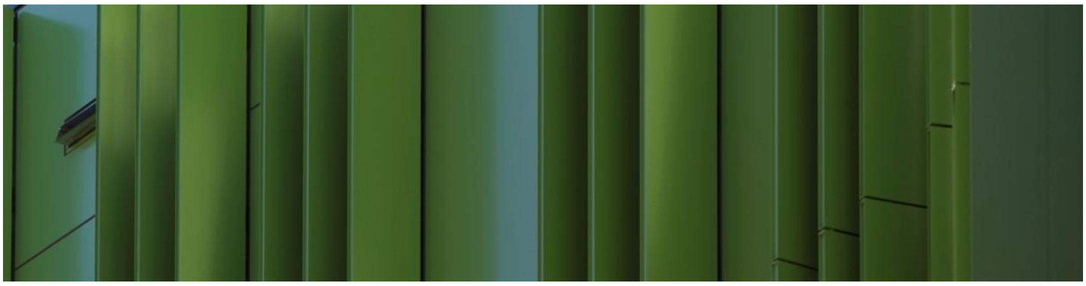
Structural



Port Kembla Copper Smelter Chimney

Military Road, Port Kembla

19 November 2013

131780

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1.0 MODELLING TECHNIQUE

The chimney was modeled based on existing drawings dated between 1963-1966 as well as demolition plans provided by Tait Condon Pty Ltd dated March – May 2013 using strand7 software. The duct penetrations in the inner and outer concrete shells of the structure were also modeled using dimensions provided in the available drawings.

The modeling involved applying wind loads to AS1170.2:2011 as well as earthquake loads to AS1170.4:2007. Due to the sensitive nature of the structure a natural frequency analysis was conducted which calculated the structure to have a first mode frequency of 0.25Hz. As such a dynamic wind and crosswind were calculated and applied.

For the earthquake analysis it was assumed the structure was founded on rock (Be soil class) with a corresponding torsion applied to the centre of mass of the structure.

A buckling analysis was also conducted to determine the structural suitability of various elements.

For the areas of the inner and outer tube which were weakened due to the drilling of holes for explosives the modulus of the materials used to model these elements was reduced to reflect the lower stiffness.

1.1 CONCRETE RESULTS

The main concern with the concrete elements of the structure was the concrete capacity under tension and compression of various load cases. It was found that the worst load case for compressive stresses was the 1.35G load case.

1.1.1 INNER TUBE

The inner tube had a maximum peak compressive concrete stresses of approximately 11MPa occurring at locations next to duct openings. Results are shown in Figure 1. When the load is averaged over a strip width of 1m either side of the opening the average stress reduces to 9MPa. Based on calculations using the information provided for concrete grade the allowable stress is 9MPa when detailed as wall and 15MPa when detailed as a column. No drawings detailing reinforcement at these locations are available however we expect there to be open ties and additional trimming bars as the openings are original to the building.

The drawings available show that the first two floors above ground level of the inner concrete tube (same height as the penetrations) being 2” thicker than the structure above which is consistent with the detailing of the outer tube.

A buckling analysis was run on the critical portions of the wall and the Euler load factor was determined as being 16, adjacent to the duct openings. Results are shown in Figure 2. The buckling load factor is sufficiently high for us to believe that the wall does not have a buckling problem.

The inner tube did not experience axial tension under any of the load cases. It did however experience local tension due to tie forces above duct openings. It was assumed that the horizontal steel has been detailed to provide resistance to these forces at these locations as they are original to the building but there are no available drawings to confirm this.

1.1.2 OUTER TUBE

The outer tube had a maximum peak compressive concrete stress of approximately 8.0MPa which is less than the allowable calculated wall stress 8.2MPa. No further analysis was required. Results are shown in Figure 3.

The outer tube did not experience axial tension under any of the load cases. It did however experience local tension due to tie forces above duct openings. It was assumed that the horizontal steel has been detailed to provide resistance to these forces at these locations as they are original to the building but there are no available drawings to confirm this.

1.2 MASONRY RESULTS

The masonry was modelled as a lumped mass as it does not provide any structural capacity.

Based on information provided in the Golder Report dated October 2013 the first few courses of brick at each level were removed to allow access to underlying asbestos sealant. The brick was then propped using jacks, layout and procedure documented by Tate Condon.

According to the drawings provided by Tate Condon there are between 15 and 25 jacks on each floor used for propping. Each jack has a locating bolt into the concrete annulus on which it sits. On each of the floors 4 jacks have stabilising plates which are assumed to provide some lateral resistance to earthquake forces in two directions. Based on the drawings provided we have calculated that the jacks have adequate capacity to resist lateral loading of the brickwork on each floor. At the top of each brick cylinder we have calculated that the existing mortar has adequate shear capacity to resist lateral loading due to earthquake.

2.0 CONCLUSIONS

Based on the information provided in the drawings the calculated forces on the walls on the outer shell are at an acceptable level to Australian Standards. It is assumed that the reinforcement has been detailed to resist any local tensile forces in the original design.

Based on the information provided in the drawings the calculated compressive forces on the inner shell are at an acceptable level to Australian Standards. It is assumed that the reinforcement has been detailed to resist any local tensile forces in the original design.

Based on the information provided the masonry does not provide any structural load bearing support to the structure. Current demolition works mean that at each floor the masonry is propped using proprietary jacks. The proposed jacks have adequate capacity for lateral loading on the masonry due to earthquake.

In conclusion we believe that the overall stability of the structure is adequate in complying with Australian Standards until demolition of the structure occurs, expected to be by April 2014.

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Richard Green
Director

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Project:	
Author: MAF	Reference: 131780

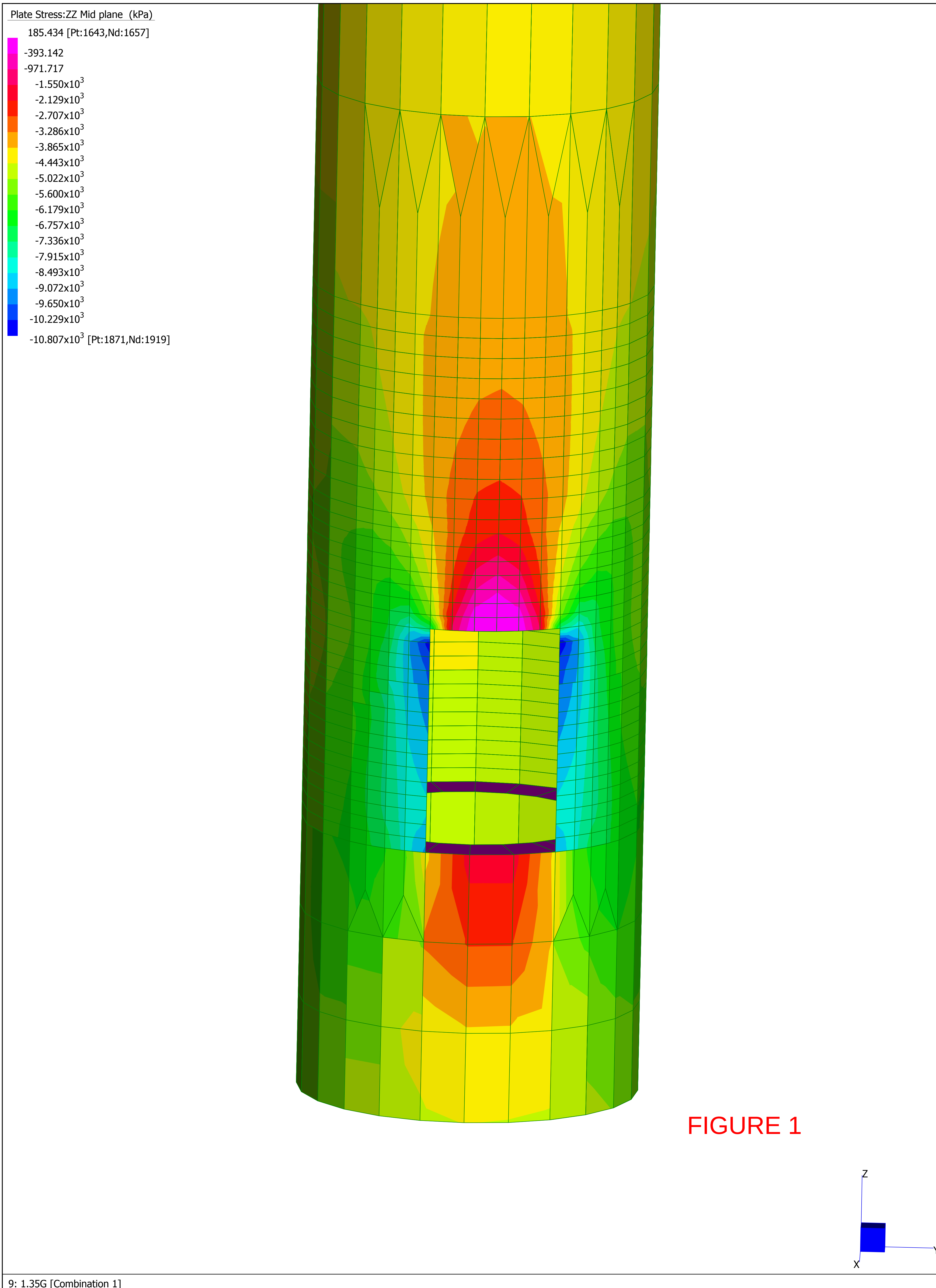

FIGURE 1

9: 1.35G [Combination 1]
1: Pinned Base
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Result file: C:\Strand Models\Port Kembla Chimney\131115 Model refined door mesh.LSA
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Title: Port Kembla Smelter Chimney	
Project:	
Author: **MAF**	Reference: **131780**

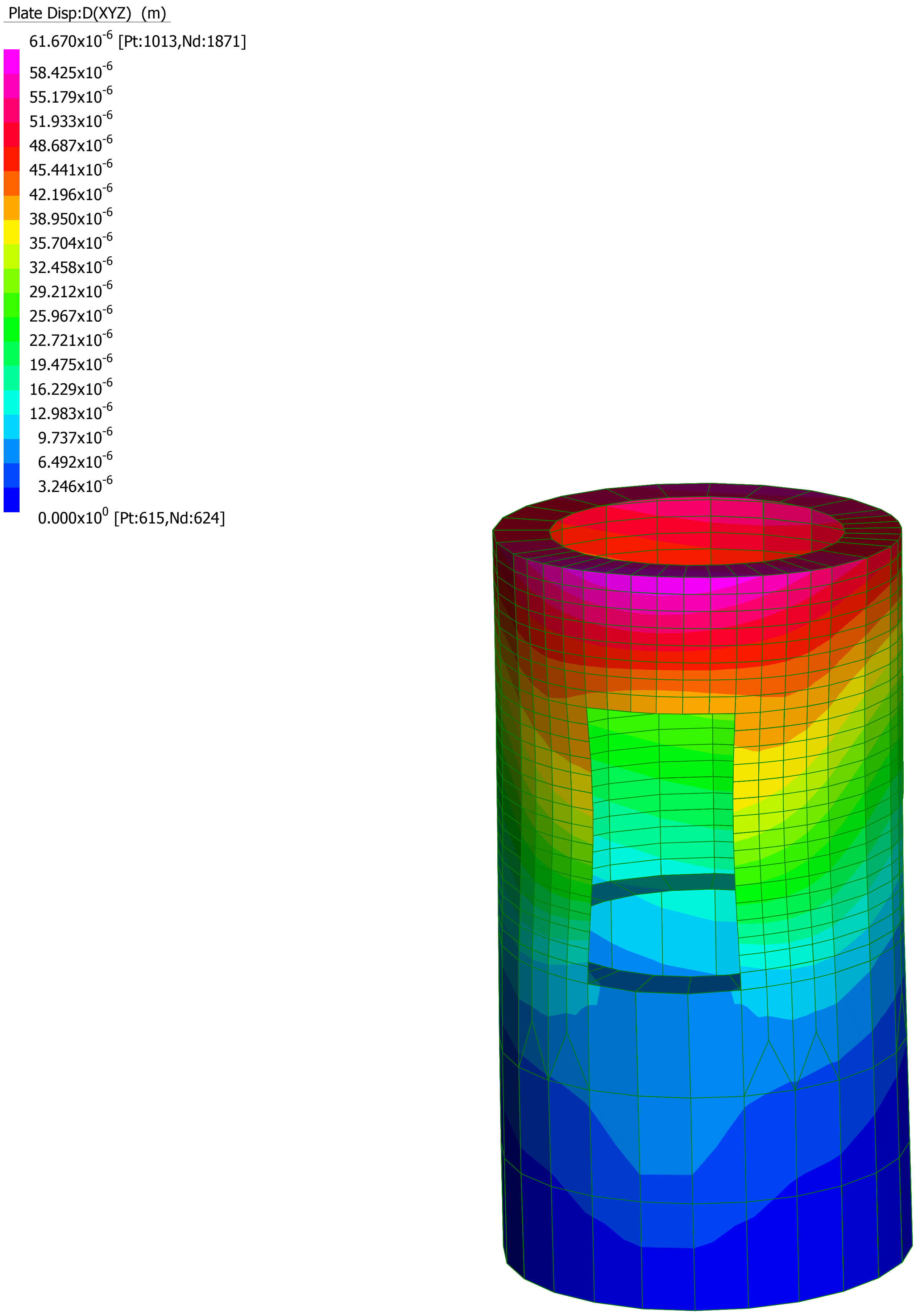
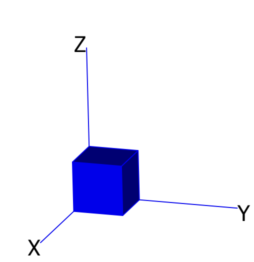


FIGURE 2



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Author: MAF	Reference: 131780

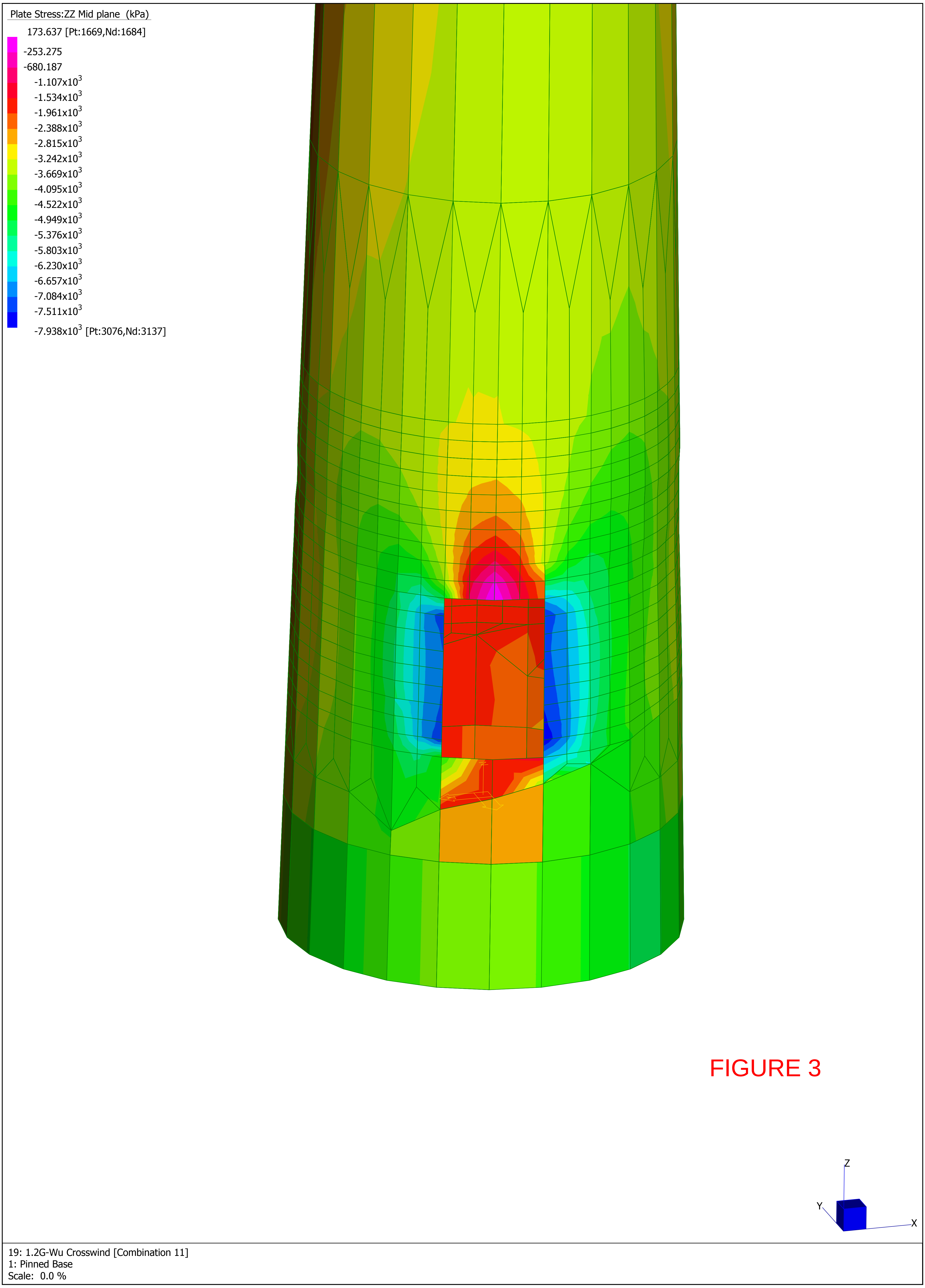

FIGURE 3

19: 1.2G-Wu Crosswind [Combination 11]
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