EXPERIENCES WITH POST-CONSTRUCTION RETESTING OF ENGINEERED CLAY FILLS

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ABSTRACT

A considerable amount of disputation, both legal and informal, arises where compacted clayey fills are retested some time after completion of the works and is predicated on two assumptions. Firstly, that once compacted, clayey fills remain unchanged thereafter and secondly that the results of post-construction retesting are more credible than the results of control testing carried out at the time of construction. The authors have been exposed to a number of cases, including legal proceedings, where earthworks having apparently been properly carried out and reputably tested during construction were retested some time after completion and assessed to be below specification. The simple conclusion often drawn by owners and their experts in such instances is that the earthworks were inadequately carried out at the time of construction. However, in the authors' view there has developed a body of factual evidence which does not support that simple conclusion. This evidence has arisen from a variety of sources involving actual construction works where multiple testing and/or retesting by a range of reputable authorities often under conditions that were less than ideal. Nevertheless they have provided a series of experiences or case histories to which geotechnical engineers, earthworks contractors, lawyers and owners should have regard and from which valuable insights and lessons may be drawn. This paper deals with the factual aspects of seven cases which, in the authors' view, challenge the simple conclusion based on the assumptions of essentially inert compacted clayey fills and the primacy of retests over tests during placement

1 INTRODUCTION

Dr Andrew Charles reminded us in his recent Rankine lecture that, despite their limitations, case histories have a vital role in geotechnical engineering, and are of far greater importance than in other branches of civil engineering (Charles, 2008). He quoted from Karl Terzaghi's presidential address to the 1st International Conference on Soil Mechanics and Foundation Engineering where he affirmed that:

> *"….successful work in soil mechanics and foundation engineering requires not only a thorough grounding in theory combined with an open eye for the possible sources of error, but also an amount of observation and of measurement in the field far in excess of anything attempted by the preceding generation of engineers."* (Terzaghi, 1936)

The traditional view of clayey fills has probably been, and still generally is, that when properly compacted they will hold that level of compaction indefinitely thereafter. The only concession to this view may be in relation to surficial material where exposure to the elements occurs. However, there have been a number of instances in recent years where the compaction of engineered clay fills, thought to have been properly compacted during construction, have apparently been found to be below the specification, some time after completion This paper describes some of those experiences and presents the results of density control testing during construction together with the results of density retesting carried out some years later. They provide a set of factual data that are to be used in a companion paper where the potential causes for an apparent loss of compaction over time will be explored.

Many of the experiences presented as case histories in this paper come from the involvement of the authors over many years in legal proceedings arising from construction issues that have been the subject of considerable effort and often significant differences of opinion by experts for the parties involved. Nevertheless, the matters are considered important to current earthworks practice and particularly to related forensic investigations judged, not least, by the amount of disputation generated. The authors expect that there may be divergent and strongly held views within the geotechnical community and have sought to provide a greater than normal level of detail so that others may, if so desired, carry out their own independent assessments. The authors would welcome additional experiences and case histories from other sources.

2 SELECTED EXPERIENCES

Engineered fills are commonly tested for compliance with the relevant earthworks specification during construction and not further tested thereafter. The authors have been involved in several matters where the state of compaction of clay fills after construction was at issue. In these circumstances *a posteriori* compaction testing of completed fills is often carried out in an attempt to establish that compaction during construction was inadequate, albeit that contemporaneous control testing and approval had often been carried out in a manner considered satisfactory at the time.

It is widely assumed within the geotechnical community that failure during retesting indicates inadequate or poor compaction at the time of construction. This assumption has not previously been questioned, or critically examined, to the best of the authors' understanding. The experiences presented were all located in New South Wales, six in metropolitan Sydney and one in the semi-arid north western area. The results from these experiences allow an examination of the inherent assumption that underlies retesting against factual data. Several of the selected earthworks projects have been the subject of legal proceedings that have gone on to final judgement and hence are in the public domain. Others were not concluded but have had broad exposure in legal circles and are widely known. To the extent that is reasonable the authors have identified the projects and the parties to these matters because it is considered that as much factual detail as possible should be provided to facilitate frank and open review of the issues.

3 RECONSTRUCTION OF FLOOD LEVEES AT BREWARRINA, NSW

The Brewarrina flood control level system, built originally in 1976 in the face of advancing floodwaters in the Barwon River, was reconstructed between October 2001 and April 2002. Reconstruction involved the rehabilitation of existing, but inadequate, Northern and Southern levees by overlays on both wet and dry sides, and the construction of 3 other new levees. The general layout of the levee system is shown in Figure 1. The existing Northern and Southern levees were rehabilitated by stripping the exposed batter faces to remove surficial instability and slumping, tension cracks, toe bulging, erosion, poor compaction, inadequate freeboard and providing wedge-shaped overlays at 1:3 batters on both upstream and downstream sides of the levees. Although assessed as structurally deficient, the existing levees were retained except for a major part of the Southern levee which had become redundant due to growth of the town area. The Charlton Road and Tarrion Creek levees are low height road embankments that extend the existing flood protection function to the west and south of the township. The North Brewarrina levee is a closed embankment protecting a separate northern section of the town on the Kamalori Highway beyond the existing levees. Total earthworks in all 5 levees involved some 75,000m^3 of stabilised, medium to highly plastic clays sourced from 4 separate borrow areas.

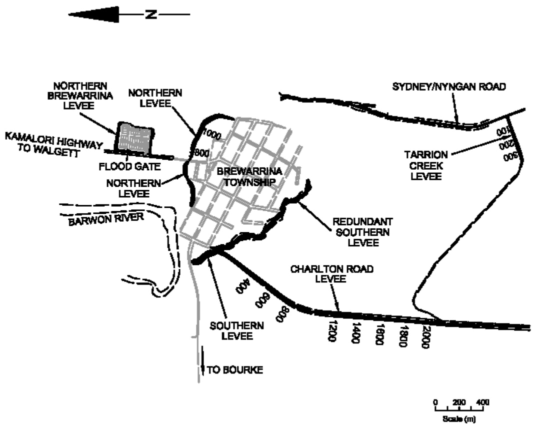


Figure 1: Layout of flood levees at Brewarrina, NSW.

The specified compaction requirement was 95% Standard Hilf with moisture variations within the range SOMC to 3% dry of optimum. Control testing was carried out by Civil Engineering Testing Services (CETS) from Muswellbrook, a NATA registered, small regional firm appointed by the contractor, Beckhaus Civil of Wee Waa. The material was specified as being clayey material selectively won from the borrow areas or from the outer faces of the Northern and

Southern levees and having a maximum linear shrinkage value of 12%. The addition of gypsum was specified when the linear shrinkage exceeded 12%. The time effects of stabilisation have not been considered further in this paper.

Control testing was at an overall rate of about 270m^3 per test, compared to the specification requirement of 150 to 600m^3 depending on the size of the lot placed. All construction control tests, by sand replacement, met the specified compaction requirement. There were several stages of post-construction compaction testing:

- In April 2002 immediately after completion of the levees, Douglas Partners carried out a limited set of 10 tests directly for the Council of the Shire of Brewarrina.
- In April 2003, Barnson, a NATA registered firm from Dubbo carried out a series of tests under the direction of GHD on behalf of the Council. Further tests were carried out in July 2003.
- In August 2003, Golder carried out a series of tests on behalf of Beckhaus Civil, in the presence of GHD and Council with selected parallel testing by Barnson on behalf of Council.

Every density ratio test result in every stage of construction control and post-construction retesting was reported with NATA certification.

3.1 DOUGLAS TESTING

Douglas carried out a series of 10 tests for Council in levee fill materials soon after the levees had been completed. There were 4 tests in the Northern levee, 2 in each of the Charlton Road and North Brewarrina levees, and 1 in each of the Southern and Tarrion Creel levees. All tests were located on or near the levee centrelines and were carried out in shallow test pits excavated to depths between 0.3 and 0.85m below surface level. Nine of the 10 ten tests were 95% of the Hilf density ratio or better and one was 94% based on the nuclear field method. All moistures were within the specification limits.

3.2 BARNSON TESTING

GHD carried out an extensive investigation programme on behalf of the Council between April and July 2003. The investigation consisted of 33 test pits, 24 of which were excavated for the purpose of investigating the density ratio of levee fill materials. Sixteen were located at 7 cross sections on the Northern levee, 6 at 2 sections on the Southern levee, and 2 on the Charlton Rd levee. Test pits were positioned along the batter slopes of the levees either close to the crests or close to the toes of the overlays and were up to 2.7m deep.

Barnson, under GHD direction, carried out 26 field density tests in 21 test pits at depths up to 1.85m. There were 16 field density tests in the bank sections of the Northern and Southern levees and 10 field density tests in culvert backfills for the Northern, Southern and the Charlton Rd levees. In July 2003, Barnson carried out another 5 field density tests in 2 test pits on the North Brewarrina levee, taking the total number of such tests nominally located in levee material to 21.

Because of the form of rehabilitation adopted for the Northern and Southern levees, accurate location of test sites was imperative to ensure that only newly placed fill, and not pre-existing levee material, was tested for density ratio. Complications associated with variations between work-as-executed and as-designed, overfilling of the overlay batters, variable stripping of the old levee faces and survey inaccuracies resulted in at least 3 and possibly as many as 9 density tests not being located in newly placed fill. Consequently, the test results from affected test locations may not be relevant to works as required under the contract.

All the Barnson field tests were carried out by the nuclear method to AS 1289.5.8.1. The April 2003 tests used an older style Troxler model 3430 and the July 2003 tests a newer model Humboldt 5001C gauge. The laboratory component was by the Hilf method nominally to AS 1289.5.7.1. There was a parallel set of 5 nuclear and sand replacement tests carried out at depths between 0.1m and 1.2m in the North Brewarrina levee in July 2003, which agreed in respect of density ratio to within better than 1% over a range from 85.1% to 94.6%. These tests reported variations only in respect of field wet density determined by nuclear and sand replacement methods with common field moistures, laboratory wet densities and moisture variations. The variations in field wet density for 4 of the 5 tests were ±0.01t/m^3 and 0.02t/m^3 for the other for densities between 1.70t/m^3 and 1.94t/m^3.

3.3 GOLDER TESTS AND GOLDER/BARNSON PARALLEL TESTS

Golder carried out a limited investigation in late August 2003 for the Contractor, with participation by GHD and Barnson on behalf of Council. A total of 9 pits was excavated in all levees together with 18 field density tests, for 7 of which Barnson carried out testing in parallel. There was reasonably good agreement in terms of density ratio with differences in 6 of the 7 tests between -0.7% and +2.5% and the remaining test a difference of +4.0%. However, there were larger, and more consistent, differences in terms of the underlying Standard maximum dry density and Standard optimum moistures and in terms of field dry densities and field moisture contents as detailed below.

The numbers of field density tests carried out during construction by CETS and in the various post-construction investigations on behalf of Council and the Contractor is presented in Table 1 in terms of general earthworks and culverts for the individual levees, by each of the parties. The Northern levee was the most intensively tested section of the works both during construction and in post-construction investigations, accounting for over 40% of the control tests and nearly half of the post-construction tests. The level of scrutiny is reasonable because it is prominently located closest to the river, was the most complex form of re-construction, and involved the largest individual quantity of earthworks. To a large extent the Northern levee is emblematic of the putative deficiencies perceived by Council for the rehabilitated Brewarrina flood control system.

Table 1: Summary of numbers of field density tests carried out during construction and in the various post-construction investigations.

Levee/ Quantity of Earthworks	Test Location	Construction Control	Post-Construction Field Density Tests					
		CETS	Douglas	Barnson	Barnson	Golder	Barnson	Total
		Oct 2001- Mar 2002	Apr 2002	Apr 2003	Jul 2003	Aug 2003	Aug 2003	
Northern 20,900m^3	Bank	105	4	13	-	7	3	27
	Culvert	16	-	5	-	1	-	6
Charlton Rd 16,800 m^3	Bank	73	2	2	-	2	-	6
	Culvert	6	-	2	-	-	-	2
North Brewarrina 16,640 my	Bank	37	2	-	5	3	2	11
	Culvert	8	-	-	-	-	-	8
Tarrion Ck 6,700 m^3	Bank	34	1	-	-	2	-	3
	Culvert	1	-	-	-	-	-	-
Southern 6,700 m^3	Bank	8	1	3	-	2	2	8
	Culvert	0	-	3	-	-	-	3
Bank total		257	10	18	5	17	7	57
Culvert total		25	-	10	-	1	-	11
Total tests		**282**	**10**	**28**	**5**	**18**	**7**	**68**

Table 2: Summary of post-construction compaction testing with control tests on the Northern levee during construction.

Date/ GTA	Areas Tested	Number of Tests			Density ratio Results			Comments
		<95%	≥95%	Total	Mean	Range	StandardDeviation	
Jan - Mar 2002 CETS	**Northern levee overlays**	**0**	**105**	**105**	**100.3%**	**95.5% - 102.8%**	**0.9%**	
	North levee culverts	**0**	**16**	**16**	**101%**	**99.4% - 102.4%**	**0.8%**	
April 2002 Douglas	**Crest**	**1**	**9**	**10**	**96.0%**	**94 - 99%**	**1.4%**	**All levees, tested for Council**
April 2003 Barnson	**Levee bank overlays**	**16**	**0**	**16**	**84.5%**	**74.2 - 94%**	**5.9%**	**N, S only, tested for Council**
	Culverts	**10**	**0**	**10**	**75.2%**	**67.6 - 83.6%**	**4.7%**	**N, S, CR levees, tested for Council**
July 2003 Barnson	**Levee bank**	**5**	**0**	**5**	**88.0%**	**85.1 - 94.8%**	**3.9%**	**NB levee only, tested for Council**
August 2003 Golder & Barnson	**17 in bank; 1 in culvert**	**6**	**12**	**18**	**93.2%**	**85.5 - 101%**	**4.4%**	**All but TC levee, tested by Golder for Contractor**
	6 in bank; 1 in culvert	**4**	**3**	**7**	**93.2%**	**87.5 -100.6%**	**5.4%**	**Parallel tests by Barnson for Council**

Notes N - Northern Levee; S - Southern Levee; NB - North Brewarrina Levee; CR - Charlton Rd Levee; TC - Tarrion Ck Levee

3.4 COMPARISON OF RESULTS FOR CONTROL TESTS AND POST-CONSTRUCTION RETESTS

A comparison of post-construction compaction testing and the results of density tests during construction, typified by those taken on the Northern levee, is summarised in Table 2.

This comparison is further detailed in terms of the distribution of density ratio for each stage of testing at the Northern Levee in Figure 2.

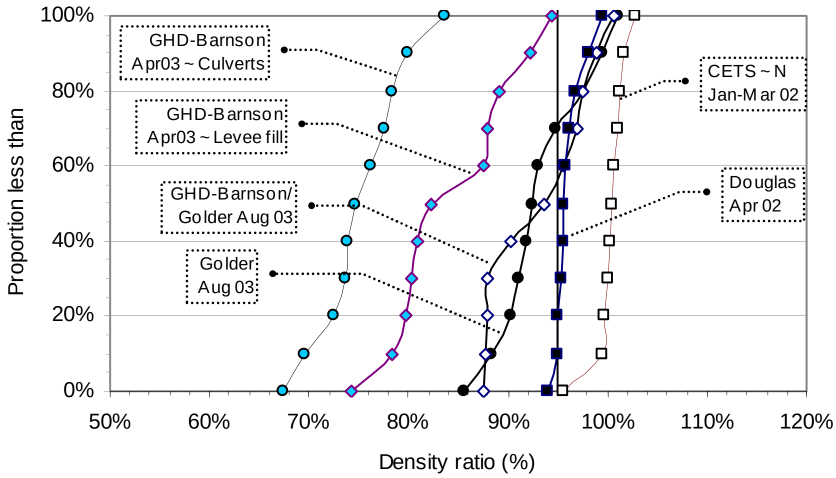


Figure 2: Distributions of compaction test results for Northern levee

The parallel tests carried out by Golder and Barnson in August 2003 consisted of independent nuclear field density tests in the same probe hole, independent determinations of field moisture by oven drying of split samples and independent Standard laboratory compactions of split samples by Golder and Barnson. The results of this testing programme showed essentially similar range, average and distribution of density ratios and confirmed the overall reliability of these results. While the differences between the two sets of 7 tests would not be insignificant in the context of contractual requirements, they nevertheless confirm the essential order of accuracy and reliability of that set of test results compared to the earlier Barnson tests. This indicates that the test procedures used in August 2003 by both parties were consistent with each other and likely to be reliable.

For the parallel test set the average Barnson's density ratio was 1.3% below the Golder average and 3 of the 7 tests were more than 2% different as indicated in the distribution of density ratio differences shown in Figure 3.

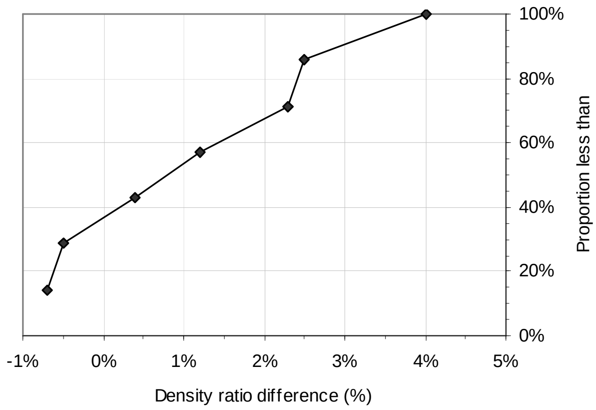


Figure 3: Distributions of differences in density ratios of the Barnson results compared to the Golder results from 7 parallel tests at the North Brewarrina levee.

The differences in density ratio arise from differences in both field and laboratory results but with the larger contribution to the differences coming from the laboratory values. Field wet density results agree in terms of average values but vary by up to ±1.6% on individual results, which is somewhat surprising given that both sets of measurements were taken in the same probe hole in each case and hence material variations should have been effectively eliminated. Field moisture contents differed on average by 0.6% and individually by between 0.2% and 1.9% with the Barnson values generally less than Golders and accentuating the differences in terms of field dry densities with an average difference of 0.5% and individual differences ranging between -0.9% and 1.8%. For laboratory compactions the differences in Standard maximum dry densities on average was 1.9% with a range of 0% to 5.1% and for Standard optimum moistures the average difference was 0.9% with a range of 0.2% to 1.9%.

The earlier investigation programme carried out by Barnson in April and July 2003 produced density ratio values that were much lower than the parallel test results. The results for the overlay sections of the levees were almost 10% below the parallel tests results and the results for the culvert sections almost another 10% lower again. The minimum density ratio recorded for the 'bank' sections was 74.2% and 5 of the 16 results were less than 80%. The minimum density ratio for the culvert sections was 67.3% and 5 of the 10 results were less than 75%. These are spectacularly low density ratios and are among the lowest that the authors have seen reported. It is possible that the levee fill and culvert backfill results might represent different compaction regimes, or populations, as the methods of construction for the existing Northern and Southern levees differed between the two. Levee filling was carried out over an extended site area using pad foot compactors, while culvert backfill was confined to a restricted area and used different compaction equipment.

The Barnson levee fill results and the Golder/Barnson parallel test results represent essentially the same compaction regime, or population. This raises the obvious question as to how two such apparently random samples taken from the one population could yield such different distributions. There are four hypotheses that are capable of explaining this difference:

- The population had a very wide distribution encompassing both sets of results and fortuitously, one had sited its tests at locations representing the lower portion of the population range and the other at locations representing the upper portion of the population range. The odds against this particular combination are very high, particularly given the number of tests on both sides. It can be inferred by means of Tukey's Quick and the Mann-Whitney Tests for non-parametric statistics (Conover, 1971) that the two data sets are extremely unlikely to be from a common population.
- Both sets of results are correct and each accurately represents the population from which it came. This would require that there had been significant change in the population over the time and would require a substantial physical change in the levees between April and August 2003. It is physically impossible that the dimensions of the levees could have changed by 10% over that time interval.
- One of the sets of density ratios is wrong and the population did not have the wide range of values implied by both sets.
- Alternatively, there may have been some combination of the second and third propositions. That is, the state of compaction of the levees changed with the seasons and some but not necessarily most, of the Barnson results were wrong.

Given the independent confirmation of the Golder tests by Barnson in the parallel test programme and because the Barnson results are so low, they invited detailed scrutiny as to the procedures and equipment used in their production and were finally rejected by Master Macready of the NSW Supreme Court because of the following inadequacies in testing protocols (NSW Supreme Court, 2004):

- Of necessity the Barnson tests were carried out in test pits, but most test pits were not wide enough to satisfy the clearance requirements of AS 1289.5.8.1 and in addition standard counts, to account for the effects of reflected radiation, were not determined at any test sites in the April and July 2003 investigations. Standard counts were taken for the parallel tests in August 2003.
- Samples for laboratory tests have to be taken by excavating a hole with vertical sides to the depth at which the probe of the nuclear gauge was located during the tests. On some occasions the samples were taken from below the probe depth.
- There was ambiguity in relation to the positioning of tests pits with respect to the interface between old and new work. Additionally, the position of the face of the pit that was logged differed from the position where the insitu density test was carried out. Because in some pits it was difficult to distinguish between newly placed material and old levee material there was doubt as to which material had been tested.

- The April 2003 test series used a hired Troxler gauge that was out of calibration and which, in 9 of the 16 tests, produced field density values less than the lower calibration limit of 1.77t/m^3. However, neither matter was thought to affect the results significantly.
- There was one instance where the actual probe depth of 250mm was wrongly input to the gauge as 200mm and the conversion to field wet density used the calibration coefficient for a probe depth of 200mm. This resulted in the density ratio for that test being reduced from 86.6% to 74.1%. This inconsistency between actual the probe depth and the probe depth used for density conversion was evident only from the manual field record and it was not possible to independently confirm that other tests in the series had not been similarly affected.
- Insufficient material was sampled from test sites to comply with the requirement of AS1289.5.7.1 for three separate compaction specimens and the one sample of material was reused in the laboratory compaction tests. The likely effect would have been to increase the peak wet density and consequently reduce the density ratio.
- Hilf compactions are required to be carried out with added moisture values between 4% dry and 6% wet. Six of the 16 tests in levee fill material were carried out in the April 2003 series were beyond these limits and may not have been reliable.

3.5 POST-CONSTRUCTION MOISTURE VARIATIONS

The variation in field moisture from placement through the various stages of investigations is shown in Figure 4. The trend is quite clear with moisture variation from optimum having dried slightly over time. The Barnson moisture variations from the April 03 investigation have not been included because that set of tests are considered suspect.

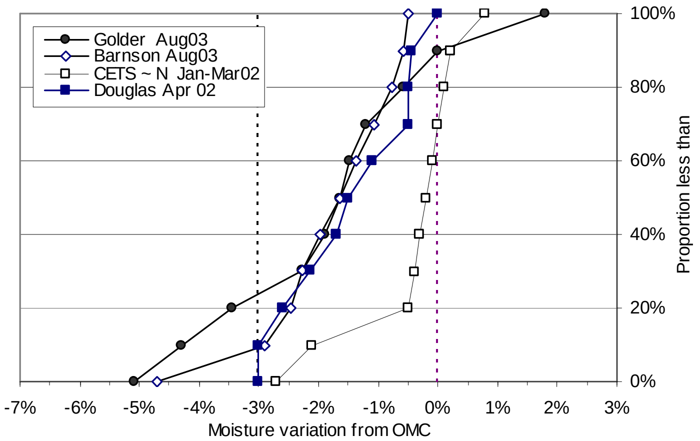


Figure 4: Distribution of moisture variations from optimum for Northern levee

3.6 CONCLUSION

Post-construction retesting of the clay fills placed as part of the Brewarrina levees rehabilitation works indicated an apparent loss of compaction that appeared to increase with time since the completion of construction. On the face of it, post-construction retesting showed about a 5% loss in compaction within months of completion and an additional loss of approximately 3% within 2 years of completion. Over the same time periods there appears to have been an initial drying of the clay fill by about 1% and thereafter slight drying overall, but with an increase in the range of moisture variations associated with a further loss of up to about 2% at one extreme and possibly with some slight increase in moisture at the other.

4 MINTO RAILWAY OVERPASS AT CAMPBELLTOWN

Approach embankments for a bridge over the Great Southern Railway immediately south of Minto Station near Campbelltown were constructed by Abigroup Contractors between October 1992 and June 1993. The embankments, constructed from clayfill, were up to about 9m in height with side slopes of 1.5H:1V and about 1km in total length. They were terminated at the over-bridge in reinforced earth abutments and because of lateral constraints associated with the Minto Main Drain No.2 the southern edge of the main carriageway was retained by about a 50m length of reinforced earth wall from the western bridge abutment. Batters were topsoiled and hydro-seeded for erosion protection. The overall arrangement is shown on Figure 5.

Clayfill used in embankment construction was obtained from the 45ha site of the Minto retarding basin located on Bow Bowring Creek, about 1 km north of the site, and comprised some 50,000m^3 of clay fill. The materials in the borrow area had been investigated in 1985 by Coffey as the source of fill for water retaining embankments and in 1989 by SMEC for use in the overpass embankments. They both reported Quaternary alluvial clays and sandy/silty clays, 2m to 3m thick over shale, with variable composition and geotechnical properties. Liquid limits varied between 32% and 69%, plastic limits between 14% and 20%, linear shrinkage between 10% and 18%, Standard maximum dry densities between 1.56t/m^3 and 1.86t/m^3, standard optimum moistures between 12.5% and 22%, Emerson classes between 1 - 2 and 5 - 8 and from highly dispersive to non-dispersive.

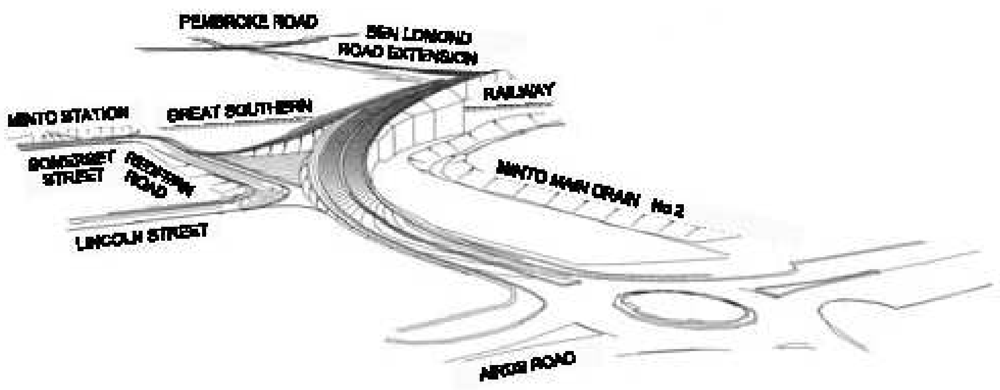


Figure 5: Layout of the road embankments for the Railway Overpass at Minto, NSW.

4.1 TESTING DURING CONSTRUCTION

During construction Level 2 control testing was carried out by Ground Test Pty Ltd, appointed by the Contractor and TestRite Laboratories Pty Ltd carried out independent audit testing on behalf of Campbelltown City Council. Both testing authorities were NATA registered and all test results were NATA certified. Ground Test carried out 97 density tests in compacted clayfill between November 1992 and March 1993 and TestRite conducted 17 density tests in clayfill between December 1992 and May 1993, with over half those tests in early December 1992. The evidence was that there was no communication between TestRite and Ground Test and that TestRite took its tests at different locations from those of Ground Test. TestRite's results were provided only to the Council and not to the Contractor. The contract specification required control testing and approval by the Superintendent of each layer of material placed in the embankments. The procedure was for a single test to be taken from each section of embankment that had been notified by the Contractor as ready for testing and the Superintendent's Representative was conscientious in requiring further compactive effort where the test results did not achieve the 95% Standard density ratio.

The distributions of control and audit testing results for clayfill compaction are compared with the contract specification of 95% standard maximum dry density (AS1289 E1.1) and moisture between 60% and 90% of standard optimum moisture content in Figure 6.

The differences in density ratio between control and audit tests arose because Ground Test reported to the Contractor only those tests which passed and areas of test failures had been reworked until compliance was achieved whereas TestRite reported all tests and failures were used by the Superintendent to have additional compaction carried out on those areas. There was good agreement between control and audit tests in respect of laboratory compaction results for maximum dry densities and optimum moisture contents. Ground Test carried out 35 3-point standard compaction tests and assumed reference values for the other 62 tests. TestRite carried out 7 3-point standard compactions on individual samples and 10 3-point tests on samples combined from 2 and 3 separate tests. A comparison of similar materials yielded almost exactly similar average results for the 31 Ground Test samples for standard maxima of 1.83 ±0.04t/m^3

(mean±standard deviation) at 15.3 ± 1.1% and for the 9 TestRite samples of 1.82 ± 0.04t/m^3 at 15.4 ± 1.2%. Hence differences in density ratio results between control and audit tests reflected differences in field dry density results rather than in Standard maxima. However, even on the raw test results it is apparent that there was substantial compliance with the compaction specification indicating a relatively high standard of earthworks control.

4.1.1 Post-Construction Performance of the Embankments

were the earliest indication of batter failures. By November 1993 cracks were present on all batter slopes and two small slips had occurred over the RE walls. Remedial works consisting of soil nailing to stabilise the batters was subsequently carried out and litigation between Campbelltown City Council and Abigroup Contractors ensued in 1998 and 1999. The adequacy of compaction during construction was at issue, particularly in relation to the compaction of clay fill within the batters.

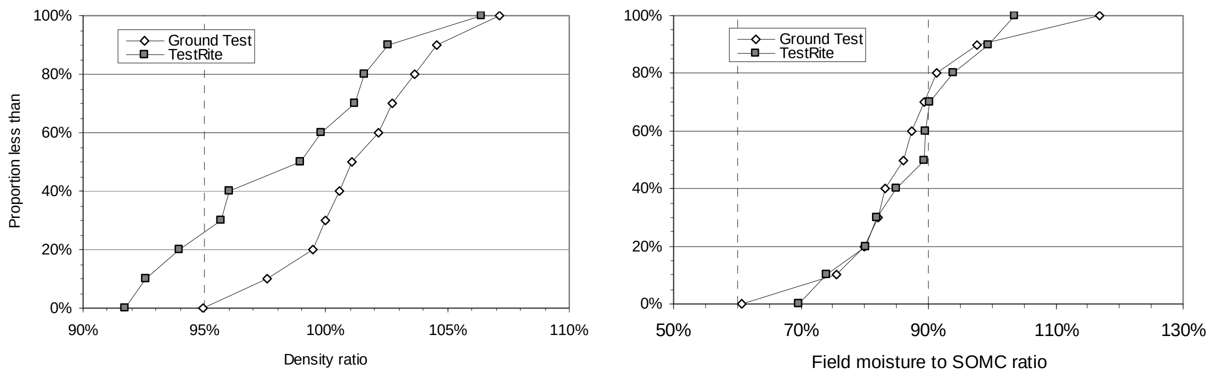


Figure 6: Distribution of density ratio and field moisture to optimum for Minto overpass embankments

4.1.4 Comparison of Results for Control Tests and Post-Construction Retests

SMEC Testing Services carried out an initial investigation of batters in September 1993 involving 5 field density tests at depths of 0.2m and a more substantial investigation of the batters in March 1996 involving 12 field density tests in 4 test pits at depths between 0.5m and 3.6m. The pits were on either side of the overpass bridge and in northern and southern batter faces. All tests were by the sand replacement method and were NATA certified. The variations in density ratio and in field moisture from placement through the various stages of investigations are shown in Figure 7. Construction results are a combination of control tests by Ground Test on behalf of the Contractor and independent audit testing by TestRite on behalf of the Superintendent's Representative.

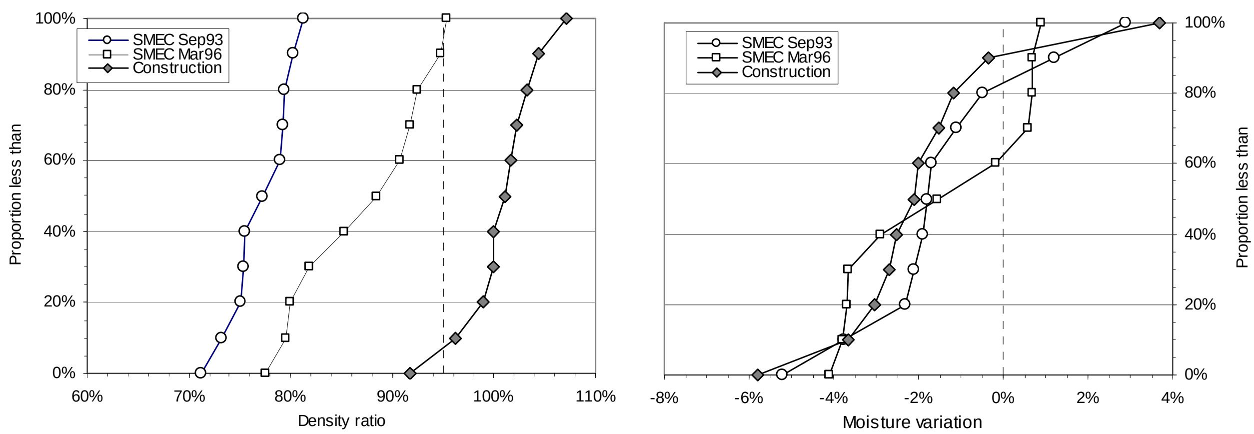


Figure 7: Distribution of density ratio and the variation in field moisture from optimum (FMC-SOMC) for Minto overpass embankments

The distribution of density ratios from the SMEC shallow (September 93) and deeper (March 96) investigations are significantly different from that of the testing carried out by Ground Test and TestRite during construction of the embankments. The results of the shallow SMEC investigation can be discounted because they were limited in number and restricted to surficial materials. The deeper SMEC testing is likely to be more representative of conditions within the batters of the embankments as it covered the northern and southern batters of the embankments and either side of the over-bridge. It is clear from Figure 7 that the compaction status of batter materials as characterised by the deeper SMEC investigation represents a significantly different set of conditions to that shown by the results of testing during construction. They quite clearly represent different populations and solely on that basis it would be tempting to conclude that compaction of the batters had been inadequate during construction. However, there are other factors that have to be taken into account before an assessment can properly be made based on all the factual evidence.

4.1.3 Like-for-Like Testing

There was only limited testing within the confines of the batters during construction, presumably because of a somewhat natural, and possibly inadvertent, bias towards the roadway structure as opposed to the edges. Out of Ground Test's 97 tests 2 were in the southern batter east of the overpass and at least another 7 were close to, and possibly within, the batters although test locations are somewhat equivocal. All these tests showed compliance with the 95% SMDD criterion. Although none of the TestRite testing was specifically identified as located within the batters, 7 of the 17 audit tests were in the kerb side lanes and would at least have been close to batters particularly for the upper levels of embankment. Three of the 7 tests were below 95% but are likely to have been reworked under the control of the Superintendent.

The Contractor gave evidence to the effect that the construction method involved overfilling to ensure compaction to the extreme edges of the final profile and cutting back to the design batter. The evidence indicates that this work was conscientiously supervised by the Superintendent who accepted each layer of compacted filling before the subsequent layer was placed.

4.1.4 Post-Construction Moisture Variations

Moisture variations from SOMC during construction and at the times of the SMEC post-construction investigations are shown in Figure 7. Moisture conditions as indicated by the distributions appear to have been substantially unchanged over the 8-month period between placement and the initial investigation. However, the distribution of moisture variations between construction, as characterised by the combined set of Ground Test and TestRite results, are markedly different from that evidenced by the SMEC investigation of March 1996 and may show the effects of climatic factors over the 3-year period. Interestingly, although the mean value has remained unchanged, the distribution have become more symmetrical with the initially wetter and initially drier proportions have increased by about equal amounts and the range has reduced by at least 2% at both extremes.

The SMEC investigation involved a range of depths below the batter faces and closer examination of the SMEC results for moisture variation from SOMC suggests the possibility of a depth related effect over the 3-year period since the embankments were completed. Figure 8 is a plot of the SMEC moisture variation data against depth below the batter face, where depth has been measured normal to the batter face.

It is expected that the upper 1m to 1.5m of the embankment might be influenced by climatic effects and that deeper sections of the embankment might tend towards "equilibrium" moisture conditions. Although the SMEC test results are limited in number there does appear to have been a significant increase in moisture of 2 to 3% at depths below the range of seasonal effects and no significant change at shallower depths. Two results from TP1 at 1.6m and 2.0m depths do not fit the suggested pattern and might if anything have dried by about 2% relative to the average placement conditions. Batter aspect may be a factor with TP1 having been on a northerly facing batter although TP3 which was on a north-easterly facing batter face does not support that proposition. TP2 and TP4 were on southerly and south-westerly facing batters.

The distributions of moisture variations from SOMC are presented in Figure 9 for placement conditions and from the March 1996 SMEC investigation with the 12 results further separated into a shallow subset of 5 and a deeper subset of 7 results.

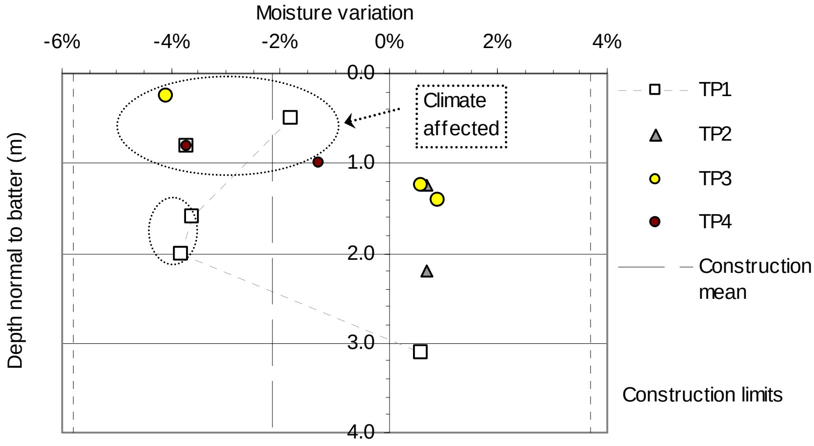


Figure 8: Variations in field moisture from optimum from the SMEC 1996 investigation plotted against depth normal to the batter faces and compared to mean and extreme moisture variations based on construction results

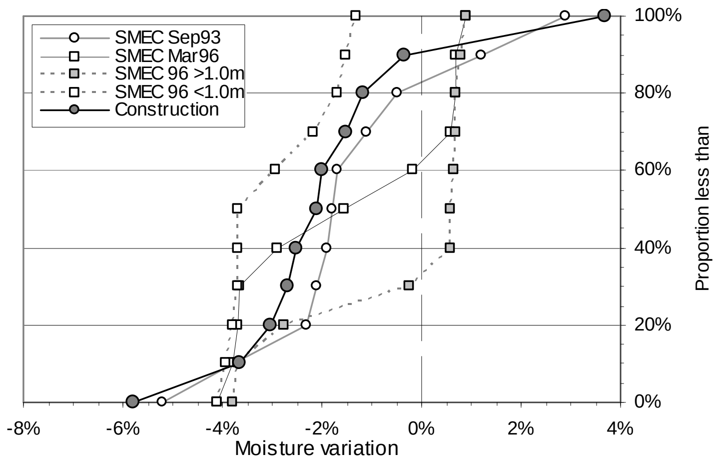


Figure 9: Distribution of the variations in field moisture from optimum (FMC-SOMC) for Minto overpass embankments

Table 3: Summarises the statistics of field moisture content less SOMC for placement conditions during construction and for the March 1996 SMEC investigation.

Statistics	**Field moisture content less standard OMC**			
	Placement conditions	**SMEC Investigation March 1996**		
		Shallow (0 – 1.0m)	**Deep** (> 1.0m)	**Combined**
Number	114	5	7	12
Mean	2.1% dry	2.9% dry	0.6% dry	1.5% dry
Standard deviation	1.4%	1.3%	2.2%	2.1%
Range	5.8% dry to 3.7% wet	4.1% dry to -1.3% wet	3.8% dry to 0.9% wet	4.1% dry to 0.9% wet
Kurtosis	+2.20	-2.71	-0.83	-2.10
Skewness	+0.32	+0.62	-1.22	0.00

These statistics show that the shape of the distribution for placement conditions was relatively peaked with a slight skew towards the positive values. The majority of the values were within the range 3% dry and SOMC. The shape of the distribution for the combined SMEC values was relatively flat and symmetrical with all values within the range 4% dry to 1% wet and was markedly different from that of the construction stage.

4.2 CONCLUSION

Mr John Muirhead, Consulting Engineer acting as court-appointed Arbitrator, found that the embankments had been uniformly compacted in accordance with the specification including those parts of the embankments which were beneath the batters. This finding was based on the consistency of density test results between the control testing for the Contractor by Ground Test and audit testing on behalf of Council by TestRite and on the supervision and acceptance of compacted filling layer by layer by the Superintendent's Representative. The failures were judged to be the result of design deficiencies including overly steep batters, the use of fill materials that were reactive and dispersive and inadequate erosion protection and the loss in density of the batter fills to have been caused in part by the "de-compaction" of reactive soils exposed to the elements.

5 HOMEBUSH AQUATIC CENTRE

5.1 INTRODUCTION

The Sydney International Aquatic Centre, constructed as part of the preparations at Homebush for the 2000 Olympics, consisted of a concrete framed structure housing the pools and associated facilities backed by a 14m high reinforced earth retaining wall. The structure was connected to the retaining wall by an access bridge. Short term extra seating was originally intended to be provided by means of an earth berm; generally triangular in section, up to 15m high and of arcuate shape in plan, over and behind the fill behind the RE wall.

In 1993, a deep clay fill consisting of residual clays, shaley clays and clayey shales won from excavations on the site was constructed behind the RE retaining wall. The work was undertaken by Thiess Contractors, under a project manager, who prior to this project, had several years experience with Coffey Partners. There were two types of fill placed:

- Fill in a reinforced earth block (REB), with reinforcing strips at about 0.7m vertically, that was required by the specification to be placed to at least 95% Standard maximum dry density (SMDD). The specification did not require any control on moisture content.
- Fill in an earth berm (Berm) that was required by the specification to be placed to at least 100% SMDD, again there was no requirement for moisture content control. The Berm formed an embankment behind and above the REB. It appears that for a short period during the middle of the project, a technician had applied the minimum requirement for the REB to the Berm.

It is noted that the more important fill, i.e. that in the REB, had the less stringent compaction requirement.

5.2 TEST PROGRAMS

Coffey Partners was engaged as the geotechnical testing authority (GTA) for earthworks control and undertook 104 tests on the fill between May 1993 and April 1994. The specification did not contain a clear definition of the frequency of testing required to be undertaken by the GTA; these tests are referred to as the "CPI " tests.

Not long after completion of construction, issues arose with performance of the reinforced earth wall that was adjacent to and intimately connected with both the fill and the Aquatic Centre. Arup was the designer of the structure and had a second suite of testing, referred to as the "Arup" tests, undertaken in 1995. These tests were undertaken by Australian Soil Testing (AST) and consisted of twelve tests undertaken completely within the Berm.

The upper levels of the REB and Berm were removed during modifications for the Olympic Games in 1998/99; the section of the Berm behind the lower portion of the REB was not removed. A further suite of testing was completed during these works by AST and is referred to as the "AST" tests. At the same time, the contractor engaged Douglas Partners (DP) to undertake parallel testing at many of the locations tested by AST, these tests are referred to as "DP" tests.

The results of these test programs are summarised in Table 4.

Table 4: Summary of insitu testing at Homebush.

	REB			BERM			
GTA	**CPI**	**AST**	**DP**	**CPI**	**Arup**	**AST**	**DP**
Number of tests	44	24	19	60	12	57	32
Test period	Aug 93 - Dec 93	Jan 99	Jan 99	May 93-Apr 94	1995	Dec 98 – Jan 99	Dec 98 – Jan 99
Test RLs	114.7-124.9	120-123.6	120-123.5	119.4-129	123.5-134.1	121.3-135.7	125.8-133.1
Field test	Nuclear	Sand	Nuclear	Nuclear	Not known	Nuclear	Nuclear
Lab test	Full compaction	Hilf	Hilf	Full compaction	Full compaction	Hilf	Hilf
DR	103.1±2.7%	87.9±20.0%	93.7±6.9%	101.7±3.7%	92.8±9.3%	95.6±4.8%	97.2±5.4%
MV	1.7±1.0%	0.1±1.2%	na	1.8±1.1%	-2.7±3.6%	0.1±1.7%	na

Note test results are given as the mean and standard deviation.

Notionally, there are four (three post construction) suites of tests undertaken by different GTAs on the same material (ie the Berm) and three suites on the REB. All test results were NATA certified. It can be seen that the distributions could hardly be more different. These differences led to a major contractual dispute and legal claim. Various opinions existed to explain the differences but these were not tested in, nor determined by, the Court and ultimately the matter was settled without resolving these differences.

5.3 THE REINFORCED EARTH BLOCK (REB)

Figure10 show cumulative plots of density ratio and moisture variation, respectively, in the REB obtained as part of the CPI and AST tests. Figures 10 (a) and (b) show all results and it can be seen that the density ratios obtained in the AST tests are very much lower than those in the CPI tests, in addition the moisture variation of the AST suite is spread about optimum and typically 2% wetter than in the CPI suite. Note in accordance with AS1287.5.4.1 moisture variations are quoted as optimum moisture content minus field moisture content (OMC – FMC) so that negative moisture variations represents field conditions wet of optimum.

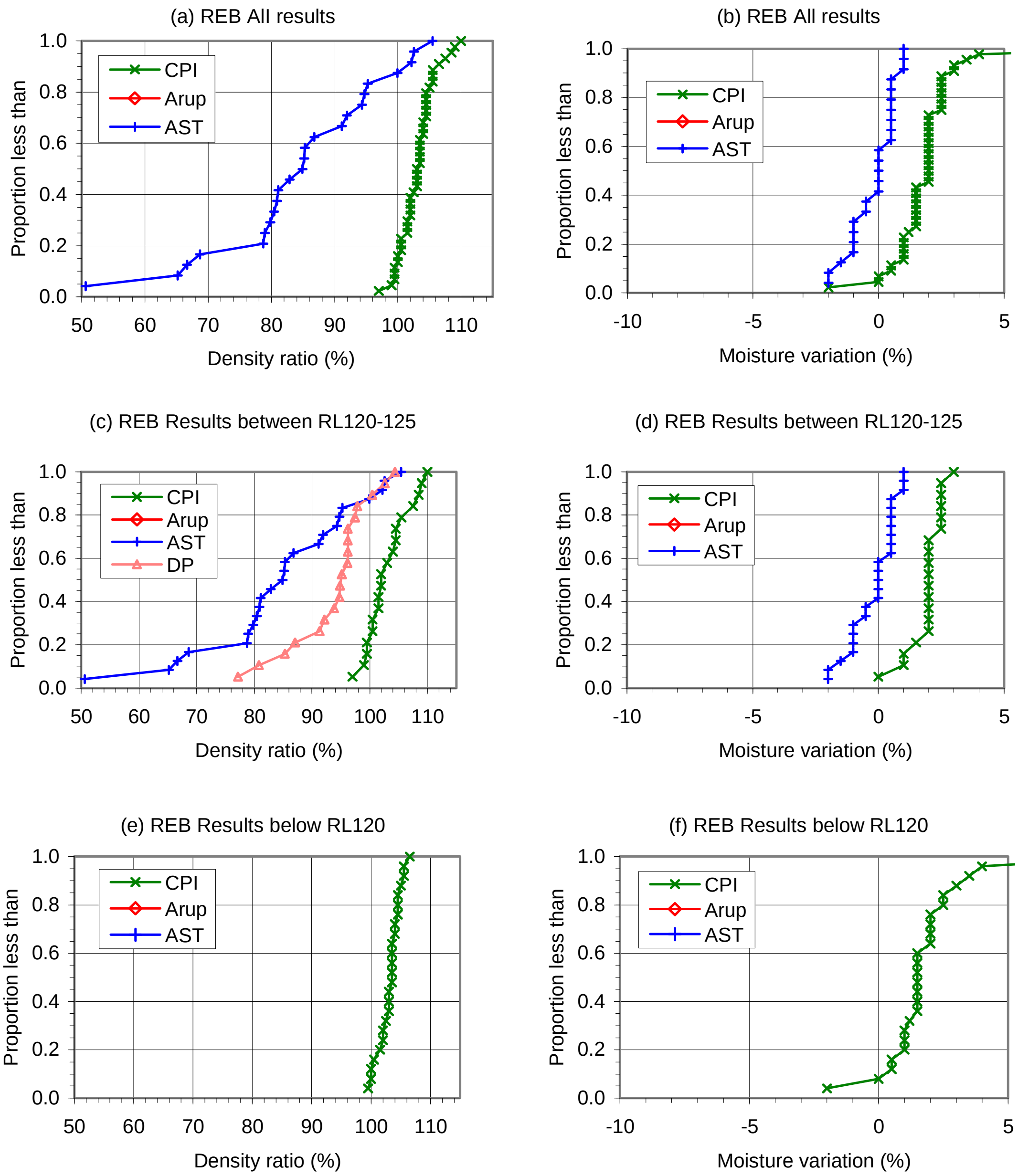


Figure 10 Cumulative distributions – Reinforced Earth Block: (a&b) All results; (c&d) Results below RL 120 m; (e&f) Results above RL 120 m

The entire REB was not deconstructed and thus the AST tests did not cover the same elevations as the CPI tests. It is useful to divide the test results into two groups, those obtained at elevations below RL 120 m, where only CPI tests exist, and those above this elevation, where the material was tested in both programs.

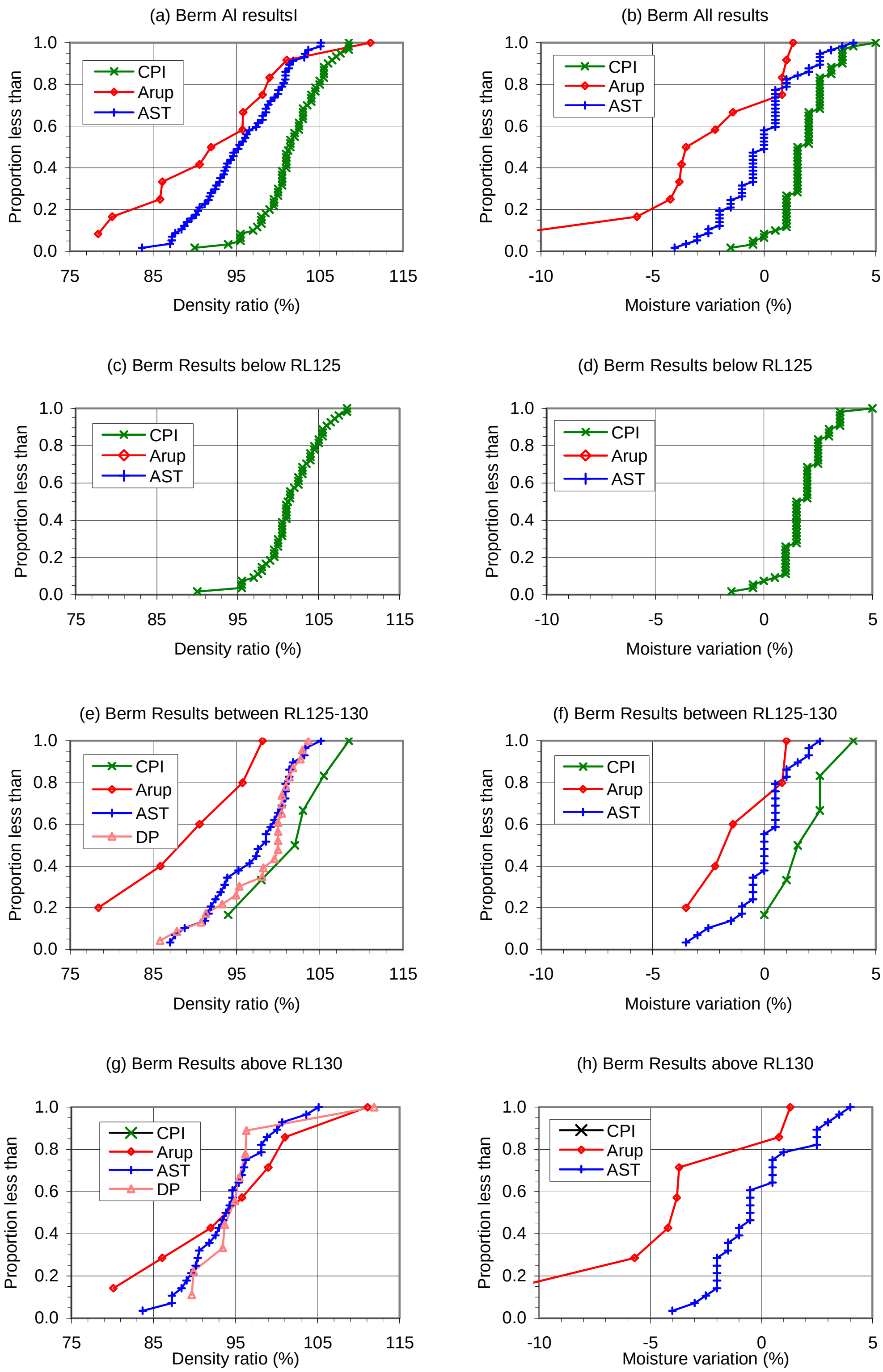


Figure 11 Cumulative distributions – Berm: (a&b) All results; (c&d) CPI results below RL 125 m; (e&f) All results between RL 125 m and RL 130 m; (g&h) Arup, AST & DP results above RL 130 m

Figures 10 (c) and (d) present cumulative plots of density ratio and moisture variation, respectively, for the tests below RL 120 m. The only test results are those of CPI and are typical of earthworks well controlled in relation to the compaction specification.

Figures 10 (e) and (f) present similar information for testing completed above RL 120 m. Figure 10 shows that, while somewhat more variable, the density ratios obtained by CPI are still consistent with earthworks that complied with the specification (given that failures were reworked). Notwithstanding this, the figure also shows that the AST test results obtained some years after construction indicate very variable earthworks with almost 30% of test results being below 80% density ratio and one as incredibly low as 50%. Douglas carried out 19 tests in parallel with AST which gave consistently higher DDR values, only one result was below 80%, and the shape of the upper 70% of the distribution (DR>90%) was similar to the CPI results albeit about 8% lower. Moisture variation results for the DP test results are not available to the authors.

5.4 THE BERM

Figure 11 shows cumulative distributions of density ratio and moisture variation, respectively, in the Berm obtained as part of the CPI, Arup and AST tests. All CPI test results are summarised on the top row plots, (a) and (b), including those that failed. However, the authors understand that all failures were "closed out" during the works by the area being reworked and, when directed, retested. It can be seen that the density ratios obtained in the AST tests are typically 6% lower than those in the CPI tests; the Arup tests are much lower again. The post construction DR results are markedly lower and more widely spread than the CPI tests, particularly the Arup results which range from 78% to 111%. The moisture variation of the AST suite is typically spread about optimum and about 2% wetter than in the CPI suite, whereas the moisture variation in the Arup tests is considerably more variable and wetter than either of the other two programs. Arup reported one result that was 11% wet.

The entire berm was not deconstructed and thus the AST tests did not cover the same elevations as the CPI tests. There were no CPI tests in the uppermost levels of the berm. The Arup tests were completed in test pits excavated from the surface and only covered a limited range of elevations. Hence, it is useful to divide the test results into three groups: those obtained at elevations below RL 125 m, where only CPI tests exist; those taken at elevations between RL 125 m and RL 130 m where all programs tested the fill; and those above RL 130 m, where the fill was tested only in the Arup, AST and DP programs.

Figures 11 (c & d) presents cumulative distributions of density ratio and moisture variation, respectively, for the tests below RL 125 m. The only test results are those of CPI and show the fill is essentially compacted to greater than 95% density ratio and some to a considerably higher ratio.

Figures 11 (e & f) show the same information for testing completed between RL 125 m and RL 130 m. Figure 11(e) shows that the density ratios obtained by CPI during construction are approximately 3% higher than that obtained by AST during deconstruction and between 9 and 15% higher than that obtained by Arup during its investigation. Figure 11 shows that the moisture content increased between 2 and 5% from dry of optimum during construction to wet of optimum during post construction testing.

Figures 11 (g & h) present the density ratios and moisture variation for the testing completed above RL 130 m where only Arup, AST and DP tests were completed. The density ratios have similar average values just below about 95% but show large ranges of 88% to 105% for AST and 80% to 111% for Arup but the moisture variation results of Arup are approximately 4% wetter than those of AST

5.5 PARALLEL TESTING

As stated previously, the Contractor engaged a separate GTA (viz DP) to undertake parallel tests to the Owner's GTA (viz. AST) during the 1999 tests. DP was not in full time attendance but completed 32 parallel tests in the Berm and 19 in the REB. Both GTAs adopted Hilf rapid compaction for the reference density, AST undertook sand cone testing and DP nuclear gauge testing for the field densities and the parallel tests were completed at essentially the same locations. Figure 12 (a) presents a plot of the density ratio pairs from the REB and Figure 12 (b) from the Berm. Figure 13 presents cumulative distributions of the difference between the density ratios in the REB and the Berm.

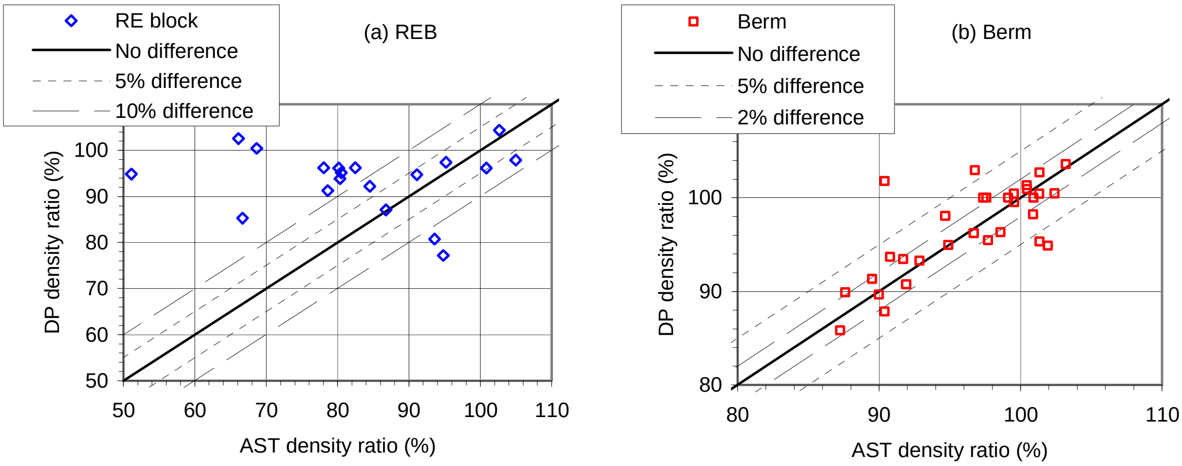


Figure 12 Density ratio of two GTA in parallel tests (a) REB; (b) Berm

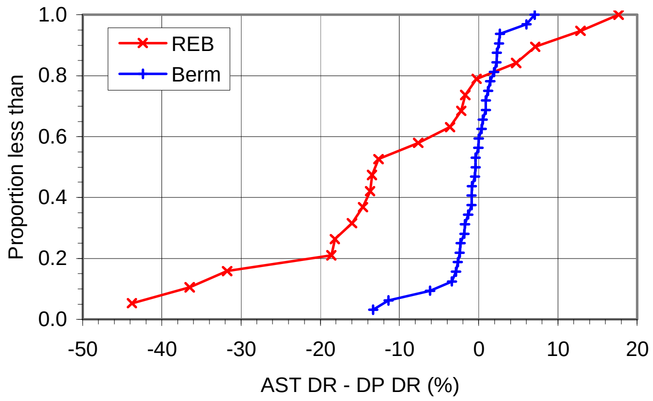


Figure 13 Cumulative histograms – Parallel test results

It can be seen that the results in the Berm differed by 2% or less in more than half of the tests (18 out of 32 or 56%) and by 5% or less in 27 of the 32 tests or 85%. In only 5 of the 32 tests (~15%) were the differences greater than 5%. This represents reasonable, but not great, agreement and suggests that the AST post-construction retests may have represented the state of compaction in the Berm at the time of deconstruction in 1999, several years after the fill had apparently been compacted to specification. On the other hand the results in the REB bear almost no relationship to one another with the statistics being almost reversed compared to those for the Berm. Only 2 of the 19 tests (10%) were within ±2% while 14 of the 19 tests (74%) differed by more than 5%. This suggests there can be little confidence that the post-construction retests reasonably represent the state of compaction behind the RE retaining wall. It is apparent that there was some aspect of the way in which parallel retesting of the REB materials was carried out that differed from that of the Berm for what were essentially the same testing protocols by the same GTAs in similar materials. The comments above are reflected in the fact that the standard deviation of the difference in the parallel tests is 3.3% for tests in the Berm and is 16% for the pairs in the REB. The difference is almost entirely due to differences in the measured field wet density.

Comparison of the construction control CPI tests and the post-construction DP retests for the REB indicates that the retest results are on average about 10% less than the DR values reported during construction

5.6 LIKE FOR LIKE TESTING

The locations of the various tests were known with sufficient accuracy to be able to assess the distance and elevation difference between any two tests. In those cases a relatively direct comparison of the test result taken at the time of construction with that taken some time later is possible and is referred to as “like for like testing”. The test programs were such that there were several tests in the post-construction AST and DP retest suites that were within 0.5 m elevation and 20 m or less in plan of a test in the construction control CPI suite. Such a pair of tests is effectively testing the same portion of fill but is not a "parallel" test. The AST-CPI test pairs are plotted on Figure 14 (a) and the DP-CPI test pairs on Figure 14 (b). The mean of the CPI tests is a density ratio of 101.8% and of the AST tests is 87.3%. In this figure some tests that are within 10 m of each other and at essentially the same level reported a density ratio of 100% by CPI during construction and less than 70% by AST during deconstruction. For the eight 10 m separation tests not one of the AST post-construction retests equalled or exceeded the CPI construction control tests and the differences in DR ranged from 1.6% to 34.8% with an average of 16.2%. For the seventeen 20m separation tests the differences in DR ranged between -6.9% and 52.9% with an average of 13.7%.

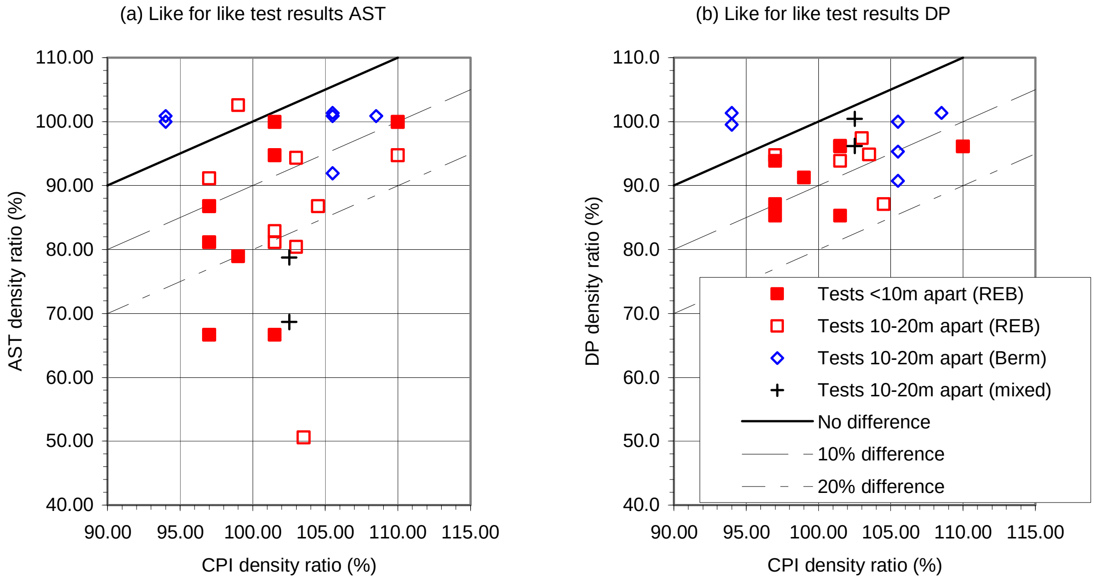


Figure 14: Like for like testing (a) comparison of AST-CPI density ratio results; (b) comparison of DP-CPI density ratio results.

5.7 CONCLUSION

The density testing carried out during construction of both the REB and the Berm are indicative of a reasonably well controlled earthworks where the fill was compacted dry of optimum. Retesting of the Berm about a year after completion indicated a reduction in DR by about 8% and a moisture increase of about 4.5% relative to optimum. Later retesting during deconstruction of the Berm indicated a loss in density of between 4% and 6% from construction with the material at about optimum. The results from parallel post-construction retesting were reasonably consistent. Although the results of parallel retesting of the REB during demolition were not consistent they nevertheless indicated a loss in compaction in the range of 10% to 15% with the material again at or about optimum.

Apart from the possible effects of working around the reinforcement, there is no readily apparent reason for the lack of consistency for REB retests when the same GTAs using the same test methods in what was virtually similar material produced test results only 10% of which agreed to within ±2% in DR. Parallel testing of the Berm in comparison yielded differences within ±2% in more than half of the tests.

Where it was possible to identify tests on essentially the same lot of earthworks that had been carried out during and after construction the retested results were consistently lower than those performed during construction by about 15% on average with considerably larger individual differences.

Overall these results confirm a trend for post-construction retesting to indicate an apparent loss in compaction over time and in some instances to the incredible extent of 30% to 50%. The general reduction in DR is not adequately explained by the moderate increase in moisture conditions.

6 BLACKTOWN RAIL UNDERPASS

In 1993 a Tensar reinforced Keystone wall was constructed as part of a rail underpass at Blacktown. Some years later concern arose over alleged movements in the wall and litigation was commenced. Three suites of density testing were completed and are summarised in Table 5.

All GTAs described the material as completely or extremely weathered shale. Cumulative distribution plots of the dry density ratio and moisture variation of the three suites of testing are provided on Figure 15. It can be seen that the distributions of the test results in 1996 and 1998 are very different from that during construction in 1993. Approximately half of the results having a DDR less than 95%, whereas all control tests during construction gave results greater than 95% and the tests indicate that the fill was 4 to 5% wetter than during construction. Figure 15 shows that the differences are essentially due to differences in the measured field densities. Again, it is possible that the working around the reinforcement has had an effect on the post construction testing.

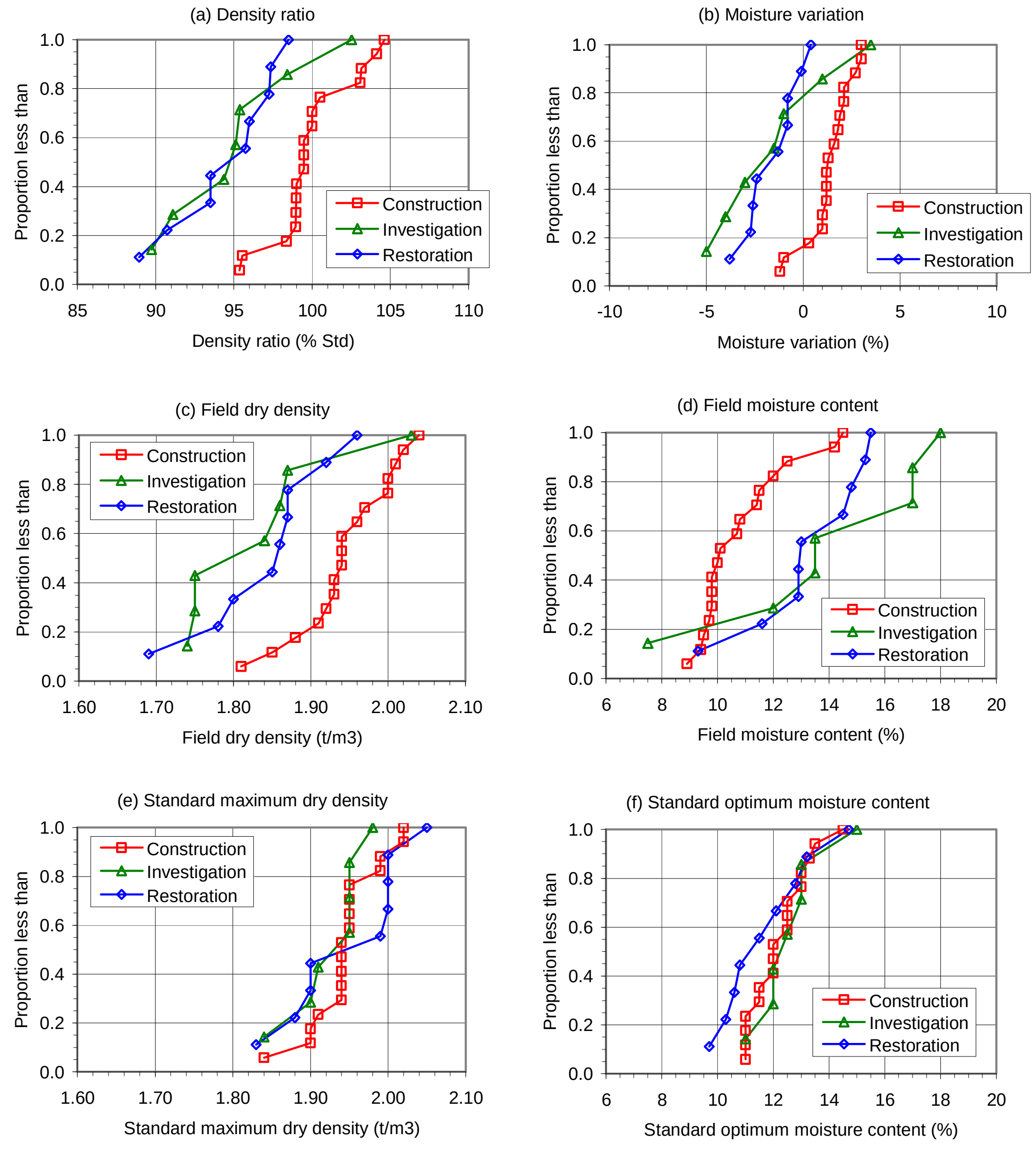


Figure 15: Cumulative distributions – (a) Dry density ratio; (b) Moisture variation; (c) Field dry density; (d) Field moisture content; (e) Standard maximum dry density; (f) Standard optimum moisture content

Table 5: Summary of testing at Blacktown

GTA	Regional Construction	Golder	Regional Restoration
Number of tests	17	7	9
Year	1993	1996	1998
Field test	Sand replacement	Nuclear	Sand replacement
Lab test	Full compaction	Full compaction	Full compaction
FDD	1.94±0.06 t/m^3	1.83±0.10 t/m^3	1.84±0.08 t/m^3
FMC	10.9±1.6%	14.1±3.7%	13.3±2.0%
SMDD	1.95±0.05 t/m^3	1.93±0.05 t/m^3	1.95±0.07 t/m^3
SOMC	12.2±1.0%	12.6±1.2%	11.7±1.6%
DR	100.0±2.9%	95.2±3.2%	94.4±4.3%
MV	1.4±1.2%	-1.6±1.4%	-1.4±3.0%

7 OUTER SYDNEY RESIDENTIAL SITE 1

As part of the initial stages of a large earthworks project at Outer Sydney Residential Site 1 up to 12 m of fill had been placed and compacted between September 2002 and September 2004. The fill comprised materials from Sydney excavations including residual clay, weathered to fresh shale and ripped sandstone. Remediated fill from site was also used in the earthworks.

The fill was placed under Level 1 control to a fill specification including compaction limits of between 98% and 102% Hilf (Standard) and a moisture variation between 2% wet and 2% dry. During the placement Hilf density testing was completed at a frequency of approximately 1 test per 500 m^3. Material which failed to meet the specification was either removed or reworked and retested. A total of 432 tests were completed. The GTA acting on the project was Soil Testing Services and was contracted to the Contractor undertaking the works. The insitu testing was undertaken using a nuclear gauge. All test results were NATA certified..

After handover a portion of the earthworks in February 2005 the developer, as part of its due diligence process, excavated two test pits, with density testing completed at various depths within the fill. The test pits were excavated using an excavator fitted with a bucket with a straight edge and no teeth. Nine (9) in situ density tests were completed by Bowler Geotechnical between 0 m and 4 m depths in one pit and seven (7) in situ density tests between 0 m and 3.8 m depth in another pit. Field density was measured using a nuclear gauge and the laboratory testing was determined by the full compaction method.

Cumulative distribution plots of the density ratio and moisture variation of the two suites of testing are provided on Figure 16 (a and b).

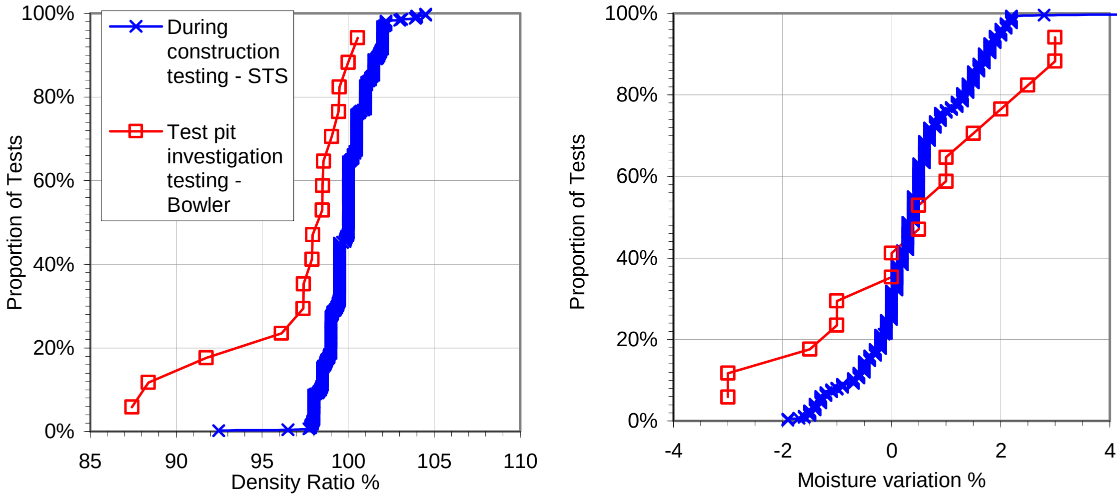


Figure 16 (a & b): Cumulative distribution plots for DR and MV of the two suites of testing at Outer Sydney Residential Site 1

It can be seen from the distributions that 75% of the test pit investigation test results are consistently about 2% lower than the during construction tests. The distributions of the moisture variation do not indicate a significant change in moisture condition of the fill material.

It is noted that the matter was resolved positively between the parties involved.

8 OUTER SYDNEY RESIDENTIAL SITE 2

Remediation of a large site in Outer Sydney Residential Site 2 required placement of fill up to approximately 6 m depth between October 2003 and October 2004. The fill comprised weathered to fresh shale and weathered to fresh dolerite.

The fill was placed under Level 1 control by Ground Technologies (GT) as the GTA to a fill specification including compaction limits of greater than 98% Hilf (Standard) and a moisture variation between 2% wet and 2% dry. During placement Hilf density testing was completed at a frequency of approximately 1 test per 500 m^3. The insitu testing was undertaken using a nuclear gauge (test method AS1289.5.8.1). Failed tests were either removed or reworked and retested. A total of 240 tests was completed in the area. The earthworks were undertaken under controlled conditions, the subgrade was inspected and approved prior to placement of fill. The earthworks and the earthworks testing were audited at regular intervals by Keighran Geotechnics and all test results were NATA certified.

Following handover of the area the developer as part of its due diligence process in November and December 2006 completed excavation of fourteen (14) test pits, with density testing completed at various depths within the fill by Soil Testing Services (STS). The maximum depth of the test pits was 1.8 m. Thirty five (35) tests were completed at various depths in these test pits. The in situ density was measured using the sand replacement method. The maximum dry density and optimum moisture content were determined by full compaction (standard). Cumulative distribution plots of the density ratio and of MV of the two suites of testing are provided on Figures 17 (a & b).

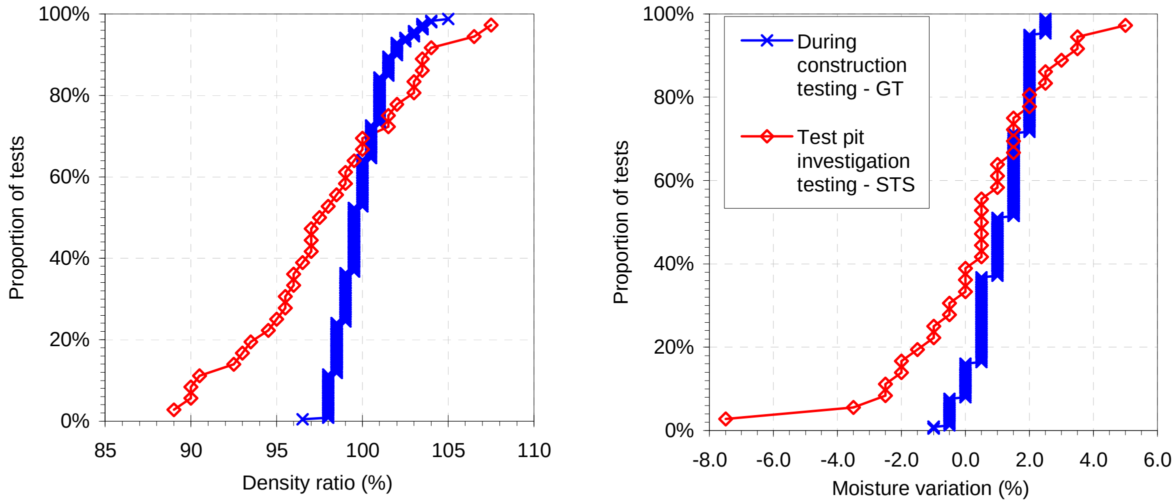
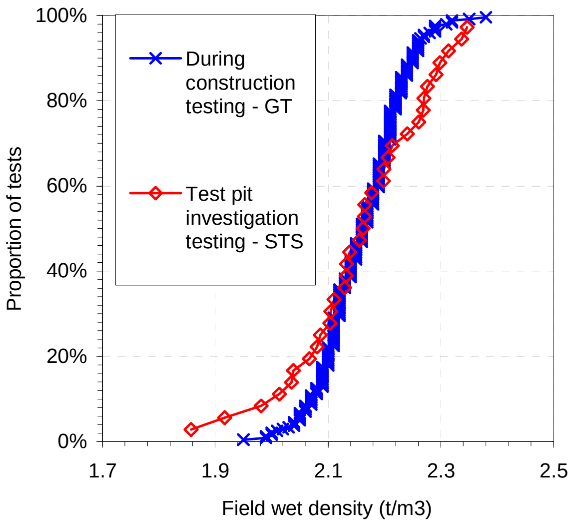


Figure 17 (a & b): Cumulative distribution plots of DR and MV for the two suites of testing at Outer Sydney Residential Site 2

Figures 18 and 19 compare the field wet densities (FWD) and peak wet density (PWD) distributions from the two test suites. The PWD of the test pit investigation tests has been calculated from the maximum dry density and optimum moisture content results reported in the test result sheets. As can be seen the peak wet density for the STS test pit testing is consistently greater than the original GT testing. The resulting STS density ratios are therefore less than density ratios from the original GT testing.

Figure 20 presents the MDD versus the OMC for the test pit investigation testing. Lines for 0%, 5% and 10% air voids are also shown on the figure for an S.G. of 2.8. As can be seen some of the test results require a particle density of up to 3.3 t/m^3 to be meaningful.

It is noted that the matter was resolved positively between the parties involved.



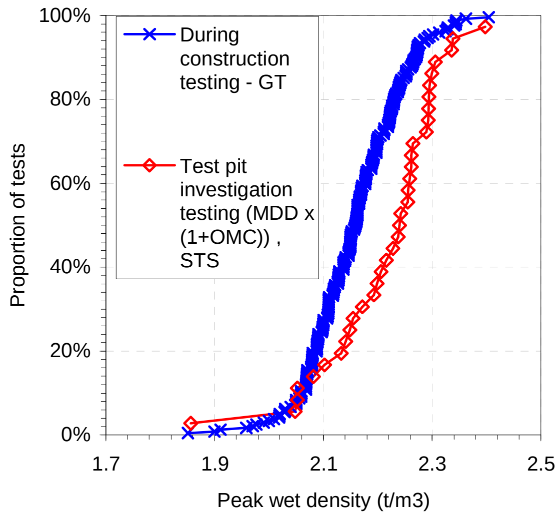


Figure 18: Cumulative frequency distribution plots of field wet density of the two suites of testing at Outer Sydney Residential Site 2

Figure 19: Cumulative frequency distribution plots of peak wet density of the two suites of testing at Outer Sydney Residential Site 2

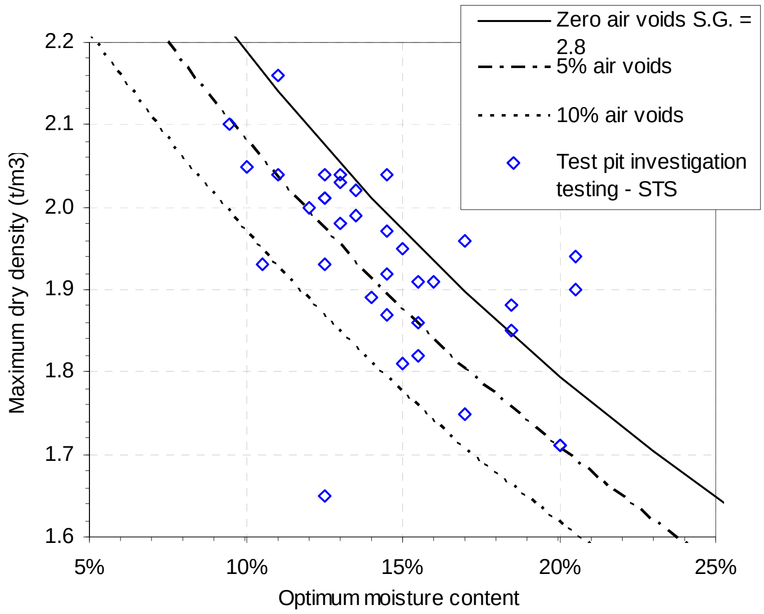


Figure 20: MDD versus the OMC for the test pit investigation testing at Outer Sydney Residential Site 2

9 SOMERSBY

As part of the investigation of a failed road embankment in Somersby, a well controlled and detailed programme of density testing of the intact portion of the road embankment was undertaken. The embankment was constructed of fill comprising clayey sand with sandstone fragments up to 50 mm in diameter. Construction records showed that the fill had been placed and compacted under controlled conditions.

Robert Carr & Associates (RCA) completed density testing at 42 locations within the road embankment fill. The tests were completed on exposed benches at various levels through the fill embankment. At each location RCA measured the field density by means of sand replacement (SR) (AS1289.5.3.1) and nuclear gauge methods (NG) (AS1289.5.8.1).

Laboratory testing was by the full compaction method (AS1289.5.1.1) and all test results were NATA certified.

At 10 of the locations tested by RCA, duplicate field density and bulk sampling were undertaken by STS. The duplicate density testing was carried out as close as possible to the original RCA test location by means of the sand replacement method (AS1289.5.3.1). The laboratory component was by the full compaction method (AS1289.5.1.1).

RCA and STS undertook the testing at the same time and the positions and levels of the in situ density tests were surveyed. Table 6 summarises the testing at Somersby.

Table 6 Summary of testing at Somersby

Location	RCA	STS	Plan distance between tests(m)	Elevation difference between tests(m)
1	SR,ND, SMDD	SR, SMDD	0.315	0.035
2 - 11	SR, ND, SMDD	-	-	-
12	SR, ND, SMDD	SR, SMDD	1.315	0.114
13	SR, ND, SMDD	SR, SMDD	0.817	0.012
14	SR, ND, SMDD	SR, SMDD	0.612	0.007
15 - 21	SR, ND, SMDD	-	-	-
22	SR, ND, SMDD	SR, SMDD	0.836	0.061
23	SR, ND, SMDD	SR, SMDD	0.598	0.007
24	SR, ND, SMDD	SR, SMDD	0.669	0.002
25	SR, ND, SMDD	SR, SMDD	0.787	0.013
26 - 34	SR, ND, SMDD	-	-	-
35	SR, ND, SMDD	SR, SMDD	0.869	0.011
36	SR, ND, SMDD	SR, SMDD	0.992	0.013
37	SR, ND, SMDD	-	-	-
38	ND, SMDD	-	-	-
39 - 42	SR, ND, SMDD	-	-	-

The testing undertaken is of particular interest as it allows comparison of duplicate testing completed at the same time at essentially the same location. In particular it allows an assessment of the effect of in situ density testing method and the variation between testing completed at two NATA certified commercial laboratories.

It is important to note that the testing at the site was undertaken as part of a coronial inquest for forensic purposes. It follows that the site preparation, in situ testing, sampling and laboratory testing was completed by the laboratories more experienced laboratory technicians striving to produce test results of the highest quality. This programme has provided a set of density test results that are rare, if not unique, and results which represent testing standards that are unlikely to be achieved in routine day to day earthworks control.

Figure 21 (a to g) are scatter plots presenting a comparison between the field measurements as completed by RCA and STS at the same locations. Figure 22 (a and b) provides a comparison between the laboratory compaction measurements by RCA and STS on duplicate samples. Figure 23 (a to d) compares the density test results by RCA and STS using the two standard in situ test procedures, namely sand replacement and nuclear gauge in situ density testing methods on adjacent test sits, referred to herein as duplicate samples.

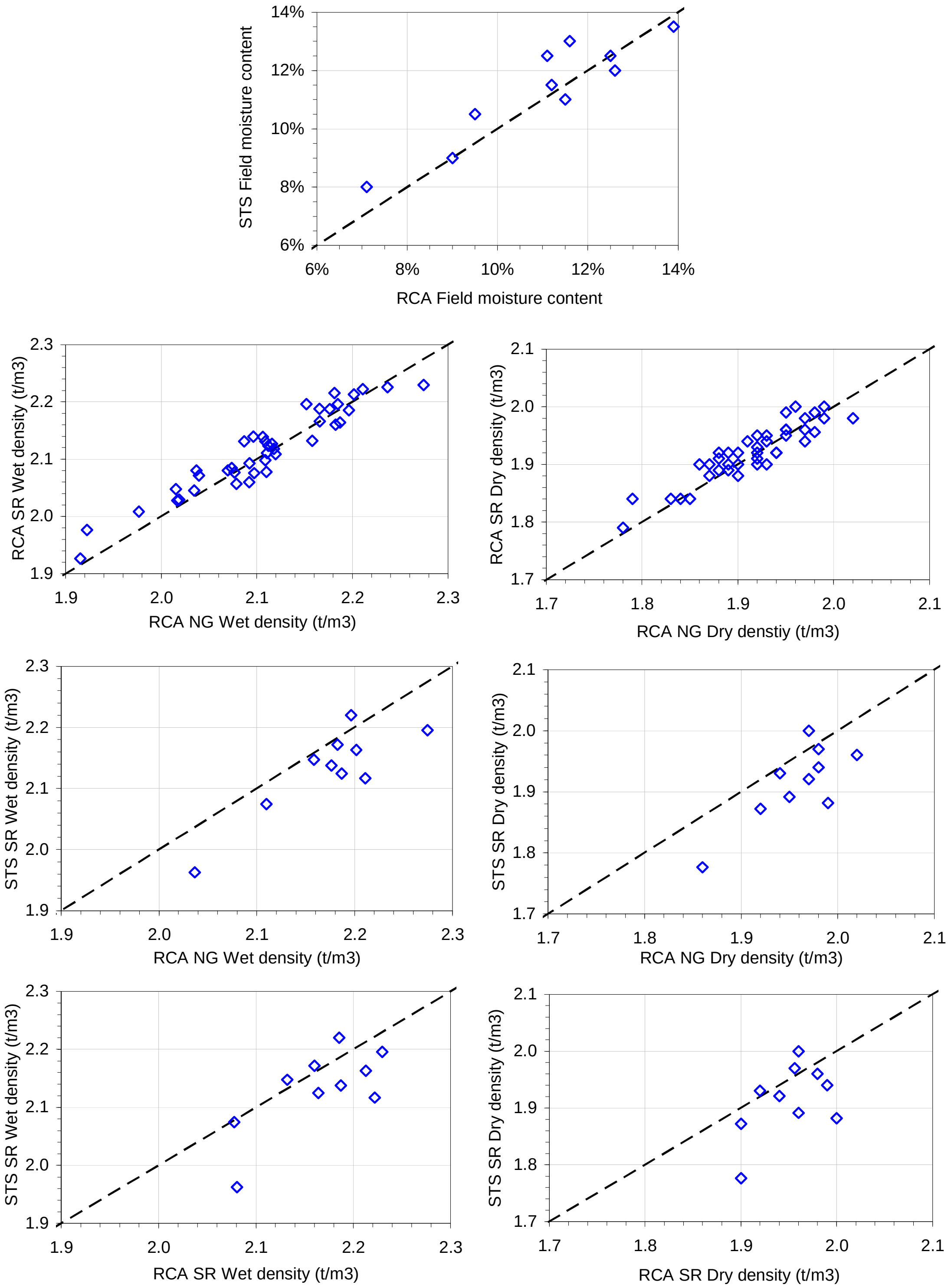


Figure 21 (a to g): Comparison of field measurements by RCA and STS using two different in situ density testing methods at the same location.

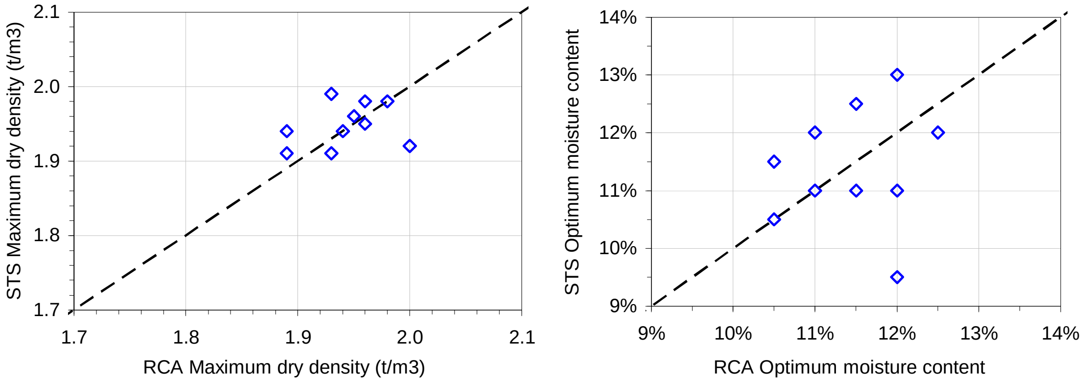
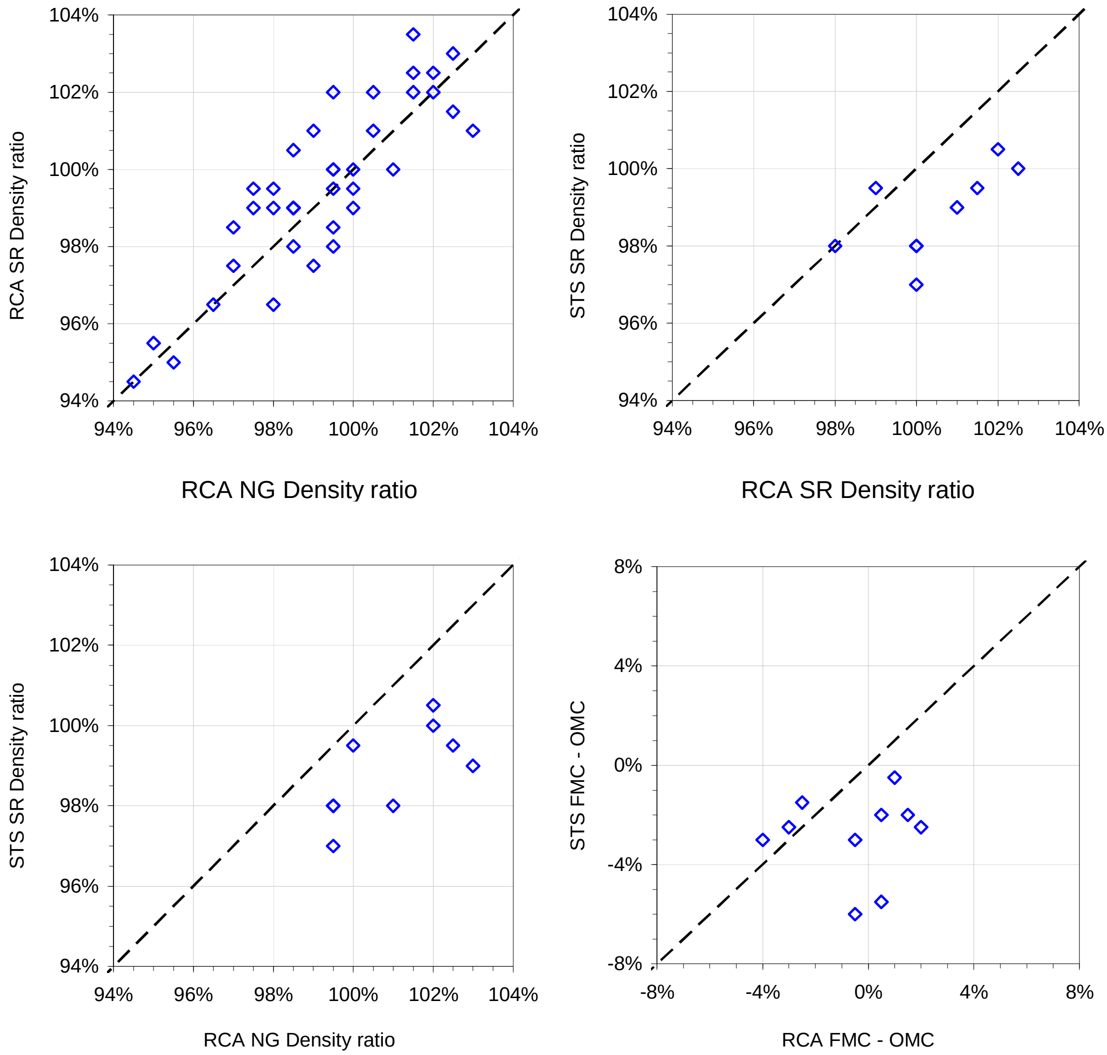


Figure 22 (a and b): Comparison of laboratory compaction measurements by RCA and STS on duplicate samples.



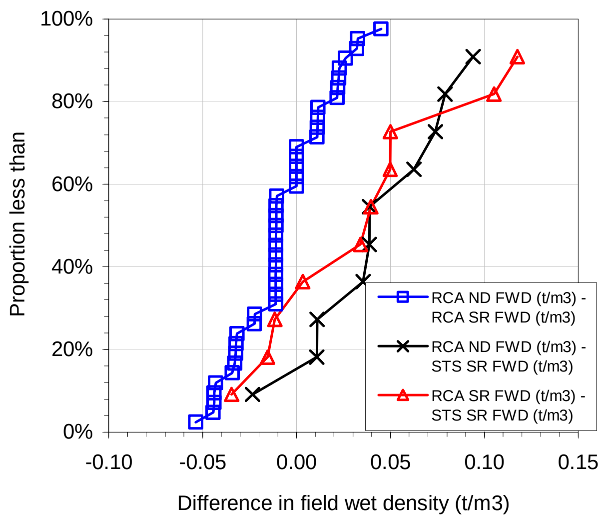
Figures 23 (a to d): Comparison of density test results by RCA and STS using various test methods on duplicate samples.

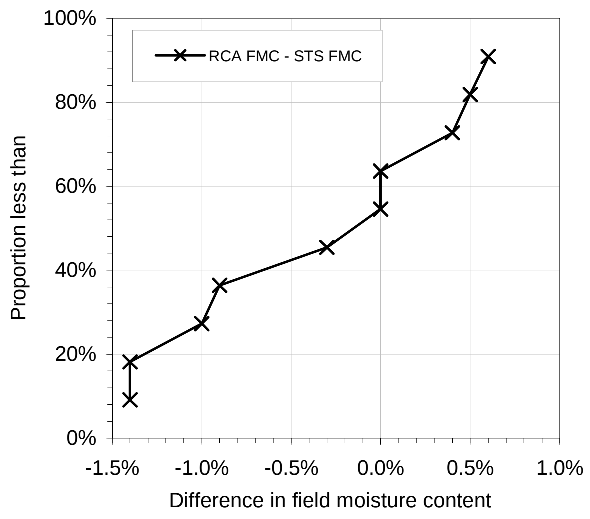
Table 7 below presents a summary of the difference in test results on the 10 duplicate samples.

Table 7 Summary of difference in test results on 10 duplicate samples at Somersby

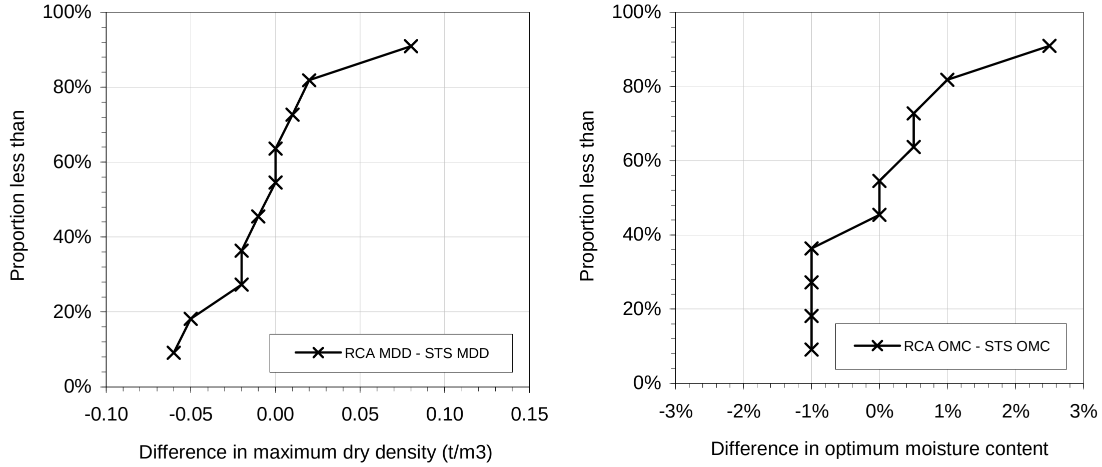
Parameter	Laboratories	Test Methods	Difference	Mean of difference	Standard deviation of difference
Field moisture content	Two laboratory (RCA, STS)	One test method (Measured in laboratory)	RCA - STS	-0.35%	0.77%
Field wet density	One laboratory (RCA)	Two test methods (ND and SR)	RCA ND - RCA SR	-0.007 t/m^3	0.024 t/m^3
	Two laboratory (RCA, STS)	Two test methods (ND and SR)	RCA ND - STS SR	0.042 t/m^3	0.036 t/m^3
	Two laboratory (RCA, STS)	One test method (SR)	RCA SR - STS SR	0.034 t/m^3	0.050 t/m^3
Standard Maximum Dry Density	Two laboratories (RCA, STS)	One test method (AS1289.5.1.1)	RCA - STS	-0.005 t/m^3	0.039 t/m^3
Optimum Moisture Content	Two laboratories (RCA, STS)	One test method (AS1289.5.1.1)	RCA - STS	0.05%	1.14%

Figures 24 (a and b), 25 (a and b) and 26 (a to c) present cumulative frequency distribution plots of the difference between field measurements, laboratory measurements and calculated values respectively for the RCA and STS testing as undertaken on each duplicate sample.





Figures 24 (a and b): Cumulative frequency distributions for the difference between field test results completed by RCA and STS on the duplicate samples.



Figures 25 (a and b): Cumulative frequency distributions for the difference between laboratory test results completed by RCA and STS on the duplicate samples.

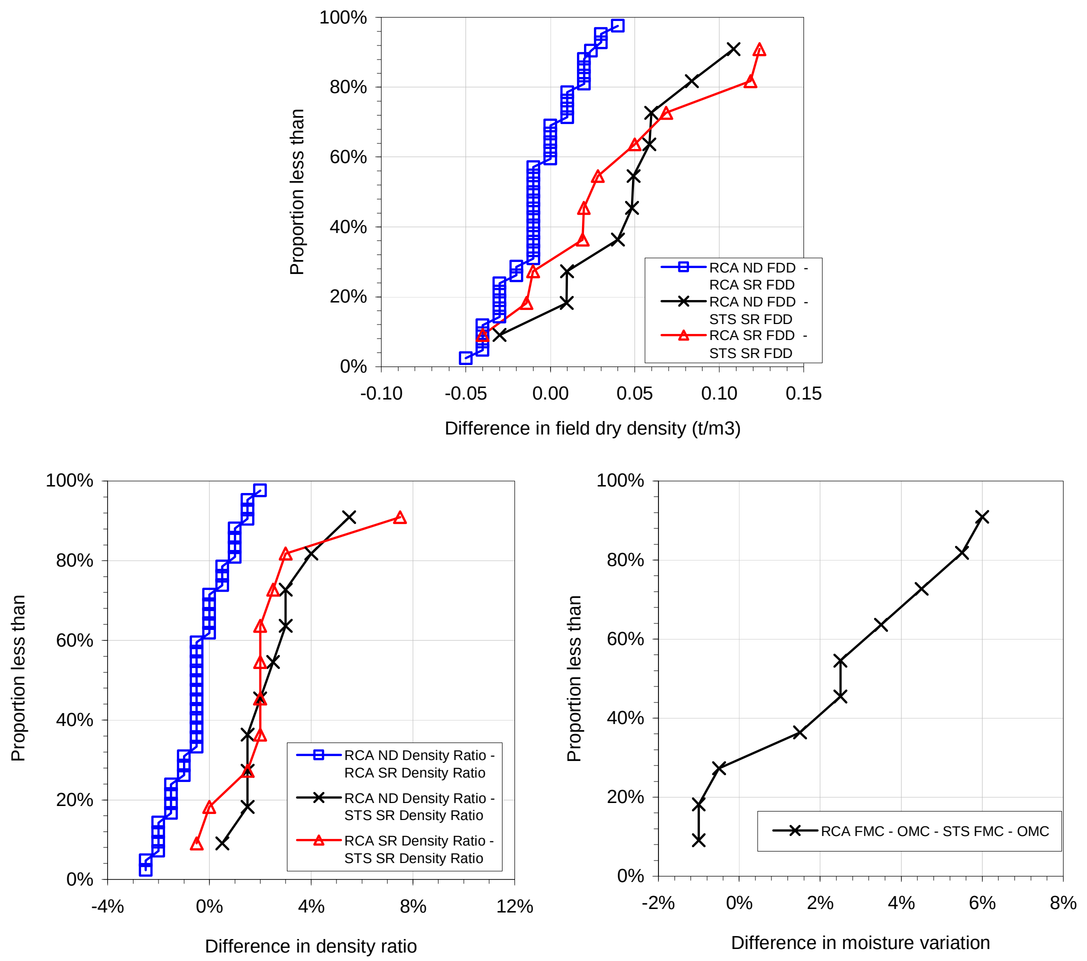


Figure 26 (a and c): Cumulative frequency distributions for the difference between calculated results from field and laboratory testing as completed by RCA and STS on the duplicate samples.

The standard test method for determination of the SMDD and OMC of a soil sample (i.e. test method AS1289.5.1.1), provides guidelines on the precision of the test (ref. Table 3 of AS1289.5.1.1). It provides limits with regards to the acceptable range of two results expressed as a percentage of the mean value. Table 8 presents the mean and 95th percentile of the difference in SMDD and OMC expressed as a percentage of the mean for the 10 duplicate samples tested by both RCA and STS. The precision limits provided in Table 3 of AS1289.5.1.1 have also been reproduced. Note 2 of Table 3 states *"The values show are based on test results from a wide variety of soils but in some cases, such as heavy clays, these values may be exceeded"*. The results in Table 9 indicate that the range of results for the testing at Sommersby falls within those expected by the test method. Figure 27 (a and b) presents the distribution of the difference in SMDD and OMC expressed as a percentage of the mean value for the 10 sets of duplicate samples.

Table 8: Repeatability and reproducibility of the tests at Somersby expressed as a percentage of the mean value

Parameter	Difference as a percentage of the mean value	Mean of difference as a percentage of the mean value	95th percentile of difference as percentage of the mean	Precision limits from AS1289.5.1.1: 95th percentile for single operator	Precision limits from AS1289.5.1.1: 95th percentile for multiple operator
Standard Maximum Dry Density	$\frac{\text{ABS (RCA – STS)}}{\text{[MEAN(RCA, STS)]}}$	1.4%	3.6%	2%	4%
Optimum Moisture Content		7.5%	16.9%	10%	20%

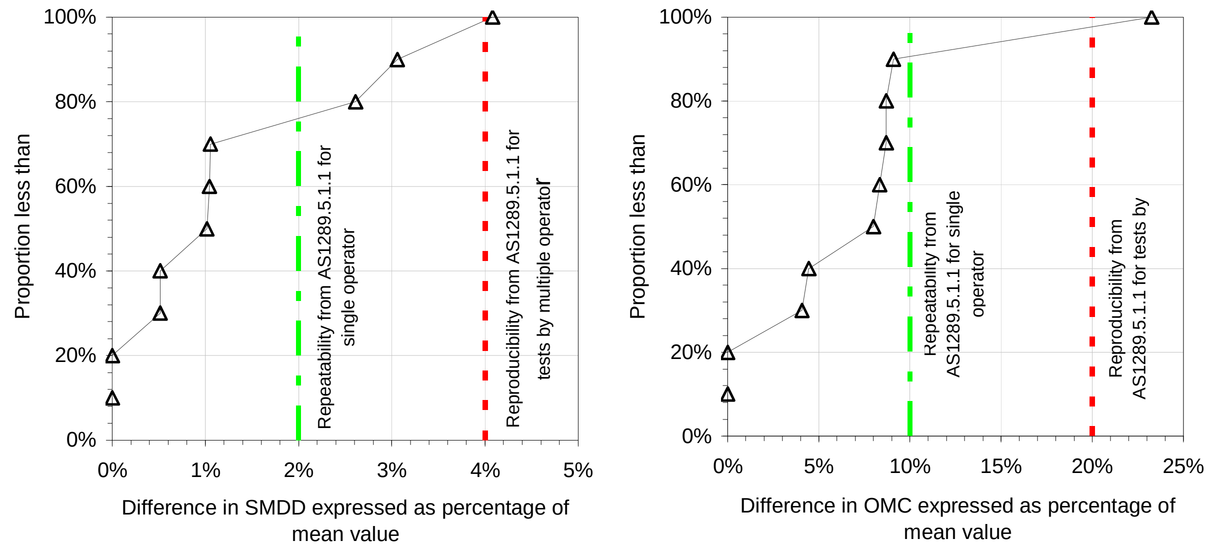


Figure 27 (a and b): Cumulative frequency distributions for the difference between SMDD and OMC as determined by RCA and STS on the duplicate samples expressed as a percentage of the mean value.

10 SIGNIFICANCE OF THESE EXPERIENCES

These examples represent a situation that is common in civil earthworks where fills have been apparently properly constructed by ostensibly competent civil contractors, with control testing by independent NATA registered geotechnical firms, under the overall supervision and approval of experienced, independent engineers. Yet, in each instance, retesting after completion has apparently produced compaction results below, and in some cases spectacularly below, the specified minimum achieved during construction. Completed fills are not usually subject to retesting, as has occurred in these cases, but the factual evidence presented here is sufficient to raise questions as to the long term performance of clay fills that do not appear to have been recognised or to be capable of ready explanation.

The purpose of this paper was to present a body of factual data that credibly portrays the "apparent loss of compaction" dilemma which arises with the post construction retesting of clayey fills. It is beyond the intended scope of the paper to canvass the factors that may be involved - that is to be the subject of a subsequent companion paper. However, it is possible to draw some preliminary conclusions that may be useful in practice for situations similar to those presented herein:

- When a post-construction retesting programme is being planned, consideration should be given to developing a methodology which will allow a direct comparison of retests with control tests carried out during construction as a means of quality control.
- Blind, parallel testing by different independent GTAs should be employed as part of any retesting programme.
- Retesting is frequently carried out in test pits excavated into the fill and particularly careful attention is required to minimise the effects of disturbance. Particularly careful attention should be applied in accurately locating the position of the retest sites.
- Because retesting is often used forensically particular care is recommended in selecting the testing protocols. The authors recommend that test standards be implemented to the fullest extent and that full laboratory compaction tests be adopted rather than the rapid procedure.
- Close engineering control of retesting by experienced geotechnical engineers and the use of well experienced geotechnicians for the execution of all phases of the retests is recommended. Retesting should not be seen as the simple straightforward performance of a routine test under normal commercial pressures but as an important component of forensic investigation. In the authors' view the level of engineering control and scrutiny has often been sadly lacking.
- It should be recognised that, post construction, near the surface density ratio may be significantly affected by climatic effects.
- It should be recognised that DR tests are unlikely to have the accuracy that the Standard precision might be assumed to imply.
- Where unusually low DR results are obtained from retesting the results should be treated with caution and a thorough review should be carried out of all testing details. The occurrence DDR values in the low 80%s and below in an engineered fill should be viewed as a potential harbinger of inadequate and/or erroneous testing procedures.
- In assessing the results of post construction retesting there should be no latent assumption that such results are more reliable than those obtained during construction without clear and well-considered reasons justifying that assumption. It has been the experience of the authors that there is commonly an implied, and apparently unwitting, bias in favour of the results post construction retesting as compared to the results obtained during construction. The evidence presented from our experiences would indicate that there should, if anything, be a positive bias in favour of the construction time test results.

It is the authors' experience that in most instances it has been virtually impossible to determine the locations of density tests carried during construction with any degree of precision. Test locations are often recorded in areal terms without coordinates or relevant distance measurements and elevations are often in terms of courses or lifts rather than as RLs. This has the effect of restricting the usefulness of these tests for forensic purposes. It is recommended that the locations and elevations of density tests carried out during placement be determined with sufficient precision so that test sites can be confidently determined post construction and that they be recorded on scaled drawings.

11 REFERENCES

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