Your ref: [0000]
**Our ref:** 12545790

**15 September 2021**

**Andrew Stone**
**Port of Newcastle Operations Pty Ltd**
**Level 4, 251 Wharf Road**
**Newcastle NSW 2300**

**46 Fitzroy Street / 65 Denison Street, Carrington – analysis of soil response to theoretical fault displacement associated with mine subsidence (Phase2 analysis)**

Dear Andrew

# 1. Introduction

This letter responds to Subsidence Advisory NSW (SA NSW) - Ref. FN09-20447N1 / TBA21-02320 dated 1 July 2021 which, following their review of the GHD 2021 report, requested either of the following approaches to provide additional justification for the residual subsidence parameters.

– *An updated report be provided detailing potential worst-case residual subsidence parameters for the site, or*
– *Additional geotechnical investigation be undertaken involving boreholes to the base of the Borehole Seam to further quantify recommended subsidence parameters for the site. Any future borehole investigation should target pillars and voids.*

This letter presents the results of geotechnical analysis of ground surface affects resulting from a 'credible worst-case' and 'absolute worst-case' mine subsidence event in the Borehole Seam at 46 Fitzroy Street / 65 Denison Street, Carrington. The analysis was commissioned by Port of Newcastle (PON) following GHD's recommendation to prepare 'plane strain elasto-plastic analysis of mine subsidence' in response to SA NSW's abovementioned request.

The purpose of the analysis and this letter is to provide supplementary information on potential mine subsidence impacts to be considered in the approval, design and construction of a proposed four-storey building. Details of the proposed development and mine subsidence investigations and assessments completed for the proposed development to date are provided in the April 2021 GHD report[1]. This letter is to be read in conjunction with this report.

# 2. Background

A four storey commercial building development has been proposed at 46 Fitzroy Street / 65 Denison Street, Carrington (Lot 33, DP 1078910) by Port of Newcastle (PON). No basement is proposed with the finished ground level to be at about 2.6 m AHD. A stormwater detention tank is understood to be proposed on the eastern portion of the site with an invert of about 0.9 m AHD.

[1] GHD report for Port of Newcastle Operations Pty Ltd. April 2021. '65 Denison Street Development – Carrington, Geotechnical and Mine Subsidence Report. Rev 0. 12545790-62329, 29 April 2021.

GHD carried out a mine subsidence desktop review for this site in 2020. This review confirmed the site and surrounding area is undermined by abandoned mine workings in the Borehole Seam of coal at about 66 m to 69 m depth. The mine working height (tops and bottoms) is expected to be the full seam thickness of 6 m based on our desktop review of historical data as reported in GHD 2020[2]. Above the seam is 20 to 22 m thickness of interbedded siltstone and sandstone and above this 44 to 49 m thickness of alluvial sand and clay.

Additional investigations were subsequently completed earlier in 2021 to assist in further determining subsidence parameters including 8 boreholes and 5 test pits. The results of these investigations and subsequent analysis were provided in the April 2021 GHD report[3]. This assessment determined pillars beneath the proposed building are interpreted to be crushed by about 1 m from a full seam thickness of 6 m based on borehole GBH6. The pillar crushing is interpreted to extend east of the proposed building and across a fault recorded on the mine plans to be about 13 m to the east of the proposed building at its closest point. Based on the subsidence induced cracks observed in borehole GBH5 and review of mine plans and other historical records, roof convergence beneath the entire site is anticipated.

Results from the investigation were communicated to Subsidence Advisory NSW who subsequently requested additional justification of subsidence parameters. An additional assessment has therefore been completed based on Subsidence Advisory's first approach to detail the 'potential worst case' residual subsidence parameters, the results of which are discussed in this letter.

This assessment utilised the same available roof convergence and historical subsidence data together with additional desktop data discovered as well as a two-dimensional (plane strain) numerical finite element analysis of the soil overburden response to stepped rockhead displacements along a line to the east of the proposed building, representing the fault. The analysis has been used to calculate and plot ground surface subsidence, strain, tilt and curvature in relation to the horizontal distance from the line (fault).

Despite the occurrence of roof convergence and hence subsidence, there is a possibility of future residual subsidence. That is, some additional subsidence resulting from a change in stress conditions or reduction in coal pillar stiffness. While residual subsidence is a recognised phenomenon, the mechanism by which it would occur at the subject site is not established and as such, only estimates of the residual roof convergence can be used to calculate a resulting subsidence profile at the ground surface together with the associated strains, curvatures and tilts.

# 3. Roof convergence and subsidence records

Comparable mining and overburden characteristics exist in areas surrounding the site where the same geological sequence, mining method and seam thicknesses exist in other parts of the Wickham and Bullock Island Colliery (W&BIC) workings. This information provides a useful guide to the range of typical roof convergence and surface subsidence where pillar failure has occurred such as at the subject site.

As reported in GHD 2020, subsidence accounts and convergence estimates from surrounding areas are:

– Up to about two feet (~0.6 m) of ground surface subsidence at Darvall Street, south west of the site where tops and bottoms were taken in every bord
– About 930 m north at Hargrave Street, three feet nine inches (~1.1 m) of subsidence was reported to have occurred in 1901/1902
– Roof convergences of between 0.1 m and 1.65 m based on geotechnical assessments by Coffey Geotechnics at Cottage Creek in 2009 about 650 m to the south.

Considering that pillar failure beneath the subject site has already occurred, the worst-case additional roof convergence was found to be a differential displacement concentrated along a vertical fault plane. The magnitude of such differential displacement was judged likely to be in the order of 0.1 m with 0.2 m being

[2] GHD report. '46 Fitzroy Street, Carrington, Mine Subsidence Assessment'. Rev 0. 21 May April 2020.
[3] GHD report for Port of Newcastle Operations Pty Ltd. April 2021. '65 Denison Street Development – Carrington, Geotechnical and Mine Subsidence Report. Rev 0. 12545790-62329, 29 April 2021.

adopted as a credible worst-case. A semi-empirical analysis of resulting ground surface subsidence for this case was presented in the April 2021 GHD report.

Following SA NSW review of the April 2021 report, further consideration has been given to the possible magnitude of differential roof convergence beneath the subject site as well as the behavior of the overburden soil in response to such movement.

A further search for data on convergence ranges in the W&BIC Borehole Seam workings returned a report by Coffey[4] in the Lee Wharf 5 area at Honeysuckle. This report presents data from 18 geotechnical boreholes through crushed pillars (from 28 boreholes in total) indicating mine roof convergence to be between 7% and 24% (average 15%, median 14%) of full seam thickness in similar W&BIC Borehole Seam workings where 'tops and bottoms' have been taken and where cover depth is about 75 m. Where cover depth is less, and/or percentage of coal extraction less, the amount of roof convergence would also be expected to be less. Geological anomalies such as a fault and the layout of the mine workings (in particular the proximity to unmined coal or un-failed pillars) would also be expected to affect roof convergence. The reported data from Lee Wharf 5 includes a fault through the site and has a similar extraction ratio and mining method and height. The 7% to 24% of full seam thickness as a range of roof convergence is comparable to the Carrington site where the full seam height is expected to be 6 m. That is, 0.4 m to 1.5 m of roof convergence could be expected at the Carrington site. Based on the observed conditions in GBH5 and GBH6, as well as review of the mine plans, most, if not all of this convergence is judged to already have occurred.

While the stepped roof convergence considered in the April 2021 GHD report is maintained as the worst-case scenario, the magnitude of this differential displacement is now considered to potentially be greater on the basis of the Lee Wharf 5 data and taking the more conservative approach of assuming roof convergence is transmitted in full to rockhead level along a discrete vertical plane.

# 4. Analysis approach and methodology

How convergence of the mine roof over a localised 'creep' area propagates through the overburden rock and soil is a function of the creep's lateral extent and shape as well as the properties of the overburden materials. In particular, the overburden rock would bridge (span) failed pillars to some degree as well as crack and open along bedding partings. The net effect would be that the amount of movement at the top of the bedrock (rockhead) about 20 to 22 m above the top of the Borehole Seam would be less than the amount of mine roof convergence. The subsidence at rockhead level would in turn be ameliorated (smoothed) as subsidence propagates through the 44 to 49 m thickness of alluvial sand and clay.

The stratigraphy and engineering properties of the soil was investigated and reported in GHD 2021. The extent of investigation allowed use of finite element methods as a means to model soil behaviour in response to a subsidence event. While the stratigraphy and intact strength of the interbedded siltstone and sandstone unit are also known with sufficient reliability to warrant numerical modelling, the subsidence induced fracturing of the rock and geometry and engineering properties of the fault or faults are not.

An 'absolute worst-case' distribution is a stepped roof convergence along the fault with the roof east of the fault dropping relative to the roof west of the fault as depicted in Figure 1, and the displacement along the mine roof transmitted to rockhead level at RL 46 m. All other roof convergence scenarios would produce less concentrated effects at ground level and hence less stain, tilt and curvature.

The transmittal of mine roof convergence to rockhead level along a vertical plane representing the fault is a conservative simplification. In realty, convergence of the mine roof resulting for pillar crushing would occur either side of the fault to varying degrees reducing the differential displacement. Additionally, the displacement would be spread over a width of several meters rather than concentrated along a vertical plane.

[4] Coffey Services Australia Pty Ltd report to University of Newcastle. 2018. 'Proposed University of Newcastle – Honeysuckle City Campus Development – Site 1 – Mine Subsidence Remediation Strategy and Numerical Analysis'. Ref. 754-NTLE213472-R06.Rev1, 16 November 2018.

Considering the range of typical roof convergence magnitudes reported, historical subsidence accounts and estimate of 1 m roof convergence at GBH6, a differential roof convergence across the fault (modelled as a vertical plane) of 1 m is considered an absolute worst-case. This could result from additional roof convergence on either side of the fault such as 0.5 m on the west and 1.5 m on the east, or only on the eastern side of the fault.

For reasons stated above, this displacement would not be transmitted in full through the rock along a discrete vertical plane and so the modelled case is conservative in this regard.

A credible worst-case is considered to be residual subsidence on the east side of the fault nearing the 0.6 m reported at nearby Darvall Street, without any additional convergence west of the fault. As significant pillar crushing is interpreted to have already occurred east of the fault, as well as on the west side of the fault, the magnitude of residual subsidence would be less than 0.6 m and reasonably in the order of 0.1 m to 0.2 m. Taking this into consideration, a maximum ground surface subsidence and hence displacement at rockhead level of 0.5 m is considered a credible worst-case.

To more accurately derive the resulting subsidence parameters for the above absolute and credible worst-case scenarios, a more sophisticated soil analysis was employed to model the overburden response to stepped displacement across a vertical plane representing the fault. A plane strain (two-dimensional) analysis using a finite element method was used to assess the resulting behavior of the soil and hence subsidence at ground surface. Ground surface subsidence profiles, curvatures, tilts and strains were calculated for the following cases:

– 0.2 m vertical displacement along a line at 46 m depth (rockhead)
– 0.5 m vertical displacement along a line at 46 m depth (rockhead)
– 0.8 m vertical displacement along a line at 46 m depth (rockhead)
– 1.0 m vertical displacement along a line at 46 m depth (rockhead).


*Figure 1 Modelled rockhead stepped displacement*

The geotechnical analysis software Phase[2] by Rocscience was used with a linear elasto-plastic soil model. That is, first, the soil behavior was approximated as being perfectly elastic to locate regions of intense strain requiring mesh refinement. Then, elasto-plastic analysis based on Mohr-Coulomb parameters was conducted, with the revised mesh. The Mohr-Coulomb parameters defined from the geotechnical investigation data were adopted for the modeling.

The outputs are presented as subsidence profiles, curvature, tilt and strain plots with maximum values for each case tabulated.

A vertical section passing through GBH6 and GBH5 (refer Figure 1) was considered for the numerical analysis. While the soil strata observed in the two boreholes are generally consistent, the soil layer thicknesses vary slightly between the two boreholes. Average layer thicknesses were considered in the numerical model, with horizontal bedding. Table 1 shows the ground model and Mohr-Coulomb material parameters adopted. Groundwater was considered nominally at RL 0.5 m AHD.

***Table 1 Adopted ground model and material parameters in the numerical modelling.***

| Unit | Layer top RL (m AHD) | Layer thickness (m) | Unit weight (kN/m3) | Effective friction angle, ϕ’ | Effective cohesion, c’ (kPa) | Young’s modulus, E’ (MPa) |
|---|---|---|---|---|---|---|
| 1 - Fill | 2.14 | 1.30 | 19 | 34 | - | 15 |
| 2b – Holocene Marine sand | 0.84 | 1.44 | 19 | 37 | 0 | 35 |
| 2c – Holocene Estuarine Clay | -0.6 | 4.7 | 17 | 26 | 0 | 5 |
| 3 – Holocene Marine sand (upper 1.5m) | -5.3 | 1.5 | 20 | 38 | 0 | 40 |
| 3 – Holocene Marine sand (below upper 1.5m) | -6.8 | 7.1 | 21 | 42 | 0 | 100 |
| 4 – Lower Holocene Clay | -13.9 | 32.1 | 20 | 28 | 6 | 30 |

Figure 2 shows the model geometry and the adopted finite element mesh. Table 2 summarises the adopted stages during numerical modelling.

***Table 2 Adopted staging for the numerical model***

| Stage | Remarks |
|---|---|
| Stage 1 - Initial stage | Develop initial stresses of the ground. Vertical to horizontal stress ratio of one was adopted for soil types except the upper three layers where ‘at rest’ lateral earth pressure ($K_o$) was adopted.<br>Reset displacements to zero at the end of initial stage. |
| Stage 2 to 6 – Apply vertical displacement of the rockhead | The full rock head movement of 0.5 m was simulated using 0.1 m movement advancement at each stage. |

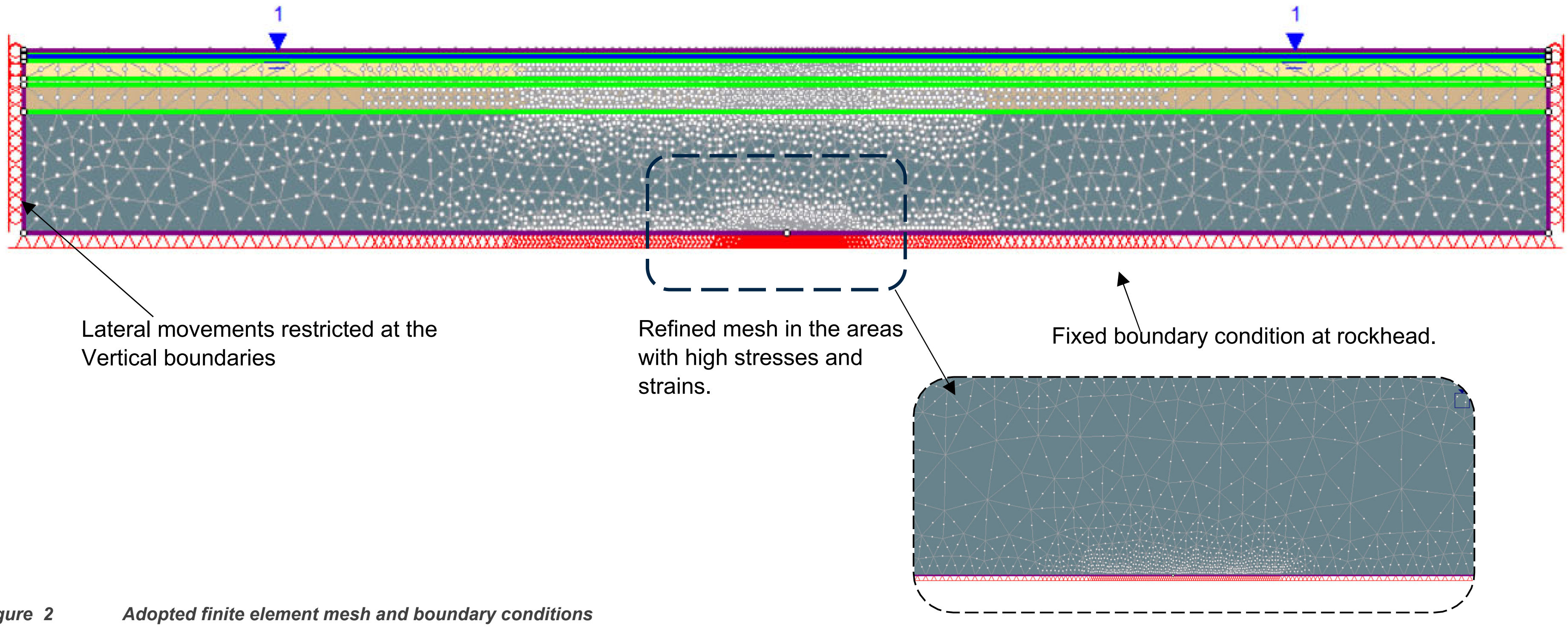


***Figure 2*** ***Adopted finite element mesh and boundary conditions***

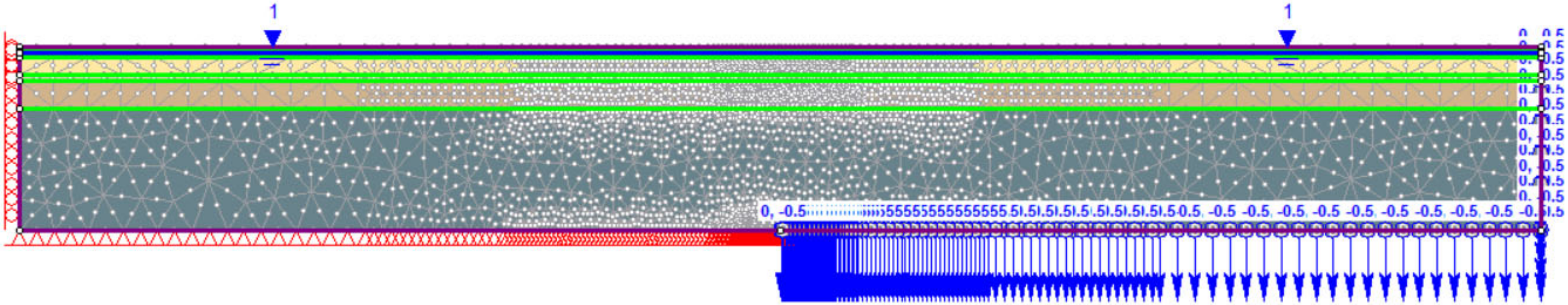


***Figure 3*** ***Simulation of rockhead movement due to mine roof convergence on the east side of the fault***

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Newcastle, New South Wales 2300
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# 5. Analysis results

The ground surface deformations were extracted from the model and the corresponding subsidence parameters (tilt, strain and curvature) calculated based on vertical and horizontal movements of the ground surface. The above subsidence parameters were calculated considering a base length of 5 m employing central differences. Figures 4 to 7 present the subsidence profile, tilt, curvature and strain for the four different magnitudes of rockhead displacement (Δ) of 0.2 m, 0.5 m, 0.8 and 1.0 m analysed.

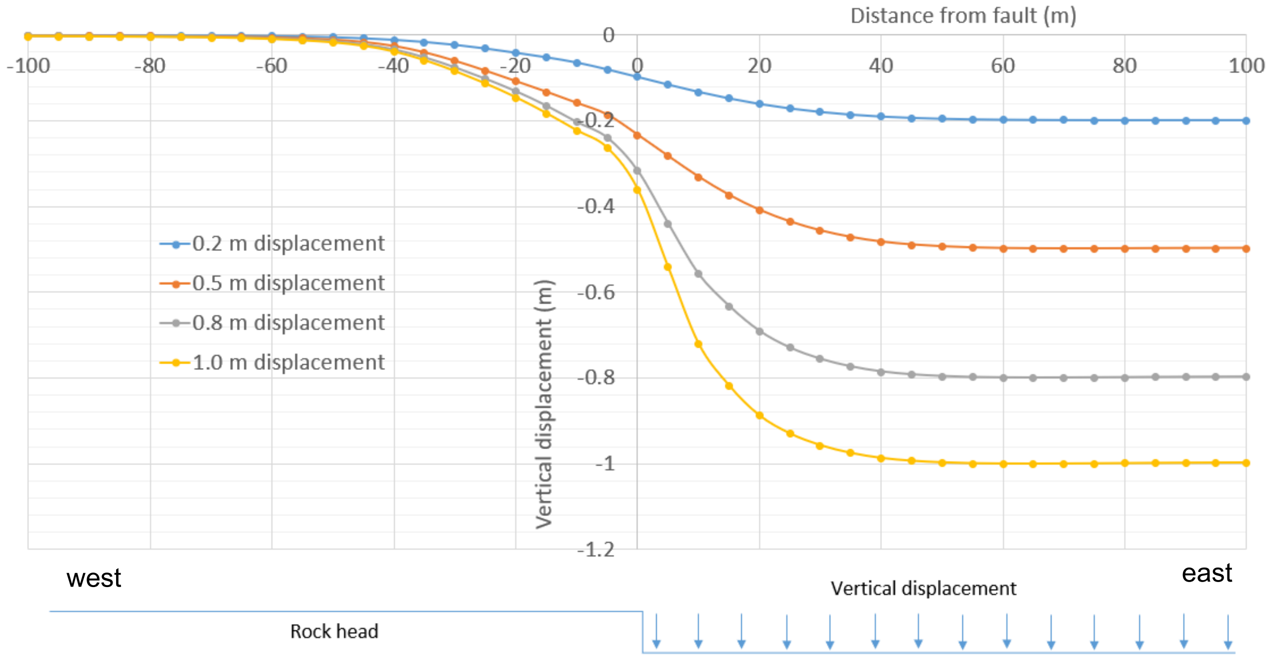


*Figure 4 Resultant surface subsidence profiles with Δ of 0.2 m, 0.5 m, 0.8 m and 1.0 m*

Figure 4 demonstrates that at 60 m horizontal distance from the modelled fault, the estimated subsidence is the full rockhead displacement which is intuitively correct for the condition of plane strain adopted. That is, the surface subsidence must eventually equal the magnitude of rockhead displacement where there is no volume change (dilation) in the soil.

Just west of the fault location there is a change in slope of the subsidence profiles which is more evident in the 0.5 m, 0.8 m and 1.0 m cases. Figure 5 and 6 present the central difference estimates of tilt and curvature based on a 5 m interval. About 10 m west of the fault the tilt is reflecting the change in slope of the displacement profile evident in Figure 4.

This change in slope is a result of a developing tensile zone in the model as discussed at the end of this section.

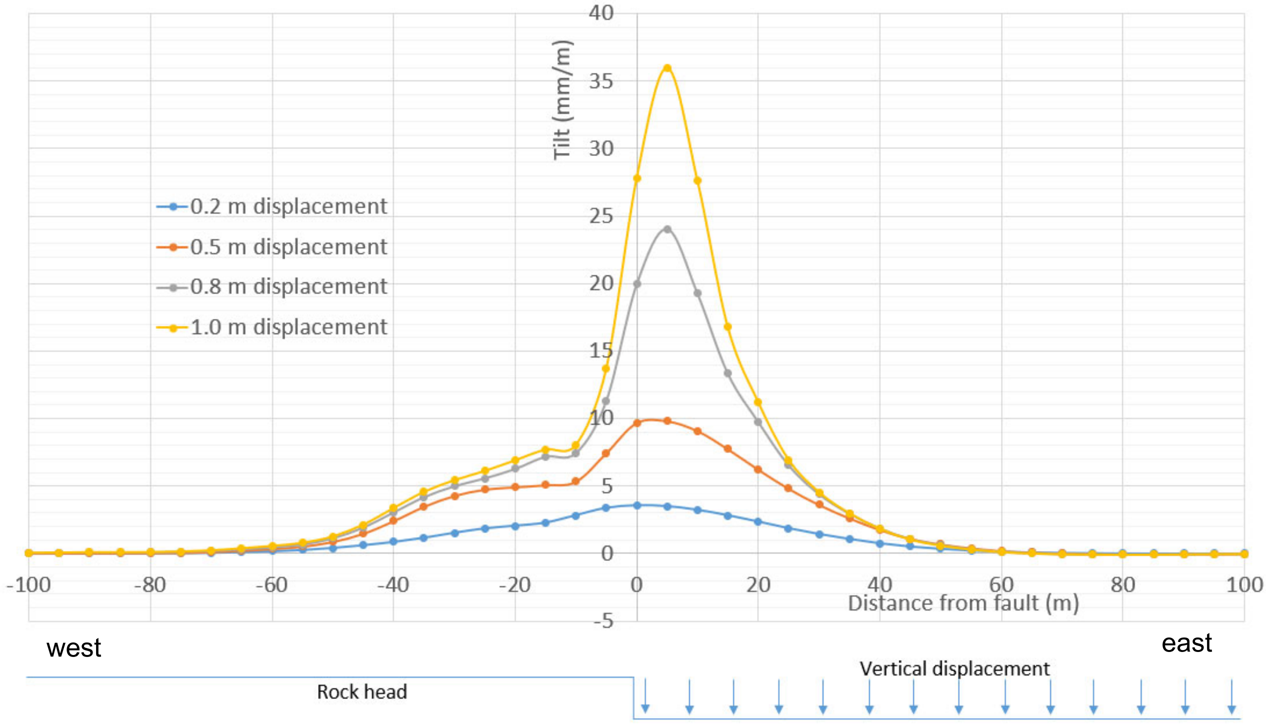


*Figure 5* *Resultant tilt profiles with Δ of 0.2 m, 0.5 m, 0.8 m and 1.0 m*

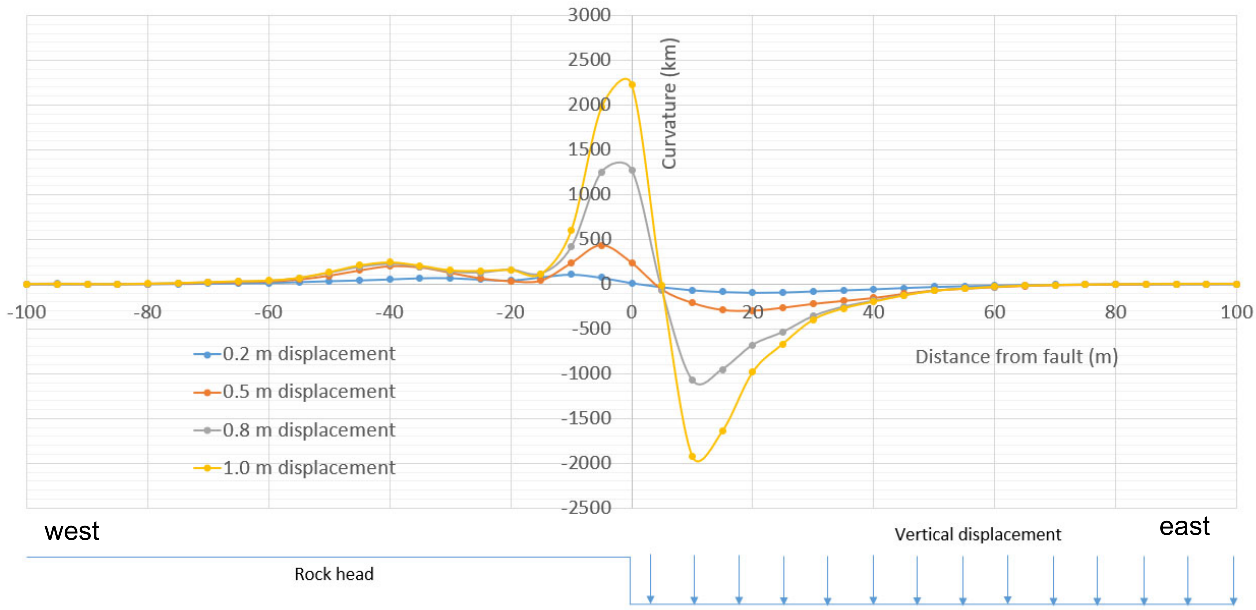


*Figure 6* *Resultant curvature profiles with Δ of 0.2 m, 0.5 m, 0.8 m and 1.0 m*

Figure 7 presents the estimated horizontal surface strain.

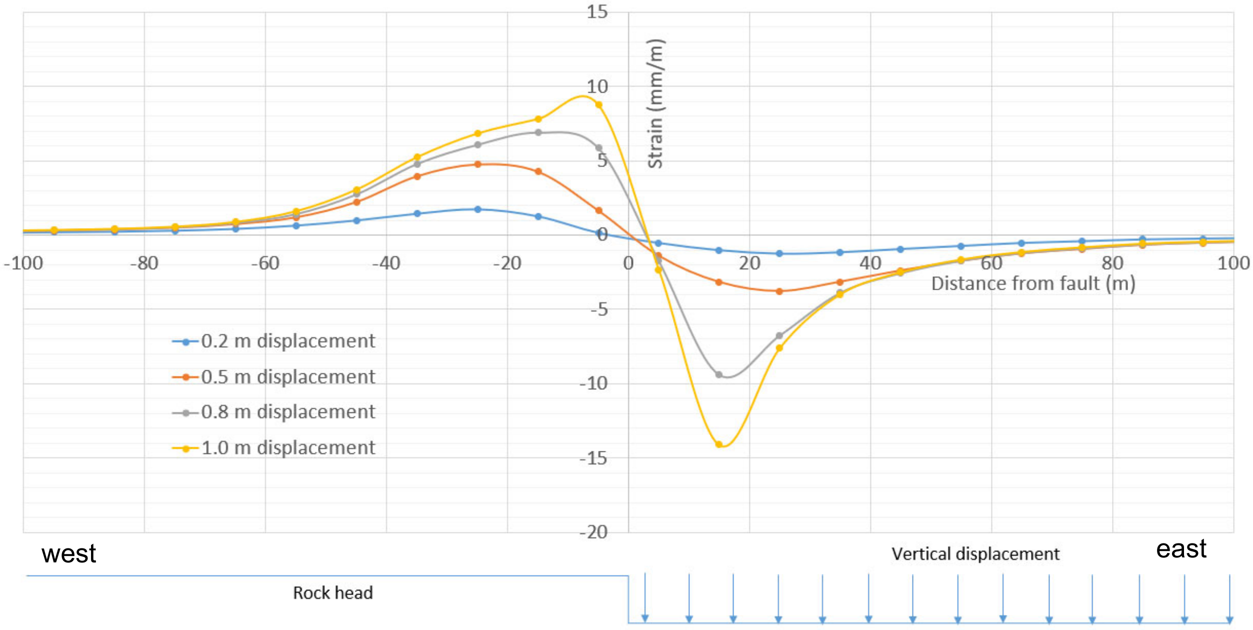


***Figure 7*** ***Resultant strain profiles with Δ of 0.2 m, 0.5 m, 0.8 m and 1.0 m***

Table 3 summarises the maximum subsidence effects for each analysis case west of the fault where the proposed building is located.

***Table 3*** ***Maximum estimated subsidence effects for each displacement magnitude Δ***

| Parameter | Rockhead movement (displacement along fault) | | | |
|---|---|---|---|---|
| | 0.2 m | 0.5 m | 0.8 m | 1.0 m |
| Maximum tensile strain E+ (mm/m) (Over 5 m bay length) | 1.75 | 4.8 | 6.9 | 8.75 |
| Maximum tilt T (mm/m) (Over 5 m bay length) | 3.6 | 9.8 | 24.1 | 36.0 |
| Minimum radius of curvature (on the west side of the fault) (km) | 9.2 | 2.3 | 0.8 | 0.4 |

For the 0.5 m case, the maximum values of tilt and radius of curvature are no less on the east side of the fault and so the parameters provided in Table 3 are applicable either side of the fault. That is, the location of the fault, should it be different to that modelled, would not change these maximum parameter values for tilt and curvature. East of the fault, the strain becomes compressive rather than tensile and has a maximum value of -3.8 mm/m.

Plastic points (crosses) and contours of shear strain are shown in Figure 8 (see over page) for the 0.5 m displacement case with a very exaggerated deformation scale to allow observation of the deformation. The circled area is a shear zone forming just outside the zone of imposed subsidence down to 14 m depth. This is in the zone adopting 'at rest' ($K_o$) initial stress. Material layering, (specifically contrasting stiffness), is also initiating a slip at 25 m depth and causing concentration of shear strain elsewhere at layer junctions in the model.

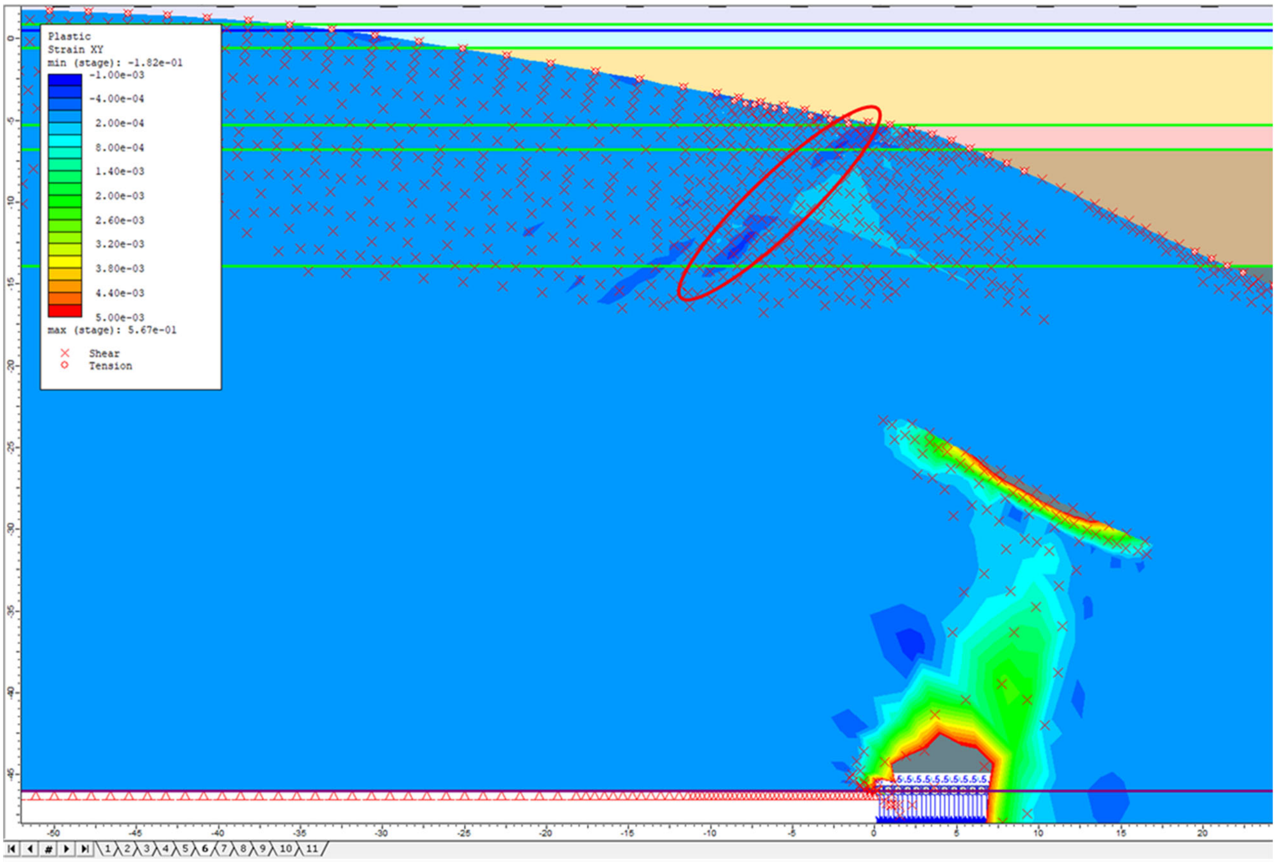


*Figure 8* ***Shear strain contours and plastic points with Δ of 0.5 m with highly exaggerated deformation scale***

The rapid change in surface profile slope (see Figure 4) and the kink in the tilt profile (see Figure 5) is attributed to the tensile zone created at the outside edge of the zone of subsidence imposed at rockhead.

Using elasto-plastic analysis based on Mohr-Coulomb parameters to estimate the soil behavior is considered a conservative approach. The actual soil behavior will be non-linear (rather than perfectly elastic – perfectly plastic) and is expected to result in a less concentrated shear zone where maximum strains and tilts would be less. Testing of soil samples would be required to adopt a more sophisticated non-linear soil model than the elasto-plastic Mohr-Coulomb model to assess this expectation.

# 6. Summary

Subsidence parameters for the proposed buildings have been revised based on the finite element analysis and are given in Table 3. The following are recommended to be adopted in design of the proposed building:

For the credible worst-case, the parameters given for the 0.5 m case being:

– Maximum tensile strain E+ of 4.8 mm/m
– Maximum compressive strain E- of 3.8 mm/m
– Maximum tilt T of 9.8 mm/m
– Minimum radius of curvature of 2.3 km.

For the absolute worst-case, the parameters given for the 1.0 m case being:

– Maximum tensile strain E+ of 8.8 mm/m
– Maximum tilt T of 36 mm/m
– Minimum radius of curvature of 0.4 km.

A compressive strain design parameter is given for the credible worst-case scenario to allow for the possibility that the 0.5 m stepped subsidence occurs anywhere beneath the site and in any direction. For example, the rock west of the fault drops relative to the east side the fault.

A compressive strain design parameter is not given for the absolute worst-case scenario as the scenario that the west side of the fault drops relative to the east side of the fault by any more than 0.5 m is not considered credible, even as an absolute worst-case.

Please see the attached letter from the Structural Design Engineers (Northrop) confirming the structure can be designed and built to accommodate these parameters.

# 7. SA NSW Merit Assessment Policy and approval conditions

For non-residential development such as that proposed, SA NSW assessment will be based on their Development Application – Merit Assessment Policy (SA NSW. 2018). The proposed building is understood to be classified as 'B3' as defined in the Policy and, as presented in the April 2021 GHD report, the site Uncertainty Factor has been assessed as 12 (i.e. High Uncertainty).

Table C3 of the Policy sets out SA NSW's "Estimated Conditions of Approval for Trough Subsidence Risk". Different conditions are given depending on whether the assessed pillar (panel) factor of safety (FoS) and pillar width to height ratio is less than or greater than nominated criteria. The conditions are based on assessed pillar factors of safety (FoS) and the assumption that the pillars have not yet failed but may do so in the future. However, as the investigation has concluded that the pillars have already failed and subsidence occurred, Table C3 may not be directly applicable.

The following approval conditions, based on those from Table C3 are recommended with reference to the credible worst-case parameters presented in Section 6. These must be confirmed by SA NSW.

> *Structure must be designed to be "safe, serviceable and readily repairable" under the predicted credible worst-case subsidence impact parameters.*
>
> *Structure must be designed to be* remain "*safe and structurally adequate*" *under the predicted absolute worst-case subsidence impact parameters.*
>
> *Submit plans prior to construction with a letter from a qualified structural engineer that the improvement will remain "safe, serviceable and any damage from mine subsidence shall be limited to 'very slight' in accordance with AS2870 (Damage Classification), and readily repairable". The subsidence impact parameters should be clearly stated.*
>
> *Demonstrate that the improvement can be designed to remain "safe, serviceable and any damage from mine subsidence shall be limited to 'very slight' damage in accordance with AS2870 (Damage Classification), and readily repairable".*
>
> *Submit an "Engineering Impact Statement" prior to commencement of detailed design for acceptance by SANSW, which shall identify the:*
>
> > *a. Mine subsidence parameters used for the design.*
> >
> > *b. Main building elements and materials.*
> >
> > *c. Risk of damage due to mine subsidence*
> >
> > *d. Design measures proposed to control the risks*
> >
> > *e. Comment on the likely building damage in the event of mine subsidence and sensitivity of the design to greater levels of mine subsidence.*

*Submit detailed design drawings prior to commencement construction with the design measures proposed to control the mine subsidence risk clearly highlighted and the design subsidence parameters clearly marked on the plan.*

*A number of permanent survey marks to AHD will be required so that building movement can be monitored should mine subsidence occur. Survey marks need to be initially surveyed and all details are to be forwarded to Subsidence Advisory NSW.*

*Following construction, sign-off from qualified engineer that improvements have been constructed in accordance with plans submitted to SANSW and in accordance with all relevant codes and standards.*

Please don't hesitate to contact the undersigned should you have any questions.

Regards

Sam Mackenzie

**Sam Mackenzie**
Technical Director - Geotechnical Engineer

0455 865 377
sam.mackenzie@ghd.com

**Attachments:**

Letter from Structural Design Engineers (Northrop)

Level 1, 215 Pacific Highway, Charlestown NSW 2290
PO Box 180 Charlestown NSW 2290
02 4943 1777
newcastle@northrop.com.au
ABN 81 094 433 100

15 September 2021

NL202453

Port of Newcastle Operations
Andrew Stone
Level 4, 251 Wharf Road
Newcastle NSW 2300

Dear Andrew,

**Re: 46 Fitzroy & 65 Denison St, Carrington – Structural Commentary to Mine Subsidence Analysis**

Northrop Consulting Engineers have reviewed the mine subsidence investigation reports prepared by GHD for the above-mentioned site and we provide the following commentary regarding the structural design for the proposed building.

The proposed building is a four-storey office building. The structure is proposed to typically consist of a reinforced concrete frame of columns and walls with post-tensioned concrete slabs. Piled foundations are likely, and will be detailed accordingly for any ground strain conditions. The proposed building does not contain a basement.

Based on the mine subsidence analysis carried out by GHD, the maximum subsidence parameters proposed for the design of the building are:

*For the credible worst-case*:

- Maximum tensile strain = 4.8 mm/m
- Maximum compressive strain = 3.8mm/m
- Maximum tilt = 9.8 mm/m
- Minimum radius of curvature = 2.3 km.

*For the absolute worst-case*:

- Maximum tensile strain = 8.8 mm/m
- Maximum tilt = 36 mm/m
- Minimum radius of curvature = 0.4 km.

Based on subsidence structural design checks carried out to date on the proposed structure; we advise that the building can be detailed to accommodate the credible worst-case parameters and remain structurally safe, serviceable and repairable with any damage limited to “very slight” as defined in Table C1 of AS2870-2011. We would propose to address this in detail in our Engineering Impact Statement, however in summary the building would be designed and detailed to accommodate the impacts from mine subsidence in the credible worst case situation using principles of serviceability design of concrete structures and ground isolation detailing.

Further to this, we advise that the building will be designed to remain structurally adequate and safe when subjected to the mine subsidence parameters associated with the absolute worst-case parameters. We would propose to address this in detail in our Engineering Impact Statement,

| | | Date |
|---|---|---|
| Prepared by | MA | 15/09/2021 |
| Checked by | NP | 15/09/2021 |
| Admin | BM | 15/09/2021 |

however in summary the building will be designed to the material strength principles in the relevant Australian Standards to adequately withstand the induced forces on the structure in this event.

We trust this meets your requirements, however should you need anything further please contact the undersigned.

Yours sincerely,

**Matthew Allen**
Associate | Structural Engineering Manager
BE (Civil) MIEAust CPEng NER (Structural)

*This certificate is provided to you for your sole benefit and only for the purpose of the 46 Fitzroy & 65 Denison St, Carrington project. You may not provide this certificate to any third party without our prior written consent. A third party may not rely on this certificate unless otherwise agreed in writing us, or required by law. To the extent permitted by law, we disclaim and exclude all liability for any loss, damage, cost or expense suffered by any third party relating to or resulting from the unauthorised use or, or reliance on, any information contained in this certificate.*

| | | Date |
|---|---|---|
| Prepared by | MA | 15/09/2021 |
| Checked by | NP | 15/09/2021 |
| Admin | BM | 15/09/2021 |