaft from the desalination plant; and
- a 2.8 km outfall pipe line extending from the desalination plant out into Fitzgerald Bay.

The tunnelling of the shaft followed by the outfall pipe is expected to generate 33,000 m^2 (or 53,000 tonnes) of tunnelling waste that would need to be transported to Olympic Dam for disposal. The waste will be transported by trucks to Olympic Dam.

The BHPB Desalination Plant is proposed for construction in 2014, being operational from 2015 onwards.

3.1 Construction

The construction of the Desalination Plant is proposed to commence in 2014. There are two parts to the construction stage:

- Construction of the buildings and infrastructure; and
- Construction of the 2.8 km outfall pipe line and the 87 metre deep shaft.

3.1.1 Desalination Plant Construction - 2014

Details provided by BHPB in the document "Scope of Work: Traffic Impact Assessment. Transport of tunnelling waste: Port Bonython to Olympic Dam", July 2010 estimate that there will be 28 vehicle movements per day generated by the construction activities. The construction period is assumed to occur within the 2014 calendar year.

Information provided by BHPB indicates that 18 of these movements relate to construction trucks and 10 movements relate to commercial vehicles (including a bus delivering daily labour to and from the site). It is assumed that the bus movement would occur during the AM and PM peak hour, and that the construction truck and commercial vehicle movements would be spread across the day. For the purposes of this assessment, as a worst case scenario, seven movements are estimated to occur in the peak hour.

Daily volumes: 14 return trips (28 movements: 18 construction, 10 commercial vehicles)
Peak hour volumes adopted for intersection impact analysis: one bus, four trucks and two commercial vehicles.

3.1.2 Tunnelling Shaft Construction - 2014

Data provided by BHPB also indicates that the drilling of the vertical shaft would take 91 days at a rate of 4 cm/hr whilst the outfall pipe, drilled at a rate of between 60 and 120 cm/hr would take between 97 days (best case timeframe) and 194 days (worst case timeframe). The overall construction period for the shafts would take a maximum of 285 days i.e. less than one year.

Traffic generation for the shaft and outfall pipe construction is shown in Table 5. All BHPB traffic carrying shaft and outfall tunnel waste from Port Bonython will travel along the Lincoln Highway to Port Augusta and then on to Olympic Dam via the Stuart Highway and Olympic Way.

Table 5 Tunnelling transport profile

	Average advance rates	Construction Time	Traffic Volume (2-way movements)
Shaft	0.04 m/ hr	91 days	2-3 trucks / day
Outfall tunnel – Worst case	0.6 m/ hr	194 days	12 trucks / day
- Best case	1.2 m/ hr	97 days	23 trucks / day

Daily volumes:
Shaft: 3 return trips i.e. 6 movements (3 CVs)
Outfall: Daily volumes: 11-12 return trips i.e. 23 movements (23 CVs, best case timeframe) or 6 return trips i.e. 12 movements (12 CVs, worst case timeframe)

Peak hour volumes assumed for intersection impact analysis: 0 cars / 5 CVs (best case timeframe) or 0 cars / 3 CVs (worst case timeframe).

3.2 Operation volumes – 2015 onwards

Once operational the desalination plant would generate eight return trips (five car; one bus and two trucks) per day. It is assumed that some movements occur during the work day. For a worst case scenario in the PM peak hour it is assumed that three car movements; one truck and the bus leaves the site.

Daily volumes: Eight return trips i.e. 16 movements (10 cars, 2 buses, 4 commercial vehicles)
Peak hour volumes adopted for intersection impact analysis: 3 cars, 1 bus, and 1 truck.

4 Port Bonython Fuel Storage and Processing Facility

The additional traffic volumes that will use Port Bonython Road and Lincoln Highway are associated with both the Desalination Plant being constructed and operated by BHPB and the Fuel Storage and Processing Facility being constructed and operated as a joint venture by Stuart Petroleum Limited and the Scott Group of Companies (Port Bonython Fuels).

Information relating to the construction and operation of this facility has been provided in the document "Port Bonython Fuel Storage and Processing facility, Traffic Impact Assessment", by QED Pty. Ltd., dated 21st May 2009. The document assumed construction occurs in 2010, Stage 1 operations commence in 2011 and Stage 2 of the fuel storage facility is constructed in 2015. From 2011 to 2020 it is proposed to increase production annually.

The proposed Port Bonython fuel storage and processing facility was approved by the South Australian government in January 2010, however as construction has not yet commenced it is assumed for this analysis that slippage of one year has occurred i.e. construction commences in 2011 and Stage 1 operations commence in 2012.

4.1 Stage 1 construction traffic volumes: 2011

Although the construction period for Stage 1 of the fuel storage facility was not outlined in the project's TIA report[4], the traffic assessment within the report suggests that works will occur across a single year. It is noted that traffic generated by the Stage 1 construction works would not conflict with the proposed BHPB construction works as the tunnelling works for the desalination plant are not expected to commence before 2014.

Based on the project's TIA report, traffic volumes expected during construction include construction employee and delivery trips, and it is conservatively assumed that delivery trips are all undertaken by heavy vehicles. Accordingly, it is assumed that there will be:

- *49 construction employee trips and 12 delivery trips / day (25 construction employee and three delivery trips in the peak hour).*

4.2 Operational traffic volumes: 2012 onwards

Once operational it is predicted that daily traffic volumes generated by the fuel storage facility will comprise, in the first year:

- *12 employee trips / day and 22 fuel tanker trips / day*
  (This equates to six employee trips and two fuel tanker trips in the peak hour).

*By 2014, with growth of the facility fuel tanker volum*es will reach:
- 34 fuel tanker trips / day (four in the peak hour).

4.3 Stage 2 construction activities: 2015

The construction of Stage 2 and operational activities during 2015 will generate:

- *49 construction employee trips and 20 construction deliveries per day*
  (28 construction employee and five construction deliveries during the peak hour);
- *12 employee trips / day and 40 fuel tanker trips / day*
  (six employee and five fuel tanker trips in the peak hour).

Traffic volumes will be greatest in 2015 when the construction of Stage 2 is occurring concurrently with Stage 1 operations. The overall total volumes would comprise:

61 light vehicles and 66 heavy vehicles (33 light and 10 heavy vehicle trips in the peak hour).

4 Port Bonython Fuel Storage and Processing facility, Traffic Impact Assessment", by QED Pty. Ltd. and dated 21st May 2009

5 Traffic Impact Assessment

Having established the existing conditions, described the proposal and other concurrent projects, assessment of traffic impacts has been undertaken. This section describes an assessment of the traffic volumes associated with a variety of scenarios, the impact on the operation of key intersections and also the impact on road links between intersections.

5.1 Traffic Volume Scenario Assessment

As discussed in Section 5, the timing of the Port Bonython Fuels project is still to be confirmed. There may also be variation in the timeframe for the desalination plant and outfall construction and commencement of operation. Traffic volumes along Port Bonython Road would permanently increase once the new PBF fuel storage facility and the BHPB desalination plant are in operation. During construction of the two facilities there would be additional traffic peaking.

Based on the above, an assessment of four traffic volume cases has been undertaken to ascertain the traffic volumes associated with a variety of scenarios. The scenarios were as follows:

With Port Bonython Fuels

- Scenario 1 – base traffic / Pt Bonython fuels / desal plant (const / operation) / tunnel waste removal (worst case)
- Scenario 2 - base traffic / Pt Bonython fuels / desal plant (const / operation) / tunnel waste removal (best case)

Without Port Bonython Fuels

- Scenario 3 – base traffic / desal plant (const / operation) / tunnel waste removal (worst case)
- Scenario 4 - base traffic / desal plant (const / operation) / tunnel waste removal (best case)

Information provided in the BHPB brief and in the TIA report for Port Bonython Fuels identified that there would be three years during which construction works and operation of the two projects would coincide providing the critical traffic volume scenarios.

Estimated daily volumes generated by the facilities were discussed in Section 3 and 4 and it has been assumed that peak hour traffic volumes were 10% of the daily volumes. Deliveries occurring during a day were conservatively assumed to part occur during the peak hour. The PM peak hour, when employee traffic departs the sites, was considered to be more critical as traffic volumes at this time were approximately twice that of the traffic expected during the morning peak hour when construction workers were heading to the site.

5.1.1 Traffic volumes during 2014

The staging of the two projects suggested that 2014 was likely to be the peak traffic year with the desalination plant construction and transfer of tunnelling waste occurring and Stage 1 of the fuel storage facility operating. Total traffic volumes along Port Bonython Road (shown in Table 6) would be the sum of:

- Existing traffic (2010 volumes conservatively growthed at 1% per annum to 2014).
- PBF operational traffic (growthed from 2011 to 2014 assuming six additional fuel tankers per day each year).
- BHPB Desalination Plant construction traffic (occurring during 2014).

Table 6 2014 traffic volume scenarios

Traffic Origin	Peak hour volumes				
	2014 Base	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**
Base traffic – 2014	42 cars 5 CVs	42 cars 5 CVs	42 cars 5 CVs	42 cars 5 CVs	42 cars 5 CVs
PBF - Stage 1 operations in 2014	N/A	6 cars 4 CVs	6 cars 4 CVs		
BHPB Desalination Plant: -Construction 2014	N/A	0 cars 7 CVs	0 cars 7 CVs	0 cars 7 CVs	0 cars 7 CVs
Removal of tunnel waste - best case - worst case	N/A	3 CVs -	- 5 CVs	3 CVs	- 5 CVs
2014 ESTIMATED PEAK HOUR VOLUME TOTALS	**42 cars 5 CVs**	**48 cars 19 CVs**	**48 cars 21 CVs**	**42 cars 15 CVs**	**42 cars 17 CVs**
TOTAL MOVEMENTS ALONG PORT BONYTHON ROAD (% inc. compared to 2014 base)	**47**	**67 (43%)**	**69 (47%)**	**57 (21%)**	**59 (26%)**

5.1.2 Traffic volumes during 2015

Given that the proposed PBF Stage 2 construction is proposed to occur in 2015, an assessment was undertaken of the 2015 traffic volumes. The TIA for the fuel storage facility indicated that some 30 additional movements would be generated in the peak hour by Stage 2 construction traffic.

The following year (2015) would include, in a worst case if concurrent:

- Existing traffic (2010 volumes conservatively growthed at 1% per annum to 2015)
- PBF operational traffic (growthed from 2011 onwards to 2015)
- PBF construction traffic (Stage 2 - 2015)
- BHPB Desalination plant operational traffic (2015 onwards)

The four scenarios reviewed in 2014 included different scenarios for transporting the tunnelling waste i.e. Scenario 1 and 3 for best case and Scenario 2 and 4 for worst case. In 2015, with the BHPB desalination plant construction finished, only two scenarios require review i.e. with PBF or without PBF:

With Port Bonython fuels

- Scenario 1 – base traffic / Pt Bonython fuels / desal plant (operation)

Without Port Bonython fuels

- Scenario 2 – base traffic / desal plant (operation)

Table 7 tabulates the peak hour traffic volumes assuming, base case with no development; Scenario 1 including Stage 2 PBF construction traffic and Scenario 2 without PBF. Scenario 1 provides the worst case in 2015 therefore the worst-case scenario for analysing traffic using the Lincoln Highway / Port Bonython Road intersection would also therefore occur during 2015.

Table 7 2015 Traffic volume case scenarios

Traffic Volumes	Peak hour volumes		
	2015 Base	**Scenario 1**	**Scenario 2**
Base traffic – 2015	43 cars 5 CVs	43 cars 5 CVs	43 cars 5 CVs
PBF - Stage 1 operations in 2015 Stage 2 construction	N/A N/A	6 cars 4 CVs 25 cars 5 CVs	
BHPB Desalination Plant: -Operation 2015	N/A	3 cars 2 CV	3 cars 2 CV
2015 ESTIMATED PEAK HOUR VOLUME TOTALS	**43 cars 5 CVs**	**77 cars 16 CVs**	**46 cars 7 CVs**
TOTAL MOVEMENTS ALONG PORT BONYTHON ROAD (% inc.)	**48**	**93 (94%)**	**53 (10%)**

5.1.3 Traffic volumes during 2018

The operation of the intersection will change over time assuming increased traffic along both Port Bonython Road and Lincoln Highway based on the assumed growth rates. A future year of 2018 has been assumed for analysis. The PBF Stage 2 operation will be operational by this time.

The year 2018 traffic volumes would include:

- Existing traffic (2010 volumes conservatively growthed at 1% per annum to 2018)
- PBF operational traffic (growthed from 2011 onwards to 2018)
- BHPB Desalination plant operational traffic (2015 onwards)

Table 8 tabulates the peak hour traffic with both developments. The traffic volumes predicted by 2018 for Port Bonython Road allows for the adopted growth rate of 1% per annum. No other development at Point Lowly has been assumed to be in place by 2018. Peak hour traffic volumes are estimated to reach 71 vehicles by 2018 which is far lower than the 93 vehicles estimated to occur in the worst 2015 scenario.

Table 8 2018 Traffic volume case scenarios

Traffic Volumes	Peak hour volumes	
	2018 Base	**BHPB and PBF**
Base traffic – 2018	44 cars 5 CVs	44 cars 5 CVs
PBF - Stage 1 and 2 operations in 2018	N/A	10 cars 7 CVs
BHPB Desalination Plant: -Operation 2025	N/A	3 cars 2 CV
2015 ESTIMATED PEAK HOUR VOLUME TOTALS	**44 cars 5 CVs**	**57 cars 14 CVs**
TOTAL MOVEMENTS ALONG PORT BONYTHON ROAD (% inc. on 2018 base volume)	**49**	**71 (45%)**

Having established the critical traffic volumes, an assessment has been undertaken at key points in the road network.

5.2 Intersection Impact Assessment

Given the expected travel patterns travelling to and from Port Bonython, it is considered that the intersection of Lincoln Highway and Port Bonython Road requires assessment.

5.2.1 Lincoln Hwy / Port Bonython Road Intersection layout

The Lincoln Highway / Port Bonython Road intersection is located along a straight section of the Lincoln Highway. Using Google maps "street view" feature, an estimation of the lane lengths at the intersection was undertaken. The assessment is shown in Figure 2.

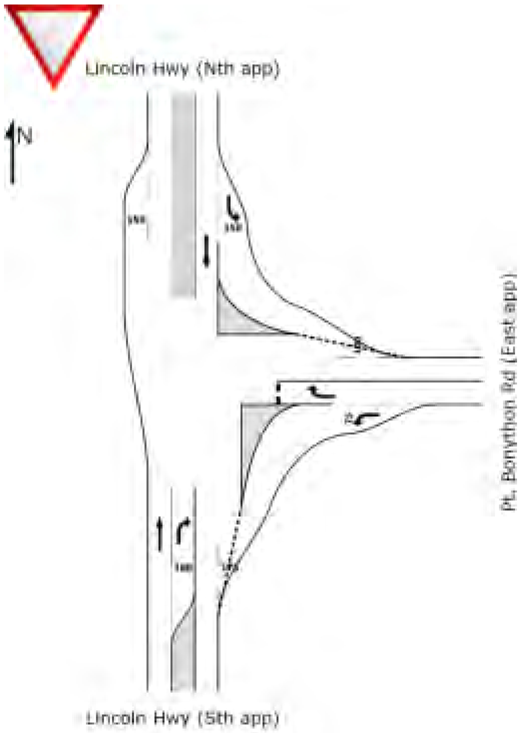


Figure 2 Layout of Lincoln Hwy Pt. Bonython Rd intersection

5.2.2 Lincoln Hwy / Pt. Bonython Rd intersection traffic volumes

Based on information provided by DTEI, the 2010 two-way traffic volumes along Lincoln Highway in 2010 are estimated at 120 cars and 16 commercial vehicles in the late afternoon peak (4 to 5pm) and 60 cars and 10 commercial vehicles in the morning peak hour (8 to 9am). The volumes in the PM peak hour were therefore conservatively assumed as the critical peak hour for the analysis of the Lincoln Highway / Port Bonython Road intersection.

It was assumed for the purpose of the analysis that traffic using Port Bonython Road in the PM peak hour comprised 80% exiting traffic and 20% arrival traffic. Other assumptions for the turning movements were provided by BHPB (and in the PBF TIA report) as follows:

- 50% of BHPB construction traffic originates from Whyalla / 50% from Port Augusta.
- 90% of BHPB operational traffic originates from Whyalla / 10% from Port Augusta.
- 100% of BHPB waste transfer traffic travels to / from Olympic Dam.
- 95% of PBF fuel tanker traffic travels to / from Port Augusta.
- Most PBF employees originate from Whyalla.

Traffic volumes at the Lincoln Highway / Port Bonython Road intersection are included in Appendix A for the following scenarios:

- 2010 Existing (no developments)
- 2014 Scenario 1 – base traffic / PBF / desal plant (const / operation) / tunnel waste removal (worst case)
- 2014 Scenario 2 - base traffic / Pt Bonython fuels / desal plant (const / operation) / tunnel waste removal (best case)
- 2014 Scenario 3 – base traffic / desal plant (const / operation) / tunnel waste removal (worst case)
- 2014 Scenario 4 - base traffic / desal plant (const / operation) / tunnel waste removal (best case)
- 2015 with PBF Stage 2 construction
- 2018 with both developments operating

5.2.3 Intersection analysis

The impact of additional traffic on the intersection Lincoln Highway / Port Bonython Road was assessed using the intersection analysis software aaSIDRA with the results summarised in Table 9. It shows that there is little impact on the intersection through increasing traffic volumes associated with the proposed developments.

The average delay for vehicles undertaking the right turn from Port Bonython Road into Lincoln Highway (considered to be the critical movement) is currently 12.2 seconds and increases to 14.2 seconds by 2018.

The level of service of the intersection operation does not worsen beyond the current LoS B for all scenarios across all future time periods. Similarly, the maximum queue distance is negligible across all scenarios, with less than one vehicle queued.

Table 9 Summary of SIDRA analysis

Scenario	**Intersection Degree of Saturation**	**Pt. Bonython Rd RT: avg. delay (secs)**	**Worst Level of Service**	**Pt Bonython Rd RT Queue distance (m)**
Existing (2010)	0.038	12.2	B	0.8
Scenario 1 (2014)	0.040	14.5	B	2.6
Scenario 2 (2014)	0.042	14.7	B	2.8
Scenario 3 (2014)	0.040	13.8	B	1.6
Scenario 4 (2014)	0.040	14.0	B	1.9
2015 – Stage 2 PBF and BHPB operational	0.060	13.6	B	1.9
2018 – BHPB and PBF Stage 2 in operation	0.040	14.2	B	2.1

5.2.4 Intersection assessment summary

The additional traffic using the intersection of Lincoln Highway / Port Bonython Road will result in a minor change in the existing operation over time. This will result in a marginal increase in queue lengths and traffic delays for traffic departing Port Bonython Road. It is emphasised that these impacts are low and are expected to be lower still outside of the PM peak hour.

5.3 Road Link Impact Assessment

Having established the impact at key intersections an assessment was undertaken of the potential traffic impact between intersections. The impact of the forecast traffic volumes along Port Bonython Road, Lincoln Highway, Stuart Highway and Olympic Way are summarised below.

5.3.1 Port Bonython Road

The traffic volumes along Port Bonython Road are currently some 400 vehicles per day, including an estimated 45 vehicles during the peak hour. The proposed construction of the BHPB desalination plant and PBF fuel storage facility will raise traffic volumes along the road however the volume changes during the peak hour are low with the estimate in 2014 - the desalination plant construction year - that peak hour volumes will reach a maximum 69 vehicles.

The highest peak hour volumes will occur in 2015 (93 vehicles) when Stage 2 of the PBF fuel storage facility is being constructed and with the Desalination Plant operating. It is estimated that the peak hour traffic volumes will reach only 71 vehicles by 2018 including both the desalination plant and with PBF Stages 1 and 2 operational. The 2015 traffic volumes are therefore the worst case scenario but such low volumes remain well within the theoretical capacity of the road.

5.3.2 Lincoln Highway

The addition of traffic associated with both the BHPB desalination plant and the Port Bonython Fuels project will result in a very minor increase in traffic along the Lincoln Highway. The base peak hour volumes during 2015 are estimated at approximately 144 vehicles per hour. Under a worst case scenario (2015) there will be an additional 45 vehicles per hour using Lincoln Highway. While the proposed volumes are a reasonable increase in percentage terms, they are well within the theoretical peak hour capacity of the Lincoln Highway (1,800 v.p.h. under ideal conditions).

5.3.3 Stuart Highway and Olympic Way

The impact of the BHPB Desalination Plant on traffic volumes along the Stuart Highway and Olympic Way would only occur during 2014, the year the plant is being constructed. The impact would be confined to trucks conveying tunnelling waste material to Olympic Dam. This traffic was not previously assessed as part of the ODX traffic outlined in the EIS for ODX and therefore not included in the Stuart Highway volumes. Information provided notes that daily volumes associated with this are:

- Vertical Shaft tunnelling: 4 movements / day and
- Outfall tunnelling: 12 (worst case) to 23 (best case) movements / day

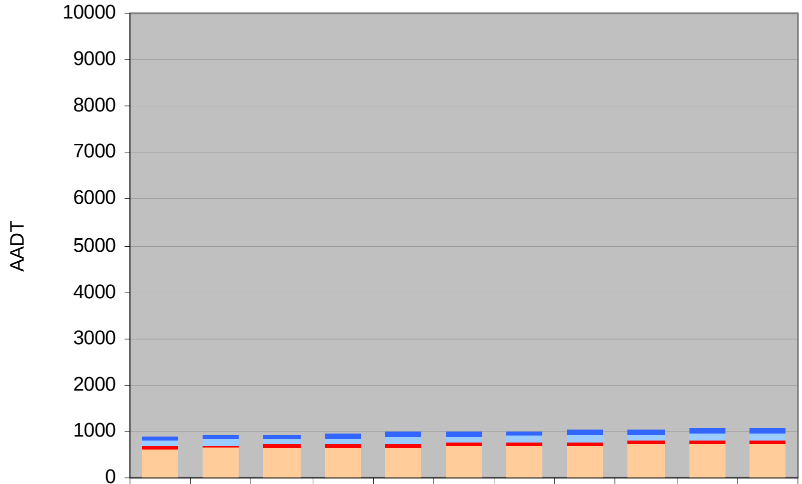
It is noted that traffic generated by the BHPB Desalination Plant construction and its operation would impact only on traffic volumes along Port Bonython Road and Lincoln Highway due to traffic travelling only as far as Port Augusta or Whyalla.

5.3.3.1 Traffic volumes on Stuart Highway

Baseline traffic and Olympic Dam Expansion (ODX) along both the Stuart Highway and Olympic Way have been reproduced here from the original EIS (Figure 3 to Figure 6).

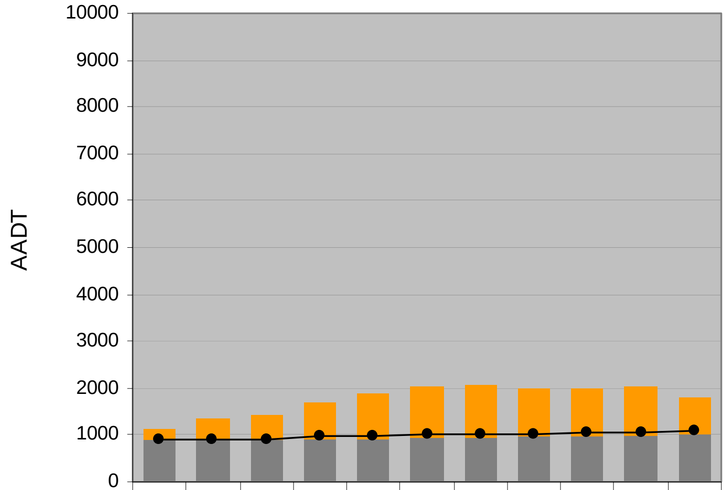
Figure 4 notes the baseline traffic in 2010 as 889 v.p.d. and by 2014 (the desalination plant construction year) the number will be 979 v.p.d. The total traffic in 2014 including base traffic and ODX traffic is predicted to be 1,940 vehicles.

The additional traffic due to trucks transferring desalination plant tunnelling material to Olympic Dam would be between 16 and 27 heavy vehicle movements / day, representing an increase in traffic of between 0.8% and 1.4% per day along the Stuart Highway compared with the total volumes expected to be operating in 2014.



	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
Heavy Vehicle - OD	73	77	72	104	104	106	108	107	105	105	105
Heavy Vehicle - Other	120	123	126	129	132	135	138	142	145	149	152
Bus/2-Axle Truck	71	72	73	75	76	77	78	80	81	82	83
Light Vehicle	626	636	646	657	667	678	689	700	712	723	735

Figure 3 – Baseline Traffic (No ODX): Stuart Highway



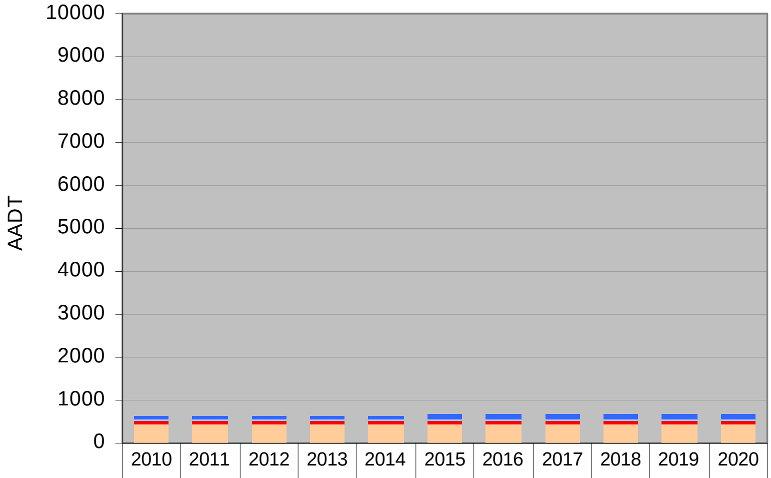
	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
ODX Proposed Expansion	222	427	534	801	961	1109	1100	1009	1015	1043	813
Base Traffic (ODX)*	889	908	884	900	915	932	950	965	979	995	1010
Baseline*	889	908	917	964	979	996	1014	1029	1043	1059	1075

Figure 4 – Total Future Traffic: Stuart Highway

For a two-lane two-way rural road, traffic volume capacity is some 1,800 v.p.h. under ideal conditions. The expected volumes are approximately 2,000 vehicles per day in 2014, or (assuming 10% of traffic travels in peak hour) 200 v.p.h. represents 11% of the theoretical capacity of the road. The effect on the addition of fewer than 30 additional vehicles in 2014 across the day is considered to be minimal.

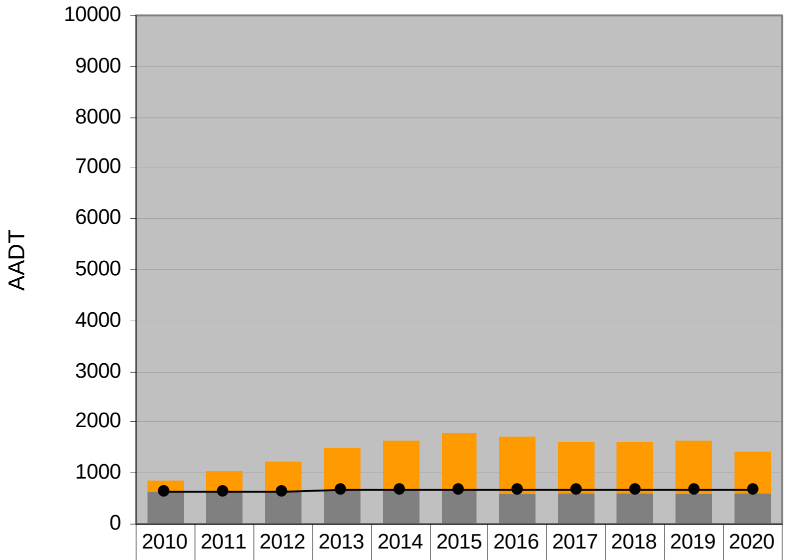
5.3.3.2 Traffic volumes on Olympic Way due to Olympic Dam Expansion

In the case of Olympic Way the proposed volumes in 2014 are expected to be 1,644 v.p.d. The addition of between 16 and 27 heavy vehicle movements per day adds between 1% and 1.6% additional traffic to Olympic Way to the traffic expected to be using the road in 2014.



	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
Heavy Vehicle - OD	73	77	72	104	104	106	108	107	105	105	105
Heavy Vehicle - Other	50	50	50	50	50	50	50	50	50	50	50
Bus/2-Axle Truck	49	49	49	49	49	49	49	49	49	49	49
Light Vehicle	458	458	458	458	458	458	458	458	458	458	458

Figure 5 – Baseline Traffic (No ODX): Olympic Way, South of Roxby Downs



	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
ODX Proposed Expansion	221	427	591	822	984	1132	1100	1009	1015	1043	813
Base Traffic (ODX)*	629	633	628	660	660	662	600	599	597	597	596
Baseline*	629	633	628	660	660	662	664	663	661	661	661

** - See Glossary for definitions of Baseline and Base Traffic (ODX)*

Figure 6 – Total Future Traffic: Olympic Way

As noted in Section 8.1.1 traffic volume capacity is some 1,800 v.p.h. under ideal conditions. The expected daily volumes along Olympic Way in 2014 (the year of the desalination plant construction) are approximately 1,650 vehicles per day or (assuming 10% of traffic travels in peak hour) 165 v.p.h. This number represents about 9% of the theoretical capacity of the road. The effect on the addition of fewer than 30 additional vehicles in 2014 across the day is therefore considered to be minimal.

6 Conclusion

Traffic associated with the construction and operation of the BHPB Desalination Plant and with the Port Bonython Fuels will add to current traffic volumes using Port Bonython Road and the Lincoln Highway.

The current daily volumes using these roads in 2010 are estimated to be 400 v.p.d. and 1,800 v.p.d. respectively. In the peak hour the volumes are 45 v.p.h. and 136 v.p.h. respectively with the expectation being that in 2014 hourly volumes along would increase by up to 22 v.p.h. In 2015, when Stage 2 works for the PBF fuel storage facility are due to commence, the peak hourly traffic volumes are expected to increase by up to 45 v.p.h. Although a reasonable increase in percentage terms based on current volumes, the proposed usage in 2015 is still low for Port Bonython Road and the Lincoln Highway with the theoretical value for a two-lane road being 1,800 v.p.h. hour per direction.

The intersection of Lincoln Highway / Port Bonython Road is estimated to currently (in 2010) operate at a Level of Service B with average delays for Port Bonython Road traffic being 12.2 seconds for right turning vehicles. SIDRA analysis for this intersection over the years 2014, 2015 and 2018 predicts the delay to increase to a maximum 14.7 seconds per vehicle during PBF construction works in 2014 and that the intersection would continue to operate at Level of Service B, despite the increase in traffic volumes. The minor increase in traffic volumes (at worst 45 v.p.h.) represents fewer than one vehicle per minute and is minor compared to the current capacity of the intersection. As such, it is considered that the intersection will continue to operate satisfactorily well into the future.

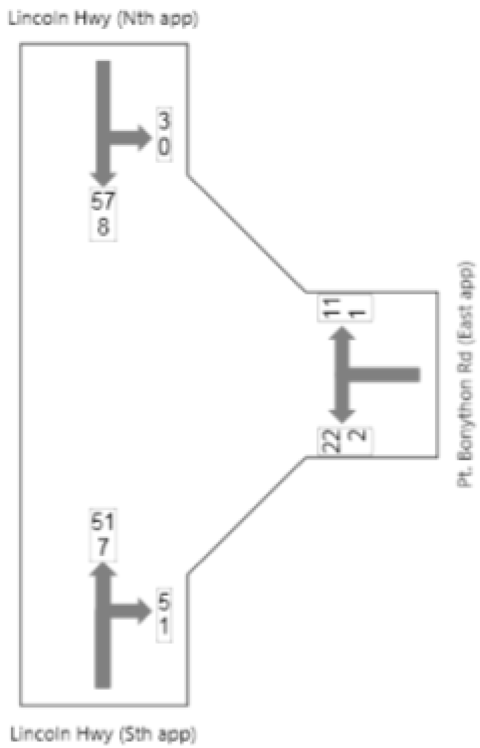
The transfer of tunnelling waste from Port Bonython to Olympic Dam will add between 16 and 27 movements per day to traffic volumes on these roads during the 2014 construction year. The effect on Stuart Highway and Olympic Way traffic operation is again considered small because these volumes add around 1% of traffic to the proposed volumes that would be operating during that year.

Appendix A

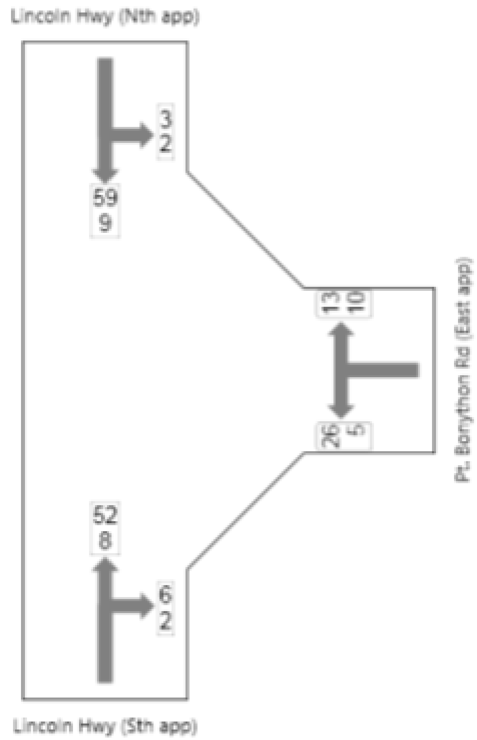
Traffic Volume Scenarios – Lincoln Hwy / Pt Bonython Rd

A1 Port Bonython Road – Traffic volume scenarios

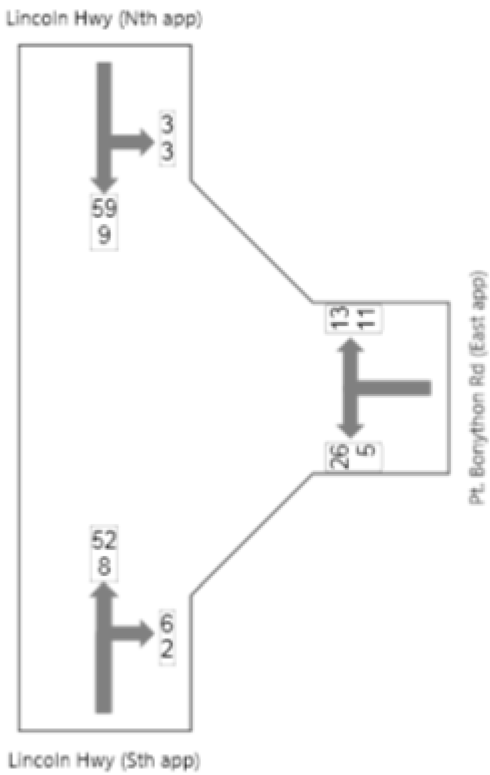
2010 Existing volumes



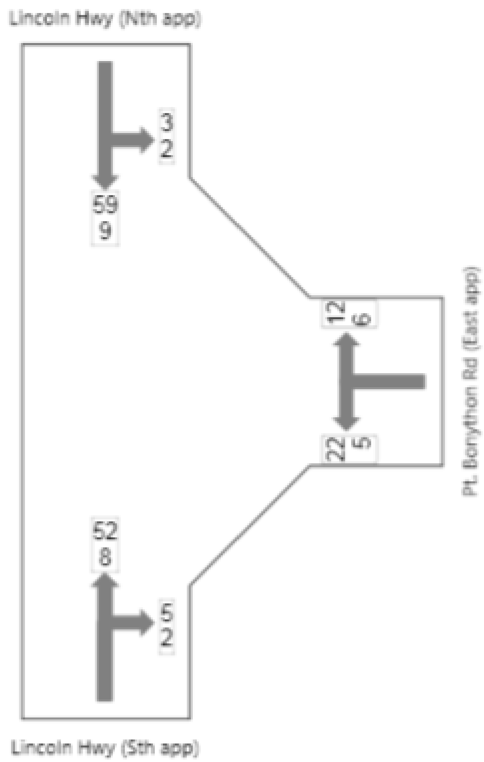
2014 Scenario 1



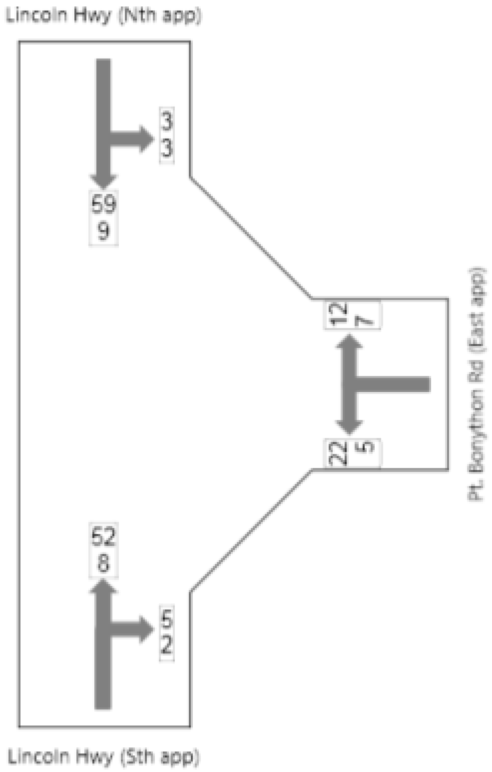
2014 Scenario 2



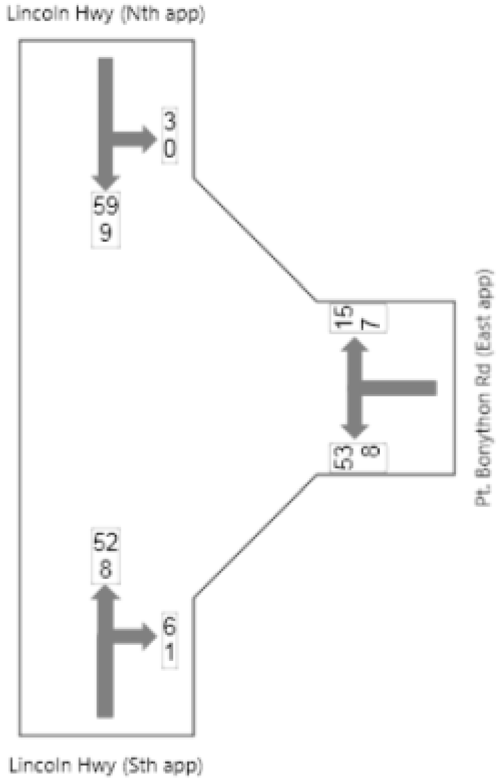
2014 Scenario 3



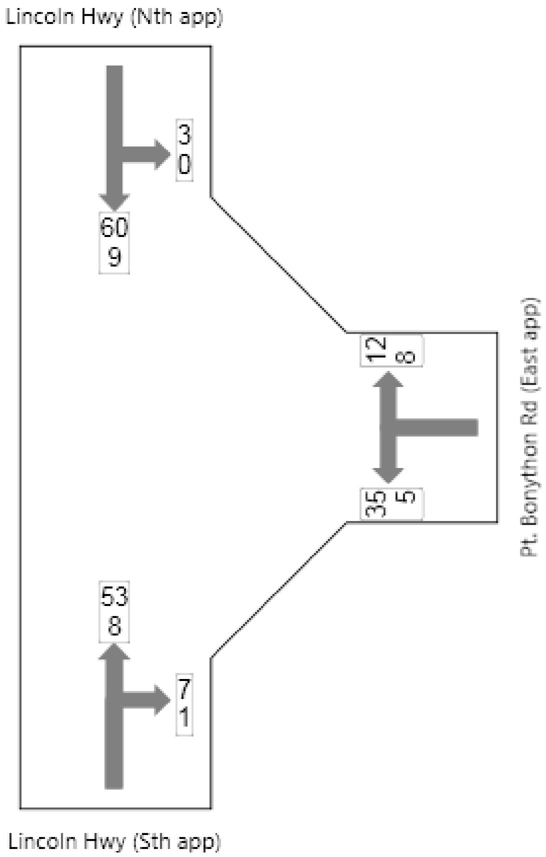
2014 Scenario 4



2015 Scenario 1 – PBF Stage 2 const.



2018 Scenario 1



Appendix B

SIDRA outputs - Lincoln Hwy / Port Bonython Rd I/S

B1 Lincoln Hwy / Port Bonython Rd SIDRA analysis

2010 Existing

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2010 EXISTING

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	62	13.6	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	6	16.7	0.006	11.2	LOS B	0.0	0.2	0.17	0.64	41.9
Approach		68	13.8	0.035	1.0	LOS B	0.0	0.2	0.02	0.06	75.6
East: Pt. Bonython Rd (East app)											
1	L	25	8.3	0.023	10.3	LOS B	0.1	0.8	0.16	0.61	42.8
3	R	13	8.3	0.014	12.2	LOS B	0.1	0.6	0.30	0.65	40.8
Approach		38	8.3	0.023	11.0	LOS B	0.1	0.8	0.21	0.62	42.1
North: Lincoln Hwy (Nth app)											
4	L	3	0.0	0.003	9.9	LOS A	0.0	0.1	0.04	0.64	43.9
5	T	68	12.3	0.038	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		72	11.8	0.038	0.4	LOS A	0.0	0.1	0.00	0.03	78.0
All Vehicles		178	11.8	0.038	2.9	NA	0.1	0.8	0.05	0.17	68.0

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

2014 Scenario 1- BHPB desal const. / waste removal (worst), with PBF

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2014-Case 1

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	63	13.3	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	8	25.0	0.008	11.6	LOS B	0.0	0.3	0.19	0.64	41.8
Approach		72	14.7	0.035	1.4	LOS B	0.0	0.3	0.02	0.08	74.4
East: Pt. Bonython Rd (East app)											
1	L	33	16.1	0.032	10.6	LOS B	0.1	1.1	0.18	0.61	42.6
3	R	24	43.5	0.039	14.5	LOS B	0.2	2.6	0.37	0.67	39.3
Approach		57	27.8	0.039	12.3	LOS B	0.2	2.6	0.26	0.64	41.2
North: Lincoln Hwy (Nth app)											
4	L	5	40.0	0.007	10.9	LOS B	0.0	0.3	0.06	0.62	43.7
5	T	72	13.2	0.040	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		77	15.1	0.040	0.7	LOS B	0.0	0.3	0.00	0.04	76.9
All Vehicles		205	18.5	0.040	4.1	NA	0.2	2.6	0.08	0.22	64.4

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

2014 Scenario 2- BHPB desal const. / waste removal (best), with PBF

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2014-Case 2

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	63	13.3	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	8	25.0	0.008	11.6	LOS B	0.0	0.3	0.19	0.64	41.8
Approach		72	14.7	0.035	1.4	LOS B	0.0	0.3	0.02	0.08	74.4
East: Pt. Bonython Rd (East app)											
1	L	33	16.1	0.032	10.6	LOS B	0.1	1.1	0.18	0.61	42.6
3	R	25	45.8	0.042	14.7	LOS B	0.2	2.8	0.38	0.68	39.2
Approach		58	29.1	0.042	12.4	LOS B	0.2	2.8	0.26	0.64	41.1
North: Lincoln Hwy (Nth app)											
4	L	6	50.0	0.009	11.1	LOS B	0.0	0.5	0.06	0.61	43.7
5	T	72	13.2	0.040	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		78	16.2	0.040	0.9	LOS B	0.0	0.5	0.01	0.05	76.3
All Vehicles		207	19.3	0.042	4.3	NA	0.2	2.8	0.08	0.22	64.1

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

2014 Scenario 3- BHPB desal const. / waste removal (worst), No PBF

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2014-Case 3

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	63	13.3	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	7	28.6	0.007	11.7	LOS B	0.0	0.3	0.19	0.64	41.8
Approach		71	14.9	0.035	1.2	LOS B	0.0	0.3	0.02	0.07	75.0
East: Pt. Bonython Rd (East app)											
1	L	28	18.5	0.028	10.7	LOS B	0.1	1.0	0.18	0.61	42.6
3	R	19	33.3	0.028	13.8	LOS B	0.1	1.6	0.35	0.66	40.0
Approach		47	24.4	0.028	11.9	LOS B	0.1	1.6	0.25	0.63	41.6
North: Lincoln Hwy (Nth app)											
4	L	5	40.0	0.007	10.9	LOS B	0.0	0.3	0.06	0.62	43.8
5	T	72	13.2	0.040	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		77	15.1	0.040	0.7	LOS B	0.0	0.3	0.00	0.04	76.9
All Vehicles		195	17.3	0.040	3.6	NA	0.1	1.6	0.07	0.19	66.0

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

2014 Scenario 4- BHPB desal const. / waste removal (best), No PBF

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2014-Case 4

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	63	13.3	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	7	28.6	0.007	11.7	LOS B	0.0	0.3	0.19	0.64	41.8
Approach		71	14.9	0.035	1.2	LOS B	0.0	0.3	0.02	0.07	75.0
East: Pt. Bonython Rd (East app)											
1	L	28	18.5	0.028	10.7	LOS B	0.1	1.0	0.18	0.61	42.6
3	R	20	36.8	0.031	14.0	LOS B	0.2	1.9	0.36	0.67	39.7
Approach		48	26.1	0.031	12.1	LOS B	0.2	1.9	0.25	0.63	41.4
North: Lincoln Hwy (Nth app)											
4	L	6	50.0	0.009	11.1	LOS B	0.0	0.5	0.06	0.61	43.7
5	T	72	13.2	0.040	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		78	16.2	0.040	0.9	LOS B	0.0	0.5	0.00	0.05	76.3
All Vehicles		197	18.2	0.040	3.8	NA	0.2	1.9	0.07	0.20	65.7

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

2015 Scenario 1 - BHPB Desalination plant operational / Stage 2 PBF construction

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2015-Stg2 PBF, Desal operational

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	63	13.3	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	7	14.3	0.006	11.1	LOS B	0.0	0.2	0.18	0.64	41.9
Approach		71	13.4	0.035	1.2	LOS B	0.0	0.2	0.02	0.07	75.0
East: Pt. Bonython Rd (East app)											
1	L	64	13.1	0.060	10.5	LOS B	0.3	2.0	0.17	0.61	42.6
3	R	23	31.8	0.034	13.6	LOS B	0.2	1.9	0.35	0.67	40.1
Approach		87	18.1	0.060	11.4	LOS B	0.3	2.0	0.22	0.63	42.0
North: Lincoln Hwy (Nth app)											
4	L	3	0.0	0.003	9.9	LOS A	0.0	0.1	0.04	0.64	43.9
5	T	72	13.2	0.040	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		75	12.7	0.040	0.4	LOS A	0.0	0.1	0.00	0.03	78.1
All Vehicles		233	14.9	0.060	4.8	NA	0.3	2.0	0.09	0.26	61.6

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

2018 – Scenario 1 BHPB Desalination plant / Stage 2 PBF operational

MOVEMENT SUMMARY

Site: Lincoln Hwy/ Pt Bonython Rd 2018-Desal,PBF operational

Three-way intersection with 4-lane major road (Give-Way control)
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Lincoln Hwy (Sth app)											
11	T	64	13.1	0.036	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
12	R	8	12.5	0.007	11.1	LOS B	0.0	0.3	0.18	0.64	41.9
Approach		73	13.0	0.036	1.3	LOS B	0.0	0.3	0.02	0.07	74.5
East: Pt. Bonython Rd (East app)											
1	L	42	12.5	0.039	10.5	LOS B	0.2	1.3	0.17	0.61	42.7
3	R	21	40.0	0.033	14.2	LOS B	0.2	2.1	0.36	0.67	39.6
Approach		63	21.7	0.039	11.7	LOS B	0.2	2.1	0.24	0.63	41.6
North: Lincoln Hwy (Nth app)											
4	L	3	0.0	0.003	9.9	LOS A	0.0	0.1	0.05	0.64	43.9
5	T	73	13.0	0.040	0.0	LOS A	0.0	0.0	0.00	0.00	80.0
Approach		76	12.5	0.040	0.4	LOS A	0.0	0.1	0.00	0.03	78.1
All Vehicles		212	15.4	0.040	4.1	NA	0.2	2.1	0.08	0.22	64.1

LOS (Aver. Int. Delay): NA. The average intersection delay is not a good LOS measure for two-way sign control due to zero delays associated with major road movements.
Level of Service (Worst Movement): LOS B. LOS Method for individual vehicle movements: Delay (HCM).
Approach LOS values are based on the worst delay for any vehicle movement.

ATTACHMENT C

Noise modelling update

ARUP

By Email & Post

Dave Winterburn
Senior Sustainability Advisor
BHP Billiton Olympic Dam Projects
Uranium CSG

Level 17
1 Nicholson Street
Melbourne
VIC 3000
Australia

t +61 3 9668 5500
d +61 3 9668 5664
f +61 3 9663 1546

will.gouthro@arup.com
arup.com.au

20 September 2010

Dear Dave

ODX Expansion
Workers Camp and Mine Maintenance Industrial Area Relocation

Introduction

The Acoustic model that was created for predicting noise levels associated with the Olympic Dam expansion as detailed in Arup report *Olympic Dam Expansion Environmental Impact Assessment Noise and Vibration Revision D (November 2008)* has been updated with the following:

- The workers construction camp (labelled as Hiltaba Village in Arup report) relocated approximately 150 m closer to the Rock Storage Facility (RSF)
- The Mine Maintenance Industrial Area (MMIA) relocated to the East, approximately 7 km from the workers construction camp and 11 km from Roxby Downs.

The noise sources modelled for the MMIA are as follows:

- 3 X Operational CAT 797B
- 1 X Ancillary vehicle (Lube truck)
- 1 X Reversing Alarm
- 1 X CAT 797B horn

In addition to this some mobile machinery is modelled at locations between the currently modelled MMIA and the RSF, however these vehicles do not have a significant impact on the overall predicted noise level.

The updated acoustic model is used to predict noise levels for the three meteorological conditions considered in the previous noise modelling.

Figure 1 shows the updated layout that has been modelled.

Results

The noise predictions using the updated acoustic model have shown that the relocation of the workers construction camp 150 m closer to the RSF has a negligible effect on the predicted noise levels at the camp.

The predicted noise levels with the relocated MMIA are provided in Table 1 below.

Table 1: Predicted Noise Levels with Updated Model

Location	Sound Pressure Level, dB(A) re 20 X 10^{-6}Pa		
	Meteorological Conditions		
	Neutral	Adverse	Temperature Inversion
Roxby Downs	34	41	44
Workers Construction Camp	38	46	48

For reference, the previously predicted noise levels without the relocated MMIA are provided in Table 2 below.

Table 2: Previously Predicted Noise Levels (for reference)

Location	Sound Pressure Level, dB(A) re 20 X 10^{-6}Pa		
	Meteorological Conditions		
	Neutral	Adverse	Temperature Inversion
Roxby Downs	33	40	43
Workers Construction Camp	32	39	42

Impact

The noise limits for noise sensitive receivers are provided in Table 3 below.

Table 3: Noise Limits

Location	External Noise Criteria	
	Day (7am to 10pm)	Night (10pm to 7am)
Roxby Downs	47 dBLAeq	40 dBL_{Aeq} 60 dBL_{Amax}
Hiltaba Village	50 dBLAeq	45 dBL_{Aeq} 60 dBL_{Amax}

The predicted noise levels for noise sensitive receivers at Roxby Downs and the Workers Construction Camp for the three meteorological conditions are assessed with respect to the criteria in Table 4 below.

Table 4: Assessment of the Predicted Noise Level

Receiver	Predicted Sound Pressure Level dB(A) re 20 X 10-6 Pa	Excess	Comments
Neutral Meteorological Conditions:			
Roxby Downs	34	0	Daytime and night-time noise criteria met
Construction Camp	38	0	Daytime and night-time noise criteria met
Adverse Meteorological Conditions:			
Roxby Downs	41	1	Daytime noise criterion is met and night-time noise criterion is marginally exceeded
Construction Camp	46	1	Daytime noise criterion is met and night-time noise criterion is marginally exceeded
Temperature Inversion:			
Roxby Downs	44	4	Daytime noise criterion is met and night-time noise criterion is exceeded
Construction Camp	48	3	Daytime noise criterion is met and night-time noise criterion is exceeded

Noise levels are predicted to increase with respect to previous modelling. The increase at Roxby Downs is in the order of 1 dB and the Workers Construction Camp is in the order of 7 dB, resulting in exceedance over the night-time noise limit.

The most significant noise source at noise sensitive receivers is the CAT797B horn. Previously, all CAT797B horns were operated in areas that had significant shielding by terrain.

Recommendations

It is recommended that the CAT797B horn is not operated during night-time hours during adverse weather conditions or a temperature inversion.

The predicted noise levels without the CAT797B horn are provided in Table 5 below.

Table 5: Predicted Noise Levels without CAT797B Horn

Location	Sound Pressure Level, dB(A) re 20 X 10^{-6}Pa		
	Meteorological Conditions		
	Neutral	Adverse	Temperature Inversion
Roxby Downs	33	40	43
Workers Construction Camp	34	41	44

Without the CAT797B horn, the predicted increase in noise level at Roxby Downs in negligible and the predicted increase in noise level at the Workers Construction Camp is of the order of 2 dB and below the noise limit for all scenarios considered.

Previous mitigation recommendations still apply at Roxby Downs for noise sources unrelated to the MMIA.

Alternative mitigation options for the MMIA include the construction of a workshop enclosure (warehouse) for testing of loud equipment such as the horn or relocating the MMIA to another location where the RSF provides adequate acoustic shielding.

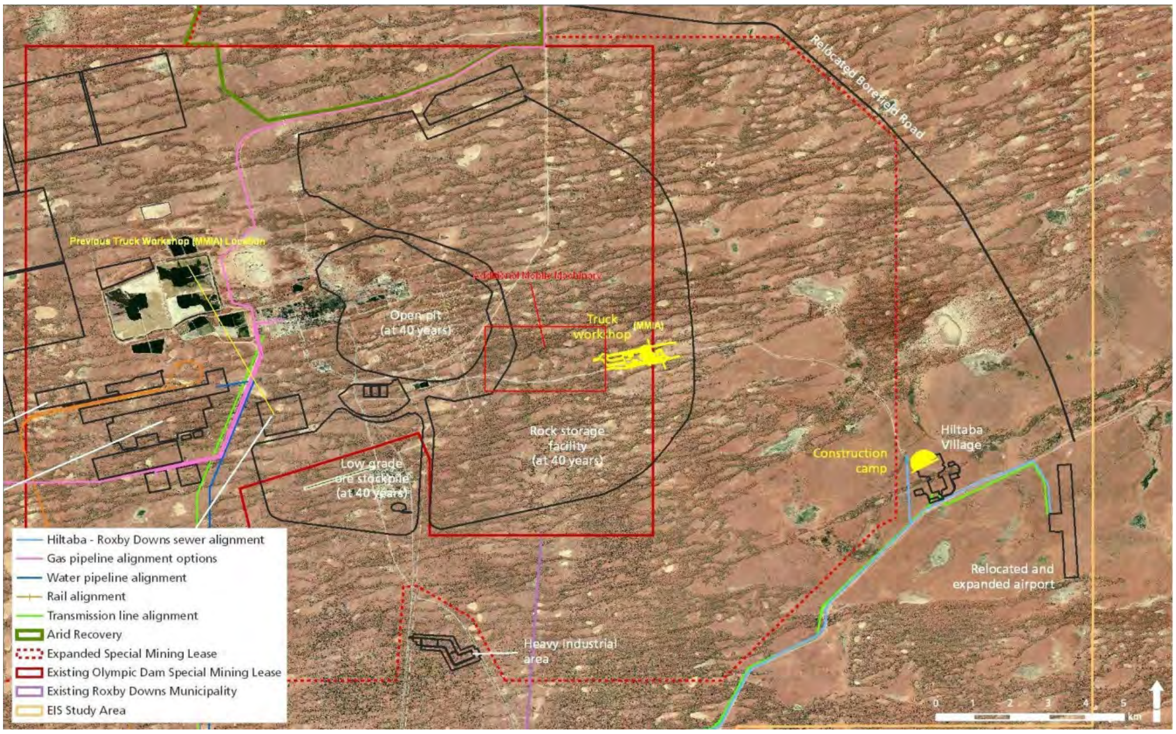
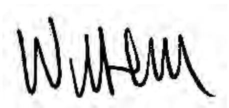


Figure 1: Updated Layout for Modelling

Yours sincerely



Will Gouthro
Acoustic Consultant

ATTACHMENT D

New entry gate – Traffic impact assessment

BHP Billiton

Olympic Dam Expansion Project

Supplementary EIS - Traffic

85200-10

Final | November 2010

New Entry Gate

Arup
Arup Pty Ltd ABN 18 000 966 165

Arup
Level 17
1 Nicholson Street
Melbourne
VIC 3000
Australia
www.arup.com

Job number 85200-10

ARUP

Contents

Appendices

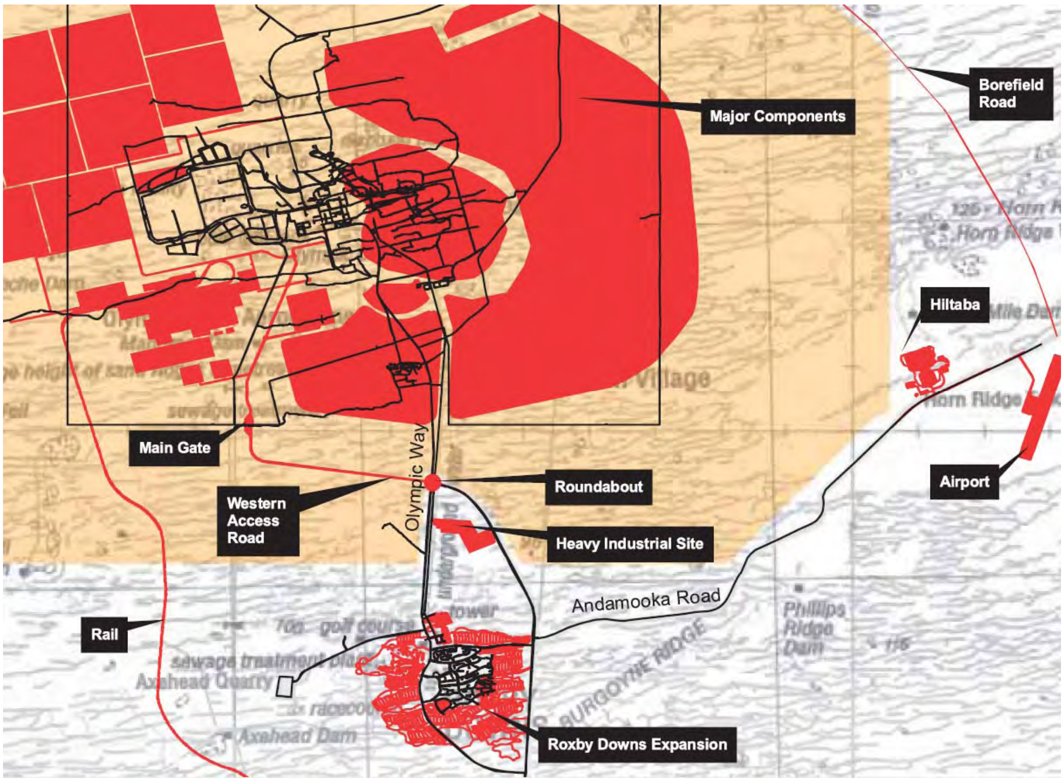
1 Introduction

1.1 Olympic Dam Expansion Draft EIS

This document provides an assessment of traffic impacts associated with the proposed second entry gate and eastern access road to be located adjacent Hiltaba Village. Specifically, it addresses traffic flows on the Axehead Road and Andamooka Road staggered T-intersection. The assessment:

- Determines new traffic patterns associated with the second access gate,
- Assesses intersection capacity and levels of service, and
- Examines the suitability of the current configuration.

Figure 1 – Olympic Dam Expansion Details (extract Draft EIS – App. Q9 – Fig. 13)



2 Background

The Olympic Dam Expansion Draft EIS (DEIS) presented the following project infrastructure requirements to be established along Andamooka Road to support the planned expansion at Olympic Dam:

- New Airport
- Hiltaba Village
- Re-routed Borefield Road.

Andamooka Road is currently not an accredited Restricted Access Route (RAV) for the operation of non standard truck configurations (i.e. B-doubles/Double Road trains or Triple road trains). The traffic anticipated to support the above facilities was anticipated to be predominately bus and light vehicles.

Additional investigations undertaken subsequent to the release of the DEIS concluded that the Axehead Road–Andamooka Road intersection would continue to operate within capacity and that the operation of one intersection would not affect the other (i.e. in respect of the ‘staggered T’ movement through the junctions). These additional investigations also found there would be a low volume of non standard truck configurations (doubles/triples) from the re-routed Borefield Road.

There were no requirements for double or triple road trains to operate along Andamooka Road for the expansion related traffic included in the DEIS. The SEIS did not assess the existing geometric layout of the Andamooka Road/Heavy Vehicle Bypass intersection and its use for RAV vehicles.

2.1 DTEI Investigations (Mace Report)

The Department of Transport, Energy and Infrastructure (DTEI) has conducted a study of Andamooka Road to understand if any modifications to this road and the intersection with the Heavy Vehicle By-pass would be required in order to accept Restricted Access Vehicles (double road vehicles).

A report for DTEI by Mace Engineering Services dated October 2008, states the following recommendations and the outcomes to allow the use of Andamooka Road by RAV vehicles.

1. That Andamooka Road (MM 00 to MM 20) be placed under A-Double (L3B) Road Train Gazette subject to the following actions:
    - The junction with the Olympic Dam to Pimba Road be upgraded to allow extra seal width for A-Double (L3B) turning movements into and out of Andamooka Road;
    - An adequate length entry lane be constructed on Olympic Dam to Pimba Road for left turning vehicles out of Andamooka Road;
    - Vegetation be removed on the inside of the curve on the approach to the junction with the Olympic Dam to Pimba Road to provide adequate stopping sight distance; and
    - DTEI’s acceptance of the lower standard shoulders MM 5.4 to MM 20 either unchanged or requiring to be upgraded to an all weather standard.
2. That Andamooka Road (MM 00 to MM 20) is placed under Higher Mass Limit Gazette.

2.2 Initial Commitment to support the New Mine Entry Gate

The traffic profile used in the DEIS Traffic Impact Assessment (TIA) was based on double road trains to support the expansion project with the opportunity to use triple road trains where appropriate so as to reduce road usage.

Using the TIA and DEIS, double road train road deliveries of construction materials, operating consumables from Pimba Intermodal Terminal would have used the Heavy Vehicle By-pass and Olympic Way to Olympic Dam. These vehicles will now turn right at the Heavy Vehicle by-pass onto Andamooka Road and return along the same route.

BHP Billiton will work collaboratively with the Department of Transport, Energy and Infrastructure to the upgrading of the Andamooka Rd to DTEI standards to allow the operation of Restricted Access Vehicles (RAV) such as B-Double, Double and Triple Road trains. Without this collaborative approach, it is illegal to operate such vehicles along Andamooka Rd. BHP Billiton would face court fines, exposure to liabilities in the event of an accident and damage the road infrastructure. This commitment is required irrespective of any other mitigating action supporting the planned new mine entry gate.

3 Geometric Conditions

The existing intersections of Andamooka Road with Axehead Road and the Heavy Vehicle Bypass comprise of two three arm intersections forming a staggered T- intersection arrangement. The two intersections:

- Are approximately 200m apart,
- Operate within a 110 km/h speed limit,
- Provide one through traffic lane in each direction (north-south),
- Allow right turn auxiliary lanes for turning traffic (approximately 90 meters (inc taper) for vehicles turning into Andamooka Road and 110 metres (inc taper) for vehicles turning into Axehead Road), and
- Provide a short left turn slip lane on the approach and a short stand-up left turn on the departure side of the Andamooka Road intersection.

Photograph 1 - Looking from Andamooka Road intersection towards Axehead Road

Auxiliary Right Turn Lanes

The design of auxiliary right turn lanes at an intersection is explained in Section 5 of AustRoads Guide to Road Design, Part 4A: Unsignalised and Signalised intersections. The significant factor in the design is the traffic speed and for these current intersections the speed limit is 110 km/h.

Current right turn auxiliary lane arrangements do not align with guidelines for a 110 km/h road. For a 110 km/h road this would be 185 metres (including taper), the right turn auxiliary lane for Andamooka Road is currently 90 metres. Likewise the Axehead Road right turn auxiliary lane which is currently 110 metres in length does not align with the guidelines.

Auxiliary Left Turn Lanes

The left slip into Andamooka Road and the left auxiliary lane exiting Andamooka Road are below recommended guidelines for cars on a 110 km/h road. Guidelines for cars on a level grade indicate approximately 160-180 metre auxiliary lanes dependent on left turn radii at Andamooka Road. The current left lanes are less than 125 metres. Further consideration is required in terms of heavy and special vehicles.

Given low through traffic along the Heavy Vehicle By-pass at Axehead Road and rare left turn movements into Axehead Road, it is considered that left lanes are not required.

The current Axehead Road and Andamooka Road staggered intersection does not conform to AustRoads Guidelines for a 110 km/h road.

4 Assumptions

The following assumptions have been used to assess the road and traffic impacts associated with the newly proposed entry gate being located adjacent Hiltaba Village.

- Revised road and traffic impacts based on information contained in the DEIS.
- DEIS project time lines, construction and operational materials, traffic volumes, workforce (2,600 operational staff in the open pit) and accommodation plans (ie Long Distance Commute (LDC) and residential workers are located in Roxby Downs) remain unchanged and used as the basis of investigation.
- Pimba Intermodal terminal operates as planned and road movements from Pimba conclude once the rail line to Olympic Dam is operational. Thereafter all freight movements to the mine area will be via the rail spur and then on internal private roads.
- The re-routed Borefield Road and new access road from the additional entry gate are constructed as priorities once construction commences so as to avoid any impacts to traffic along Borefield Road north of Olympic Dam.
- In agreement with DTEI, the Andamooka Road, intersection of Andamooka Road and the heavy vehicle by pass are upgraded and approved for RAV activities.

- Traffic distribution assumptions consists of: Roby Downs/Long Distance Commute traffic accessing the new entry gate adjacent Hiltaba Village via Axehead Road and Andamooka Road, heavy vehicles from Pimba accessing via the new entry gate adjacent Hiltaba Village via Andamooka Road, and Hiltaba Village traffic accessing the Western Access Road via Andamooka Road and the heavy vehicle by-pass.
- Profile for open pit workforce has been assumed as shown in Table 1.

Table 1 Open Pit Workforce Profile

Year		**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Workforce		400	800	1200	1600	2000	2500	2500	2500	2500
Town	60%	240	480	720	960	1200	1500	1500	1500	1500
LDC	40%	160	320	480	640	800	1000	1000	1000	1000

- Using the profile for open pit workforce as shown in Table 1 the following assumptions as shown in Table 2 have been used to determine a traffic profile for light vehicles and buses from Roxby Downs that would use the Axehead Road and Andamooka Road link via the staggered intersection (AADT) which is presented in Table 3. The mode shares used are considered robust with the potential to bus significantly more Roxby Downs residents.

Table 2 Open Pit Workforce Mode Share Assumption

Workers on shift	**% of workforce**		**25%**
Mode of travel	Residential Town workers	Light Vehicle	80%
		Bus	20%
	LDC workers	Light Vehicle	10%
		Bus	90%
Utilisation	Passengers	Light Vehicle	1
		Bus	40

Table 3 Mine Workforce Traffic Profile (AADT)

Year		**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
AADT	LV	104	208	312	416	520	650	650	650	650
	Bus	5	10	14	19	24	30	30	30	30
	Total	109	218	326	435	544	680	680	680	680

- Profile from the DEIS for double road trains of construction, operational consumables and supplies for mining which will turn right onto Andamooka Road (prior to rail line operation in 2016) is shown in Table 4. Deliveries would be distributed over a 350 day year and over a 12 hour period daily.

Table 4 Annual Heavy Vehicle movements supportive of Construction, Operational Suppliers and Consumables for Mining

Annual One Way Trips **Year**	**1**	**2**	**3**	**4**	**5**	**6**
Pioneer miner	-	-	-	-		
Mining equipment	159	941	95	803	119	74
Mining consumables	360	2510	4877	6832	8334	8129
MMIA	613	1,089	740			
Camp village	700	700	700			
Misc units (contractor offices)		70				
Cement	53	131	394			
Light vehicles - contractors	18	28	3			
General freight NOE	3	125				
Total of One Way Trips (pa)	1906	5595	6809	7635	8453	8203

- Shift assumptions - Open mine workforce will consist of township residents and LDC (located at Roxby Downs), consistent workforce numbers required 24 hours a day at the open pit mine, 4x4 roster for the open mine workforce; therefore, only 25% of total open mine workforce will work any period.
- Peak Hour factors for workforce trips (only a certain proportion of trips will occur within the peak hour, the following factors are based on traffic flows under current mine operations) - 64% of traffic during AM commute period will occur during the peak hour , and 58% of traffic during PM commute period will occur during the peak hour.

Peak traffic periods occur within year 6 (2015) and year 11 (2020). In line with the DEIS, the assessment concentrates on anticipated traffic movements under these conditions.

5 Traffic Flows

The new entry gate adjacent Hiltaba Village would result in a reassignment of traffic away from Olympic Way and the Western Access Road. Consequently, it is anticipated that traffic volumes would increase along Axehead Road and Andamooka Road.

For consistency with the DEIS, traffic flows are calculated for:

- 2015 as this is prior to the commencement of rail freight operations, and
- 2020 as this signifies full workforce conditions.

The following figures outline the anticipated peak hour traffic volumes with the new entry gate. 2020 conditions represent the highest traffic conditions for the Andamooka Road and Axehead Road staggered intersection. Traffic flows without the new entry gate are illustrated in Appendix Q9 of the DEIS.

Figure 2 – 2015 with New Entry Gate

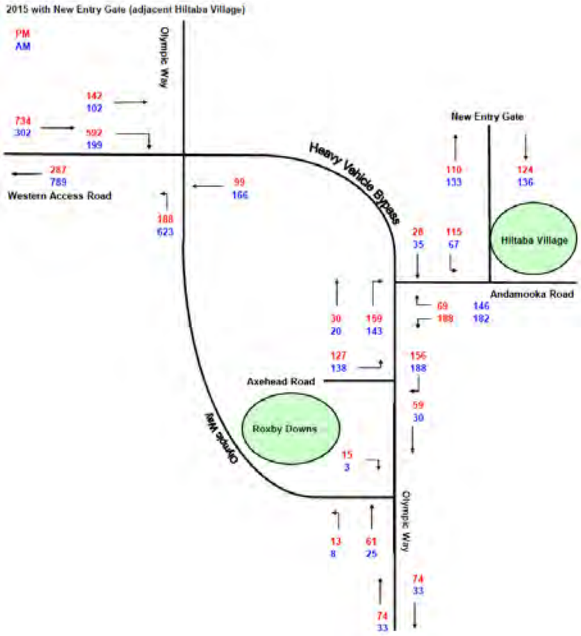


Figure 3 – 2020 with New Entry Gate

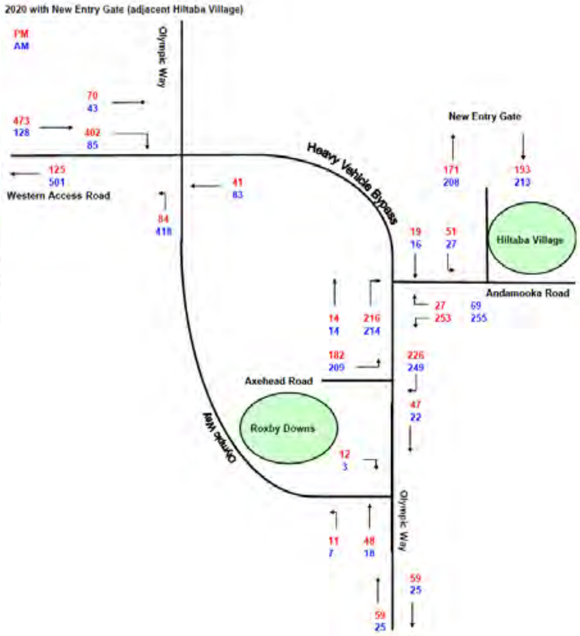


Table 5 indicates the inbound traffic volumes to the Axehead Road/Andamooka Road intersection for the base line (no development) and with the Olympic Dam Expansion inclusive of the new entry gate (adjacent Hiltaba Village).

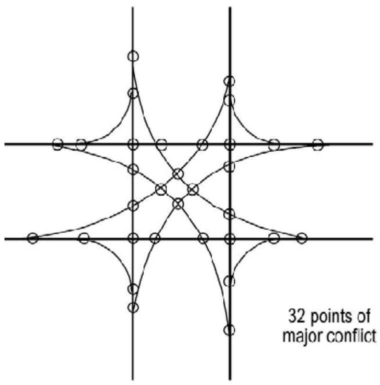
Table 5 Inbound Traffic Flows–Axehead Road/Andamooka Road Staggered Intersection

	Base Line	**Olympic Dam Expansion With New Entry Gate**
2015 AM	115	593
2015 PM	121	588
2020 AM	113	594
2020 PM	117	580

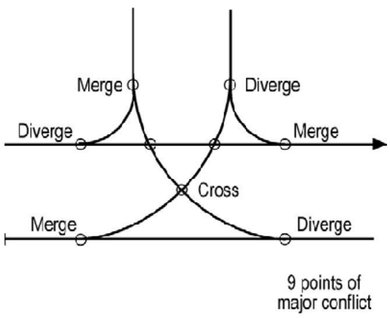
With the new entry gate, traffic volumes increase through the Axehead Road and Andamooka Road staggered intersection, whilst traffic volumes north of Roxby Downs decrease.

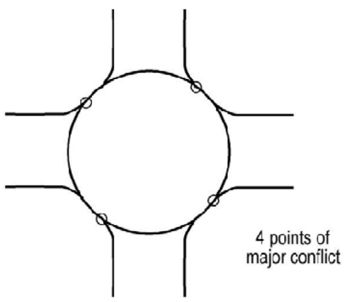
6 Change in Traffic Patterns

The assessment highlights a change in the dominant traffic flows at the Andamooka Road/Axehead Road staggered intersection. The Base line traffic flows as outlined within the DEIS – Appendix Q9 demonstrated no dominant traffic flow with priority given to vehicles travelling along the Heavy Vehicle By-pass. Development of Hiltaba Village and the new entry gate result in a notable east-west movement which becomes the dominant traffic flow. The Heavy Vehicle By Pass accommodates relatively low volumes.



This raises the question of whether the current intersection requires an upgrade to accommodate the change in traffic patterns from both an operational and a safety perspective. Based on traffic patterns alone, it would be reasonable to change the priority of the intersection to better accommodate and reduce delay to east-west movements. This also reduces the risk of conflict if the majority of vehicles do not need to turn. However, this would conflict with the existing road hierarchy and B-Doubles and road trains would continue as a north-south movement.





A change in intersection configuration can also simplify decision making and reduce points of conflict. The basic types of conflict are merging, diverging or weaving at intersections. At a cross intersection there are 32 points of conflict, whereas at a T-intersection this reduces to 9 points of conflicts. At a single lane roundabout the number of conflict points reduces further to 4. An extract from the Guide to Traffic Management – Part 6 (2007) to the left illustrates the points of conflict for typical intersections.

As outlined within the Draft Route Assessment Report (Mace Report), there have been a number of incidents relating to the staggered intersection under pre-2008 conditions (report date, exact 5 year crash period not highlighted).

> “Crash statistics for the past five years on Andamooka Road show that 15 crashes have occurred within the section from MM 00 to MM 20. Eleven of the crashes have been casualty crashes including one fatality. Fourteen of the fifteen crashes have been single vehicle with the other crash being head on. All vehicles involved in the crashes were light vehicles except one 8 tonne rigid truck.”

Traffic patterns are anticipated to change at the Andamooka Road and Axehead Road staged intersection.

7 Intersection Capacity Assessment

An intersection analysis of the two T intersections (Andamooka Rd / Heavy Vehicle Access Road and Axehead Rd / Heavy Vehicle Access Road) was undertaken using SIDRA Intersection (Version 5.0). The assessment compares base traffic assignment and volumes in the DEIS with the anticipated traffic with the new entry gate.

Intersections are assessed on their:

- Degree of Saturation (DoS) which is a simple volume/capacity ratio, a ratio of less than 0.85 is considered within capacity, and
- Level of Service (LoS) which is a measure of delay for an intersection, a LoS of D is considered within acceptable limits although a LoS of C is preferred and more comfortable for drivers

Table 6 – SIDRA LoS Definitions (Extract from Sidra Manual – Table 11.1A)

Level of Service	Control delay per vehicle in seconds (d)		
	Signals	Roundabouts	Stop and Giveway / Yield Signs
A	d ≤ 10	d ≤ 10	d ≤ 10
B	10 < d ≤ 20	10 < d ≤ 20	10 < d ≤ 15
C	20 < d ≤ 35	20 < d ≤ 35	15 < d ≤ 25
D	35 < d ≤ 55	35 < d ≤ 50	25 < d ≤ 35
E	55 < d ≤ 80	50 < d ≤ 70	35 < d ≤ 50
F	80 < d	70 < d	50 < d

Table 7 and Table 8 show the intersection operating conditions for Andamooka Road and Axehead Road respectively. The following tables illustrate the Level of Services for the worst turn movement which relates to the delay. The worst movement is predominantly the right turn out of Andamooka Road. Detailed results are included in Appendix A.

Table 7 - Andamooka Rd / Heavy vehicle bypass

Scenario	**DoS**	**Queue length (metres)**	**Worst Average Delay (seconds)**	**Worst LoS**
AM Peak				
2015 baseline (no expansion)	0.045	2.0	14.2	B*
2020 baseline (no expansion)	0.045	2.0	14.9	B
2015 Expansion (new entry)	0.304	15.3	24.4	C
2020 Expansion (new entry)	0.288	9.1	23.2	D
PM Peak				
2015 base (no expansion)	0.026	1.0	14.4	B*
2020 base (no expansion)	0.026	1.0	14.4	B
2015 Expansion (new entry)	0.218	10.7	28.7	D
2020 Expansion (new entry)	0.287	9.1	27.5	D

* Change in LoS from Draft EIS reflecting LoS criteria adopted in the current SIDRA version 5

Table 8 - Axehead Rd / Heavy vehicle bypass

Scenario	**DoS**	**Longest Queue length (metres)**	**Worst Average Delay (seconds)**	**Worst LoS**
AM Peak				
2015 base (no expansion)	0.033	1.0	8.7	A
2020 base (no expansion)	0.032	1.0	8.7	A
2015 Expansion (new entry)	0.157	5.6	10.6	B
2020 Expansion (new entry)	0.221	7.7	11.1	B
PM Peak				
2015 base (no expansion)	0.022	0.5	8.8	A
2020 base (no expansion)	0.023	0.6	8.8	A
2015 Expansion (new entry)	0.137	4.7	11.2	B
2020 Expansion (new entry)	0.194	7.0	11.6	B

To summarise the capacity assessment:

- The Degree of Saturation is well within operational requirements for all movements (within 0.85)
- In the 2015 PM peak, with the new entry gate operating the Level of Service for traffic turning right out of Andamooka Road changes from LoS C to LoS D i.e. the average wait for right turning vehicles out of Andamooka Road exceeds 25 seconds. Whilst LoS D is an acceptable level, this is not a comfortable delay.

The staggered intersection at Andamooka Road and Axehead Road operates well within the DoS although right turning vehicles out of Andamooka Road have a delay that falls within a less comfortable but acceptable LoS D.

8 Link Assessment

Level of Service (LoS) is a measure of operational conditions within a stream of traffic. Austroads 1988 provides a description of each LoS as outlined in Table 9. The actual traffic volumes that result in each LoS are dependent on a number of factors, including number of traffic lanes provided in each direction, traffic speeds, road width, and the proportions of buses and heavy vehicles.

Table 9 – Link Level of Service Definitions

Level of Service	Description
A	Is a condition of free flow in which individual drivers are virtually unaffected by the presence of other drivers. Freedom to select desired speeds and to manoeuvre within the traffic stream is extremely high, and the general level of comfort and convenience provided is excellent.
B	Is in the stream of stable flow and drivers still have reasonable freedom to select their desired speed and to manoeuvre within the traffic stream, although the general level of comfort and convenience is a little less than with LoS A.
C	Is also in the zone of stable flow, but most drivers are restricted to some extent in their freedom to select their desired speed and to manoeuvre within the traffic stream. The general level of comfort and convenience declines noticeably at this level.
D	Is close to the limit of stable flow and is approaching unstable flow. All drivers are severely restricted in their freedom to select their desired speed and to manoeuvre within the traffic stream. The general level of comfort and convenience is poor, and small increases in traffic flow will generally cause operational problems.
E	Occurs when traffic volumes are at or close to capacity, and there is virtually no freedom to select desired speeds or manoeuvre within the traffic stream. Flow is unstable and minor disturbances within the traffic stream will cause break-down.
F	Is in the zone of forced flow.

Table 10 and Table 11 illustrates the anticipated link LoS for Andamooka Road and Olympic Way north of Roxby Downs for the AM and PM peak. Total two way peak hour traffic volumes are indicated although these don't have a direct correlation with the LoS as these are dependent on a number of factors including directional split; for further details see Appendix B.

Axehead Road attracts less traffic and would perform better than Andamooka Road. As indicated within the DEIS, Olympic Way is anticipated to fall into a link LoS of D during the peak construction year and thereafter improve as the Olympic Dam expansion falls into a steady operational state.

Table 10 – AM Mid Block Level of Service

Road Link	Two way peak hour traffic			Level of Service		
	Base Line (2015)	Peak (2015)	Steady State (2020)	Base Line (2015)	Peak (2015)	Steady State (2020)
Andamooka Road (adjacent the Heavy Vehicle Bypass)	63	538	565	A	C	B
Olympic Way (adjacent Opal Road)	706	822	503	C	D	C

Table 11 – PM Mid Block Level of Service

Road Link	**Two way peak hour traffic**			**Level of Service**		
	Base Line (2015)	Peak (2015)	Steady State (2020)	Base Line (2015)	Peak (2015)	Steady State (2020)
Andamooka Road (adjacent the Heavy Vehicle Bypass)	68	531	565	A	B	C
Olympic Way (adjacent Opal Road)	670	780	486	C	D	C

The link LoS falls within acceptable operational limits and under ongoing operational conditions, traffic is stable.

Appendix A

Intersection Assessment

A1.1 Baseline (no expansion)

See Draft EIS – Appendix Q9 for baseline Andamooka Road/HV bypass baseline

2015 Axehead / Heavy vehicle bypass

MOVEMENT SUMMARY **Site: 2015 AM Bypass/ Axehead**

Roxby Downs Bypass Rd / Axehead Rd 2015 AM peak - base flows only
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	7.7	LOS A	0.0	0.0	0.12	0.56	49.1
2	T	8	50.0	0.006	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		9	44.4	0.006	0.9	LOS A	0.0	0.0	0.01	0.06	99.8
North: HV Bypass (nth app)											
8	T	11	40.0	0.007	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	42	0.0	0.033	8.3	LOS A	0.1	1.0	0.06	0.65	48.6
Approach		53	8.0	0.033	6.6	LOS A	0.1	1.0	0.05	0.52	56.9
West: Axehead Rd											
10	L	11	0.0	0.011	7.6	LOS A	0.0	0.3	0.06	0.58	49.5
12	R	5	0.0	0.006	8.7	LOS A	0.0	0.2	0.19	0.60	48.1
Approach		16	0.0	0.011	8.0	LOS A	0.0	0.3	0.10	0.59	49.0
All Vehicles		78	10.8	0.033	6.2	NA	0.1	1.0	0.05	0.48	59.0

MOVEMENT SUMMARY **Site: 2015 PM Bypass/ Axehead**

Roxby Downs Bypass Rd / Axehead Rd 2015 PM peak - base flows only
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	7.6	LOS A	0.0	0.0	0.07	0.57	49.4
2	T	24	34.8	0.015	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		25	33.3	0.015	0.3	LOS A	0.0	0.0	0.00	0.02	106.1
North: HV Bypass (nth app)											
8	T	28	29.6	0.017	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	17	0.0	0.013	8.4	LOS A	0.1	0.4	0.10	0.63	48.4
Approach		45	18.6	0.017	3.1	LOS A	0.1	0.4	0.04	0.24	80.0
West: Axehead Rd											
10	L	22	0.0	0.022	7.7	LOS A	0.1	0.5	0.10	0.57	49.2
12	R	5	0.0	0.007	8.8	LOS A	0.0	0.2	0.22	0.60	48.0
Approach		27	0.0	0.022	7.9	LOS A	0.1	0.5	0.12	0.57	49.0
All Vehicles		98	17.2	0.022	3.7	NA	0.1	0.5	0.05	0.28	74.5

2020 Axehead / Heavy vehicle bypass

MOVEMENT SUMMARY

Site: 2020 AM Bypass/ Axehead

Roxby Downs Bypass Rd / Axehead Rd 2020 AM peak - base flows only
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	7.7	LOS A	0.0	0.0	0.11	0.56	49.1
2	T	7	42.9	0.005	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		8	37.5	0.005	1.0	LOS A	0.0	0.0	0.01	0.07	98.6
North: HV Bypass (nth app)											
8	T	11	30.0	0.006	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	41	0.0	0.032	8.3	LOS A	0.1	1.0	0.05	0.65	48.6
Approach		52	6.1	0.032	6.6	LOS A	0.1	1.0	0.04	0.52	57.1
West: Axehead Rd											
10	L	11	0.0	0.011	7.6	LOS A	0.0	0.3	0.05	0.58	49.5
12	R	5	0.0	0.006	8.7	LOS A	0.0	0.2	0.19	0.60	48.2
Approach		16	0.0	0.011	8.0	LOS A	0.0	0.3	0.10	0.59	49.0
All Vehicles		76	8.3	0.032	6.3	NA	0.1	1.0	0.05	0.48	58.7

MOVEMENT SUMMARY

Site: 2020 PM Bypass/ Axehead

Roxby Downs Bypass Rd / Axehead Rd 2020 PM peak - base flows only
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	7.6	LOS A	0.0	0.0	0.07	0.57	49.4
2	T	22	28.6	0.013	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		23	27.3	0.013	0.3	LOS A	0.0	0.0	0.00	0.03	105.7
North: HV Bypass (nth app)											
8	T	26	24.0	0.016	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	17	0.0	0.013	8.4	LOS A	0.1	0.4	0.09	0.64	48.4
Approach		43	14.6	0.016	3.3	LOS A	0.1	0.4	0.04	0.25	78.9
West: Axehead Rd											
10	L	23	0.0	0.023	7.7	LOS A	0.1	0.6	0.09	0.57	49.3
12	R	5	0.0	0.007	8.8	LOS A	0.0	0.2	0.21	0.60	48.1
Approach		28	0.0	0.023	7.9	LOS A	0.1	0.6	0.11	0.57	49.0
All Vehicles		95	13.3	0.023	3.9	NA	0.1	0.6	0.05	0.29	73.0

A1.2 Olympic Dam Expansion - New Entry Gate

2015 Axehead Rd / Heavy Vehicle Bypass road – New Entry

MOVEMENT SUMMARY — Site: 2015 AM Bypass/ Axehead

Roxby Downs Bypass Rd / Axehead Rd 2015 AM peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	8.3	LOS A	0.0	0.0	0.29	0.54	48.2
2	T	26	24.0	0.016	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		27	23.1	0.016	0.3	LOS A	0.0	0.0	0.01	0.02	106.2
North: HV Bypass (nth app)											
8	T	32	20.0	0.018	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	198	1.6	0.157	8.4	LOS A	0.8	5.6	0.11	0.63	48.4
Approach		229	4.1	0.157	7.3	LOS A	0.8	5.6	0.10	0.55	53.9
West: Axehead Rd											
10	L	145	2.2	0.146	7.8	LOS A	0.6	3.9	0.15	0.56	49.0
12	R	5	0.0	0.009	10.6	LOS B	0.0	0.3	0.41	0.64	46.5
Approach		151	2.1	0.146	7.9	LOS B	0.6	3.9	0.15	0.56	48.9
All Vehicles		407	4.7	0.157	7.0	NA	0.8	5.6	0.11	0.52	54.3

MOVEMENT SUMMARY — Site: 2015 PM Bypass/ Axehead

Roxby Downs Bypass Rd / Axehead Rd 2015 PM peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	8.1	LOS A	0.0	0.0	0.26	0.54	48.4
2	T	78	10.8	0.043	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		79	10.7	0.043	0.1	LOS A	0.0	0.0	0.00	0.01	108.7
North: HV Bypass (nth app)											
8	T	62	13.6	0.035	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	164	1.9	0.131	8.7	LOS A	0.7	4.7	0.19	0.63	48.0
Approach		226	5.1	0.131	6.3	LOS A	0.7	4.7	0.14	0.46	59.8
West: Axehead Rd											
10	L	134	2.4	0.137	7.9	LOS A	0.5	3.8	0.19	0.57	48.7
12	R	5	0.0	0.009	11.2	LOS B	0.0	0.3	0.45	0.65	45.9
Approach		139	2.3	0.137	8.1	LOS B	0.5	3.8	0.20	0.57	48.6
All Vehicles		444	5.2	0.137	5.7	NA	0.7	4.7	0.13	0.41	62.0

2020 Axehead Rd / Heavy Vehicle Bypass road – New Entry

MOVEMENT SUMMARY

Site: 2020 AM Bypass/ Axehead

Roxby Downs Bypass Rd / Axehead Rd 2020 AM peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	8.5	LOS A	0.0	0.0	0.34	0.54	47.9
2	T	20	15.8	0.011	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		21	15.0	0.011	0.4	LOS A	0.0	0.0	0.02	0.03	105.0
North: HV Bypass (nth app)											
8	T	23	13.6	0.013	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	262	2.0	0.207	8.4	LOS A	1.1	7.7	0.10	0.64	48.4
Approach		285	3.0	0.207	7.7	LOS A	1.1	7.7	0.09	0.58	51.6
West: Axehead Rd											
10	L	220	2.4	0.221	7.9	LOS A	0.9	6.2	0.26	0.51	48.3
12	R	5	0.0	0.009	11.1	LOS B	0.0	0.3	0.45	0.65	45.9
Approach		225	2.3	0.221	8.0	LOS B	0.9	6.2	0.27	0.51	48.3
All Vehicles		532	3.2	0.221	7.5	NA	1.1	7.7	0.16	0.53	51.5

MOVEMENT SUMMARY

Site: 2020 PM Bypass/ Axehead

Roxby Downs Bypass Rd / Axehead Rd 2020 PM peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: HV Bypass (sth app)											
1	L	1	0.0	0.001	8.4	LOS A	0.0	0.0	0.32	0.54	48.0
2	T	51	10.4	0.028	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		52	10.2	0.028	0.2	LOS A	0.0	0.0	0.01	0.01	107.9
North: HV Bypass (nth app)											
8	T	49	10.6	0.027	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
9	R	238	2.2	0.189	8.6	LOS A	1.0	7.0	0.16	0.63	48.2
Approach		287	3.7	0.189	7.1	LOS A	1.0	7.0	0.13	0.52	55.2
West: Axehead Rd											
10	L	192	2.7	0.194	7.9	LOS A	0.8	5.5	0.16	0.56	48.9
12	R	5	0.0	0.010	11.6	LOS B	0.0	0.3	0.47	0.66	45.5
Approach		197	2.7	0.194	8.0	LOS B	0.8	5.5	0.17	0.57	48.8
All Vehicles		536	3.9	0.194	6.7	NA	1.0	7.0	0.13	0.49	56.1

2015 Andamooka Rd / Heavy Vehicle Bypass road – New Entry

MOVEMENT SUMMARY

Site: 2015 AM peak Bypass/ Andamooka

Roxby Downs Bypass Rd / Andamooka Rd 2015 AM Peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Heavy vehicle bypass (Sth App)											
2	T	21	10.0	0.011	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
3	R	151	4.9	0.129	18.8	LOS C	0.6	4.3	0.12	0.76	66.5
Approach		172	5.5	0.129	16.5	LOS C	0.6	4.3	0.10	0.67	71.6
East: Andamooka Rd (East App)											
4	L	192	3.8	0.210	18.5	LOS C	0.9	6.5	0.21	0.72	65.6
6	R	154	19.9	0.304	24.4	LOS C	1.8	15.3	0.53	0.87	58.4
Approach		345	11.0	0.304	21.1	LOS C	1.8	15.3	0.35	0.78	62.2
North: Heavy vehicle bypass (Nth App)											
7	L	71	43.3	0.083	21.8	LOS C	0.4	4.4	0.32	0.72	64.5
8	T	37	5.7	0.020	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		107	30.4	0.083	14.3	LOS C	0.4	4.4	0.21	0.47	79.0
All Vehicles		624	12.8	0.304	18.7	NA	1.8	15.3	0.26	0.70	67.5

MOVEMENT SUMMARY

Site: 2015 PM peak Bypass/ Andamooka

Roxby Downs Bypass Rd / Andamooka Rd 2015 PM Peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Heavy vehicle bypass (Sth App)											
2	T	32	13.3	0.018	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
3	R	167	4.4	0.143	18.8	LOS C	0.6	4.8	0.11	0.77	66.6
Approach		199	5.8	0.143	15.8	LOS C	0.6	4.8	0.09	0.64	73.2
East: Andamooka Rd (East App)											
4	L	198	3.7	0.217	18.6	LOS C	0.9	6.8	0.23	0.72	65.4
6	R	73	42.0	0.208	28.7	LOS D	1.0	10.7	0.57	0.90	54.1
Approach		271	14.0	0.218	21.3	LOS D	1.0	10.7	0.32	0.77	61.9
North: Heavy vehicle bypass (Nth App)											
7	L	121	25.2	0.123	20.5	LOS C	0.6	5.8	0.32	0.72	64.5
8	T	29	14.3	0.017	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		151	23.1	0.123	16.5	LOS C	0.6	5.8	0.26	0.58	72.6
All Vehicles		620	13.6	0.218	18.4	NA	1.0	10.7	0.23	0.68	67.9

2020 Andamooka Rd / Heavy Vehicle Bypass road – New Entry

MOVEMENT SUMMARY — Site: 2020 AM peak Bypass/ Andamooka

Roxby Downs Bypass Rd / Andamooka Rd 2020 AM Peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Heavy vehicle bypass (Sth App)											
2	T	15	21.4	0.009	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
3	R	225	2.3	0.191	18.6	LOS C	0.9	6.4	0.15	0.74	66.3
Approach		240	3.5	0.191	17.5	LOS C	0.9	6.4	0.14	0.69	68.8
East: Andamooka Rd (East App)											
4	L	268	2.0	0.289	18.2	LOS C	1.3	9.1	0.21	0.69	65.6
6	R	73	15.9	0.140	23.2	LOS C	0.7	5.8	0.48	0.83	59.9
Approach		341	4.9	0.288	19.3	LOS C	1.3	9.1	0.27	0.72	64.3
North: Heavy vehicle bypass (Nth App)											
7	L	28	40.7	0.036	22.1	LOS C	0.2	1.8	0.38	0.72	63.9
8	T	16	13.3	0.009	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		44	31.0	0.036	14.2	LOS C	0.2	1.8	0.24	0.46	79.1
All Vehicles		625	6.2	0.288	18.2	NA	1.3	9.1	0.22	0.69	67.0

MOVEMENT SUMMARY — Site: 2020 PM peak Bypass/ Andamooka

Roxby Downs Bypass Rd / Andamooka Rd 2020 PM Peak with new site entry
Giveway / Yield (Two-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Heavy vehicle bypass (Sth App)											
2	T	15	35.7	0.009	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
3	R	227	2.3	0.193	18.6	LOS C	0.9	6.5	0.13	0.75	66.4
Approach		242	4.3	0.193	17.5	LOS C	0.9	6.5	0.13	0.71	68.9
East: Andamooka Rd (East App)											
4	L	266	2.0	0.288	18.2	LOS C	1.3	9.1	0.18	0.72	65.9
6	R	28	40.7	0.078	27.5	LOS D	0.4	3.8	0.52	0.84	55.8
Approach		295	5.7	0.287	19.1	LOS D	1.3	9.1	0.21	0.73	64.8
North: Heavy vehicle bypass (Nth App)											
7	L	54	21.6	0.057	20.5	LOS C	0.3	2.4	0.36	0.72	64.1
8	T	20	26.3	0.012	0.0	LOS A	0.0	0.0	0.00	0.00	110.0
Approach		74	22.9	0.057	14.9	LOS C	0.3	2.4	0.26	0.53	75.5
All Vehicles		611	7.2	0.287	18.0	NA	1.3	9.1	0.18	0.69	67.7

Appendix B

Link Assessment

B1 AM - Link Level of Service

2015 Base Line AM

In accordance with Austroads 1988. Roadway Capacity - Part 2

Olympic Way (adjacent Opal Road)

North	499	Pt	0.01983		$(v/c)_i$	0.43	
South	207						
Total	706						
Buses	0	Eb	2	Table 3.4	f_w	0.89	Table 3.3
% Buses	0						
Trucks	14	Et	3	Table 3.4	f_{hv}	0.961853	
% Trucks	2						
		f_{hv}	0.961853		SF_i	855.4669	vph
Level of Sevice	C						
Traffic Limit (vph)	855						

Andamooka Road (adjacent the Heavy Vehicle Bypass)

East	11	Pt	0		$(v/c)_i$	0.15	
West	52						
Total	63						
Buses	0	Eb	2	Table 3.4	f_w	0.75	Table 3.3
% Buses	0						
Trucks	0	Et	3	Table 3.4	f_{hv}	1	
% Trucks	0						
		f_{hv}	1		SF_i	261.45	vph
Level of Sevice	A						
Traffic Limit (vph)	261						

2015 AM - Olympic Dam Expansion

In accordance with Austroads 1988. Roadway Capacity - Part 2

Olympic Way (adjacent Opal Road)

North	623	Pt	0.049878		$(v/c)_i$	0.64	
South	199						
Total	822						
Buses	38	Eb	2	Table 3.4	f_w	0.83	Table 3.3
% Buses	5						
Trucks	41	Et	3	Table 3.4	f_{hv}	0.872611	
% Trucks	5						
		f_{hv}	0.872611		SF_i	1077.247	vph
Level of Sevice	D						
Traffic Limit (vph)	1077						

Andamooka Road (adjacent the Heavy Vehicle Bypass)

East	210	Pt	0.01487		$(v/c)_i$	0.43	
West	328						
Total	538						
Buses	64	Eb	2	Table 3.4	f_w	0.94	Table 3.3
% Buses	12						
Trucks	8	Et	3	Table 3.4	f_{hv}	0.87055	
% Trucks	1						
		f_{hv}	0.87055		SF_i	817.7607	vph
Level of Sevice	C						
Traffic Limit (vph)	818						

2020 AM - Ongoing Olympic Dam Operation

In accordance with Austroads 1988. Roadway Capacity - Part 2

Olympic Way (adjacent Opal Road)

North	418	Pt	0.081511		$(v/c)_i$	0.43	
South	85						
Total	503						
Buses	32	Eb	2	Table 3.4	f_w	0.83	Table 3.3
% Buses	6						
Trucks	41	Et	3	Table 3.4	f_{hv}	0.815235	
% Trucks	8						
		f_{hv}	0.815235		SF_i	676.1849	vph
Level of Sevice	C						
Traffic Limit (vph)	676						

Andamooka Road (adjacent the Heavy Vehicle Bypass)

East	241	Pt	0		$(v/c)_i$	0.27	
West	324						
Total	565						
Buses	32	Eb	2	Table 3.4	f_w	0.94	Table 3.3
% Buses	6						
Trucks	0	Et	3	Table 3.4	f_{hv}	0.946399	
% Trucks	0						
		f_{hv}	0.946399		SF_i	558.2155	vph
Level of Sevice	B						
Traffic Limit (vph)	558						

B2 PM - Link Level of Service

2015 Base Line PM

In accordance with Austroads 1988. Roadway Capacity - Part 2

Olympic Way (adjacent Opal Road)

North	194	Pt	0.035821		$(v/c)_i$	0.43	
South	476						
Total	670						
Buses	0	Eb	2	Table 3.4	f_w	0.89	Table 3.3
% Buses	0						
Trucks	24	Et	3	Table 3.4	f_{hv}	0.933148	
% Trucks	4						
		f_{hv}	0.933148		SF_i	889.9321	vph
Level of Sevice	C						
Traffic Limit (vph)	890						

Andamooka Road (adjacent the Heavy Vehicle Bypass)

East	34	Pt	0		$(v/c)_i$	0.12	
West	34						
Total	68						
Buses	0	Eb	2	Table 3.4	f_w	1	Table 3.3
% Buses	0						
Trucks	0	Et	3	Table 3.4	f_{hv}	1	
% Trucks	0						
		f_{hv}	1		SF_i	278.88	vph
Level of Sevice	A						
Traffic Limit (vph)	279						

2015 PM - Olympic Dam Expansion

In accordance with Austroads 1988. Roadway Capacity - Part 2

Olympic Way (adjacent Opal Road)

North	188	Pt	0.052564		$(v/c)_i$	0.64	
South	592						
Total	780						
Buses	40	Eb	2	Table 3.4	f_w	0.83	Table 3.3
% Buses	5						
Trucks	41	Et	3	Table 3.4	f_{hv}	0.864745	
% Trucks	5						
		f_{hv}	0.864745		SF_i	1067.535	vph
Level of Sevice	D						
Traffic Limit (vph)	1068						

Andamooka Road (adjacent the Heavy Vehicle Bypass)

East	274	Pt	0.015066		$(v/c)_i$	0.27	
West	257						
Total	531						
Buses	64	Eb	2	Table 3.4	f_w	1	Table 3.3
% Buses	12						
Trucks	8	Et	3	Table 3.4	f_{hv}	0.869067	
% Trucks	2						
		f_{hv}	0.869067		SF_i	545.3222	vph
Level of Sevice	B						
Traffic Limit (vph)	545						

2020 PM - Ongoing Olympic Dam Operation

In accordance with Austroads 1988. Roadway Capacity - Part 2

Olympic Way (adjacent Opal Road)

North	84	Pt	0.084362		$(v/c)_i$	0.43	
South	402						
Total	486						
Buses	30	Eb	2	Table 3.4	f_w	0.83	Table 3.3
% Buses	6						
Trucks	41	Et	3	Table 3.4	f_{hv}	0.812709	
% Trucks	8						
		f_{hv}	0.812709		SF_i	674.0898	vph
Level of Sevice	C						
Traffic Limit (vph)	674						

Andamooka Road (adjacent the Heavy Vehicle Bypass)

East	241	Pt	0.014159		$(v/c)_i$	0.43	
West	324						
Total	565						
Buses	32	Eb	2	Table 3.4	f_w	0.94	Table 3.3
% Buses	6						
Trucks	8	Et	3	Table 3.4	f_{hv}	0.921697	
% Trucks	1						
		f_{hv}	0.921697		SF_i	865.8056	vph
Level of Sevice	C						
Traffic Limit (vph)	866						