SHALLOW GROUND IMPROVEMENT FOR A NEW RAIL DEPOT / MAINTENANCE FACILITY IN SYDNEY

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ABSTRACT

A new rail maintenance and stabling facility comprising rail sidings and associated maintenance buildings / facilities is to be constructed in a heavily urbanised area in Sydney, NSW. The site will be on a raised earthworks development platform for flood mitigation purposes.

Ground conditions include variable non-engineered surficial fill over up to 4m thickness of variable density alluvium (dune sands) overlying dense to very dense Botany Sands above deep alluvium to over 40m depth with a shallow groundwater level. Some site areas have been previously loaded by former development.

Identified geotechnical risk relates to potential unacceptable ground surface settlement of the development due to embankment loading of low density variable surface soils against the need to limit total and differential settlement of rail track-slab and building foundations within specified design and operational tolerances for a design life of 75 years.

A ground improvement solution was required to improve these surface soils to reduce the potential for both total and differential settlement under loading from the proposed earthworks and structures. The achievement of differential settlement criteria is critical to the long term performance of the track slab system, particularly at the interfaces with buildings.

This paper describes the site ground conditions, the ground improvement options considered and the selection of high energy impact compaction (HEIC) as the optimal solution for the proposed site development. HEIC trials including performance validation of the ground improvement are described. Site specific Specifications were then developed for HEIC construction phase works. The trials also included noise and vibration monitoring to define construction control limits and buffer zone distances from sensitive adjacent areas including residential properties and a heritage building.

Significant cost savings and construction programme benefits are realised utilising shallow ground improvement methods compared to the other ground improvement options considered.

1 INTRODUCTION

The site of the proposed new rail maintenance and stabling facility comprises a large area of land constrained by existing roads, and areas of residential and commercial development. Early on in the design process it was recognised that the near surface soil profile presented a risk of unacceptable settlement under loading from the proposed building and rail infrastructure with a lower risk of bearing capacity failure. Isolated areas of the site had been developed previously, primarily as a now disused road and also with isolated foundations for a footbridge and low rise buildings.

This paper presents the rationale for the selection of HEIC as the adopted ground improvement technique for the site, the design process including a field trial and some of the advantages over other potential ground improvement techniques and foundation solutions. HEIC was adopted due to its suitability for the site conditions, whilst providing advantages in savings of time and materials.

A site trial was carried out to assess the effectiveness of the HEIC technique in densifying the near surface soils and to determine the required number of passes of the HEIC plant in order to meet the design intent. An area was established for the trial that was representative of site conditions and the *in-situ* density of the soils measured after a set number of passes of the HEIC roller.

2 SITE CONDITIONS

2.1 SITE OVERVIEW

The proposed maintenance and stabling facilities are to occupy a site of 3.4ha located on a parcel of disused land within Sydney's urban area. The site surface is relatively level with a maximum elevation range of between 1-3.5m over a length of approximately 360m in a north-south direction. Existing site constraints include shallow buried services, particularly sewerage pipelines, and adjacent sensitive buildings including a heritage building and residential and light industrial development.

The existing site surface is to be generally raised by between 0.5m to 2.0m thickness by the construction of engineered bulk earthworks fill to form a raised level development platform for infrastructure development and flood protection purposes.

2.2 GEOLOGY, GROUND AND GROUNDWATER CONDITIONS

The geological conditions are typified by variable thickness fills over Quaternary sediments. Holocene aeolian dune sands overlie extensive and deeper transported alluvial soils of the Botany Basin sequence. The Holocene Dune sediments form the upper part of the Botany Basin sediments and mainly comprise loose sands representing localised aeolian dune deposits. Pleistocene Botany sands make up the majority of the Botany Basin sediments and comprise aeolian, medium grained quartz sand deposited in a dune environment with lenses of organic material, peat, silt and clay. Pleistocene basal sediments are generally found as dense sands and stiff fissured clays above the Triassic Hawkesbury sandstone bedrock at over 45m depth. The geology and geological history of the Botany basin area is usefully summarised by Hatley (2004).

The geotechnical model developed for the site is summarised below in Table 1 based upon information from relevant geotechnical investigation exploratory holes.

Table 1: Inferred site geotechnical conditions

Geotechnical Unit	*Level range (m, AHD)* (*1)		*Inferred thickness (m)*	*Typical description*
	Top	*Base*		
Unit 1 Fill	28.1 – 31.6	26.7 – 30.6	0.15 – 2.0	Fine to medium grained sand and sandy gravel with occasional silt, trace clay. Asphalt and road pavement materials or topsoil encountered at some locations
Unit 2b: Holocene Sediments - Dune Sand	26.7 – 30.6	24.4 – 27.9	1.2 – 4.3	Loose to medium dense fine to medium grained uniformly graded sands
Unit 3a: Pleistocene Sediments - Botany Sands	24.4 – 27.9	1.7 – 18.1	11.3 – 22.6	Dense to very dense fine to medium grained uniformly graded sands with an average thickness of 15m
Unit 3c: Pleistocene Basal Alluvium	12.4 – 18.1	-16 to -19	27 – 32	Dense to very dense sands, with bands of clayey sand and stiff to very stiff clay between approximately 1 and 4m thick
Unit 6a: Weathered Sandstone	-16 to-19	-	>10m	Extremely to highly weathered, very low strength sandstone

Note: (*1) AHD = metres Australian Height Datum.

SPTs indicated relatively lower densities in the upper zone (of most significance for shallow footings and pavements).

Hydrogeological modelling of the site and the Botany Sands groundwater aquifer based upon information from relevant exploratory holes indicated a hydraulic gradient across the site in an approximate NNE to SSW direction with maximum predicted groundwater levels of between RL 27.5m AHD and RL 26m AHD. Recommended groundwater level for design was 1.6 - 2.0m below ground level.

3 FOUNDATIONS AND CONSTRUCTION REQUIREMENTS

Rail sidings and approach roads and a number of buildings are proposed at the site with the maintenance building approximately 90m long and 25m wide. Due to the large plan areas of the proposed structures, they will induce stresses within the ground to considerable depth. The available geotechnical investigation information generally indicates that 2 to 3m thickness of dune sand was encountered across the entire site beneath Unit 1 fill (where present). The Unit 2b dune sand was noted to be typically loose to medium dense, with dynamic cone penetrometer (DCP) tests indicating that the relative density of this material is highly variable across the site. Some areas of the site have not been previously subjected to load associated with previous development, therefore it is considered that development of the site presents a potential geotechnical risk in terms of differential settlement.

At the project tender stage, it was considered that there was potential for deep layers of compressible material to be present at the site. The understanding of the ground conditions in the area at the time suggested that these materials could comprise clay and peat interbeds within Unit 3a (Botany Sands) and deeper Unit 3c (Pleistocene Alluvium). A review of the available geotechnical investigation information within the extent of the proposed site and surrounding areas suggests that extensive deposits of peat and highly compressible organic clay are not likely to be present in the site area. Subsequent geotechnical investigations have indicated that thin bands of stiff clay (typically less than 200mm thickness) are present at the interface between Unit 3a and Unit 3c, with thicker layers encountered at depth within Unit 3c. These materials are considered to be over-consolidated based on the geological history of the site and this was confirmed by oedometer testing. In consideration of this and based on the proposed additional loading due to bulk earthworks and from structures, any settlement within these layers is likely to be elastic and of secondary importance.

3.1 PREDICTED SETTLEMENT DUE TO EARTHWORKS

Bulk earthworks are proposed to raise the site levels by between 0.5m and 2m above current site ground levels. The importation and placement of bulk engineered earthworks fill will result in settlement of the site formation embankments primarily during construction.

One dimensional settlement calculations were carried out using available borehole and friction-cone penetrometer (CPT) data. At each location, the settlement prediction was based upon the proposed fill height at that location. Settlement predictions were based upon elastic modulus values derived from CPT cone resistance, qc. Calculations have also been undertaken using Standard Penetration Tests (SPTs) where equivalent CPT cone resistances have been calculated from the SPT 'N' values and based on mean grain size, D_{50}.

3.2 CONSTRUCTION TOLERANCES

Rail track formation is based on limiting rail deflections by controlling track foundation modulus and stiffness during design. Construction constraints are taken as the long term allowable settlement and deflection limits, which are in addition to construction tolerances and are 'post construction' limits. Differential track settlement and twist are of prime importance. Permissible geometric performance limits for rail track for geotechnical design at the site are taken as follows:

- Vertical (differential) settlement – 1:1000 over a 3m chord length;
- Horizontal alignment – 1:10,000 over a 5m chord length;
- Horizontal alignment – 1.5:10,000 over a 10m chord length;
- Twist – 1:1500 over a 3m chord length.

The target is to achieve the highest geometric quality possible during and after construction thereby limiting future maintenance requirements, Selig and Waters (2000).

Construction tolerances for buildings constructed on ground bearing raft foundations with thickened edge beams varied depending on building and plant requirements; a guidance value for differential deflections was taken as < 3.5% across the width of the foundation.

4 GROUND IMPROVEMENT

4.1 OPTIONS CONSIDERED

A ground improvement solution is therefore required to improve the surface soils at the site to reduce the potential for both total and differential settlement under loading from the proposed earthworks, structures and train loading. The achievement of differential settlement criteria is critical to the long term performance of the proposed rail track slab system, particularly at the interfaces with buildings. The following ground improvement options were considered during preliminary design and were ruled out for the reasons detailed below:

- Shallow surface compaction - traditional static or vibratory rollers: not suitable for compaction of site soils to required depth due to influence of less than half a metre to a metre, Hausmann (1994);
- Vibro-compaction or dynamic compaction: not suitable for improving soils at shallow depth;
- Excavation and re-compaction: not an economic solution for the large urban site area and not considered suitable for a site with shallow groundwater table/works below groundwater level; and
- Deep foundations - piles (screw or CFA): suitable for the site but considered uneconomic due to required quantities and anticipated construction duration.

4.2 PROPOSED SHALLOW GROUND IMPROVEMENT - HEIC

The feasibility of shallow ground improvement by high energy impact compaction (HEIC), which is also referred to as rolling dynamic compaction, was investigated following consideration of the site ground conditions, the proposed works and associated applied loadings and the published results of HEIC on other sites.

HEIC utilises a towed non-circular (4-sided “square” or 5-sided) drum rotating about one corner and falling to impact the ground with high impact forces causing compaction as shown in Figure 1. The average speed of the towed roller is 10km/hr and it can cover approximately 3500m^2/hour, Broons (2010). It is available as an 8 ton or 12 ton module.

Figure 1: Dynamic rolling module towed in a frame by a 4-wheel drive tractor and cross section of a four-sided HEIC module (Bierbaum *et al*., 2010)

Several previous researchers have assessed the effectiveness and performance of the HEIC impact roller. HEIC has been proven to densify granular soils up to approximately 2.5m below the surface (or greater for some sites), with slight improvement noticed below this depth in some cases (Hausmann, 1994; Bouazza and Avalle, 2006; Jaksa *et al.*, 2012). Research conducted by Bierbaum *et al*,. (2010) based upon the results of CPT testing and numerical modelling suggest that the influence zone of the impact roller is up to a depth of 3 to 3.5m for specific fill materials. This is significantly more than the typical depths of compaction influence of less than half a metre to a metre obtained by conventional static and vibratory rolling (Hausmann, 1994; Bouazza and Avalle, 2006; Bierbaum *et al*., 2010). Bierbaum *et al*. (op cit) noted that influence depths for impact rollers were dependent upon soil stress distribution as influenced by friction angle, cohesion and Poisson’s Ratio and cannot therefore be defined for all cases.

It has been demonstrated by Avalle and Carter (2005) that the strength of the ground can be improved below the water table using HEIC. In this case study, which was from a site with similar ground conditions to the proposed site, the most significant improvements in relative density were recorded below the water table. The depth of ground improvement was seen to reach at least 3m below the surface. The specification for impact rolling was to perform at least 20 passes in conjunction with settlement monitoring and geotechnical testing using the DCP and the CPT. In view of the proximity of existing nearby structures, a brick factory building within 3m of the southern boundary and residences about 15m to the north, vibration monitoring was also undertaken. CPT Cone tip resistance increased from around 5MPa to around 10MPa in this zone of improvement.

Scott and Suto (2007) reported that ground improvement using impact rolling proved successful in achieving the required 100kPa bearing capacity on a site covered with 1.5 to 2m of uncontrolled fill. The fill consisted of gravelly clay with some sand and building demolition debris with significant amounts of organic material over natural soils consisting of fine to medium grained sand. Groundwater was encountered across the site typically 1.5 metres below ground level near the natural ground surface.

Vibration effects due to HEIC plant have been assessed by Bouazza and Avalle (op cit) and Avalle (op cit) based upon variable distance vibration monitoring, from the impact. Their studies indicated that human perceptibility commences at about 0.1-0.5mm/s, but people can become annoyed by vibrations even lower than that, and many people can tolerate vibrations up to 5-10mm/s. At a distance of 10m from the impact module, peak particle vibration was typically <1mm/s based on 25 case study sites. A simple expression, $v = 100 / D$, is suggested for initial evaluation of the potential magnitude of ground vibrations resulting from the use of the 8t “square” impact roller (where v is the peak particle velocity in mm/s and D is the distance to the impact module in metres).

Consultation with a recognised HEIC industry expert was undertaken to obtain an understanding of local experience as relevant to the site (Avalle, *pers comm*, 2015) as presented below:

- Treatment by HEIC will result in the level of the treated area dropping due to the effect of soil densification with a typical treatment rate of 4 – 5000m^2 per day;
- The establishment of a controlled and monitored compaction trial area prior to construction will enable the determination of the number of passes required to achieve a nominated post treatment strength;
- Vibration monitoring undertaken during a compaction trial would enable the establishment of a relationship between vibration and distance from buildings or other sensitive receivers;
- Peak particle velocity associated with vibration from the HEIC module will decrease significantly beyond a distance of approximately 10m from the HEIC plant;
- A running surface is required for the HEIC plant (either 200 to 300mm of granular fill or an existing pavement).

This review indicated that the ground conditions at the site and required post improvement strengths are well suited to ground improvement by HEIC. The currently available geotechnical information suggests that the majority of settlement would occur in the shallow surficial soils including fill materials, which could be improved using HEIC, with the key points summarised below:

- HEIC has been successfully used elsewhere in the Sydney basin;
- Studies have indicated that vibrations are likely to be within indicative limits set by British and German Standards (Australian Standards AS2670.1-2001 and AS 2187.2–2006 may provide limited application) Bouazza and Avalle, *op cit*.;
- CPTs undertaken post treatment indicate the cone resistance, q_c may double post treatment;
- HEIC treatment will likely densify the soils to a degree such that they will provide adequate bearing capacity for most of the anticipated loads.

5 HEIC COMPACTION TRIAL

5.1 OVERVIEW

A field compaction trial was carried out under full-time supervision of a geotechnical engineer at the site using a Broons BH-1300 impact roller. This is a four sided roller with a mass of 8.5 tons and was towed by a tractor. Figure 1 illustrates the plant used.

The field trial area measured approximately 50m by 20m and was located in an area that typically represented the site ground conditions.

Existing geotechnical information indicated that the area was underlain by loose to medium dense dune sands. In situ testing was carried out utilising both CPT and a DCP. The tests were carried out after every five passes at ten locations as indicated in Figure 2.

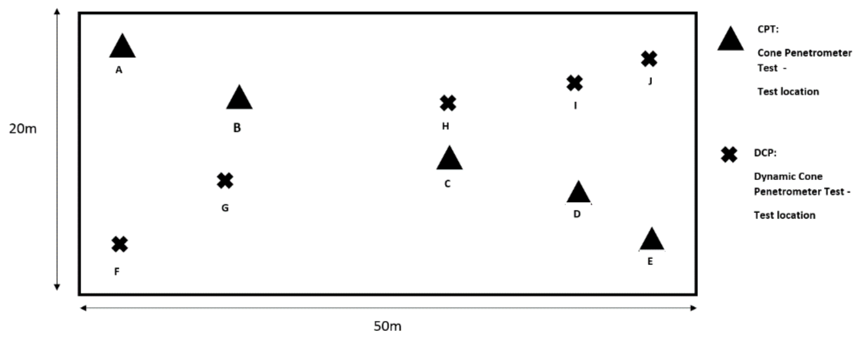

Figure 2: HEIC trial area indicating monitoring locations

5.2 SITE OBSERVATIONS

The following observations were noted during the trial:

- The trial area was underlain by materials associated with pavement construction and these were visually noted to deteriorate during the trial;
- An increased level of surface rutting was noted as the trial progressed
- The running surface of the trial area was wetted down before day 2 of the trial.

5.3 HEIC DATA ASSESSMENT

The depth of ground improvement and quantification of the degree of ground improvement achieved during the field compaction trials was assessed by evaluation of the CPT and DCP test data using the method described by Dove *et al* (2000) as described below (CPT and DCP are interchangeable in the expressions used). The degree of ground improvement reflected by the CPT tip resistance (q_c) data can be evaluated using the Improvement Index for Densification (I_d):

$$I_d = \{(q_{c\,after})/(q_{c\,before})\} - 1 \qquad (1)$$

where:

$q_{c\,before}$ = initial CPT soil strength (before ground improvement)

$q_{c\,after}$ = final CPT soil strength (following successive ground improvement(s))

This approach is sensitive to small q_c variations over depth with changing stratigraphy. The area under the CPT tip resistance profile can be normalised as follows:

$$I_d = \{(A_{\,after})/(A_{\,before})\} - 1 \qquad (2)$$

where:

$A_{\,before}$ = area under pre-ground improvement q_c plot

$A_{\,after}$ = area under post-ground improvement(s) q_c plot(s)

$A_{\,before}$ and $A_{\,after}$ are determined by numerical integration of a series of trapezoidal segments over depth from the q_c vs. depth plot as follows:

$A = (\Delta x/2)\,(q_{c1} - q_{c2})$

where:

q_{c1} , q_{c2} = two adjacent q_c values at depth increment Δx (dependant on data set in the range 0.02m<Δx<0.5m)

The target degree of ground improvement to be achieved during the field compaction trials was $I_d = 0.4$ based upon assumed soil strength gain from settlement calculations. This value was adopted based upon local experience of using HEIC in similar soils.

Surface settlement was determined as average settlement (mm) for each sequence of HEIC roller passes for the trial area.

5.4 EFFECTIVENESS OF COMPACTION

The HEIC trial was designed so that the effectiveness of the HEIC might be assessed principally by review of densification and induced settlements, if any, resulting from the applied compactive effort.

The degree of ground improvement based on CPT tip resistance was assessed by calculating the improvement index, I_d as described above. I_d was plotted at each location at 0.02m intervals. A typical plot of the variation in I_d with depth below ground level is shown within Figure 3. Similar profiles were observed across the test site.

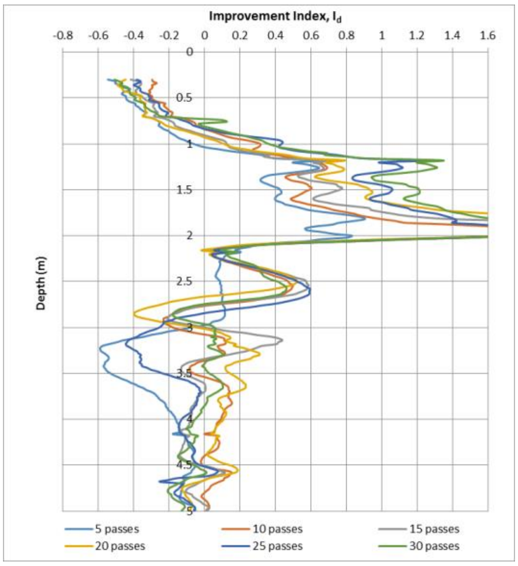

Figure 3: Typical plot of improvement index I_d with depth

There is some variation in the reported I_d values which is consistent with the CPT interpretation. At CPT D, initial cone resistances were very low between 1 and 2m, where material was noted as loose silty sand. This became medium dense to dense following completion of the trial.

A negative improvement index, implying loosening of the soils was recorded at depths of up to around 1m below ground level. After the initial 5 passes of the HEIC plant, I_d values improved with increases in the number of passes of the HEIC plant. This is supported by the DCP test results, which were undertaken in the top 1.2m below ground surface. The reduction in I_d values is attributed to the break-up of existing pavement materials under initial passes of the HEIC plant. The presence of these materials is demonstrated by the high near surface cone resistances and blow counts recorded by the CPTs and DCPs, respectively. The maximum I_d was observed between approximately 1.2m and 2.5m depth, which is consistent with the results presented by Avalle and Carter (op cit) which were undertaken on a nearby site, where the ground conditions are similar to those encountered at the trial compaction site. Localised negative I_d values at around 3.5m depth occurred alongside overall I_d improvements and may be attributed to minor variations in local stratigraphy and CPT test location offsets.

The target I_d of 0.4, which was adopted in design, was achieved at all CPT test locations after 20 passes of the HEIC plant in the top 2.5m below the site surface, indicating that 'effective compaction' has been achieved. The mean I_d has been calculated for each of the test locations and the results are summarised within Table 2.

Table 2: Mean I_d summary between 0.3m and 2.5m

Improvement index I_d after 20 passes				
CPT A	CPT B	CPT C	CPT D	CPT E
0.48	0.49	0.40	1.23	0.62

It should be noted that an impact roller is not a finishing roller due to the corrugations left following HEIC and the shearing and kneading effect as the module travels over the surface, as indicated in Figure 4. There is always a need for final trimming, watering and rolling of HEIC compacted areas with conventional compaction plant to achieve a suitable running surface.

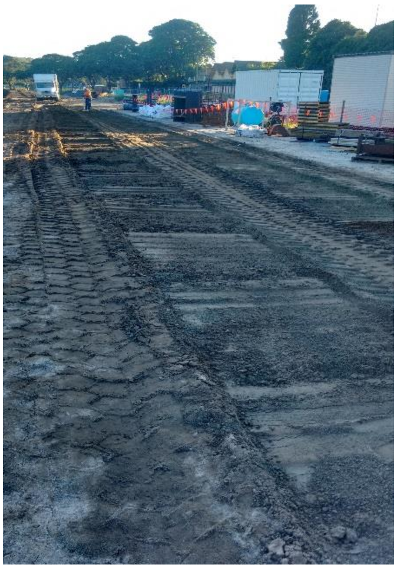

Figure 4: Typical post HEIC module surface corrugations

5.5 SETTLEMENT

Settlement monitoring was carried out to monitor changes in ground surface level as the HEIC trial progressed. The results are illustrated in Figure 5, which show the changes in reduced level and the amount of settlement after each increment of 5 passes, respectively. The settlement monitoring survey took place at the same locations as the CPTs indicated on Figure 2.

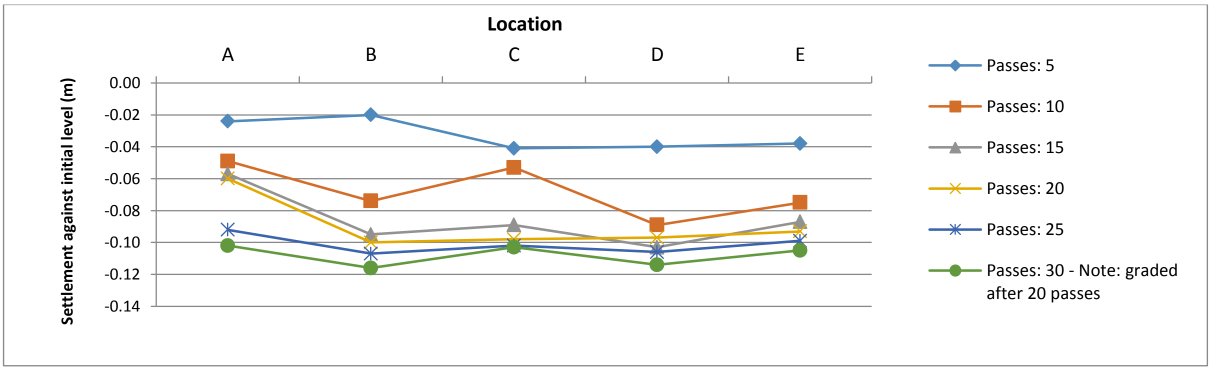

Figure 5: HEIC settlement survey results

The survey results indicate that the rate of settlement slowed after 15 to 20 passes of the HEIC plant, indicating that 'effective refusal' has been achieved, Avalle (2004). A review of the survey data in conjunction with the in situ geotechnical test results suggests a good correlation between settlement and observed level of improvement.

5.6 NOISE AND VIBRATION

Noise and vibration measurements were undertaken concurrently with the HEIC trial to establish site laws in order to determine allowable HEIC offset distances in relation to the potential to cause adverse noise and or vibration impacts to nearby sensitive receivers. Construction noise management levels and vibration control criteria were established specifically for the site by acoustics and vibration specialists. Noise management levels for residential receptors were set at 54 and 43 LAeq (15 minutes) dBA for day-time and night-time, respectively. Construction vibration control criteria for various structures were set at between 2 to 15 mm/s PPV (operator warning level) and 3 to 20 mm/s PPV (operator halt level) based on information in DIN 4150.3 (1999). In addition, human response levels were based upon information presented in DECCW (2006).

Vibration site control measurements showed that the four-sided impact roller can safely operate at distances >10m from light framed structures and >15m from heritage structures. The measured noise and vibration measurements obtained from the trial can be used for construction stage planning purposes taking into account 8 hour working duration and cumulative noise and vibration effects due to other site activities.

The HEIC trial was a pre-requisite to the bulk earthworks at the site informing the HEIC construction phase compaction requirements in the earthworks specification developed for the site. The developed method specification was based upon the results of the HEIC trial with appropriate compliance testing included for the works, with reference to Avalle (2004), relevant Australian Standards (AS 2007, 2011) and RMS (2015).

6 CONCLUSIONS

The results of the site trial were conclusive in suggesting that the site trial would achieve the required level of densification to prevent excessive settlement and potential bearing capacity failure of the loose shallow surface soils. The technique has distinct advantages over a piled solution as it can be completed quickly, offering significant programme and cost savings. No temporary works are required other than the maintenance of a suitable running surface. High energy impact compaction (HEIC) is also a sustainable solution to ground improvement so that it does not require imported and often expensive construction materials such as concrete or steel.

There is significant potential for further use of HEIC within the Sydney basin area and beyond on sites where loose soils are present near to the site surface. It is suitable for use on sites for commercial and residential buildings, for transportation infrastructure, landfill restoration and has also been employed for mine site restoration and rehabilitation including new road formation across mine spoil dumps in NSW, (GHD 2013).

7 ACKNOWLEDGEMENTS

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The opinions expressed in this paper are those of the authors.

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