Characterisation of reinforced granular transitions on freight railways for static and dynamic effects

Caractérisation des transitions granulaires renforcées pour les voies ferrées de fret sous effets statiques et dynamiques

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ABSTRACT The study demonstrates the significance of reinforced granular transition zones in controlling the settlement of heavy freight rail formations. The paper focuses on the behaviour characterisation of the transitions for static and dynamic effects through performance monitoring by conventional single-point (total station) and automated, real time (Pile Driving Monitors) surveillance systems, respectively. The track static stiffness was analysed in relation to the measurements of the permanent settlement of the ballast and the substructure (soil creep). The quantification of the vertical deformation under repeated freight movements provides a basis for the estimation of the track dynamic stiffness. The change in the track stiffness due to the presence of transitions was studied as part of this paper. The effects of the different transition configurations were also analysed.

RÉSUMÉ L'étude démontre l'importance des zones de transition granulaire renforcée dans le contrôle du tassement de formations de fret ferroviaire. Le document met l'accent sur la caractérisation du comportement des transitions pour les effets statiques et dynamiques grâce à respectivement un suivi classique de la performance en un seul point (station totale) et un suivit automatisé (systèmes de surveillance en temps réel, Moniteurs Pile de conduite). La rigidité statique de la voie ferrée a été analysée avec les mesures du tassement permanent du ballast et de la sous-structure (tassement du soil). La quantification de la déformation verticale sous les mouvements répétés de transports de fret fournit une base pour l'estimation de la raideur dynamique de la voie ferrée. La variation de la rigidité de la voie ferrée en fonction de la présence de transitions a été étudiée dans le cadre de ce document. Les effets de configurations de transition différentes ont également été analysés.

1 INTRODUCTION

A new 36 kilometre unelectrified bi-directional freight line was commissioned early 2013 in the southern metropolitan area of Sydney. The A$1 billion rail infrastructure was delivered as part of a much larger program of works to improve the efficiency and cost-effectiveness of rail freight services along the North-South Rail Corridor between Melbourne, Sydney and Brisbane. There was a major bottleneck in the rail freight network in southern Sydney where the freight trains share existing rail lines with the Sydney metropolitan passenger services. A third track (capable of supporting up to 48 train paths daily) was formed in the existing rail corridor for separating the freight services from the passenger train movements.

The rail project also involved station upgrades along with the construction of track support structures (in the form of track slabs, rail bridges, viaducts and culverts). The structures bear on piles socketed into rock, thus a much stiffer system in comparison to the immediate approach embankments.

The approaches are formed by compacted granular fill (up to 6.5 m high) overlying alluvium, residual soil and rock. Layers of soft to firm alluvium were evident. The depth to the underlying rock within the study area generally ranges from 10 m to 15 m below ground level.

Reinforced granular transitions were adopted to achieve better compatibility of deformation between sections of track with differing stiffness. Transition provisions were mainly at both sides of each structure. Base course material was used to form the transitions. Both high strength uniaxial geotextile (with a yield tensile strength of 200 kN/m) and biaxial geogrid (with a yield tensile strength of 20 kN/m) were selected as reinforcement.

2 TYPES OF TRANSITION

In general, the transitions have the shape of an inverted trapezium and are consisted of 20 mm nominal size densely graded aggregate base course (DGB20). The degree of compaction for the base course was at least 100 % of the Standard Proctor Maximum Dry Density. Governed by site physical constraints and specific design criteria, the transitions were formed in variable thicknesses with different types of basal reinforcement. Nonwoven geotextiles were placed to separate the base course from the adjacent rail formation.

The different types of transition include the following:

- Type 1 involves a partially reinforced granular transition, up to 3.3 m thick. Only the upper 1.25 m of the transition is reinforced by four layers of 200kN/m uniaxial high-strength woven polyester geotextile (Figure 1).
- Type 2 incorporates a granular transition fully reinforced by multiple layers of 20 kN/m biaxial geogrid (at 0.3 m vertical spacing). The transition extends up to 6.5 m below ground level. See Figure 2.
- Type 3 is a replication of Type 2 but with a reduced thickness of 3 m and 200 kN/m geotextile as reinforcement (Figure 3).
- Type 4 is a shallow transition, only 0.7 m thick (Figure 4). 200 kN/m geotextile was adopted to reinforce the transition.

Type 1, 3 and 4 transitions extend over a length of typically between 4.5 m and 6 m to the rear of the structures. Type 2 transition extends about 12 m beyond the bridge abutment.

The study involved the assessment of the track stiffness associated with Type 1 and 2 transitions under static and dynamic effects. The stiffness characterisation of the track support with Type 3 and 4 transitions was limited to static behaviour.

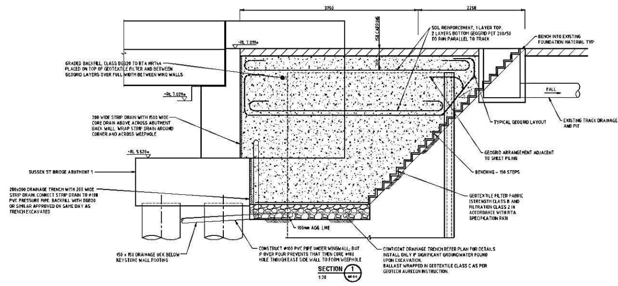

Figure 1. Partially reinforced transition using high-strength geotextile (Type 1).

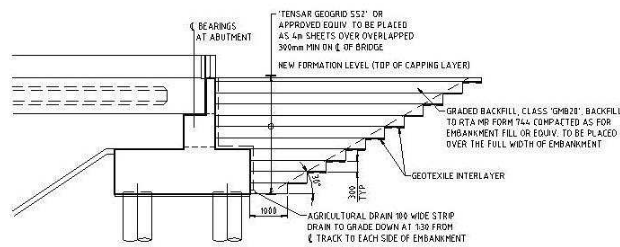

Figure 2. Fully reinforced transition using biaxial geogrid (Type 2).

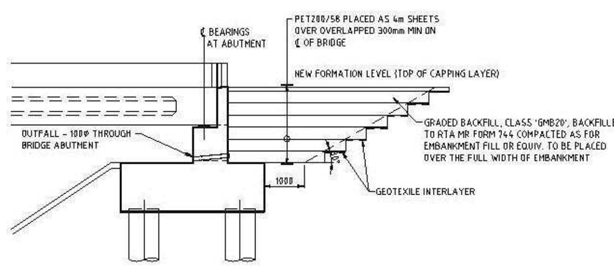

Figure 3. Fully reinforced transition using high strength geotextile (Type 3).

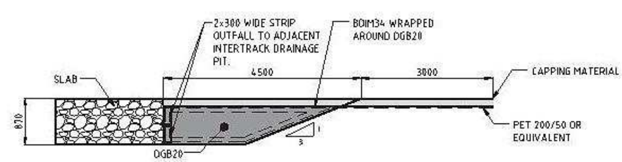

Figure 4. Shallow reinforced transition using high-strength geotextile (Type 4).

3 CONVENTIONAL SINGLE-POINT MOVEMENT MONITORING

To quantify the permanent vertical displacements of the railway, a conventional survey of reflective targets on the rail track using the total station was undertaken. The space between the survey points is typically between 2 m and 3 m. Readings were taken at daily intervals initially and were subsequently increased to weekly intervals until movements reach stabilisation. This type of movement monitoring was performed on all types of transition.

To differentiate the displacements contributed by the railway substructure beneath track ballast, settlement plates were installed in the four foot (between the rails) at the formation capping level and were then surveyed regularly. This type of monitoring was only applied to Type 2 transition.

The results from the particular monitoring were relied upon in the assessment of the static track support stiffness.

A selected graph illustrating the profiles of the permanent settlements within Type 1 transition is given in Figure 5. Similar movement trends were observed within Type 2, 3 and 4 transitions with the transition specific movement profiles highlighted in Table 1.

Table 1. Overview of conventional movement monitoring results.

Type of Transition	Target Location	Monitoring Results
1	Rail level	The rail embankment was constructed in September 2012 with no transition. The monitoring was only commenced in mid February 2013, thus only residual settlement (some 30 mm) was recorded. Site inspection revealed that the track has settled some 100 mm overall. The transition introduced in mid April 2013 was able to control the settlement rate (see Figure 5). The track has only settled up to 15 mm overall. Minor horizontal displacements were recorded. No significant increases in the recorded movements following rain events. The abrupt increase in track settlement rate immediately after the transition construction (as observed in mid April 2013) is associated with the compaction of ballast under train loading.
2	Rail level	The rail embankment was also constructed in September 2012 but with transition. Total vertical displacements of up to 20 mm were measured. Minor horizontal displacements were recorded. No significant increases in the recorded movements following rain events.
	Formation level	Settlement plates within the four foot recorded vertical movements of less than 10 mm.
3	Rail level	The rail embankment was constructed in September 2012 with no transition. No known track settlements were reported. The transition was introduced in mid April 2013. The measured total vertical displacements were generally in the range of 10 mm to 15 mm. Minor horizontal displacements. Movements do not respond to rain. No significant increases in the recorded movements following rain events.
4	Rail level	The rail embankment was constructed in September 2012 with no transition. No known track settlements were reported. The transition was introduced in mid April 2013. Total vertical displacements were measured to be in the range of 10 mm to 15 mm. Minor horizontal displacements. Movements do not respond to rain. No significant increases in the recorded movements following rain events.

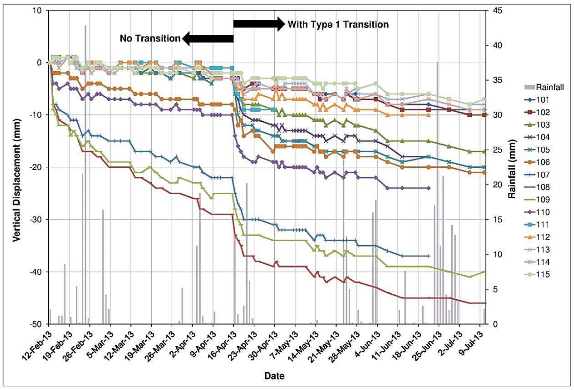

Figure 5. Permanent vertical displacements at rail level within Type 1 transition.

4 AUTOMATIC REAL TIME DYNAMIC MOVEMENT MONITORING

Dynamic testing techniques developed for pile driving have been adopted to monitor the track movements under the influence of freight rail rolling stock in real time (FSG 2013).

The dynamic monitoring was undertaken for different levels of train loading but was only within Type 1 and 2 transitions. Measurements were carried out at both track sleeper and substructure (formation) levels. The measurement results were used as the basis to estimate the dynamic track stiffness.

The monitoring results are summarised in Table 2 and have shown the following:

- Within Type 1 transition, the peak downward vertical deflections measured at sleeper level range from 0.88 mm to 4.26 mm, with the average values varying from 0.67 mm to 4 mm. At formation level, the measured peak and average vertical deflections are 0.2 mm and 0.85 mm, respectively.
- Within Type 2 transition, the measured peak downward vertical deflections at sleeper level range from 4.03 mm to 4.82 mm, with the measured average values varying from 3.61 mm to 4.09 mm. At formation level, the measured peak vertical deflections vary from 0.16 mm and 1.40 mm, with the measured average values ranging from 0.25 mm to 1.04 mm.
- The measured upward vertical deflections are typically less than 1 mm for both Type 1 and 2 transitions.
- The measurements from the Pile Driving Monitors have consistently returned to a ‘zero’ reading at the end of each event. The typical graph output from the dynamic movement monitoring is provided in Figure 6.

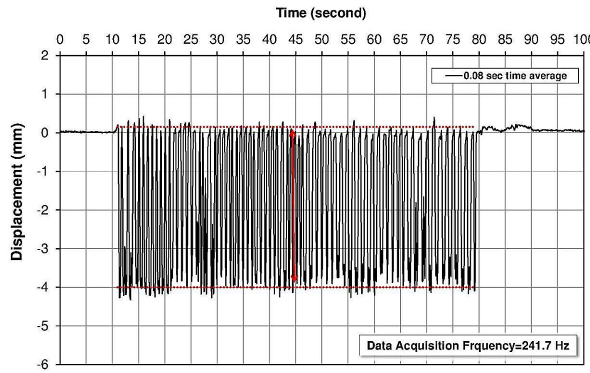

Figure 6. Typical graph output from dynamic monitoring.

Table 2. Dynamic movement monitoring results.

C1	C2	C3	C4	C5	C6	C7
10:20am 12/07/13	1,370	63.1	S	1	-4.26	-4.00
					-0.88	-0.67
11:49am 12/07/13	1,340	60.6	S		-1.35	-1.02
			F		-0.20	-0.85
1:19pm 11/07/13	1,380	32.6	F	2	-1.40	-1.04
			S		-4.03	-3.61
3:06pm 11/07/13	1,650	43.1	F		-0.16	-0.25
			S		-4.82	4.09

Note: C1 = Event; C2 = Engine mass in kN; C3 = Train speed in km/h; C4 = Monitor location; C5 = Type of transition; C6 = Peak vertical deflection in mm; C7 = Average vertical deflection in mm; S = Sleeper level and F = Formation level. Negative values indicate downward movements while positive values infer upward movements.

The experience based performance criteria indicated by the maintainer of the freight line are summarised in Table 3.

Table 3. Experience based performance criteria (Aurecon 2013).

Vertical Displacements (δ_v)	Track Performance Condition
$\delta_v \leq 4$ mm	New track
4 mm < $\delta_v \leq$ 6mm	Acceptable performance
δ_v > 6 mm	Poor track quality

The adopted performance criteria are consistent with those by Lundgren et al. in 1970 for durability. The study suggested the track deflection in the range between 3 mm and 6 mm for heavy track to give requite combination of flexibility and stiffness.

The dynamic movement monitoring suggests satisfactory performance for both Type 1 and 2 transitions. The monitoring has also inferred that Type 1 is relatively stiffer than Type 2 (apparent at track level). In reference to the performance criteria, the deflections measured at track sleeper level under dynamic train loading within Type 1 transition reflects the quality of a new track whilst Type 2 transition gives acceptable track performance.

5 BEHAVIOUR CHARACTERISATION

The fact that the measurements from the Pile Driving Monitors have consistently returned to a 'zero' reading at the end of each event has suggested the elastic behaviour of the track under cyclic (dynamic) loading. A linear relationship between the track dynamic stiffness and the corresponding strain is illustrated in Figure 7. The following observations were made:

- The vertical deflections measured correspond to small to medium strains.
- Significantly stiffer reaction was identified at the track substructure level in comparison to that at the track sleeper level, likely due to the contributions from the different track components above the substructure which include the track steel rails, concrete sleepers and ballast.
- At the track sleeper level, Type 1 transition is generally stiffer than Type 2 transition. No apparent differences in the resultant stiffness between Type 1 and 2 transitions at the track substructure level were identified.

As illustrated in Figure 8, the loading and unloading stiffness exhibited different support behaviours at the track sleeper and substructure levels. The ratio between the loading and unloading stiffness at the track substructure level was found to be generally greater than unity. The ratio was less than 1 at the track sleeper level. The particular phenomenon stresses the significance of the contribution from the track components above the substructure to the track vertical movements.

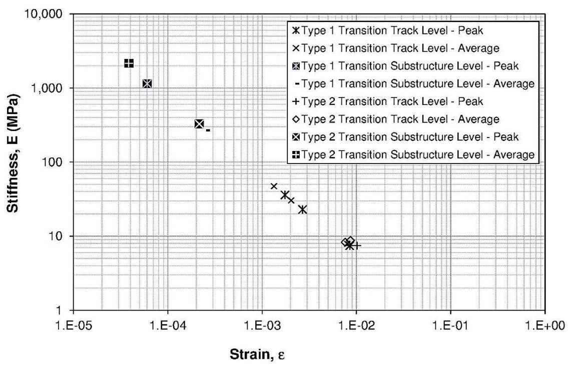

Figure 7. Stiffness as a function of strain.

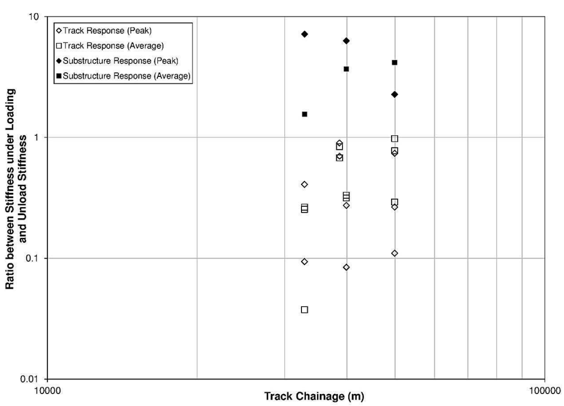

Figure 8. Dynamic stiffness as a function of load-unload cycles.

The graph in Figure 9 highlights the significance of the transitions. With no transitions, the behaviours of the track support were generally consistent with the past studies (Alpan 1970, also Benz & Vermeer 2006). Typically the ratio between the dynamic and static stiffness for the study areas with no transitions ranges from 4 to 10. The transitions resulted in significant increases in dynamic stiffness with the ratio between the dynamic and static stiffness varying from 20 to 70. The graph also demonstrated the stiffness at the track substructure level analysed based on the conventional survey to be within design limit.

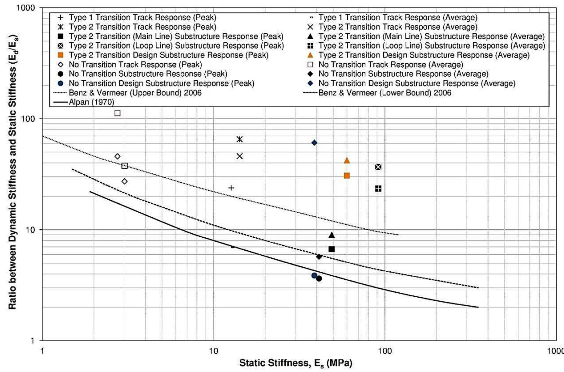

Figure 9. Correlation between stiffness ratio E_d/E_s and static stiffness (E_s).

6 CONCLUSIONS AND RECOMMENDATIONS

The study demonstrated the significance of transition zones in controlling the abrupt changes in track vertical stiffness within approaches to structures. Significantly reduced total settlements, limited to the order of 20 mm, were measured for transitions up to 6.5 m thick under site specific conditions. The static stiffness corresponding to settlements of such magnitude reflects a dense track support. A 3.3 m thick approach embankment formed by clays with no reinforcements is known to be associated with a total settlement of the order of 100 mm.

The hypothesis that in short-term the rail track under train cyclic loading behaves elastically is supported through the observations of the dynamic monitoring. The contribution from the compression of foundation stratum under the self-weight of the rail embankment to the permanent settlement of the track is considered to be greater than that from the train loading. The dynamic monitoring also demonstrated that in the short term the contribution from the track components to the track vertical displacements under train cyclic loading are greater than that from the track substructure.

The study highlighted the effectiveness of basal reinforcements within the influence zone of train loading. Type 1 transition (with high strength reinforcements mainly within the influence zone) has resulted in a stiffer response than Type 2 (with full reinforcements beyond the influence zone). A further study is recommended for gaining in-depth understanding on the performance of basal reinforcements beyond the train loading influence zone which can potentially lead to improvements in transition configuration design.

The study also indicated the correlation of transition configuration with structure vertical stiffness. Shallow transitions less than 1 m thick (Type 4) were introduced at both sides of a floating track slab while the transitions at both sides of a piled rail bridge were extended to the full height of the bridge abutment (Type 1 to 3). The different transition configurations have demonstrated satisfactory performance. The writer recommends that the behaviour of shallow transitions in particular be further studied.

The train speeds recorded at the time of monitoring range from about 33 km/h to 63 km/h, which are slower than the allowable limit. Faster train speeds are known to be associated with greater dynamic wheel loads. The dynamic monitoring in this study did not facilitate the analysis of the effects of differing train speeds. Further research is suggested for the establishment of track performance criteria under dynamic train loading in consideration of train speed.

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