A CALL FOR UPDATING SOME ROAD BUILDING PRACTICES "ARE WE THINKING IN THE 21ST CENTURY OR STILL LANGUISHING IN THE DISTANT PAST?"

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ABSTRACT

Criticism is levied at some current specifications, which are often wrong, out of date or ambiguous. The aim of the paper is to highlight the deficiencies and to propose solutions. Compacted layer thickness and moisture in fills should not be specified. Cut, fill and spoil relative volumes must be defined. Nuclear gauge peg error must be recognized. Equipment and test method variance should be assessed. Porosity and interlock should replace density requirements. A mathematical assessment of Modified density is proposed. Current G1 grading requirements should be modified for heavy vehicle loadings.

1. INTRODUCTION

Throughout the years of the writer's experiences as a road engineer he has become painfully aware of some practices relating to construction of roads today which are quite old fashioned and which appear to emanate from "rule of thumb" and "grandfather practices". The aim of the paper is to highlight the deficiencies and to propose solutions.

Drawing attention to some of these out of date or off line practices may, I hope, initiate a debate as to whether a "think again" in road construction techniques is perhaps long overdue. The writer has taken the liberty of illustrating below some of the conditions relating to present day Road Contracts that surprisingly require a re-think. In all fairness many contracts do not suffer from all the instances cited here but far too many contracts and Road Engineers still follow and think along these too old and in the author's opinion outdated lines.

2. COMPACTION

2.1 A Re-Think Relating to Fill Compaction

Although COLTO provides for compaction of layers of both below and above 200mm in thickness, often specifications for fill compaction demand that all layers shall be not more than 200mm loose. A contractor using a vibrating roller which successfully compacted earth fill in lifts of 750 to 1000mm was refused payment because the specified lift thickness was not met! Image the contractor's chagrin when having to construct several fills of over 30m in height! If required density is met, surely the work should be accepted, even if the specification relating to layer thickness is not met. All too often good work is turned down because the method is not met. A sound specification should contain a provision that allows work to be accepted if the methods employed achieve specified requirements even if they differ from specified methods. Although it is quite acceptable to require foundation layers to be done in compacted layers of say 150mm, the lift thickness in fills should be left to the party doing the work to fit in with the equipment he proposed to use. This is even

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more true when using impact compaction.

2.2 Compaction at OMC

It is still common practice amongst many road engineers to specify that compaction shall be done at OMC, which in the author's opinion is quite wrong. The moisture content at which compaction must be done surely rests with the contractor and he should be entitled to use whatever moisture suits his machine and enables him to achieve required density. It is not unknown to the writer where a contractor's compacted work well above specified density was rejected because it was not compacted at OMC, which was a specified requirement. It is a fact that when a vibrating roller is used for compaction, an excess of water above say a 50% degree of saturation can and does actually reduce the roller's efficiency.

The term OMC generally refers to the point of moisture content during the laboratory Modified test at which effective saturation (S = 80%) is taking place. This moisture content relates to the falling mass used in the laboratory and bears no relationship with field compaction equipment. It may be excusable to use this optimum moisture content to achieve a maximum density when a steamroller was used but modern vibrating rollers use an entirely different compactive technique where lubrication is replaced by vibratory effort. Compaction moisture must surely rest with the contractor, as he should be entitled to use whatever moisture he finds suitable for the equipment he proposes to use.

The specification should allow compaction moisture content to be at the contractor's discretion and omit any clause that requires compaction to be done at OMC or any other specified moisture content. Any contractor when tendering for road works should take note of these handicaps which are often present in current project specifications and draw attention to these shortcomings during tendering or as soon as is possible if his tender is accepted but prior to finally signing the contract agreement.

2.3 Cut, Fill and Loose Soil

Many specifications and schedules of quantity do not clearly define a relationship between a cubic meter of cut, a cubic meter of fill and a cubic meter of spoil. Any specification, which does not schedule or define these relationships is sorely lacking. Unless clearly agreed by all parties concerned to the contrary I suggest that all future related measurements in earth works should be in accordance with the schedule as depicted in Table 1. Note that the unit of measurement here clearly defines each cubic meter of material.

Table 1 Suggested relationships between various cubic meters of material

State	Unit	Rock	Coarse grained soil P075 > 50%	Fine grained soil P075 < 50%
Natural	Cm^3	1.0	1.0	1.0
Loose	Sm^3	1.75	1.25	1.4
Compacted	Fm^3	1.4	0.8	0.8

Naturally, should any of these relationships prove to be out of line they may be changed provided all parties are in full agreement.

3. SAND REPLACEMENT AND NUCLEAR GAUGE ERRORS

The writer is a strong advocate for accuracy, fairness and consistency in specifications as well as clarity and the avoidance of ambiguity. All too often specifications do not always follow this rule and the contractor may often be penalized for shortcomings over which he has no control.

The sand replacement test for density is generally not accepted when testing density for acceptance of crushed stone layer work as it is subject to error when the walls of the test hole show galleries, which may be caused when large aggregates are removed from the sides of the hole. These may not be filled by the replacement sand, which has an angle of repose of about 30^o. The author is in agreement here but very careful preparation of the test hole can and has eliminated this error. As the alternative to the sand replacement test the Nuclear Gauge is specified for use. This is fair enough but I have yet to see the specification which allows for “peg errors” when making the hole within the layer for inserting the radioactive probe. The insertion of a peg into a compacted stone layer destroys interlock and consequently the density of the layer to be measured. The greater the coarseness and density of the material the greater is this disturbance and a drop in density occurs. In over 300 tests performed by the writer to illustrate probe hole peg errors these can range from below 1,5 to over 4 percentile points in Relative Compaction depending on the particle size and degree of porosity within the layer tested. It is a concern that all too often a contractor’s work is rejected or he is expected to compact to a density of over 90% of the Standard Control Density for the material because the nature of the test procedure destroys the very density it is to measure. Small wonder acceptable density is so often difficult to achieve. Even drilling a hole for the nuclear probe also produces errors in coarse-grained materials.

Although the sand replacement errors are almost impossible to assess the peg- error can generally be measured in sufficient tests to obtain an acceptable average value. This is done by constantly re-establishing a series of pre-formed test holes during roller passes. Placing the nuclear probe in these holes will give a reasonably error-free reading. To demonstrate the magnitude of error, a newly established peg hole about 350mm from any preformed hole with the position of the test gauge rotated 180^o so as to cover the area previously tested will give a reading less than that obtained using the preformed hole. This difference is directly as a result of the disturbance caused by the peg when driven into the fully compacted material. An average of about six to ten such errors may be regarded as the “Peg-error” which, when added to all density readings for the material will compensate for peg disturbance. This error increases with: Density; Particle size; Particle shape as well as Moisture content and should be re-established when any of these conditions change.

4. RELATIVE DENSITY OF PARTICLES

The standard density used in judging a G1 layer should be Bulk Relative Density for the material and NOT solid or apparent density (COLTO still specifies 88 percent of Apparent Density for G1 layers instead of Bulk Density, which is quite wrong!). Although apparent relative density is relatively easily obtained in the laboratory it does not represent the actual relative density of the soil particles. The volume of any cracks or fissures in soil particles is part and parcel of the particle volume and does not contribute to the voids or porosity within the soil matrix. If the fissures are filled with air the relative density of the particles is Bulk relative density (G_{bk}) and when filled with water the relative density is slightly greater than G_{bk} and is in fact Saturated relative density (G_{sat}). If q represents the ratio of the mass of water in the fissures to the particle mass, G_{sat} is given by:

$$G_{sat} = G_{bk}(1 + q)$$

When a soil is compacted, density in itself is not a measure of strength. The degree of particle interlock or particle togetherness (L) definitely is. Interlock is given by: density divided by bulk relative density when the soil is not saturated and saturated relative density when saturated (This latter case is theoretical only as density here assumes moisture in the fissures but surface dry solids). As the soil we deal with is generally on the dry side of saturation it is safe to use G_{bk} for the relative density of the particles. As the Degree of Interlock (L) and its alternative Porosity (n) are direct measures of strength it recommended that Degree of Interlock or Porosity should replace the term Density when soil strength is considered. The degree of interlock (L) is given by:

$$L = D/G_{bk}$$

And porosity is given by:

$$n = 1 - L$$

It is a pity that the indicator tests which include Atterberg limits and grading do not also include Relative Density as the use of this parameter with density gives a direct measure of strength. Current practice requires field density to be compared with a standard density, which in turn after extensive laboratory tests has been connected to a strength value (ie. CBR).

5. STATISTICAL ANALYSIS FOR ACCEPTANCE

Errors in the measurement of field densities for acceptance purposes (or for the inevitable variances in Relative Compaction values for the different test positions within a completed layer section) are accounted for in COLTO in section 8000 where statistical analysis is applied. While the writer quite readily accepts this procedure it is in his opinion not quite fair to the contractor who does the work.

The variance as mentioned above is the result of several separate variances all of which contribute to this final variance. These different variances may be present in any one or more of the following conditions:

a. Within the soil making up the layer tested.
b. Within the method of testing the density of the layers.
c. Within the method of testing the standard for calculating relative compaction.
d. Within the Workmanship Capability of the Contractor.

It is interesting to note that the contractor only has control of the last item for variance and yet his work may and often is rejected while being based on all four of the above conditions for variance. When testing equipment is faulty or testing techniques are slack or materials are highly variable, the contractor may be unduly penalized for factors outside his control. Surely, any final statistical form of acceptance or rejection should penalize the contractor only for that over which he has control, i.e. workmanship. He is penalized quickly enough when his work fails to meet requirements. He should likewise be compensated for work done satisfactorily to satisfy specified requirements.

When field density is accepted statistically it ensures that only a specified small amount of work, which should be rejected, is in fact accepted. In order to meet this requirement the contractor has to compact the soil to above the specified minimum due to soil, testing and workmanship variances. The soil and testing variances are outside the contractor’s control and work done by him to overcome these should surely be paid for. These variances can be estimated as discussed in the following sections.

5.1 Density Instrument Variance

Although tests for density in the field may vary from test to test the repetition of tests at one spot also produces variances. A series of one spot density tests, which eliminates soil variance, done with a freshly calibrated nuclear gauge merely by re-pressing the starter button without moving the instrument involving some 20 repetitions of readings yielded the results depicted in Table 2.

Table 2 Results of repetitive nuclear gauge readings

Soil type	No. of repetitions	Mean dry density (kg/m^3)	Range kg/m^3	Standard deviation kg/m^3	Standard Variance, %
Loosely compacted gravel	20	1792	16	12	0,67
Compacted gravel	21	2057	8o	18	0,88
Sandy sub-base compacted	21	1985	15	13	0,65

Sand Replacement Test variance can be assessed by a series of repetitive density determinations of the standard sand used in the replacement tests. The following argument applies to the sand replacement tests as well as to the nuclear gauge tests for density.

In COLTO Section 8200 an acceptable limit La is defined as:

$$La = Ls + Sn\,.Ka$$

Where Ls may be taken as the specified minimum requirement mean of n tests making up a statistical lot, Sn is the standard deviation and Ka is a factor depending on the number of tests done and the allowed permissible percentage of unacceptable conditions. Note that the product Sn.Ka represents the additional requirement needed to ensure that the permissible percentage of unacceptable conditions is not exceeded.

Consider as a case study the compacted gravel in Table 2 above for a 15% risk allowance and 21 tests where Sn = 18kg/m^3, Ka = 0,57, any specified field density limit Lfs must be increased to Lfa where:

$$\begin{aligned} Lfa &= Lfs\ (1 + 18 \times 0{,}57/2057) \\ &= Lfs \times 1{,}0050 \end{aligned}$$

This implies that any field density measured as Lfs for this gravel should be further compacted to Lfa (=1,0050Lfs) in order to ensure an acceptable failure risk of not more than 15% due to instrument variance.

5.2 Modified Test Variance

A series of repetitive Modified density tests on seven identical soil samples showed a standard deviation of 13 kg/m^3 and a range of 60 kg/m^3. These tests do not include soil variance if it is assumed that all samples tested were identical. If say samples of soil taken from n test positions for field density were now tested for Modified density the standard deviation for this lot may be even larger than the above single-soil series, as soil variance would also be included here.

Consider again a case study of the compacted gravel in Table 2 above and where six density test positions were randomly chosen and that Modified tests were done on gravel samples taken from these tests positions yielded a standard deviation of 22 kg/m^3. For n = 6 and the risk factor of 15 %, from COLTO, Ka = 0,358. If the mean value of the Modified Density was 2188 kg/m3 the acceptance limit Lma here is:

$$\begin{aligned} Lma &= Lms - Sn.Ka \\ &= Lms\ (1 - 22 \times 0{,}358/2188) \\ &= Lms(1 - 0.0036) = 0{,}9964Lms \end{aligned}$$

Note that the term Sn.Ka is negative here since the modified density is a divisor. This ensures that a higher compaction must be achieved to limit the failure risk to not more than 15 %.

This statistical treatment of the Modified density results accounts for variance in both the testing as well as the soil.

To ensure that all variances resulting from testing procedures within the density test readings namely the nuclear gauge, the Modified density testing procedure and the soil variances are properly accounted for the relative compaction limit must be increased from a specified limit value Rts to a higher acceptable value Rta, where Rta = Lfs/Lms and:

$$\begin{aligned} Rta(=Lfa/Lma) &= Rts(1+ 0{,}0050)/(1 - 0{,}0036) \qquad (1) \\ &= 1{,}0086Rts \end{aligned}$$

The fraction 0.0086Rts here is the shift above the specified minimum value to allow for test variances (Vt).

5.3 Workmanship variance included

This relative compaction limit Rta does not include workmanship variance and in order to include all variances each of n test points must be assessed for relative compaction. These values in our case study of 6 tests formed a new lot which yielded a standard deviation of 1,62% and a mean value of 95,04%. Our study case had a specified relative compaction requirement of 95%Rs. In order to satisfy the 15% risk and to include all variances Rs must now be increased to an acceptable Ra as follows:

$$\begin{aligned} Ra = Rs + Sn.Ka &= 95 + 1{,}62 \times 0{,}358 \qquad (2) \\ &= 95 + 0{,}58 = 95{,}58\% \end{aligned}$$

The fraction 0,58 here represents the mean shift to allow for test and soil variances as well as workmanship variance: Vt + Vw = Vtw.

Equation (1) above gives a value for Rta as:

$$Rta = 1{,}0086 \times 95 = 95.82\%: \qquad (3)$$

These two values shown in equations (2) and (3) are not necessarily equal values. In practice, their differences may be interpreted as follows:

Ra > Rta: Workmanship variance Vtw dominates and dictates the acceptance limit,

Ra = Rta: Testing and soil variances Vt are less than anticipated,

Ra <Rta: Estimated testing and soil variances Vt dominate and dictate acceptance

5.4 A serious consideration

In our case study the mean field relative compaction is 95,03%, which is less than the acceptance limit of 95,82% which dominates acceptance and the contractor’s work would be rejected. However, the following argument should rightfully be considered.

If variances are accepted to dictate assurance that unacceptable values of not more than 15% are accepted, then surely the contractor has in all fairness a right to ensure that otherwise acceptable work, which is rejected, should not exceed 15%. This implies that in our case study the mean field relative compaction of 95,03% Rm should be adjusted for test and soil variances outside his control to a higher mean Ram. Thus:

$$Ram = Rm \times 1{,}0086 = 95{,}847\%$$

As this is above the acceptance limit of 95.82% the section should be approved. Note that this form of acceptance can only be considered when:

* The field mean value Rm exceeds the specified limit and
* The acceptance limit is governed by testing and soil variances.

Surely, it is within the contractor's right to ask for work to be accepted when these two conditions are met.

6. A SUGGESTED ALTERNATIVE TO THE MODIFIED AASHTO COMPACTION TEST

Apart from the variability of the Modified test value used to determine relative compaction in the field, because of the variability of many soils it is common practice to determine a Modified density on a sample of soil taken at the point of testing of the field density. If this practice is rigorously followed this could mean that for a common series of 6 test positions for the testing of any one section of work six modified tests must be performed. As each test requires approximately 30-40 kg of soil this implies that about 200 kg of test material must be removed from the layer concerned for laboratory testing. Apart from the enormous craters that are now dug in the finished layers the performance of six modified tests in a field laboratory is not done overnight and three to four days may elapse before final results are available for acceptance judgments. Can this really be afforded?

The writer has formulated a "one-shot" method of field-testing which he would like to see being universally used but a fair amount of research may be needed prior to general acceptance. A study of the Modified and Proctor density tests has shown the following to be reasonably acceptable:

a. The peak density for these tests occurs when the degree of saturation is 80%.
b. The "wet leg" of the test plots follows the 90% degree of saturation line.
c. For moisture contents below the true optimum moisture content the "dry leg" of the plot follows the mirror image of the 90% Saturation line.

Based on these observations a mathematical formulation has been produced which relates the density of a single compaction test at a moisture content within 2 to 1 percent of optimum M/C and the probable Modified or Proctor density for the soil tested and the compactive effort applied.

The 90% and 80% degree of saturation lines for only one soil is dependent not only on density and moisture content but also on the relative density of the soil. A relatively simple test is proposed for determining the effective relative density. Water is placed in a measuring cylinder about half full. About 250g of dry soil is added, and shaken to remove air bubbles. The increase in volume divided into the mass of dry soil gives an effective relative density. This test can be performed in the field in a matter of minutes.

The mathematical relationship between a "dry leg" density (D_D), the corresponding moisture content (W_D), the effective relative density (G) and the maximum "peak" density

D_M is given by:

$$\frac{1}{D_M}=\frac{0.56}{D_D}+\frac{0.44}{G}+0.63\,W_D \quad (D_M \,\text{and}\, D_D \text{ in tonnes/m}^3)$$

Although this "one shot" method may not give the exact modified or Proctor densities, it is most likely that the differences are in fact less than the possible replication errors inherent in the actual tests as illustrated above.

7. RELATIVE COMPACTION IN QUESTION

If a foundation layer were compacted to say 98% of Modified density it would generally be accepted as giving acceptable strength for the layer. Is this actually a valid acceptance? The density of a layer does not in itself show the strength of the layer as this has to be compared with a second or standard density for the some soil to which a strength value has been attached. The famous Coulomb equation: $S = c + \sigma \tan\varphi$ is the measure of a soil shear resistance or strength and nowhere in this equation does density appear. The effective pressure, σ, is exerted between the particles and φ is the angle of shearing resistance at the points of contact.

7.1 Interlock

The term*:* c, called cohesion in geomechanics is in fact the shearing resistance generated by the degree of particle interlock. If in the above equation σ is zero the last term vanishes and c remains as the only parameter, which represents strength. However due to compaction and overburden pressure σ is seldom zero and $\sigma\tan\varphi$ is thus active. When water is present in the soil φ is low as the water reduces friction. Hence the addition of water to the soil to augment compaction and assist in developing a high value for c the shearing resistance or its parameter particle interlock (L), where:

L = dry density/relative density of the soil particles = D/G

Interlock L can be likened to the degree of togetherness of the soil particles; the greater the value of L the greater the soil strength.

7.2 Porosity

A second parameter, which is directly related to interlock within a soil matrix is known as porosity; namely the proportion of a soil volume that is not filled by the particles themselves. It is in fact the voids content (n) within the soil. The value for n is given by:

$$n = 1 - L$$

We can thus state that the value of n is also a direct measure of the soil's possible strength. A low value for *n* is a high strength component while a high value is an indication of poor strength. Note that in a G1 layer of crushed stone the value of n is not more than 12% (88% of Gbk) showing excellent strength. A loose un-compacted fill may have a value for n of 33 to 39%, while a good gravel base layer should have an n value or porosity of not more than 18% or even better 15%. Note here that n does not include voids within the soil particles themselves.

It is quite evident from the above argument that the value of n or L is a far better measure of strength than a soil's density. To emphasize this argument even further consider the following discussion.

7.3 Density in question

Take two soils A & B where soil A has a particle relative density (G) of 3 and soil B that of 2.5. Assume also that in both cases tests for relative compaction showed values of 98%. A lay assessment may assume that in both cases the required density having been achieved will yield satisfactory strength for a gravel base (98% being the generally accepted minimum RC for a base layer). Let the Modified Density for both soils be 2.092 t/m^3. At 98% RC each soil would have a field density of 2.050 t/m^3. For the A and B soils the Table 3 shows some remarkable differences.

Table 3 Some Soil Parameters for soils A and B

Soil type	Relative Density G	Modified Density D_M t/m^3	RC (%)	Field density D_F (t/m^3)	Interlock L (%)	Porosity n (%)
A	3.0	2.092	98	2.050	0.68	32
B	2.5	2.092	98	2.050	0.82	18

If a good base course material requires a porosity of not more than 18% only B soil should be acceptable. As soil A also satisfies the specification of a RC of not less than 98% it would be accepted but in fact its strength is not even up to that of a selected layer where n would be in the order of 30%.

Is it not about time that we concentrated more on the true measure of the road foundation materials rather than the present inadequate assessment of Relative Compaction by specifying a maximum porosity n or a minimum interlock L?

7.4 Density and Porosity Criteria

In the past, particularly where new roads or gravel roads were being constructed to a surfaced standard the gravels used in the foundation layers were of selected quality, as the borrow areas of such materials were in the main plentiful. With the advent of the present day multi-wheel vehicles and the rehabilitation of existing roads it is vital that a re-think be applied to the gravels used in new foundation layer construction. Density in the field compared with the Modified density for the material, even if the Relative Compaction is satisfied is in the writer's opinion not enough on its own to adequately satisfy base-course quality. Porosity must also play a role. Table 4 gives suggested limiting values for porosity for the normal foundation layers.

Table 4 Suggested limiting criteria for RC, n, and Gg.

Layer	Soil Group Max. Gg	Rel. Comp. Min RC (%)	Max Porosity at Mod. Density (%)	Maximum n in layer (%)
Gravel Base untreated	G4,9	98	18	19.6
Gravel Base stabilized	G5,9	97	21	22.5
Gravel Sub-base	G6,9	95	24	26
Selected layer	G7,9	93	27	30.5
Top of fill or formation	G8,9	90	30	35.5

8.

9. THE SOIL GROUPS AND POROSITY

Soil types with relation to strength and grading are the Gg series where G4 to G10 are good gravel to relatively poor soils respectively. Consider a quartzitic gravel and a fine soil, such as a G4 and a G9 respectively. The Modified density for the G4 gravel may well be between 2,170 and 2,250 t/m^3. This would indicate a Modified porosity n_m of 0,15 to 0,18. Similarly the G9 soil may show a Modified density of 1,775 to 1,850 t/m3. The porosity here would be 0,30 to 0,33. Soil groups falling between the G4 and the G9 would show proportional Modified porosities. A mathematical relationship between a soil group Gg and its probable porosity n_m can now be established:

$$Gg = (n_m - 0{,}03)/0{,}03 \qquad (4)$$

Where the suffix g = soil group, not relative density.

If Gg contains a fraction, the fraction shows the position within the group range. A value of G5,9 for Gg, for example would indicate a soil within the Group 5 but bordering pretty close to a Group 6 soil.

Let the achievable porosity at Modified Density be n_m and at any other Relative Compaction be n_{98}, n_{95} n_{90} etc. Table 5 shows suggested values of n (n_{rc}) for G4 to G10 soils at various degrees of RC. These values were calculated from equation 4 and:

$$n_{rc} = \{1 - (1 - n_m)RC\}/0{,}03 \qquad (5)$$

Table 5 Suggested values for n_{rc} for soil groups at Relative Compaction (R.C.)

Group Gg	G4	G5	G6	G7	G8	G9	G10
Mod. n_m	<18	≥18-- <21	≥21-- <24	≥24-- <27	≥ 27-- <30	≥ 30-- <33	≥ 33
n_{98}	19.6	19.6-22.5	22.5-25.5	25.5-28.5	28.5-31.4	31.4-34.3	34.3
n_{97}	20.5	20.5-23.4	23.4-26.2	26.2-29.2	29.2-32.1	32.1-35.	35
n_{95}	22.1	22.1-25	25-27.8	27.8-30.7	30.7-33.5	33.5-36.4	36.4
n_{93}	23.7	23.7-26.5	26.5-29.3	29.3-32.1	32.1-34.9	34.9-37.7	37.7
n_{90}	26.2	26,2-28,9	28,9-31,6	31.6-34,3	34,3-37 0	37,0-39,7	39,7

This table indicates that higher group soils cannot readily be compacted to a porosity equal to the low n values of say G4 or G5. A G7 soil for example at 100% Modified Density yields a porosity between 24 to 27%. If this soil is to be compacted to give a porosity of 22.5 say which a G5 would yield at an RC of 95%, the degree of compaction would be to over 103% Modified Density for the soil.

10. GRADING LIMITS AMENDMENT FOR CRUSHED STONE

The present COLTO specification for the grading limits for G1, G2 and G3 crushed stone layers is presently not in the best interest of higher strength and lower compactive effort, particularly where modern multi-wheel vehicles are the order of the day.

An amended grading based on the Talbot value of $P_{ds} = (\frac{D_s}{53})^{0.5\, and\, 0.3}$ instead of $P_{ds} = (\frac{D_s}{37.5})^{0.5\, and\, 0.3}$ is more appropriate. This produces a grading more in keeping with a stronger layer for current heavy multi-wheel vehicles. The present G1 grading is basically 84-96% smaller than 26.5mm, which is not ideal for a G1 in the writer's opinion. It will be interesting to discover what stone suppliers would have to say about these suggested new limits.

11. CONCLUSIONS

It is my sincere hope that all parties concerned with road construction give serious thought to the present shortcomings embodied in our specifications and control of work relating to road- and other earthworks and that a serious attempt be made for a future re-think! The factors that need to be seriously considered are:

- Compaction and density measurements;
- The use of bulk relative density rather than apparent relative density for judging G1 base compaction;
- Incorporation of test procedure variability in judging field compaction;
- Use of the "one-shot" field density evaluation;
- Substitution of porosity or interlock for relative compaction;
- Increase the maximum size of G1 crushed stone from 37.5 mm to 53 mm.