Geotechnical Design of the Rapahoe Bridge No. 1 North Abutment

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Tranz Rail Ltd proposes to build a replacement rail bridge over the Grey River at Greymouth on the West Coast of New Zealand. The north end of the proposed bridge lands at a skew to the existing cut rail line bench. Consequently, a pile is required to support the north abutment and retaining walls are required, perpendicular to the railway alignment at the abutment and along the outer edge of the railway formation north of the abutment, to extend the railway bench on fill. The preferred option for the retaining walls consists of precast panels founded on a reinforced concrete beam cast against and anchored to the rock face. The post tensioning of the anchors and friction between the uneven rock face and the concrete beam provides vertical support.

This paper presents details of the geotechnical design of the north abutment including the investigations undertaken, the ground conditions encountered, selection of the abutment location, options considered, selection of design parameters, load cases considered and detailed design. The design included finite element analysis of the retaining wall.

INTRODUCTION

Tranz Rail Ltd proposes to build a replacement rail bridge over the Grey River at Greymouth (Figure 1). This will provide a reliable, high quality link between the Rapahoe Branch Line and the Midland Line. The Rapahoe Branch Line is principally used to collect coal from the coal mines in the Rapahoe area. The Midland line takes the coal across the South Island to the deep water port at Lyttleton (Christchurch). The existing rail bridge was built in 1889 and has, in recent years, had extremely high and increasing maintenance requirements, as well as severe operating restrictions due to its deteriorating condition.

FIGURE 1 - LOCATION PLAN

The stage is quickly being reached when it will no longer be viable to maintain the existing bridge in a state to enable its safe continuing use. Tranz Rail Ltd has therefore undertaken geotechnical investigations and design relating to construction of a new bridge, with a target date for commissioning the replacement bridge of mid-2005.

PROPOSED REPLACEMENT BRIDGE

The proposed new bridge is approximately 285 metres long. On the northern side of the river, the new bridge would leave the alignment of the existing line approximately 50 metres upstream of the existing bridge. The new line would then swing in an arc over the river in a south-easterly direction to rejoin the Midland Line approximately 150 metres upstream of the existing bridge on its southern side.

The bridge is likely to consist of 1.8m diameter reinforced concrete, circular single stem piers/piles, supporting a reinforced concrete hammerhead cap beam. The piles are likely to be founded on rock sockets. The superstructure is to comprise a reinforced and post-tensioned concrete U-beam spanning, simply supported between the hammerhead cap beams.

The north end of the bridge consists of a link span between the Number 11 pier and north abutment. The link span lands on a skew to the existing cut rail line bench. The proposed new alignment of the rail line requires the existing cut bench to be widened over a length of approximately 20m.

FIGURE 2 - PROPSED ALIGNMENT

SITE INVESTIGATIONS

The site investigations for the northern abutment design consisted of:

- Geological mapping of the rock face along the existing rail line cut bench at the proposed northern abutment location;
- Inspection by divers of the river bed below the proposed north abutment location; and
- Drilling of two boreholes from access platforms constructed below the existing rail line cut bench.

GROUND CONDITIONS

The observed ground conditions are summarised in Section on Figure 3. Photograph 1 show the rock face at the north abutment location. The riverbank and existing railway bench comprise exposed limestone rock with local areas of fill over-spill. Railway ballast blankets the railway bench.

The rock batters above and below the existing railway platform consist of light grey unweathered sand textured limestone. It is moderately strong to strong with widely spaced persistent joints. It is massively bedded with some thinner silty marl beds dipping into the face at 20°. The sandy limestone is more resistant to weathering with bed thickness ranging from 0.5 to 2.0m, while the marl beds are typically 0.5m thick. There are no persistent rock defects along the bedding layers.

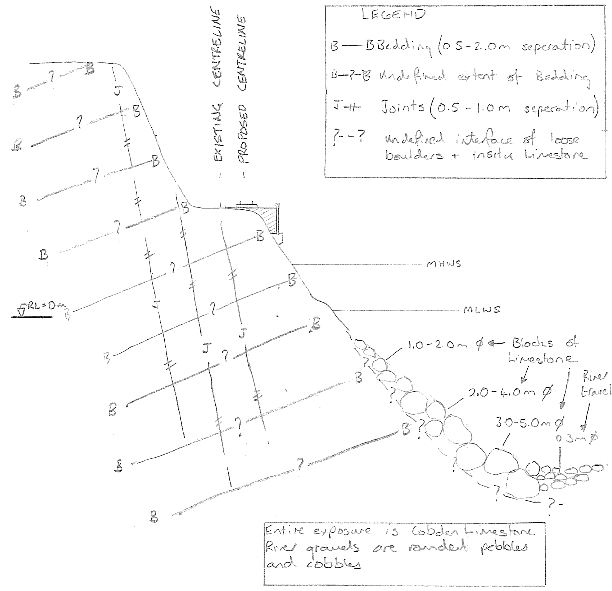

FIGURE 3 - GEOLOGICAL CROSS SECTION

Two major joint sets are present. The strike and dip of these joints sets are approximately 020/70°E and 290/80°N. The joints are very persistent but generally minor with limited soil and root infill. Near the water line the joints are more open and eroded into cavities in places. Joint spacings were closer and rock mass less competent southwest of the proposed abutment compared to the rock mass at the proposed abutment.

Cobden Cave is located 60m south-west of the proposed abutment. There is no evidence of similar extensive solution cavities in the direct vicinity of the proposed abutment. Solution erosion of joint surfaces in this area has resulted in joints being open (typically 50mm wide) extending 0.5m to 1.0m into the rock face. Solution erosion of a joint 5m southwest of the proposed abutment has resulted in a cavity of greater than 0.5m in width and extending 2m in from the general face of the rock surface.

PHOTOGRAPH 1 - NORTH ABUTMENT LOCATION

A joint controlled wedge failure of the rock face is evident near the proposed north abutment location. Adjacent to this the rock has dilated and relaxed and the rock batter is marginally stable.

Divers reported that below water level, they observed a batter slope at 35^o to 45^o extending to the riverbed at approximately 10m below mean sea level. This batter slope comprised boulders of rock ranging from 1m to 5m in size. The riverbed was observed at the toe of the slope with bed material comprising 300mm size rock cobbles. No rock exposure was reported. It is inferred that the boulders are overspill from the excavation of the road and railway benches and rock fall from the slope.

DESIGN LOADING

Load combinations considered as part of the ultimate limit state design included:

1.8EP + 1.8LL
1.35EP + 1.0EQ

where: EP=Earth Pressure
LL=Live Load
EQ=Earthquake Effects

The super imposed dead load for railway tracks, sleepers and ballast was 8.1kPa. This is equivalent to 450mm depth of ballast at $18kN/m^3$, which allows for overtopping of ballast.

The Tranz Rail Design Brief alternative axle case gave the highest live load of 230kN on axles spaced at 4.6m centres.

The seismic coefficient of 0.42g used in design was calculated in accordance with NZS 4203:1992 and the Transit New Zealand Bridge Manual.

DESIGN OPTIONS

The preliminary design was developed in conjunction with the Structural Engineer for the bridge superstructure. Assessment of the original location for the proposed abutment revealed that a retaining wall of greater than 12m height and extending below water level and through the rock boulder riverbank slope would be required. This was considered impractical. Also, at the location of the proposed abutment the geological mapping had identified that the rock slope was marginally stable.

Consequently, the abutment location was relocated north reducing the height of retaining wall required and avoiding the less competent rock identified. Pier 11 remained at its originally proposed location and the length of the link span between Piers 11 and north abutment was increased from 10m to 16m to accommodate the new abutment location. The abutment location selected required a maximum retained height of 5.0m. This retained height reduces to 1.5m over a distance of 10m to the north.

It is proposed that a ground beam cast onto the existing rail line cut bench supports the north end of the link span. A single pile will support the downslope end of the ground beam. The ground beam will resist lateral loads from the bridge superstructure through friction. Vertical post tensioned ground anchors are proposed to increase the frictional resistance between the ground beam and the underlying rock.

The proposed new alignment of the rail line requires the existing cut bench to be widened by nearly 2m at the proposed abutment location and tapering to no widening over a length of approximately 20m. To support the widening two retaining walls are proposed; one perpendicular to the rail tracks at the abutment location, and a second on the outer edge of the railway formation north of the abutment.

The following options for the construction of the abutment walls were been considered:

- Cast insitu concrete
- Sprayed concrete
- Precast concrete

The precast concrete option on a cast in-situ concrete ground beam was selected. The retaining wall is to be founded on a reinforced concrete beam cast against and anchored to the rock face. The post tensioning of the anchors and friction between the uneven rock face and the concrete beam provides vertical support. The retaining wall will comprise precast concrete slabs keyed into the in-situ concrete beam and anchored back. Granular backfill and subsoil drainage will be placed behind the behind the wall.

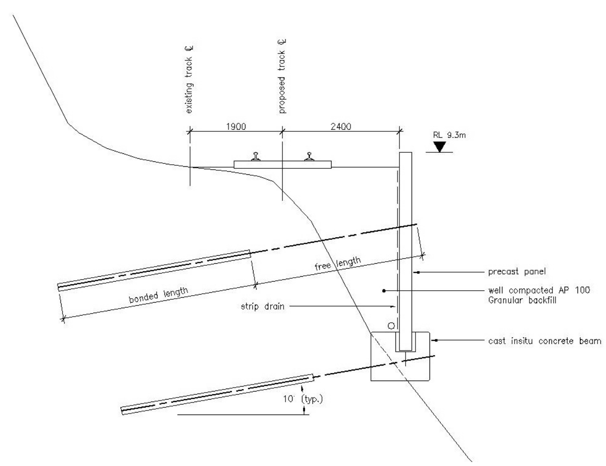

FIGURE 4 - PROPOSED WALL CROSS SECTION

Ultimate grout to country bond (τ_u) used for the design of the anchors was 800kPa. A strength reduction factor of 0.5 was applied for ultimate limit state design and anchor testing is to be undertaken during construction. An anchor free length of 2m beyond the rock surface has been provided to ensure that the bond length is beyond the open joints in the rock. The possibility of solution cavities within the rock cannot be discounted. Anchors would need to be extended through cavities if they are encountered. The anchors have been detailed with double corrosion protection consistent with BS 8081:1989.

WALL DESIGN

The wall was designed using limit state design principles. Limit state design requires that the design resistance effects (R*) be greater than the design action effects (S*)[1]. For this design the trial wedge method was used to assess the stability of the wall. The design actions were determined by assessing the forces acting on each wedge of soil. The weight of soil and live load were increased by the appropriate load factors. The design action is the resultant shear force on the base of the soil wedge (S).

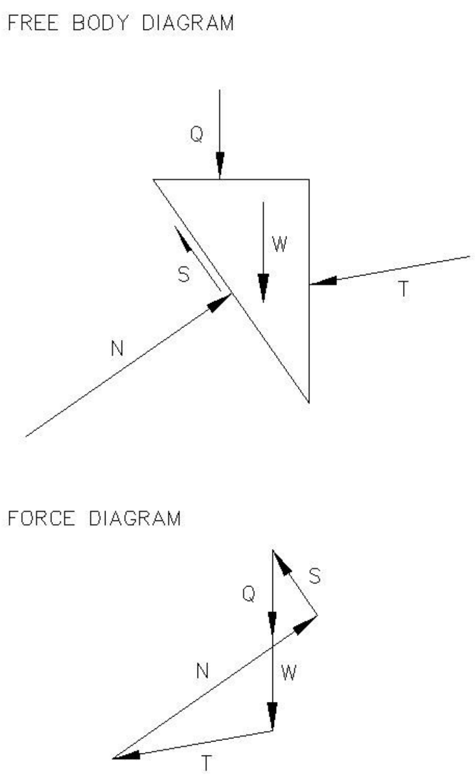

FIGURE 4 – Trail Wedge Analysis

The design resistance was assessed as the available friction at the base of the wedge (R) multiplied by the strength reduction factor (Φ). This was assessed to be:

$$\Phi R = \Phi N \tan f'$$

where
N = normal force on the base of the soil wedge
f' = internal friction angle

The minimum tension (T) required in the anchors was determined so that the R was greater than S. The minimum tension required was found to be 310kN/m. Figure 4 below summarises this analysis.

In the analysis ϕ'=35° was assumed being the angle of friction between the backfill and rock. The actual resistance along this base will be significantly greater this value as at the base of the wall the friction is between cast in situ concrete and rock.

The prestress force for the upper anchor was determined by calculating the "at rest" earth pressures on the precast panel and was found to be 80kN/m. The remainder of the 310kN/m anchor load required for stability of the wall will be applied to the lower anchor.

The proposed retaining wall also analysed using the finite element software PLAXIS Version 7.2. PLAXIS is a finite element code developed specifically for the analysis of soils and rocks. The PLAXIS analysis considered the following:

1. Initial Conditions
2. Excavation and construction of foundation beam, placement of wall panels, backfilling and prestressing of anchors
3. Live Load Case
4. Seismic Load Case

The PLAXIS analysis found a factor of safety greater than 2.4 for the live load case and 1.25 for the seismic load case. From the PLAXIS model the structural design actions for the precast panels were calculated for the two design load cases.

ROCK FACE STABILITY

Assessment of the major rock joints and bedded using stereographic projection indicated that stability of the rock face at the location of the proposed abutment should not be a significant issue as:

- The bedding dips into the rock face.
- The major joint sets are steeper than the rock face so that wedge failures are generally not kinematically possible.
- The rock anchors used to support the wall will also stabilise any potential wedge failures.

A joint controlled wedge failure of the rock face is evident on the slope above the proposed pier 11. The observed joint orientation at this location was different than that observed at the location of the north abutment. Assessment of these joints orientations indicated that a wedge failure was possible. Adjacent to this the observed failure the rock has dilated and relaxed and the rock batter is marginally stable. This loose rock will be scaled from the slope during construction. Detailed mapping of the excavated rock face is proposed for during construction. If the mapping identifies additional potential wedge failures then rock bolting will be undertake.

CONCLUSIONS

Tranz Rail Ltd proposes to build a replacement rail bridge over the Grey River at Greymouth on the West Coast of New Zealand. Tranz Rail Ltd has undertaken geotechnical investigations and design relating to construction of a new bridge, with a target date for commissioning the replacement bridge of mid-2005. The north end of the proposed bridge lands at a skew to the existing cut rail line bench. Consequently, a pile is required to support the north abutment and retaining walls are required, perpendicular to the railway alignment at the abutment and along the outer edge of the railway formation north of the abutment, to extend the railway bench on fill.

The site investigations for the northern abutment design consisted of geological mapping of the rock face, inspection by divers of the river bed and drilling of two boreholes. The rock batters above and below the existing railway platform consist of light grey, moderately strong to strong, unweathered sand textured limestone with widely spaced persistent joints.

The preferred option for the retaining walls consists of precast panels founded on a reinforced concrete beam cast against and anchored to the rock face. The post tensioning of the anchors and friction between the uneven rock face and the concrete beam provides vertical support.

The wall was designed using limit state design principles. An initial analysis was undertaken using the trial wedge method to assess the stability of the wall and to find anchor prestress loads. Finite element software was used to check the stability of the wall and calculate the design actions for the precast panels.

Assessment of the defects at the location of the proposed retaining wall indicates that stability of the rock face should not be a significant issue. A joint controlled wedge failure of the rock face is evident on the slope, south of the proposed abutment, above the proposed pier 11. Adjacent to this the rock has dilated and relaxed and the rock batter is marginally stable. Scaling and rock bolting will be undertaken to improve the stability of the rock face at this location.

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REFERENCES

1. Australian Standard AS4678-2002, "Earth-retaining Structures", February 2002.