CHAPTER 5

DIRECT SHEAR TESTS

5.1 Introduction

The shear strength of an aggregate needs to be examined when the aggregate is to be used as fill to a structure and if an aggregate is to be used as road sub-base material, an investigation of the shear strength is also useful. Measurements of shear strength should be made on samples which are in a similar condition to that expected on site. As the state of packing of the particles is an important influence on the shear strength of a material, the density of the aggregates was the main parameter under investigation in this research. Direct shear box tests were conducted on three aggregates - limestone, demolition debris and crushed concrete - to obtain the angle of friction of the materials in different test conditions. A 300mm shear box test, developed by the Pavement Materials and Construction Division of TRRL to ascertain the suitability of various aggregates for use as road sub-base, was also carried out on the materials.

5.2 300mm shear box

The shear box tests were conducted in a large shear box located at the Ground Engineering Unit of the Transport and Road Research Laboratory. The internal dimensions of the shear box were 300mm x 300mm x 179mm and the two halves of the box were made of steel and plated for protection against corrosion. The arrangement was similar to that of the standard Casagrande 60mm shear box. The top and bottom platens were both ridged and the areas of the platens were slightly smaller than the area of the shear box. The top platen was heavier than the lower one and its flat top allowed a load cell to be placed on it. This type of shear box (see Plate 5.1) is produced by Wykehan Farrance for testing aggregates and materials containing particles as large as 37.5mm.

The shearing force was generated by an electrical motor driving a mechanical screw jack via a 42 speed gear box with speeds ranging from 0.125×10^{-3} mm/min to 6.1mm/min. Rapid and reverse movements were performed manually by disengaging the clutch and using the hand wheel at the front of the shear box unit. The proving ring which measured the shear force on the original system had been replaced by a 10 tonne load cell by TRRL.

The vertical force was exerted by a fully self-contained hydraulic pressure system. In the original shear box, the seals between the load piston and its outer cylinder were inflatable causing large friction forces to develop during loading. Consequently, the friction angles recorded were 3° greater than they should have been. The load piston was replaced by one containing PTFE seals which had very low friction resistance. When these new seals had been installed, the hanger system on the shear box was capable of falling under its own weight whereas it did not before. Using the same hydraulic system and the new load piston, it was found by TRRL (Brady, Awcock and Wightman, 1983) that the measured friction angles were comparable with results from a Casagrande 60mm shear box. It was concluded therefore that friction between the seals and the cylinder had been reduced considerably. This improvement was carried out by the staff at TRRL before this research began.

The horizontal cross beam and tie rods for vertical loading pivoted on a universal joint which allowed both the beam and the rods to be swung out of the way when the shear box halves were being placed in position. The vertical force was measured by a 20kN load cell which was placed between the vertical cross beam and the top of the box during a test. The horizontal displacement of the box and the vertical displacement of the top platen were measured using linear variable differential transformers (LVDTs).

The measurement of displacement and force was controlled by a Commodore Graphic 8296d computer. The programmable switching unit contained a series of relays to which were attached the 10V power supply, the LVDTs and the load cells. The output voltages from these devices were monitored using a digital voltmeter. All devices were checked when a set of readings was taken every 30 seconds.

An uninterruptable power supply, with a back-up supply of 240V for 2 hours, supported the computer and peripherals. The forces were converted by the computer into vertical and shear stresses and subsequently the friction angles were calculated. The readings were printed on a MUTEK PPM Printer. The shear and normal forces were plotted against horizontal displacement on a standard plotter.

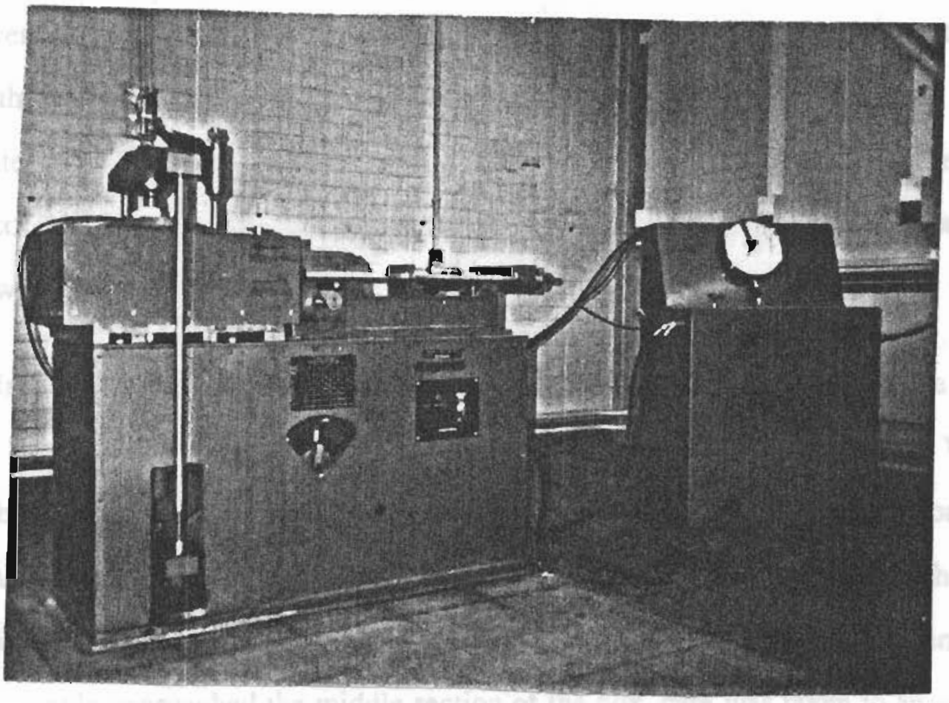


Plate 5.1 300mm shear box apparatus

5.3 Preparing the samples

The optimum moisture content (OMC) and peak dry density ($\rho_{d,peak}$) were obtained using the BS 5835 compactibility test for graded aggregates (1980). Each of the aggregates to be tested in the shear box was mixed at a moisture content just below its OMC. It was considered, if the samples were compacted at OMC, that low density samples would be difficult to obtain. By varying only one parameter i.e. density, the data from the tests could be analysed more easily. The moisture contents for limestone, demolition debris and crushed concrete, at which the tests were carried out, were 3%, 10% and 7% respectively.

The test procedure was conducted using a method described by Head (1982) and by following some suggestions by Brady (1989). At the beginning of each test the two halves of the shear box were cleaned and coated with a thin layer of oil. The two halves and the bottom platen were then placed in the chamber and the top half was bolted to the shear yoke. A portion of material weighing 5kg was placed in the box, levelled roughly with a palette knife and then compacted. To vary the density of the samples, various methods of compaction were used which are listed in Table 5.1.

The flat rectangular foot attached to the vibrating hammer was 1/6th of the area of the box so the foot had to be moved around on the material until the surface was level. When each layer had been compacted the surface was broken up gently using the edge of a palette knife with a scraping action so that no possible shear failure planes would exist in the material. The next 5kg portion of material was then added and compacted as before. When the height of the sample approached the middle section of the box, care was taken to ensure that the surface of a compacted layer did not coincide with the split in the shear box. The last portion was added so that its level was 5mm-10mm above the top of the box. The compaction time for this layer was half that of previous layers. The top platen was then lowered onto the material and was made level using the vibrating hammer.

State of material	Method of compaction
Loose	Tamped 30 times with a 20mm diameter rod
Lightly compacted	Vibrating hammer for 2 seconds
Moderately compacted	Vibrating hammer for 6 seconds
Dense	Vibrating hammer for 10 seconds

Table 5.1 Methods of compaction

5.4 Running the shear box test

The output of the unloaded vertical load cell was read four times and the mean was taken as the zero load reading. The total weight above the shear plane was calculated by adding the weights of the top platen, load cell and the soil above the shear plane to the weight of the top half of the box. The vertical stress caused by this weight was deducted when the total vertical stress to be exerted on the sample was calculated. The load cell was then placed on the top platen and the cross beam was manoeuvred onto a ball bearing on top of the load cell until the hanger system was central and level. The pressure was increased in the hydraulic system until the required vertical force reading was reached.

The protrusion of the platen above the top of the box was then measured eight times around the perimeter using callipers so that the total volume of the sample could be calculated. A volume correction was made assuming that the granular material filled half the ridges of the top and bottom platens. The density of the sample could then be determined.

The vertical LVDT was placed in position on the cross beam and zero readings were taken on it and on the shear displacement LVDT. When a zero reading had also been taken on the shear load cell, the clutch was engaged and shearing was started. A slow rate of displacement of 0.117mm/min was decided upon so that the peak shear stress would not be missed.

5.5 Test conditions of aggregates

Two test series were conducted on each of the aggregates; the first, series A, consisted of varying the density of the aggregate samples and the second, series B, involved varying the vertical stress. Preparation of samples at peak dry density was avoided because it was considered that this would cause a considerable amount of particle crushing and therefore the highest dry density achieved was $0.9\rho_{d,peak}$. The densities of the aggregates were also expressed in terms of relative density so that the data could be analysed in a conventional manner. Relative density is defined as follows:-

$$I_d = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \qquad \text{...Eqn 5.1}$$

where e_{max} is the maximum voids ratio and is achieved when a cylinder containing the dry material is inverted quickly (Bolton, 1986),

e_{min} is the minimum voids ratio and is achieved at the maximum achievable density obtained by vibration and

e is the voids ratio of the material.

The data which Bolton (1986) examined were from tests on sands but the tests in this research were conducted on aggregates which contained particles from about 0.050mm to 37.5mm in size. The limestone, demolition debris and crushed concrete had coefficients of uniformity

(C_u) of 23, 35 and 14 respectively. Bolton (1986) obtained the minimum density by inverting a 75mm diameter cylinder containing sand and measuring the resulting density. It would not be possible to carry out this test on the aggregates described above.

A test for measuring the minimum density of gravelly soils is described by Head (1980). Material is tipped quickly from a bucket into a 152mm diameter mould, similar to that used in the CBR test (BS 1377, 1975). The material in the mould is weighed and the density of the material in this condition corresponds to the minimum density which can be obtained. This test can be conducted on materials containing particles up to 20mm in size. When the test was carried out on the aggregates, it was found that the densities were not low enough to be the minimum densities obtainable. Similar densities were obtained in the shear box when some compaction had been conducted using a tamping rod. According to Jones (1989), this type of minimum density test could only be used as a guide and may not be accurate. It was concluded from the results that the preparation of very loose samples of these types of well graded materials would be difficult.

The method used for the determination of $\rho_{d,peak}$ was the BS 5835 compactibility test (1980). This test was found to cause some crushing of the aggregate particles (Chapter 4). Due to the problems in measuring minimum and maximum densities accurately, some assumptions for I_d were made. These assumptions were influenced by the results of the compaction tests conducted on the materials.

It can be seen from results reported by Bolton (1986) that the critical state plane strain angle of friction (ϕ_{cv}) is difficult to obtain at $I_d < 0.22$. Therefore an I_d of 0.35 was assumed for the loosest samples of aggregate in the shear box tests and an I_d of 0.9 was used for the densest samples. These limits of I_d were chosen using the results of the tests conducted at varying density. By using the dry densities of the shear box samples, I_d could be found by linear interpolation. The particle grading of each of the aggregate samples after shear testing was compared with the particle gradings of samples taken from the stockpiles of aggregate. These results can be seen in Figures 5.1, 5.2 and 5.3 where it is apparent that some crushing did occur, particularly in the recycled aggregate samples.

The vertical stress, at which series A was carried out, was $50kN/m^2$ and the other test conditions, including the dry density (ρ_d) and the relative density (I_d), are listed in Table 5.2. The demolition debris, when it was obtained from the supplier, contained particles greater than 37.5mm. Two tests were carried out using the full grading to establish whether the larger particles made a considerable difference to the shear strength. These tests are listed in Table 5.2 as D13 and D14. All other tests were conducted on samples containing particles smaller than 37.5mm.

The tests in series B were carried out at similar densities but the vertical stress (σ_v) was varied from $50kN/m^2$ to $200kN/m^2$. The test conditions for this series are listed in Table 5.3. Some tests were also performed on samples when the chamber surrounding the box was filled with water to determine whether pore suctions were developing in the unsaturated samples. The conditions for these tests are listed in the second part of Table 5.3.

TEST SERIES LA			TEST SERIES DA			TEST SERIES CA		
TEST No.	ρ_d (kg/m^3)	I_d	TEST No.	ρ_d (kg/m^3)	I_d	TEST No.	ρ_d (kg/m^3)	I_d
L1	2103	0.9	D1	1700	0.76	C1	1805	0.9
L2	2063	0.85	D2	1674	0.72	C2	1729	0.78
L3	1991	0.75	D3	1645	0.68	C3	1714	0.76
L4	1768	0.44	D4	1550	0.54	C4	1665	0.68
L5	1705	0.35	D5	1480	0.44	C5	1528	0.47
			D6	1418	0.35	C6	1450	0.35
			D13	1802	0.9			
			D14	1441	0.39			

Table 5.2 Test conditions for Series A

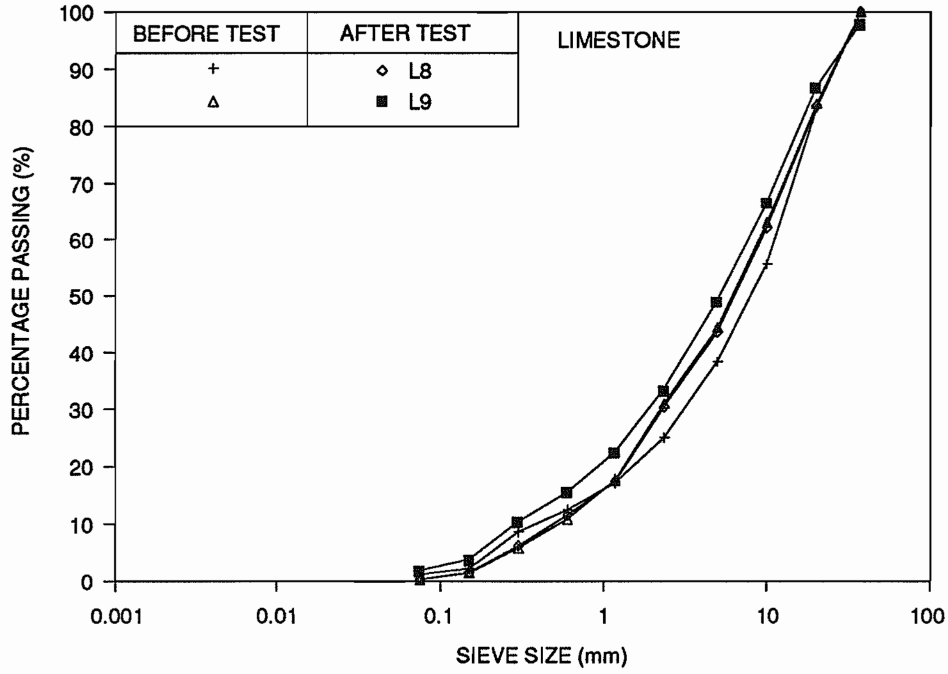


Figure 5.1 Particle gradings of limestone samples

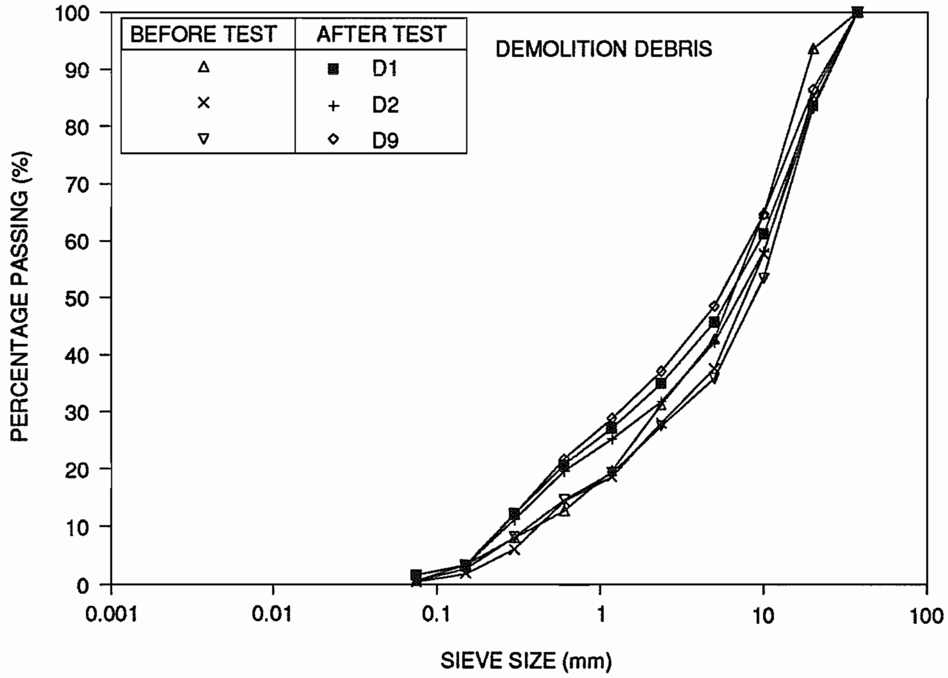


Figure 5.2 Particle gradings of demolition debris samples

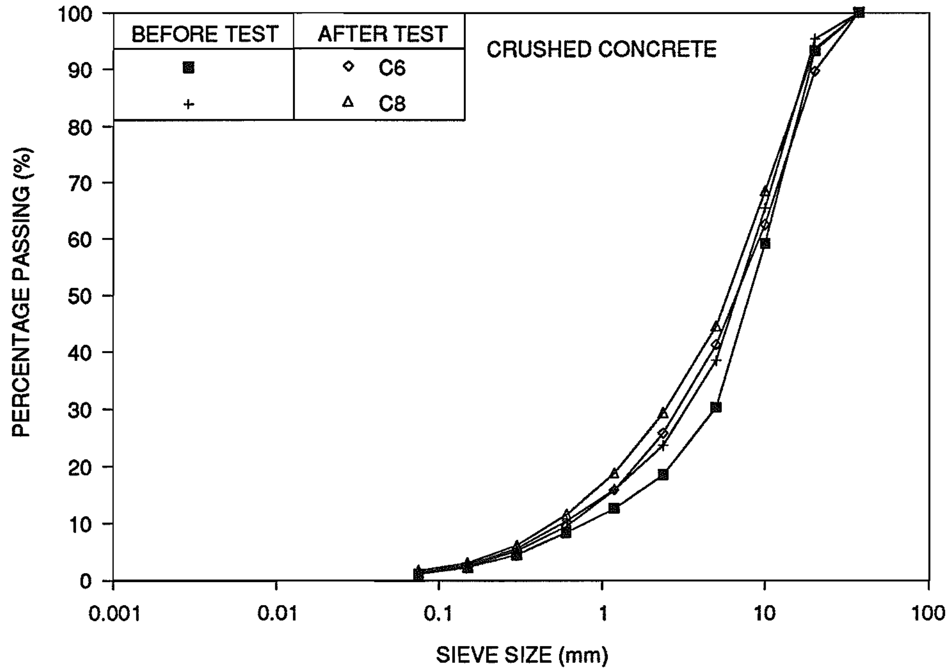


Figure 5.3 Particle gradings of crushed concrete samples

Note: In these figures, L = limestone, D = demolition debris and C = crushed concrete. The test details are listed in Tables 5.2 and 5.3.

σ_v (kN/m^2)	TEST SERIES LB			TEST SERIES DB			TEST SERIES CB		
	TEST No.	ρ_d (kg/m^3)	I_d	TEST No.	ρ_d (kg/m^3)	I_d	TEST No.	ρ_d (kg/m^3)	I_d
50	L2	2063	0.85	D4	1550	0.54	C2	1729	0.78
50				D2	1674	0.72			
75	L7	2081	0.87	D7	1591	0.6	C7	1662	0.68
100	L8	2104	0.9	D8	1573	0.58	C8	1731	0.78
150	L9	2106	0.9	D9	1651	0.69	C9	1708	0.75
200	L10	2097	0.89	D10	1640	0.67	C10	1650	0.66
100	L11	2009	0.77	D11	1535	0.52	C11	1662	0.68
200	L12	2127	0.93	D12	1656	0.7	C12	1664	0.68

Table 5.3 Test conditions for series B

5.6 Leighton Buzzard sand tests

Shear box tests were conducted on 14-25 Leighton Buzzard sand to confirm that the 300mm shear box yielded results which were comparable with those obtained in other research. It also proved useful to compare the results for sand with those of the tests on aggregates. The extreme voids ratios, e_{min} and e_{max}, for the sand were 0.49 and 0.78 respectively. Its particle grading is listed in Table 5.4. Two series of tests were carried out on the sand; series SA at varying density and series SB at varying vertical stress. The test conditions are listed in Table 5.5.

SIEVE SIZE (mm)	PERCENTAGE PASSING (%)	OTHER INFORMATION
2.36	100	$D_{10} = 0.64$mm
1.18	83.1	$D_{60} = 0.99$mm
0.60	0.67	$C_u = 1.5$
0.30	0.11	

Table 5.4 Particle grading of Leighton Buzzard sand

Note: $D_{10} = 0.64$mm means that 10% of the sand grains are smaller than 0.64mm.

TEST SERIES SA				TEST SERIES SB			
σ_v (kN/m^2)	TEST No.	ρ_d (kg/m^3)	I_d	σ_v (kN/m^2)	TEST No.	ρ_d (kg/m^3)	I_d
50	S1	1682	0.7	50	S1	1682	0.7
50	S2	1621	0.5	100	S4	1707	0.78
50	S3	1520	0.12	200	S5	1737	0.88
				200	S6	1679	0.7

Table 5.5 Test conditions for Leighton Buzzard sand

The dense samples were prepared by raining the sand from a height of 400mm above the top of the box. The loose samples were prepared by tipping the sand gently from a small container allowing no free fall and avoiding sloping of the sand surface during placement. The measured densities of the loose samples were quite accurate but, because of the loss of some particles when the sand was falling, measurement of the density of the dense samples may not have been exact.

5.7 Results

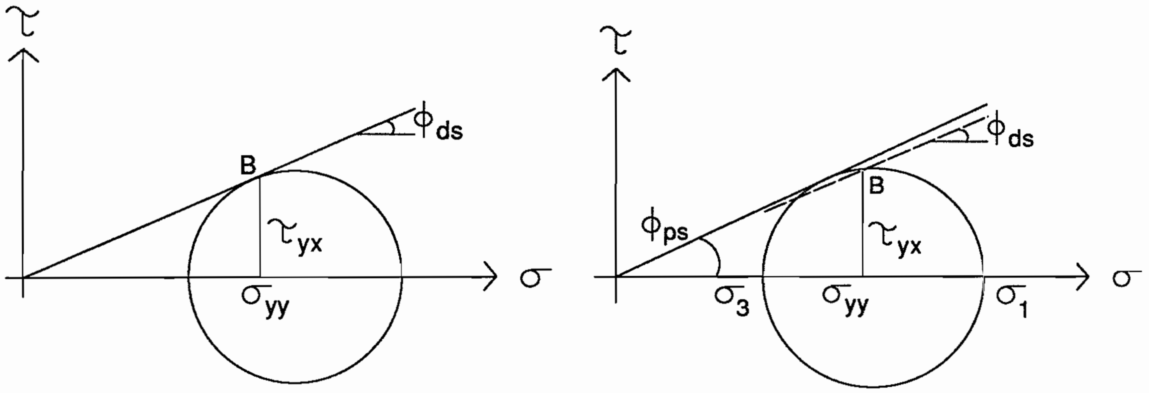
When the vertical force has been applied in a shear box test, shearing is started. Failure is assumed to have occurred when a peak in the curve relating shear stress to the shear displacement of the box has been observed. The standard approach used to interpret results from a shear box test is summarised in Figure 5.4a. The shear stress (τ_{yx}) and the vertical stress (σ_{yy}) are measured on the central plane.

The direct shear angle of friction is defined as

$$\phi_{ds} = \tan^{-1}\left(\frac{\tau_{yx}}{\sigma_{yy}}\right) \qquad \ldots Eqn \quad 5.2$$

Stress measurements could not be made on the central plane so the boundary measurements of the shear and vertical stresses were used. In this work, the shear stress is denoted by τ and the vertical stress by σ_v.

Palmeira (1987) discussed another method for the interpretation of results of shear tests. Jewell (1980) and Dyer (1985) reported that the horizontal plane in the centre of a shear box is a direction of zero extension. This observation and the assumption that the axes of principal stresses and principal strain increments coincide are used to form the method of interpretation illustrated in Figure 5.4b. The coincidence of the axes is fundamental to the theory of plasticity and was found to be true by Stroud (1971) in a simple shear box and by Dyer (1985) in a direct shear box. The direct shear angle of friction (ϕ_{ds}) is not measured on the plane of maximum stress ratio and therefore underestimates the maximum angle of friction which can be obtained. The plane strain angle of friction (ϕ_{ps}) is the angle measured on the plane of maximum stress ratio (see Figure 5.4b). The two angles of friction can be related by the angle of dilation (ψ).



ϕ_{ds} is the direct shear angle of friction

(a)

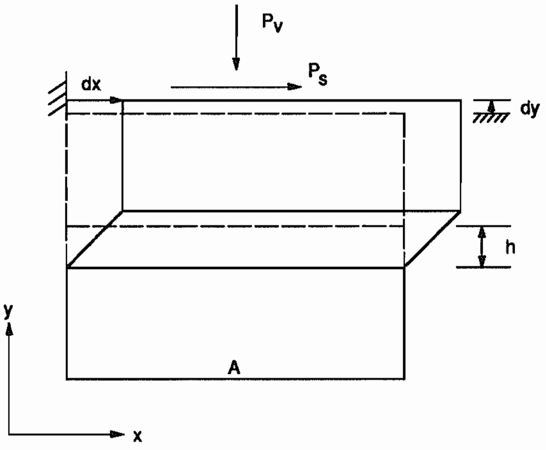
ϕ_{ps} is the plane strain angle of friction

(b)

Figure 5.4 Shear test parameters

Note: σ_1 and σ_3 are the major and minor principal stresses

During shearing of a dense sample of material, it can be seen in Figure 5.5 that the band of material at the centre of the shear box dilates. To obtain the rate of dilation, the horizontal and vertical displacements are measured during a shear box test. The dense sample will dilate until it reaches a state when, during further shearing, the rate of dilation remains zero. This normally occurs towards the end of a test and is known as the critical state. If the sample is very loose, it will compress until it also reaches a condition in which the rate of dilation is zero. The angle of friction of the material in this state is termed the critical state angle of friction.



The vertical stress $= \sigma_v = P_v/A$ where A is the area of the shear box and P_v is the vertical force.

The shear stress $= \tau = P_s/A$ where P_s is the shear force.

The vertical strain increment $= dy/h = d\varepsilon_{yy}$ and the shear strain increment $= dx/h = d\gamma_{yx}$ where h is the deforming zone of the sample.

Therefore the rate of dilation is

$$\frac{dy}{dx} = \frac{d\varepsilon_{yy}}{d\gamma_{yx}} = \tan\psi \qquad \ldots Eqn \quad 5.3$$

Figure 5.5 Definitions for the direct shear test (after Jewell, 1989)

The basic shear box data for series LA and LB are shown in Figures 5.6 to 5.8. The critical state direct shear angle of friction $(\phi_{ds})_{cv}$ tends to 42^0 when estimated from the extrapolation of the data at the end of the tests. It can be seen that a wide range of density was examined in series LA and as expected the data obtained from the dense samples showed an early and pronounced peak in shear strength but the loose samples achieved maximum shear strength gradually towards the end of the tests. The demolition debris data are shown in Figures 5.10 to 5.13 where $(\phi_{ds})_{cv}$ also tends to about 42^0. The data from the tests on the samples containing particles larger than 37.5mm are shown in Figures 5.10 and 5.12. The peak stress ratio $(\tau/\sigma_v)_p$, where τ is the shear stress and σ_v is the vertical stress, for test D13 appears to be much higher than for the other tests. This may be partly due to the fact that in this test I_d was 0.9 whereas the highest I_d for tests D1 to D6 was 0.76. However, if the data for L1 in Figure 5.6 are examined, where $I_d = 0.9$, it can be seen that the $(\tau/\sigma_v)_p$ value is much lower than the high value of 2.2 which was obtained for D13.

Palmeira (1987) found that the direct shear angle of friction (ϕ_{ds}) was not significantly affected by the ratios L/D_{50} and H/D_{50}, where L was the length and H was the height of the shear box, for values varying from 38 to 1250 and from 20 to 1250 respectively. L/D_{50} was 30 and H/D_{50} was 18 for the demolition debris containing particles greater than 37.5mm where D_{50} for the material was 10mm. The particles therefore were too large to be tested satisfactorily in the 300mm shear box. This may have also contributed to the high $(\tau/\sigma_v)_p$ value for test D13. If a large particle lay directly in the plane of shear, then the measured shear stress would be much higher than if the sample was more uniform. The result from the test on the loose sample containing the large particles was similar to the results of the other tests in the DA series. For the tests on aggregates containing particles less than 37.5mm, the L/D_{50} values for limestone, demolition debris and crushed concrete were 49, 49 and 37 and the H/D_{50} values were 29, 29 and 23 respectively. Therefore for these tests the ratio of the scale of the box to the particle size was within the limits which Palmeira (1987) suggested.

The basic shear box data for crushed concrete are shown in Figures 5.14 to 5.17 where the trends in results appear to be similar to those for limestone and demolition debris. The sand results, illustrated in Figures 5.18 to 5.21, show that the peak stress ratio and dilation rates were much lower than for the aggregates and the critical state was reached after 15mm-20mm shear displacement whereas the aggregates had yet to reach the critical state after 30mm shear displacement. The sand reached the critical state quickly due to the smaller particle size and the greater uniformity of the material. The value of $(\phi_{ds})_{cv}$ for the sand, measured at the end of the tests, was between 29^0 and 30^0.

The relationships between the peak direct shear angle of friction $(\phi_{ds})_p$ and $\rho_{d,peak}$ for the four materials are shown in Figure 5.22. Due to the difference in specific gravity of the materials (listed in Chapter 3) the $\rho_{d,peak}$ values of limestone were much higher. However, the $(\phi_{ds})_p$

values of limestone were similar to those of demolition debris and crushed concrete although the densities of these materials were much lower. Even at a low density of 1420kg/m^3, the $(\phi_{ds})_p$ values of demolition debris and crushed concrete were 44^0 and 37^0 respectively. $(\phi_{ds})_p$ is plotted against I_d in Figure 5.23 where it can be seen that the relationships for the three aggregates are similar although crushed concrete had lower $(\phi_{ds})_p$ values. It was noticed during the compaction of crushed concrete that the fines in some tests formed sticky lumps and did not disperse evenly throughout the whole samples. This may have had some effect at low density when perhaps the large particles could not interlock closely. Demolition debris performed better than limestone at high values of I_d.

The peak direct shear angle of friction values ($(\phi_{ds})_p$) for the total A series are plotted against the rate of dilation (dy/dx) in Figure 5.24 to obtain an estimate of $(\phi_{ds})_{cv}$ for each of the aggregates. A good approximation for $(\phi_{ds})_{cv}$ can be obtained for the materials where the lines cross the abscissa. Using this method, $(\phi_{ds})_{cv}$ for the aggregates ranged between 37.5^0 and 40^0 with limestone achieving the highest value and crushed concrete the lowest. These $(\phi_{ds})_{cv}$ values appear to be lower than the approximation of 42^0 made earlier. The estimate of $(\phi_{ds})_{cv}$ from the end of the tests is likely to be less accurate because the critical state had not been reached in most cases (see Figures 5.6 to 5.17).

The relationships between $(\phi_{ds})_p$ and σ_v for series LB, DB, CB and SB are shown in Figure 5.25. The relationships were similar for demolition debris and limestone where $(\phi_{ds})_p$ did not vary very much. The data for sand followed a similar trend. However, crushed concrete had a slightly higher $(\phi_{ds})_p$ value at 50kN/m^2 and then $(\phi_{ds})_p$ decreased as σ_v increased. The range of $(\phi_{ds})_p$ obtained for all aggregates was quite small. It can be concluded from the data therefore that $(\phi_{ds})_p$ was influenced by density but was not very dependent on vertical stress.

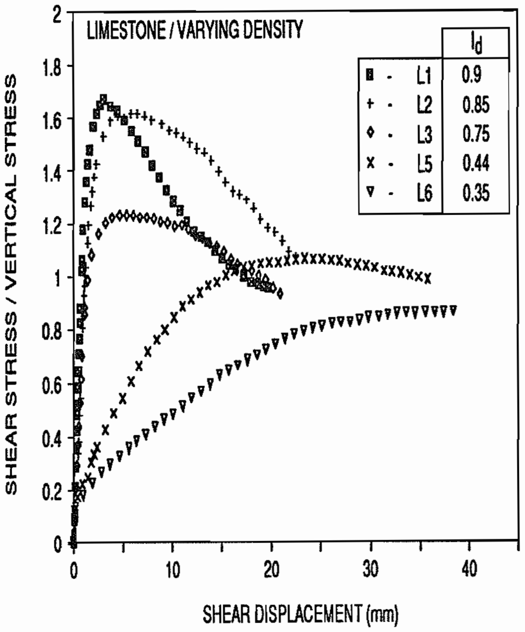


Figure 5.6 Stress ratio data for series LA

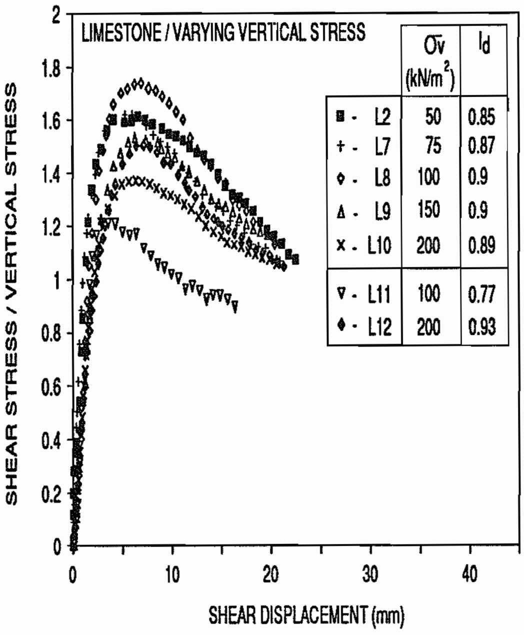


Figure 5.7 Stress ratio data for series LB

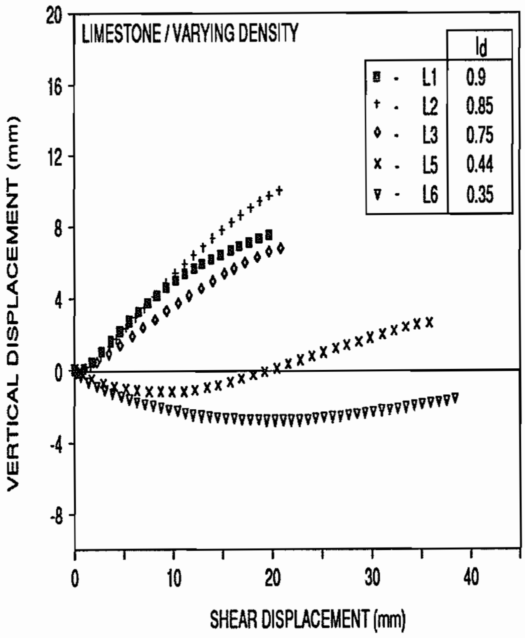


Figure 5.8 Vertical displacement data for series LA

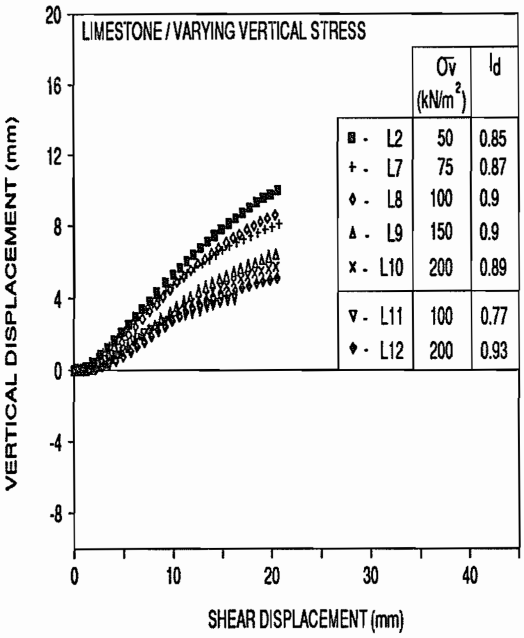


Figure 5.9 Vertical displacement data for series LB

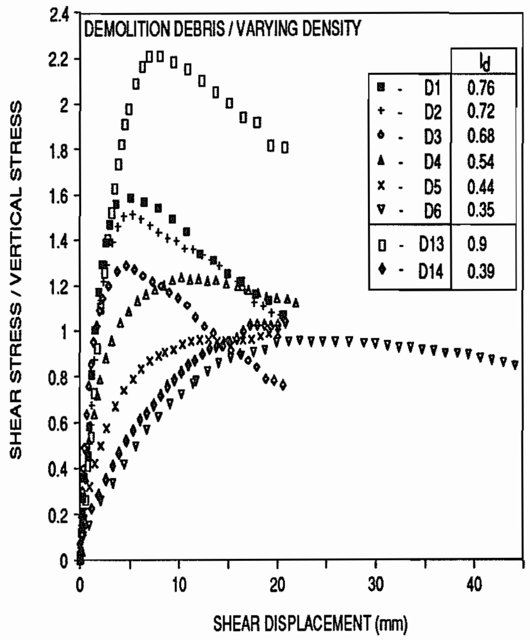


Figure 5.10 Stress ratio data for series DA

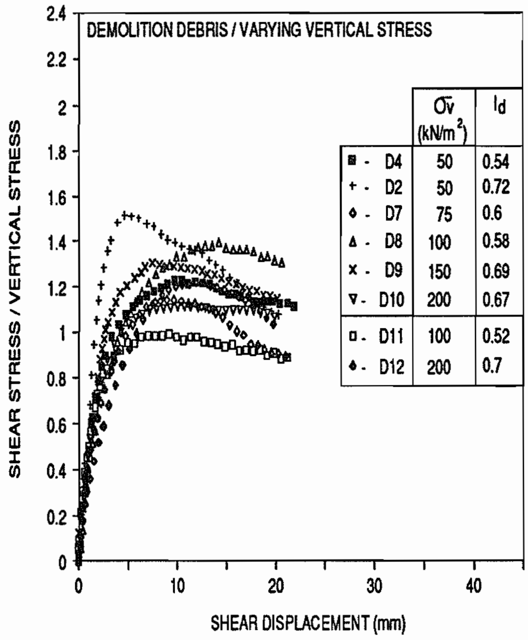


Figure 5.11 Stress ratio data for series DB

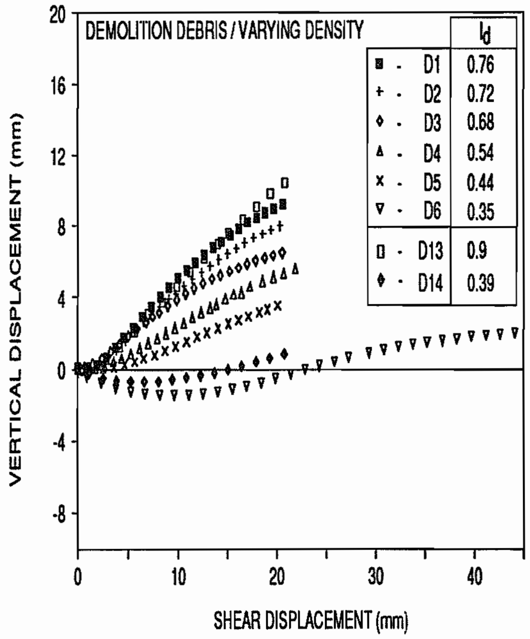


Figure 5.12 Vertical displacement data for series DA

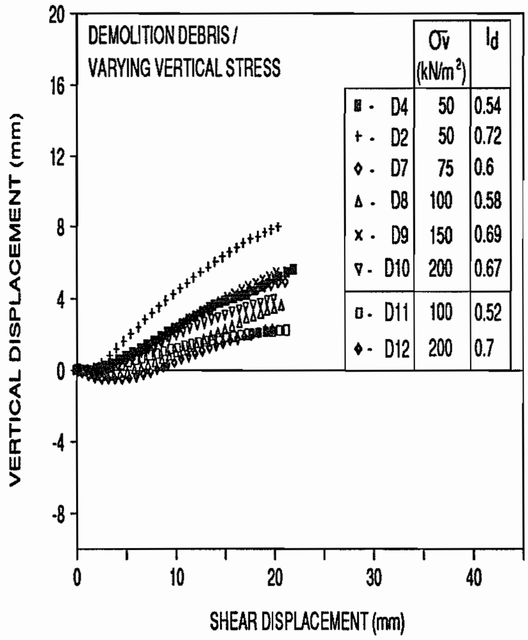


Figure 5.13 Vertical displacement data for series DB

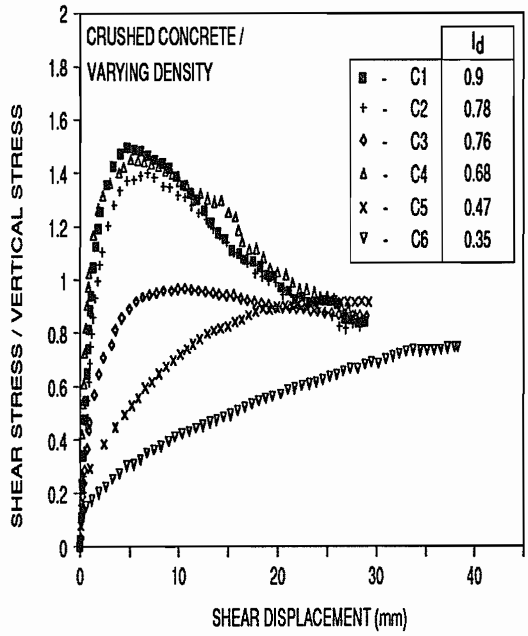


Figure 5.14 Stress ratio data for series CA

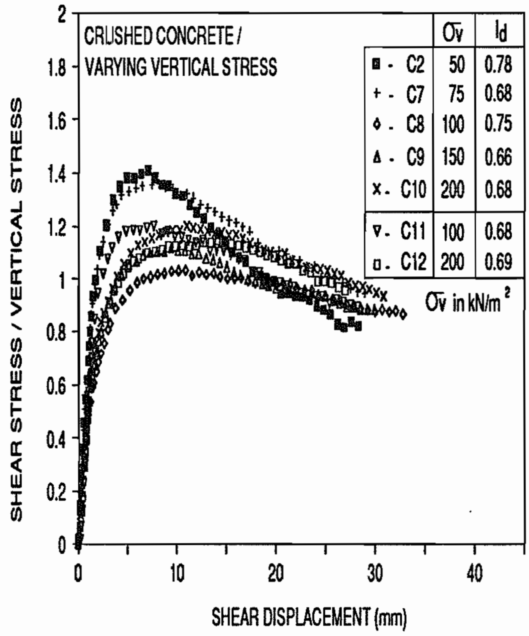


Figure 5.15 Stress ratio data for series CB

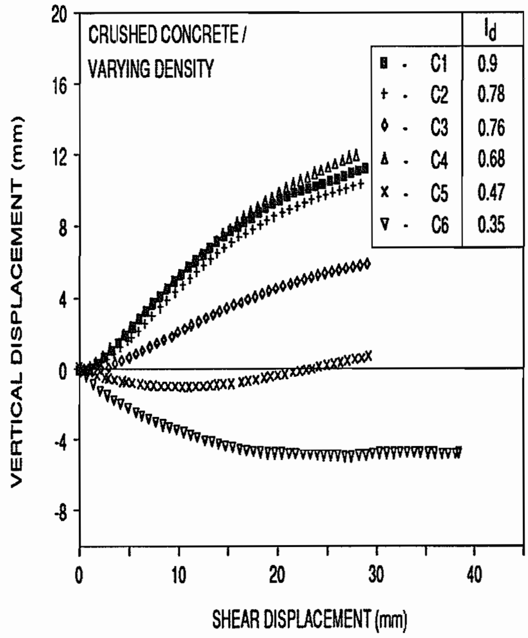


Figure 5.16 Vertical displacement data for series CA

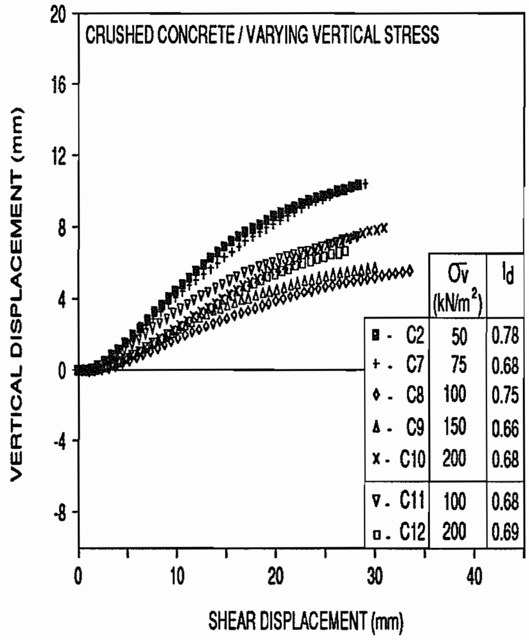


Figure 5.17 Vertical displacement data for series CB

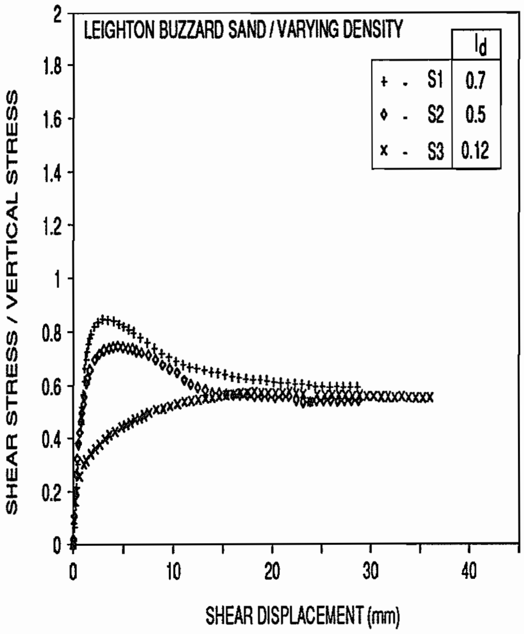


Figure 5.18 Stress ratio data for series SA

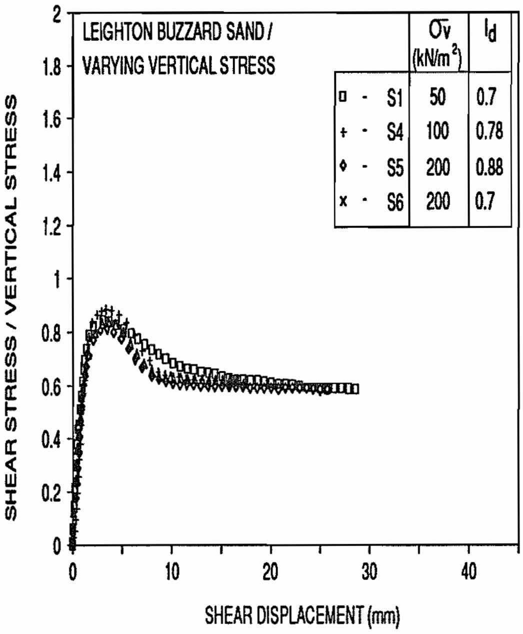


Figure 5.19 Stress ratio data for series SB

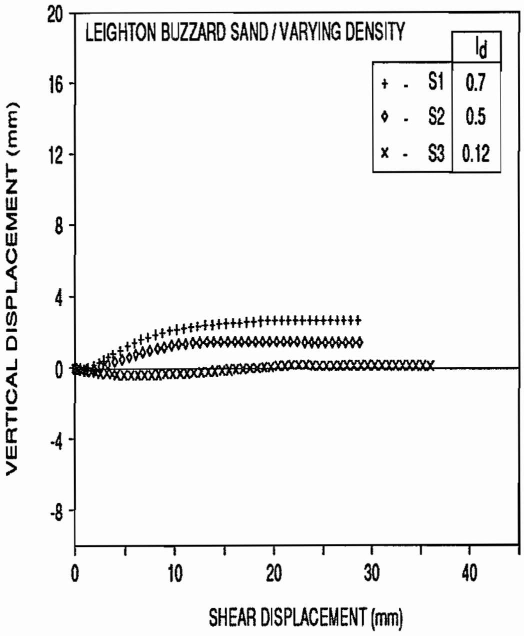


Figure 5.20 Vertical displacement data for series SA

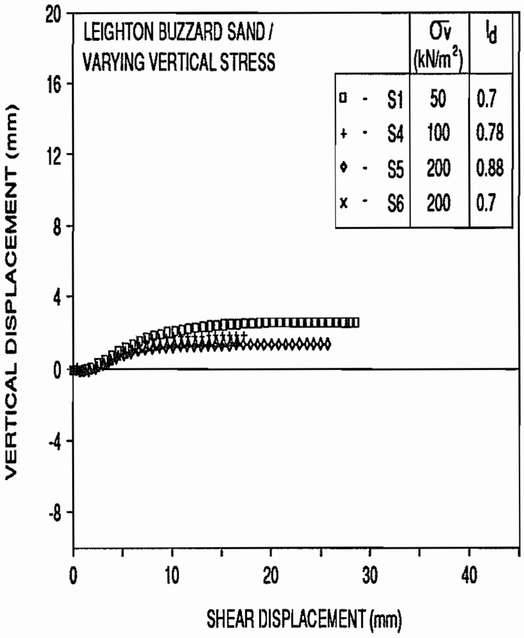


Figure 5.21 Vertical displacement data for series SB

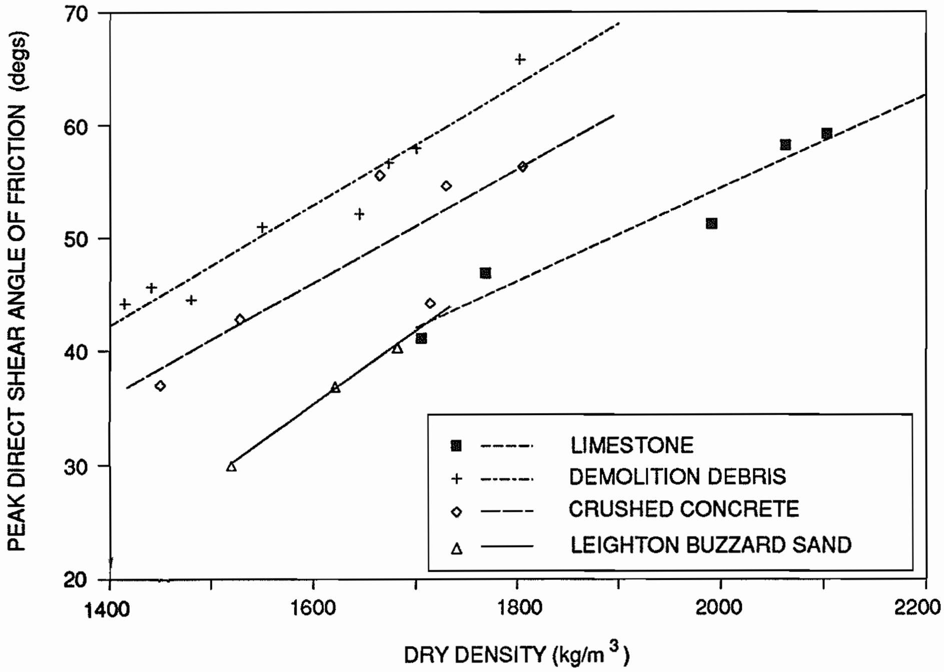


Figure 5.22 Influence of dry density on the peak direct shear angle of friction

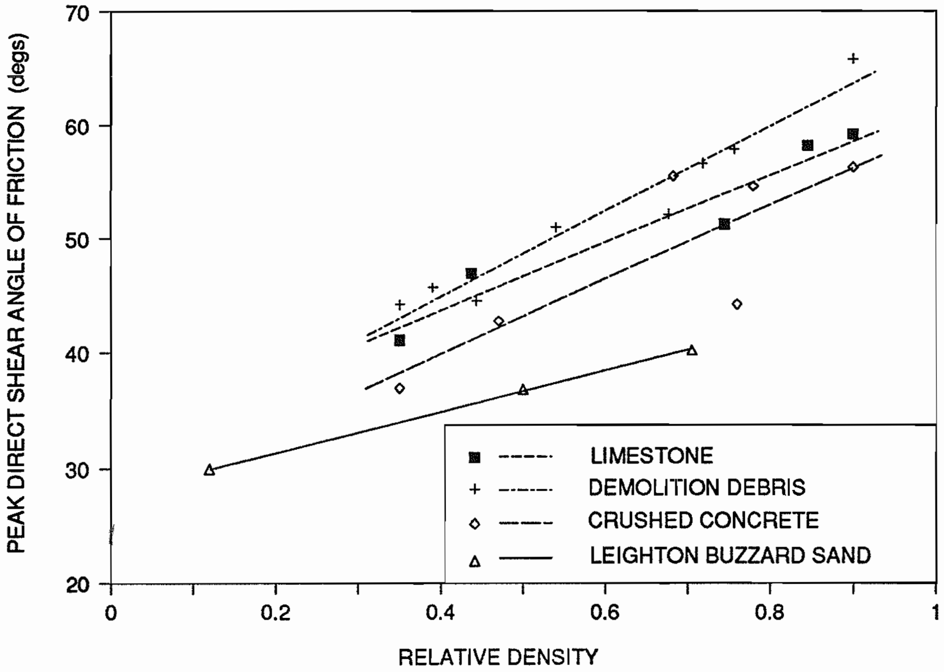


Figure 5.23 Influence of relative density on the peak direct shear angle of friction

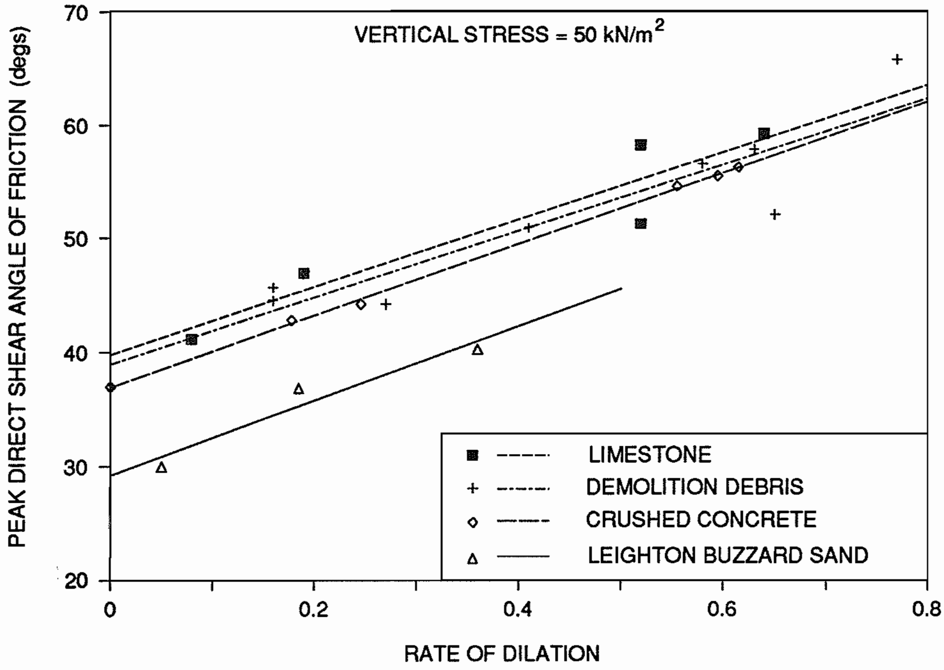


Figure 5.24 Peak rates of dilation for series A

In Figure 5.25, it can be seen for tests on limestone and demolition debris, conducted when the outer chamber of the shear box was flooded, that at a vertical stress of 100kN/m^2 the shear strength was lower than the results of similar tests conducted in a dry chamber. However, at a σ_v of 200kN/m^2, the shear strength appeared to be slightly higher. If the shear strength had dropped considerably in all tests conducted in the flooded chamber, it could be concluded that surface tension in the pore water of the unsaturated samples was causing high forces between the particles. These forces would have been reduced when the chamber was flooded and therefore the shear strength would also have been reduced. This phenomenon is normally associated with fine grained materials. Considerable scatter existed in the data and the effect of increased lubrication or surface tension in the pore water could not be determined quantitatively.

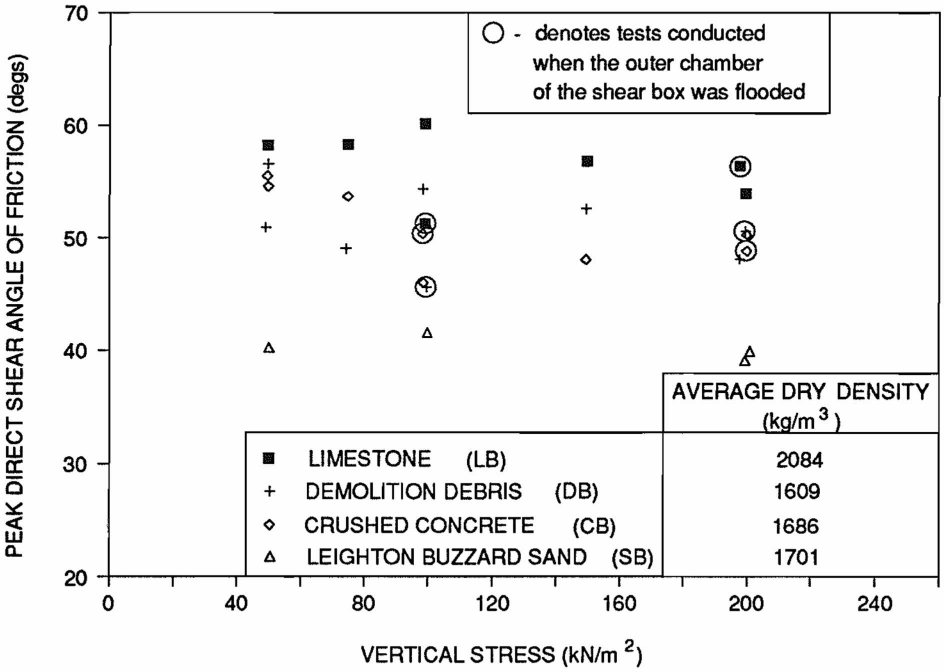


Figure 5.25 Influence of vertical stress on the peak direct shear angle of friction

5.8 Shear box tests for road sub-base materials

Earland and Pike (1985) produced a TRRL report describing a shear box test for the examination of the stability of granular sub-bases. They also made recommendations that the test could be used as a standard test for aggregates and suggested that it should be included in the British Standard for aggregate testing. A classification system for aggregates was established by comparing the results of a series of shear box tests with trafficking trials conducted on several aggregates. In the trafficking trials, conducted by Earland and Pike (1985), a 150mm thick layer of each aggregate was placed on a 250mm thick capping layer to provide a strip 5m in width. The lorries used for trafficking were restricted to channelled wheel paths. The $(\tau/\sigma_v)_p$ values from the shear box tests, which were conducted on the same materials in similar conditions, were plotted against the surface deformation caused by 1000 standard axles in the trials. From this plot, Earland and Pike (1985) produced the following classification system for aggregates:-

(a) **Low strength:** $(\tau/\sigma_v)_p$ less than 1.9. Materials in this category would not be suitable in sub-base layers but might be stable enough to be used as capping layer materials.

(b) **Medium strength:** $(\tau/\sigma_v)_p$ between 1.9 and 2.8. Materials achieving these values may be stable in favourable conditions but their performance should be checked by a preliminary trafficking trial designed to simulate real conditions of service.

(c) **High strength:** $(\tau/\sigma_v)_p$ above 2.8. Materials achieving these values should produce satisfactory sub-bases under normal construction traffic. A trafficking trial should be considered when exceptionally heavy lorries or plant are used, when there is extensive working in continuously wet weather or when poor drainage conditions exist.

For this shear box test (Earland and Pike, 1985), the aggregate was placed in a 300mm shear box between 96% and 98% of $\rho_{d,peak}$, where $\rho_{d,peak}$ was determined by the BS 5835 compactibility test (1980). The vertical stress (σ_v) exerted on the material during the test was 10kN/m^2 which represented the surcharge expected to be placed on a sub-base on site. The rate of shear, specified by Earland and Pike (1985), was 1.0mm/min. Under these test conditions, $(\tau/\sigma_v)_p$ was reached rapidly and one of the major advantages of the test is that it is quick and the quality of an aggregate can be determined in a few hours. The mean of two tests is taken as the final result, if the difference between the two is less than 0.3.

This test was performed on limestone, crushed concrete and demolition debris to determine whether they could be used as satisfactory sub-base materials with regard to the classification system. The results are listed in Table 5.6.

AGGREGATE TYPE	$(\tau/\sigma_v)_p$
Limestone	3.2
Demolition debris	2.5
Crushed concrete	1.9

Table 5.6 Peak stress ratio values obtained using the TRRL shear box test

The result for crushed concrete was lower than that of demolition debris but both materials were in the medium strength category. Limestone, as expected, would provide a sub-base of high strength. The result for limestone was confirmed by Earland and Pike (1985) who also carried out similar tests on limestone from the same source in Somerset and found $(\tau/\sigma_v)_p$ to be 3. With regard to stability, the recycled aggregates could be used as road sub-base material, provided that a preliminary trafficking trial was carried out in conditions similar to those expected on site.

Earland and Pike (1985) found that the difference between $(\tau/\sigma_v)_p$ results for tests conducted on similar samples was 0.3. From the results of the tests conducted on limestone, demolition debris and crushed concrete, it was found that the differences were 0.3, 0.13 and 0.07 respectively.

5.9 Analysis

A flow rule analysis using Taylor's energy correction (1948) is presented in Section 5.9.1 for the data from the tests on the aggregates and sand which were presented in Section 5.7. In Section 5.9.2, the data is interpreted and analysed using Bolton's (1986) dilatancy index.

5.9.1 The flow rule analysis

A flow rule relates strains and stresses during the plastic flow of a material and therefore the balance of energy in a direct shear test can be examined by a flow rule. A soil element in the central region of the shear box after an increment of shear strain is examined. By using the energy correction proposed by Taylor (1948), it can be derived that the increment of energy per unit volume of this soil element is

$$\frac{\tau_{yx}}{\sigma_{yy}}+\frac{d\varepsilon_{yy}}{d\gamma_{yx}}=m \qquad \ldots Eqn \quad 5.4$$

where τ_{yx}/σ_{yy} is the stress ratio measured on the central plane

and

m is a constant which equals the stress ratio when the rate of dilation is zero.

Assuming that the horizontal is a direction of zero extension and that the axes of principal stresses and principal strain increments coincide, Eqn 5.4 reduces to

$$\frac{\tau_{yx}}{\sigma_{yy}}-\tan\psi=\sin\phi_{cv} \qquad \ldots Eqn \quad 5.5$$

where ψ is the angle of dilation and

ϕ_{cv} is the critical state angle of friction (Jewell, 1989).

As measurements could not be made on the central plane, the boundary measurements of the stress ratio (τ/σ_v) and rate of dilation (dy/dx) were used in the analysis. For each test, the values between $(\tau/\sigma_v)_p$ and τ/σ_v at the end of the test were plotted against dy/dx. A selection of these graphs is presented in Figures 5.26 to 5.30. If there was a uniform zone of deforming material in the tests, then these plots should give 1:1 lines which would intersect the abscissa at $\sin \phi_{cv}$ (Jewell, 1989). This was not expected for the test data presented here because when a free top platen is used, rotation is apparent due to the non-symmetrical arrangement of the shear box (Jewell, 1989). Jewell (1989) also noted that measurement of the rate of dilation on the boundary of a shear box underestimates the rate of dilation on the central plane and he also found that even when a symmetrical direct shear box is used i.e. with a fixed top platen, the measured stress ratio is still higher than in a simple shear box arrangement (Jewell 1989). These conclusions by Jewell (1989) would suggest that the plot of τ/σ_v against dy/dx would give higher slopes than 1:1 for the data presented here.

In Figures 5.26 to 5.30, it can be seen that some data cross a 1:1 line and some have higher slopes. The data shown are for tests on reasonably dense material. The results from tests on loose samples were difficult to interpret as little data existed beyond the $(\tau/\sigma_v)_p$ value. For the following analysis a reasonably accurate value of ϕ_{cv} was needed.

Looking at Figures 5.26 and 5.27, it can be seen that two values of ϕ_{cv} could be obtained depending on whether a 1:1 line was drawn from the intersection of the data points with the abscissa or whether the line was drawn from the point of maximum rate of dilation back to the abscissa. Both lines were drawn on the graphs of test results where the data had a slope of less than 1 and consequently lower and upper limits of ϕ_{cv} were found. For plots where

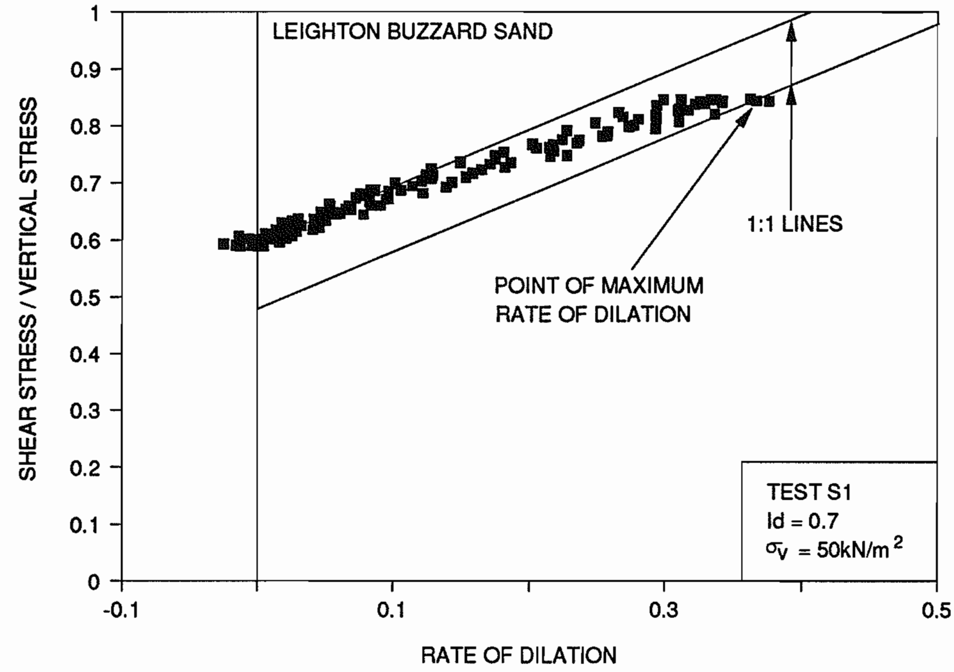


Figure 5.26 Relationship between stress ratio and rate of dilation for S1

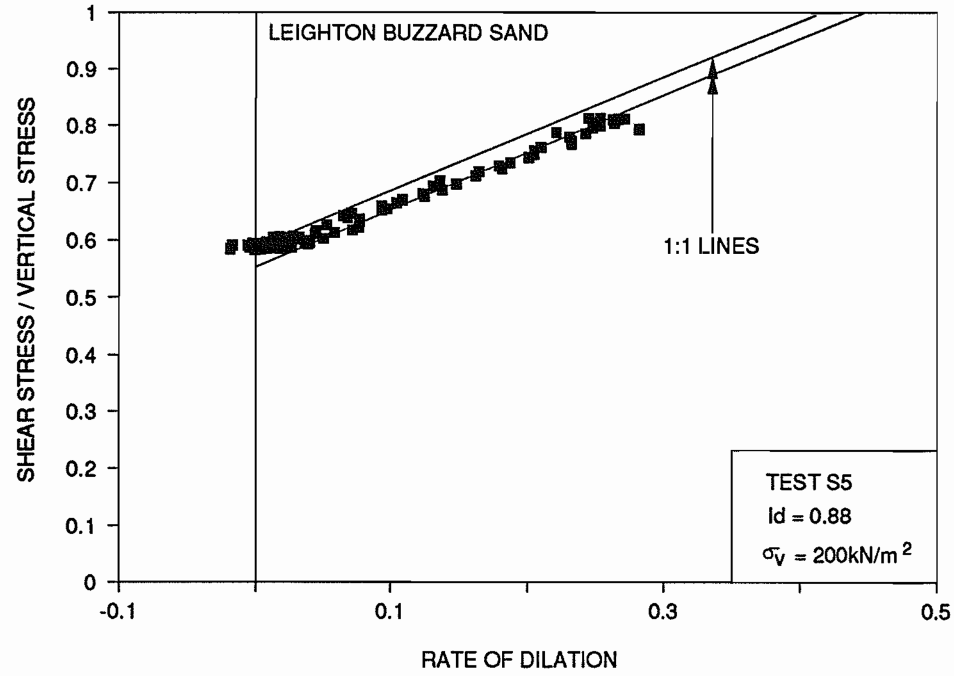


Figure 5.27 Relationship between stress ratio and rate of dilation for S5

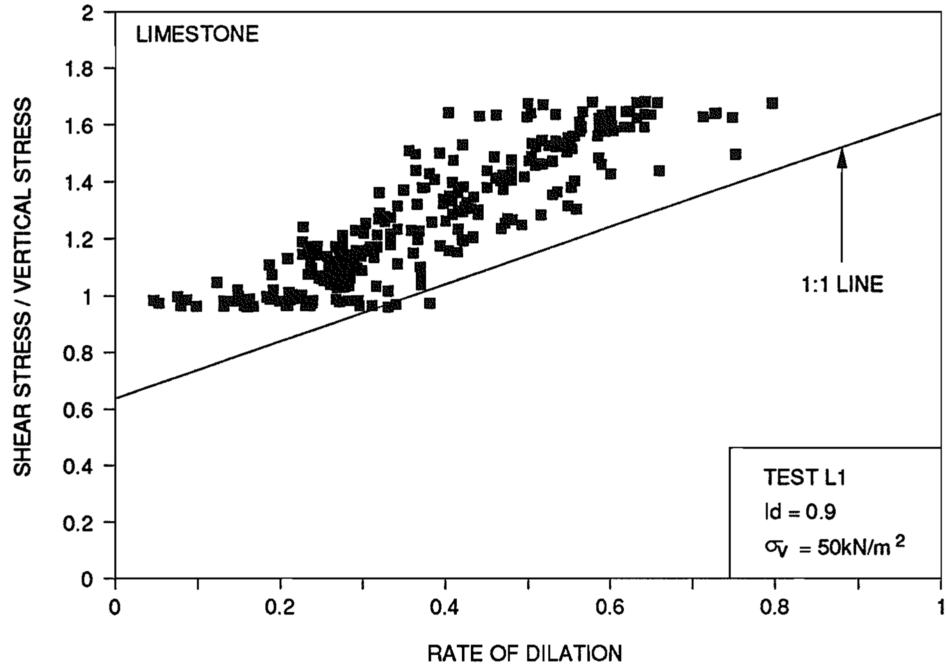


Figure 5.28 Relationship between stress ratio and rate of dilation for L1

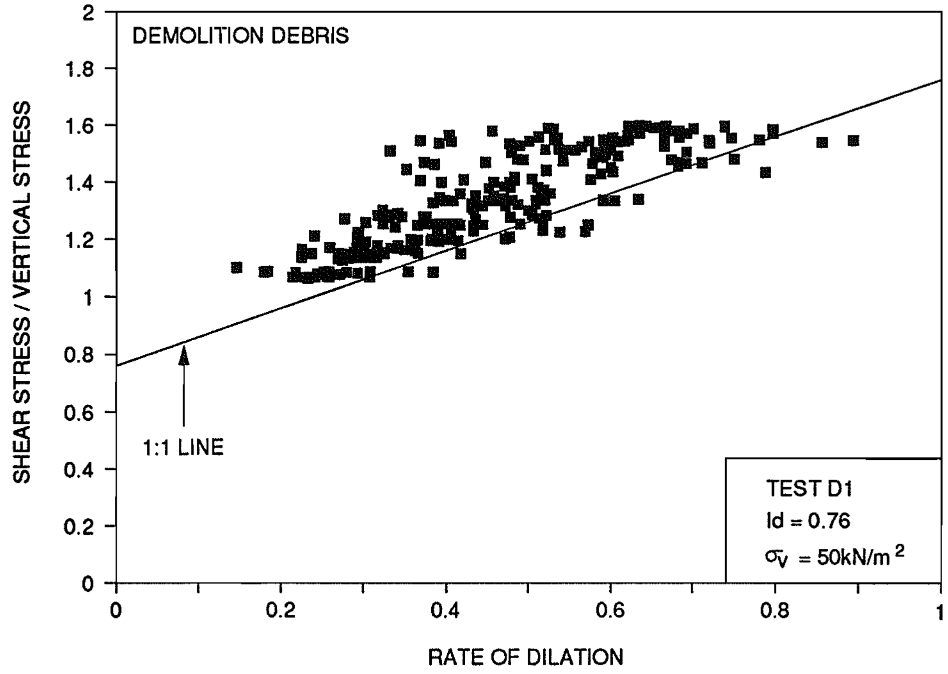


Figure 5.29 Relationship between stress ratio and rate of dilation for D1

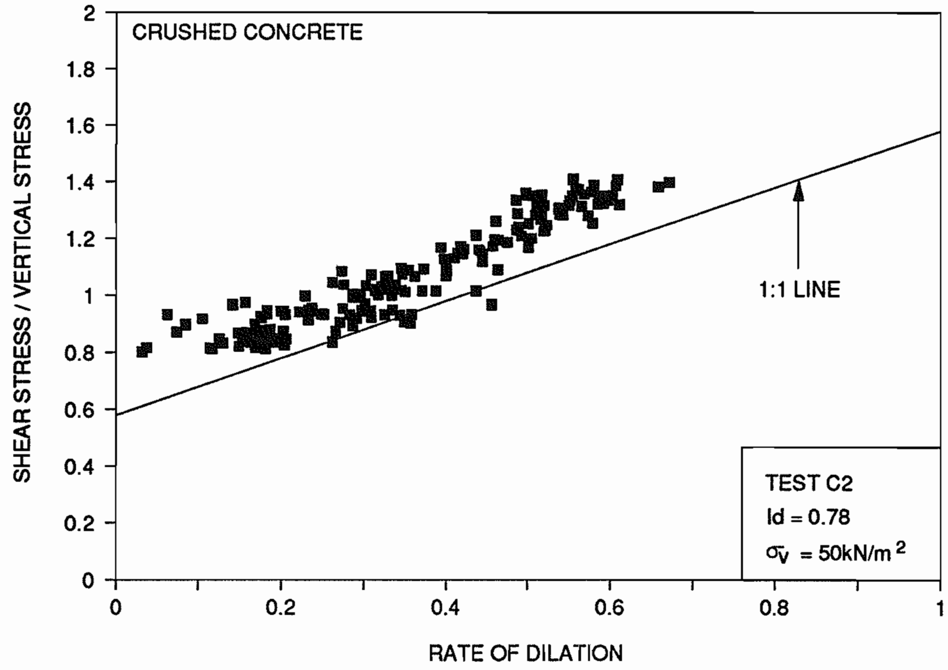


Figure 5.30 Relationship between stress ratio and rate of dilation for C2

the slope was greater than 1, the expected intersection of the data with the abscissa was taken to be ϕ_{cv}. This method of estimating ϕ_{cv} was similar to that used by Jewell (1989). The average upper and lower values of ϕ_{cv}, which are used in the following analysis, are listed in Table 5.7.

MATERIAL TYPE	ϕ_{cv} (degrees)	
	UPPER LIMIT	LOWER LIMIT
Leighton Buzzard sand	34.5	27
Limestone	45	35
Demolition debris	47	36
Crushed concrete	49	37

Table 5.7 Upper and lower limits of ϕ_{cv} used in the analysis

5.9.2 Analysis using dilatancy index

By using the values of ϕ_{cv} in Table 5.7 and the $(\phi_{ds})_p$ values for each of the tests, the peak plane strain angle of friction $(\phi_{ps})_p$ could be found using the following equation derived by Rowe (1969).

$$\tan\phi_{ds} = \tan\phi_{ps}\cos\phi_{cv} \qquad \ldots Eqn \quad 5.6$$

Bolton (1986) developed a consistent treatment of both density and confining pressure in shear box tests by using a dilatancy index (I_r) which he defined as follows:-

$$I_r = I_d(Q - \ln p') - R \qquad \ldots Eqn \quad 5.7$$

where I_d = relative density,

Q = a constant depending on material type,

p'= mean effective stress and

R = a constant = 1.

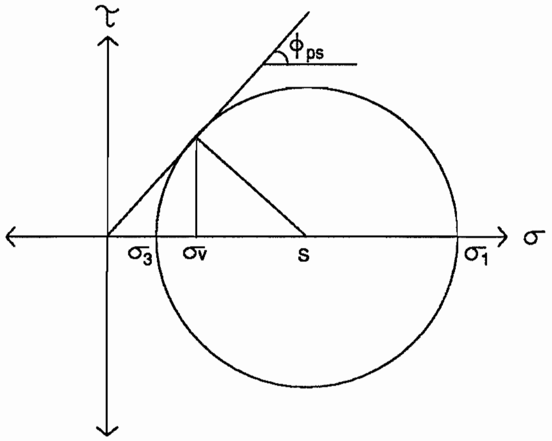
Bolton (1986) stated that Q was dependent on the compressibility and mineralogy of the particles of material and found Q = 10 for quartz and felspar sands. It was suggested that for other materials Q could range from 5.5 for chalk to 8 for limestone.

To find I_r, the mean effective stress p' must be calculated. A formula for p' can be derived using the geometry of the Mohr circle of stress shown in Figure 5.31.
The mean principal stress is defined as

$$s = \frac{\sigma_1 + \sigma_3}{2} \qquad \ldots Eqn \quad 5.8$$

and the maximum shear stress is

$$t = \frac{\sigma_1 - \sigma_3}{2} \qquad \ldots Eqn\ 5.9$$



σ_1 and σ_3 are the major and minor principal stresses.

Figure 5.31 Mohr circle of stress

Stroud (1971) found in a simple shear box arrangement that the intermediate stress σ_2 was 0.74s. By substituting this value into the equation for mean effective stress

$$p' = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \qquad \ldots Eqn\ 5.10$$

it can be concluded that s is a good approximation for p'. From the geometry in Figure 5.31, the mean effective stress can be defined as

$$p' = \frac{\sigma_v}{\text{Cos}^2 \phi_{ps}} \qquad \ldots Eqn\ 5.11$$

Each of the aggregates was dealt with separately but both series A and B for each material were used in the analysis, regardless of vertical stress or density. Assuming a start value of Q = 10, I_r was calculated for each test using Eqn 5.7. The I_r values were then plotted against the corresponding $(\phi_{ps})_p$ values. To examine the accuracy of the chosen value of Q, the following calculations were made.

Bolton (1986) stated that for plane strain

$$(\phi_{ps})_p - \phi_{cv} = 5I_r \qquad \ldots Eqn \quad 5.12$$

For a range of $(\phi_{ps})_p$ from 35^0 to 70^0, I_r values were calculated using Eqn 5.12 and the resulting linear relationship between $(\phi_{ps})_p$ and I_r was drawn on the same plot as the I_r obtained from the experimental data. By varying the value of Q in the calculations, the points on the graph were made to have the best fit to the calculated line.

This procedure was carried out for each material for the upper and lower limits of ϕ_{cv} and the results of the analysis can be seen in Figures 5.32 to 5.35. An intermediate value of ϕ_{cv} was obtained from these graphs. The determination for Leighton Buzzard sand in Figure 5.32 was influenced by the universal acceptance that ϕ_{cv} for Leighton Buzzard sand is 33^0. The final values of Q and ϕ_{cv} from the analysis are listed in Table 5.8 along with ϕ_{cv} values which were determined from the loose heap test (Cornforth, 1973).

Cornforth (1973) stated that an approximation of ϕ_{cv} could be made by tipping a material quickly to form a loose heap and excavating from the toe until a smooth slope is formed. The angle of this slope to the horizontal is ϕ_{cv} and Cornforth (1973) found that it could be measured to an accuracy of 1^0.

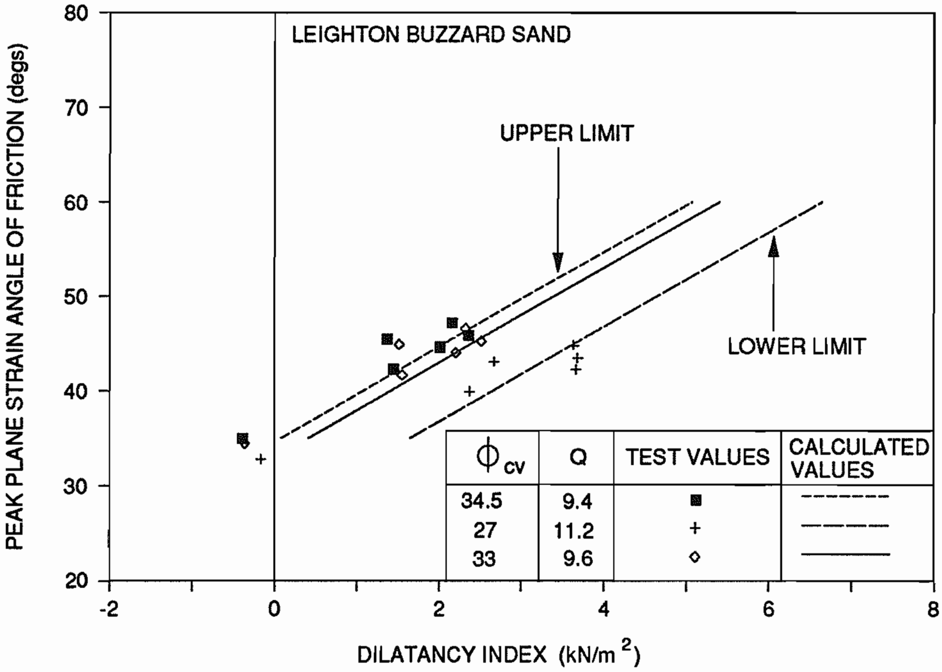


Figure 5.32 Determination of Q for Leighton Buzzard sand

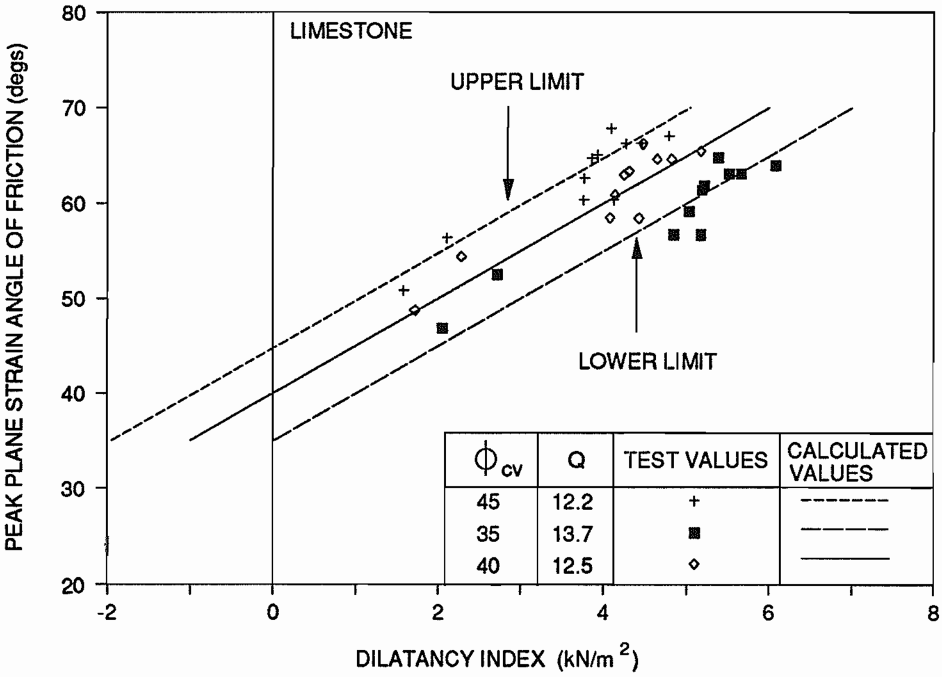


Figure 5.33 Determination of Q for limestone

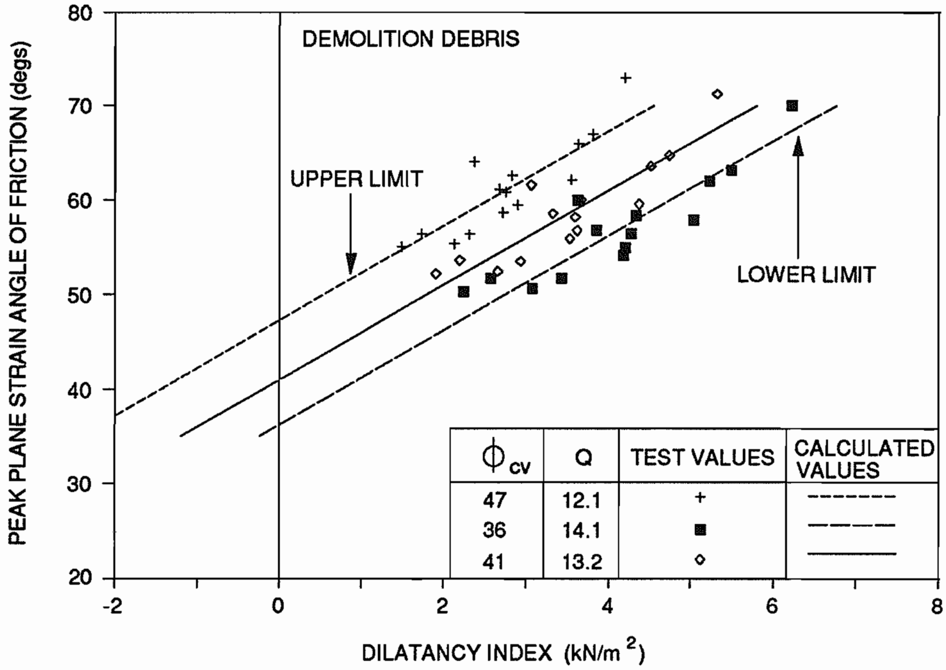


Figure 5.34 Determination of Q for demolition debris

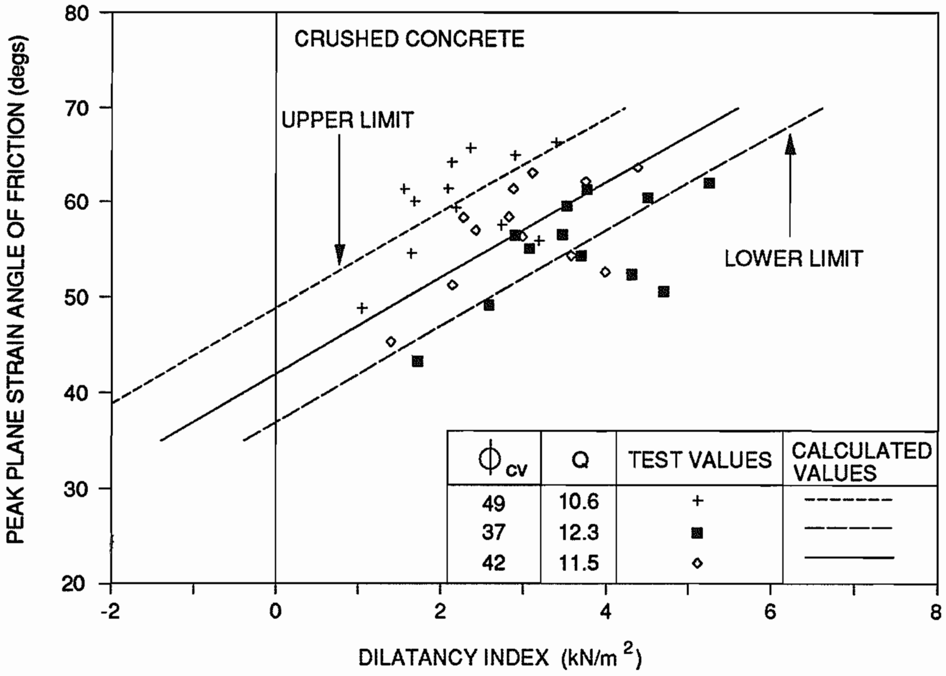


Figure 5.35 Determination of Q for crushed concrete

MATERIAL TYPE	ϕ_{cv} (degrees)	Q	ϕ_{cv} (degrees) Loose heap
Leighton Buzzard sand	33	9.6	33
Limestone	40	12.5	42.5
Demolition debris	41	13.2	37
Crushed concrete	42	11.5	39.5

Table 5.8 Results of analysis on shear box test data

A summary of the test data and the results of the analysis can be seen in Figures 5.36 to 5.39. For each material, calculated values of $(\phi_{ps})_p$ were plotted against p' for a range of I_d between 0.2 and 1. To obtain these calculated values, I_r was first determined for a range of I_d and p' using Eqn 5.7 and then $(\phi_{ps})_p$ was found using Eqn 5.12. The I_d of the test samples were also plotted as points on the figures. In general, the calculated values appeared to match the test data quite well.

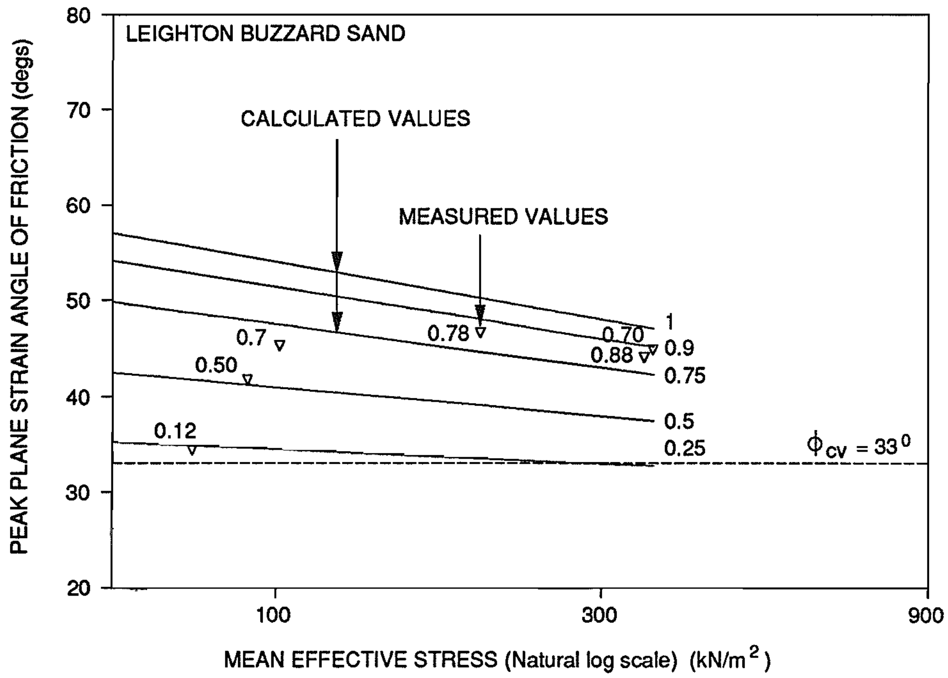


Figure 5.36 Comparison between calculations and experimental data for Leighton Buzzard sand

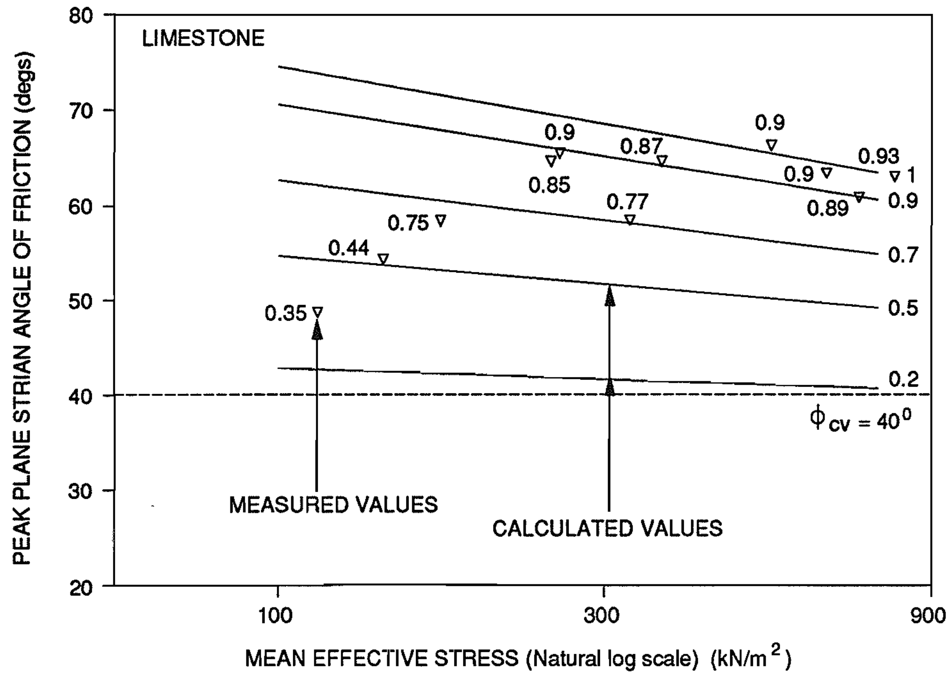


Figure 5.37 Comparison between calculations and experimental data for limestone

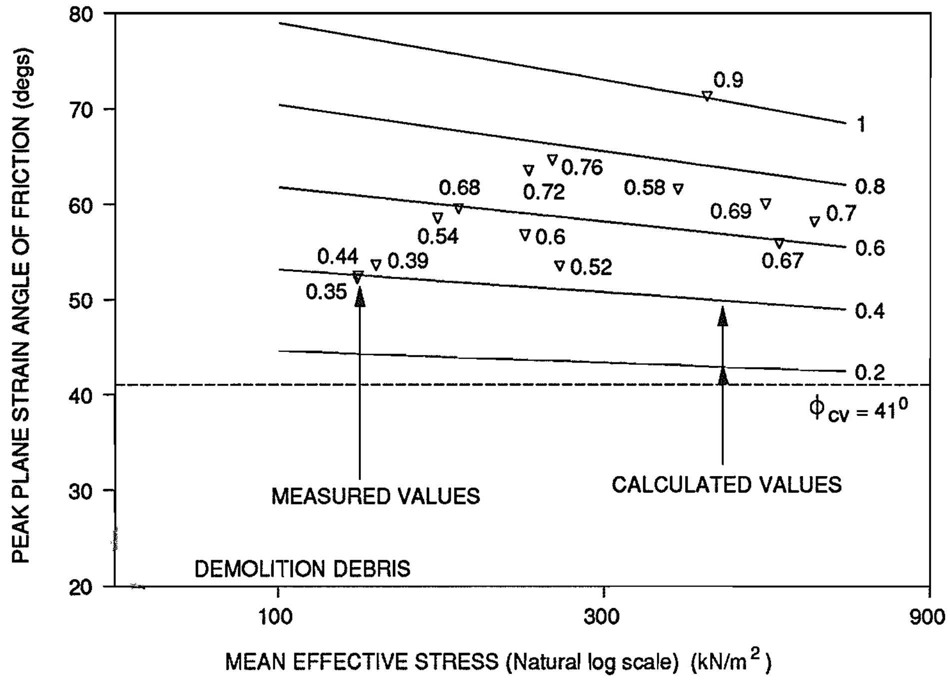


Figure 5.38 Comparison between calculations and experimental data for demolition debris

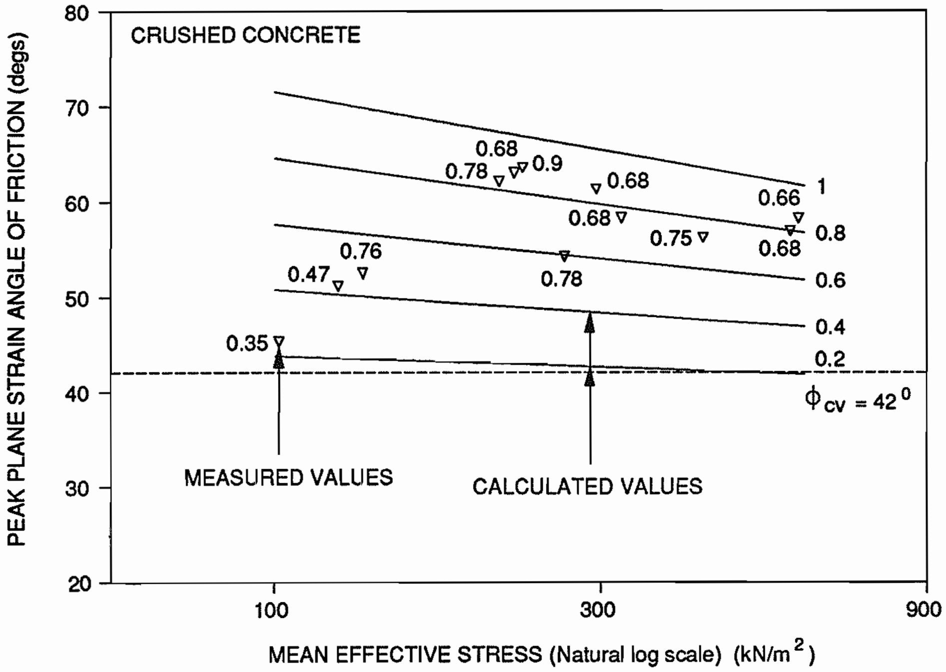


Figure 5.39 Comparison between calculations and experimental data for crushed concrete

5.10 Discussion

It is clear from Figure 5.22 that demolition debris and crushed concrete could be used successfully as backfill to structures because $(\phi_{ds})_p$ for both materials was found to be quite high, particularly for dense samples. These recycled aggregates could be considered as lightweight and this property along with high shear strength could be useful in operations where high quality gravel is used at present. In the TRRL report by Naish (1988), an investigation of the cost of backfill around bridge abutments and retaining walls was summarised. It was estimated that the use of a lower quality fill instead of granular fill, as defined by the Specification for Highway Works (1986), would reduce the cost of each structure by about £18,000 which would represent a reduction from 9% to 5% of the total cost of the structure. The use of recycled aggregates might reduce this cost further depending on the location of a particular site and the cost of transport.

Some tests, conducted when the chamber surrounding the shear box was full of water, exhibited low values of $(\phi_{ds})_p$ compared with the results of tests carried out when the chamber was dry. These lower values could be explained if pore suction had developed in the materials or if more lubrication between particles was provided by the additional water. However, some samples which were tested when the chamber was flooded exhibited higher shear strength. This is difficult to explain and it can only be concluded that the scatter in the data, caused by the variation in particle size of the samples, is too large to determine whether pore suction existed. Pore suction is normally associated with materials containing small grains and is unlikely to be a significant factor in the tests on the aggregates described earlier.

Bolton (1986) stated that Q depends on the mineralogy and compressibility of a material and suggested that Q should be lower for a material containing soft grains. It is not clear in his paper whether this recommendation was made specifically for small grained, uniform materials. It appears from the results quoted earlier in Section 5.9.2 that Q may be dependent on the coefficient of uniformity, the ratio of the shear box dimensions to the maximum particle size or angularity.

It was found by Bolton (1986) for tests on sands that Eqn 5.12 could be used between the limits of $0 < I_r < 4$ and he suggested that the upper limit of 4 should be used unless good low stress data was obtained for a particular material. This recommendation was not followed in this research because much of the test data was obtained at low stress levels and would have had to be ignored. All data from the tests on aggregates fitted the correlations well but these data suggest that the limit of

$$(\phi_{ps})_p - \phi_{cv} = 20° \qquad \ldots Eqn \quad 5.13$$

is too conservative for well graded aggregates and should be extended to

$$(\phi_{ps})_p - \phi_{cv} = 25° \qquad \ldots Eqn \quad 5.14$$

i.e. raising the upper limit of I_r to 5. It is agreed that Bolton's (1986) recommendation should be adhered to for sand as the data from this research were within the limits of Eqn 5.13.

When comparing the ϕ_{cv} values in Table 5.8, it can be seen that demolition debris achieved a value of 37^0 in the loose heap test which was 4^0 lower than that obtained in the analysis. However, the loose heap test result for limestone was 2.5^0 higher than the result from the analysis and for crushed concrete it was 2.5^0 lower. The accuracy of the loose heap test on well graded aggregates therefore was lower than the 1^0 accuracy which Cornforth (1973) found for tests on sands. However, it would be expected that the slope of a heap of material containing non-uniform and large particles would not be as well defined as that of a heap of sand where the particles are small and uniform. The mass of sand tested in the loose heap test was 2kg whereas the mass of aggregate tested was 15kg. The results of the tests on the aggregates suggest that an even larger quantity of aggregate would be required to obtain an accuracy of 1^0.

5.11 Conclusions

(i) There appeared to be little difference between the shear strength of limestone and that of the recycled aggregates and friction angles between 54^0 and 58^0 were obtained at high densities.

(ii) With regard to Earland and Pike's (1985) shear box test for the determination of the suitability of aggregates for use as road sub-base material, limestone would be classed as high strength and the recycled materials as medium strength aggregates.

(iii) The effect of flooding the outer chamber of the shear box apparatus caused a reduction in strength for some tests but an increase in others. The effect of pore suction could not be determined quantitatively due to the scatter in the results. It is likely that this scatter was caused by the variation in particle size and content of the aggregates.

(iv) Although Bolton (1986) suggested that materials containing grains which were softer than quartz or felspar should have Q values (constant in the formula for dilatancy index) of less than 10, the Q values determined for the aggregates in this study ranged between 11.5 and 13.2.

(v) The critical state angles of friction of the aggregates were found to be between 40^0 and 42^0 but the difference between the calculated and the measured values using the loose heap test varied between 2.5^0 and 4^0. It was concluded that the 1^0 accuracy which Cornforth (1973) found for sands in the loose heap test could not be achieved for well graded aggregates unless very large quantities of material were used.

CHAPTER 6

FROST SUSCEPTIBILITY TESTS

6.1 Introduction

No materials placed within 450mm of any road surface in Britain should be susceptible to frost, as defined by the Transport and Road Research Laboratory test described by Roe and Webster (1984). Although long periods of freezing are uncommon in the British climate, severe winters do occur. These winters include 40 consecutive or near consecutive days of frost, the last one having been the winter of 1962/63.

It has been found that there is an increase in the number of road failures during and following severe winters. Deterioration of a road pavement can occur in three ways.

a) When water penetrates the road surface, damage can be caused by the expansion of water as ice forms. This type of deterioration can be avoided by better construction and maintenance techniques and particularly if the road surface is sealed.

b) A more serious type of damage can be caused to road surfaces by the formation of ice lenses in the lower layers which causes the road pavement to heave.

c) When a pavement has been damaged by either of the ways described in **a)** or **b)**, a further loss of strength may occur when the ice thaws because the material will have a higher moisture content and therefore a reduced bearing capacity. In this condition, the road is more likely to fail under traffic loading.

The frost heave test, described by Roe and Webster (1984), is a laboratory test simulating frost heave in the field. In this research, the frost heave of limestone and recycled aggregates was examined because their potential use within 450mm of a road surface would require them not to be susceptible to frost, as stipulated by the Specification for Highway Works (1986).

6.2 The explanation of frost heave

The frost heave process was described by Croney and Jacobs (1967) and a summary of their explanation is given below. During an isolated cold night, when the temperature of the air above the road surface is several degrees below freezing point, the zero isotherm may penetrate the pavement as much as 100mm. If, during the following day, the temperature of the air rises above 0^0C, the low temperatures in the pavement will also rise above freezing point within a few hours. However, if the temperature remains below 0^0C for about three days, the zero isotherm may penetrate up to 300mm and following a ten day period, penetration may be more than 375mm. From measurements made by Croney and Jacobs (1967) during the winter of 1962/63, it was deduced that a much slower rate of penetration occurred at depths greater than 375mm. If after ten days, for example, the air temperature rises above freezing point for a few days this would not reduce the maximum penetration of the frost in the pavement but the temperature in the top 100mm could rise above 0^0C. During a particularly severe winter, the surfacing and the top part of the base may be subjected to many cycles of freezing and thawing but the lower layers would remain frozen and the freezing front (boundary of the freezing zone) would continue to move downwards.

The layers in a road pavement which are most susceptible to frost heave are the sub-base and the subgrade. The pore spaces in these layers are generally large enough to accommodate the expansion of water contained in them when freezing starts. However, during freezing more water rises from the unfrozen material by capillary action. The pore spaces do not have the capacity to hold this extra water when it expands on freezing. Figure 6.1 illustrates the situation in a road pavement when the top layer becomes frozen. Croney and Jacobs (1967) explained the capillary movement which causes the upward flow of water as follows:-

At temperatures above 0^{0}C, the water in moist porous materials has a negative pressure or suction which is due to the surface tension and absorption forces (Croney and Jacobs, 1967). Suction increases rapidly with decreasing moisture content. Croney and Jacobs (1967) conducted tests under a 150mm thick concrete pavement in Harmondsworth in January 1963. It was found in these tests when water froze in the pores of a material that a pressure gradient developed between the ice and the unfrozen water below. Above the level of the zero isotherm the suction increased to a value between 10 and 100 times the suction below the freezing zone. It was concluded from these tests that the rate at which the water flowed upwards depended on the permeability of the unfrozen material and the amount of water present.

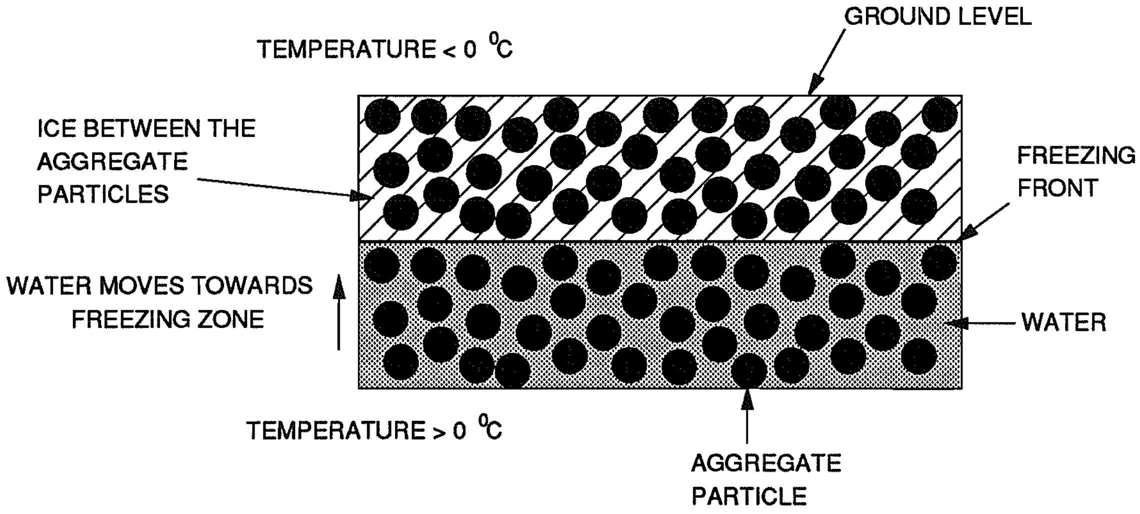


Figure 6.1 Situation in a road pavement when freezing starts

It was found by Croney and Jacobs (1967) that clean granular materials do not have the potential to hold enough water to cause frost heave. Frost heave is inhibited in clays because of low permeability even though clays are capable of retaining large quantities of water. Burns (1977) conducted full scale tests on road pavements constructed in 1.5m deep test pits at TRRL. It was concluded from these tests that the frost heave of most materials would be significantly reduced if the water table was low.

When a road pavement thaws, the water which was drawn up during freezing drains downwards at a rate which is dependent on the permeability of the material. As the water drains, it draws some smaller particles with it causing a decrease in the dry density of the material. Therefore if frost heave occurs again at a later stage the material should be capable of holding more water than before and this is likely to cause further damage to the pavement.

6.3 Development of the frost heave test

The development of a frost heave test started before 1967 and its progression is listed in a series of TRRL reports. The first report was written by Croney and Jacobs (1967) but this report did not describe the test in such a way that it could be carried out in a standardised manner. Therefore another report SR318 was produced by TRRL in 1977 to provide a description of the test which was less ambiguous. In 1981, some changes were introduced to make the test more stringent and another report MM64 was written by TRRL. The existing standard test description is SR829 which was written by Roe and Webster in 1984.

The aim of the report by Croney and Jacobs (1967) was to classify materials as frost susceptible, marginal or not frost susceptible. It included an appendix containing typical frost heave results for a range of materials. The object of this information was to help engineers to decide on the suitability of particular aggregates for pavement construction.

The test was only to be carried out on materials which fell in the marginal category so that tests and costs would be kept to a minimum. However the test was included as a compliance test in Road Note 29 written by the DoE in 1970.

6.4 Frost heave testing

The frost heave tests were carried out at Nottingham University on carboniferous limestone, demolition debris and crushed concrete. The tests involved placing compacted specimens of material in a chamber with the bottom ends of the specimens in contact with water. The temperature of the water was maintained at a constant temperature of 4^0C but the air temperature was reduced well below freezing to -17^0C. The resulting heave of the aggregate was measured after 96 hours.

6.4.1 Preparing the test specimens

The procedure for preparing the specimens is described in detail by Roe and Webster (1984). Trial samples were made a few days before the test to confirm that samples of adequate stability could be produced at the optimum moisture content (OMC) and peak dry density ($\rho_{d,peak}$) values which were obtained using the compactibility test for aggregates described in BS 5835 (1980). The material was oven-dried and particle grading tests were conducted on the samples before and after preparation of the frost heave specimens. Particles greater than 37.5mm in size were removed. The amount of water required to bring the material to its optimum moisture content was added and mixing took place in a concrete mixer. The aggregate was then allowed to equilibrate overnight in a sealed container.

Each sample was compacted in a tapered, steel cylindrical mould of internal diameter 102mm with end plugs, as shown in Figure 6.2. The bottom end plug was placed on the floor and the steel cylinder was slid over it. The required mass of material needed for the specimen had

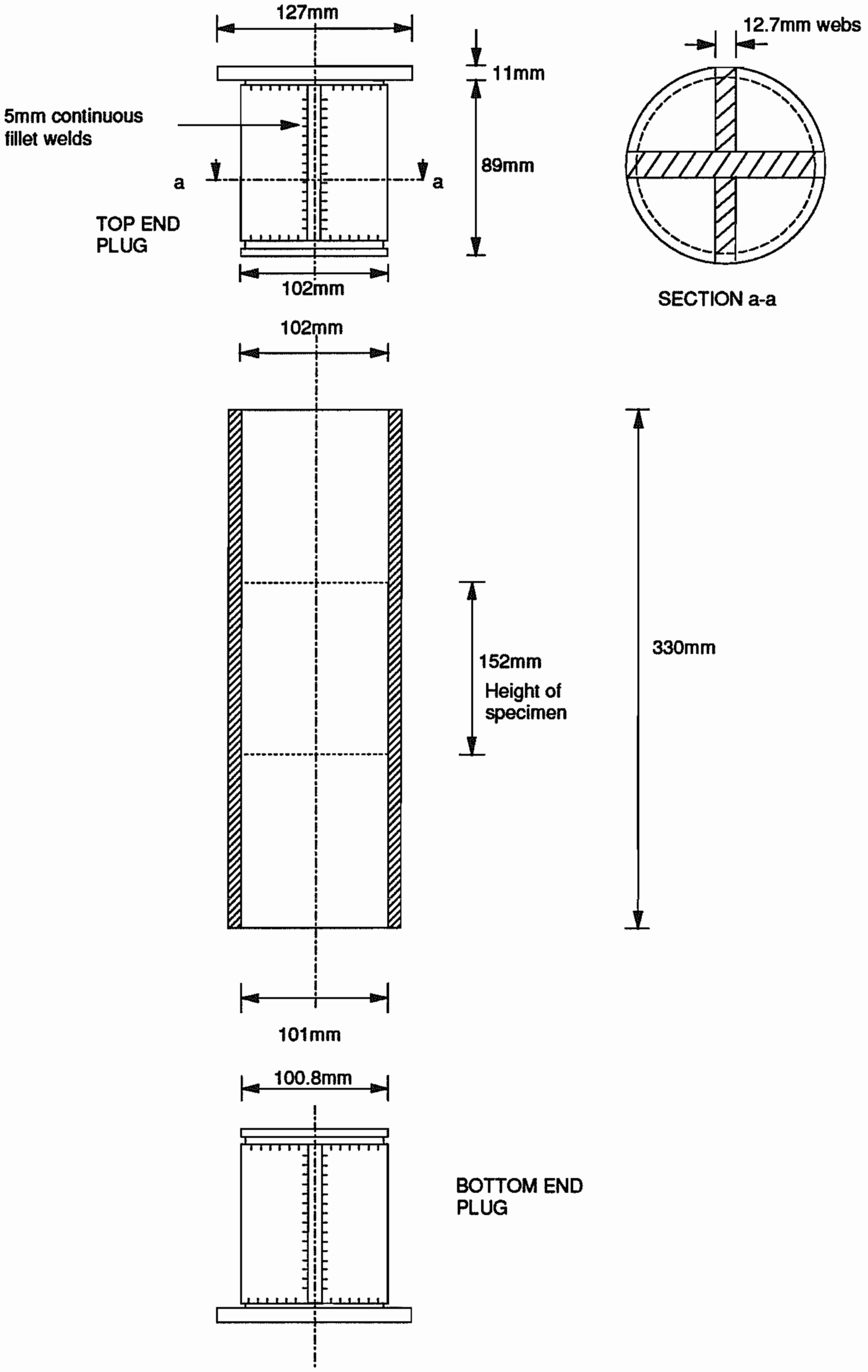


Figure 6.2 Mould and end plugs for sample preparation (after Roe and Webster, 1984)

been calculated beforehand by using the volume of the container and $\rho_{d,peak}$. One third of this mass was placed in the mould and levelled roughly. The layer was compacted using a vibrating hammer connected to a flat, circular plate (illustrated in Plate 6.1) until the material was about 185mm from the top of the mould. Another portion of the material was added and compacted until the top of the layer was 135mm from the top of the mould. The last third was added and compacted slightly before the top end plug was inserted into the cylinder. The vibrating hammer was applied to the plug and compaction was continued until the gap between the mould and the plug caps was not greater than 4mm in total. The height of the sample was 154mm with a 2mm tolerance.

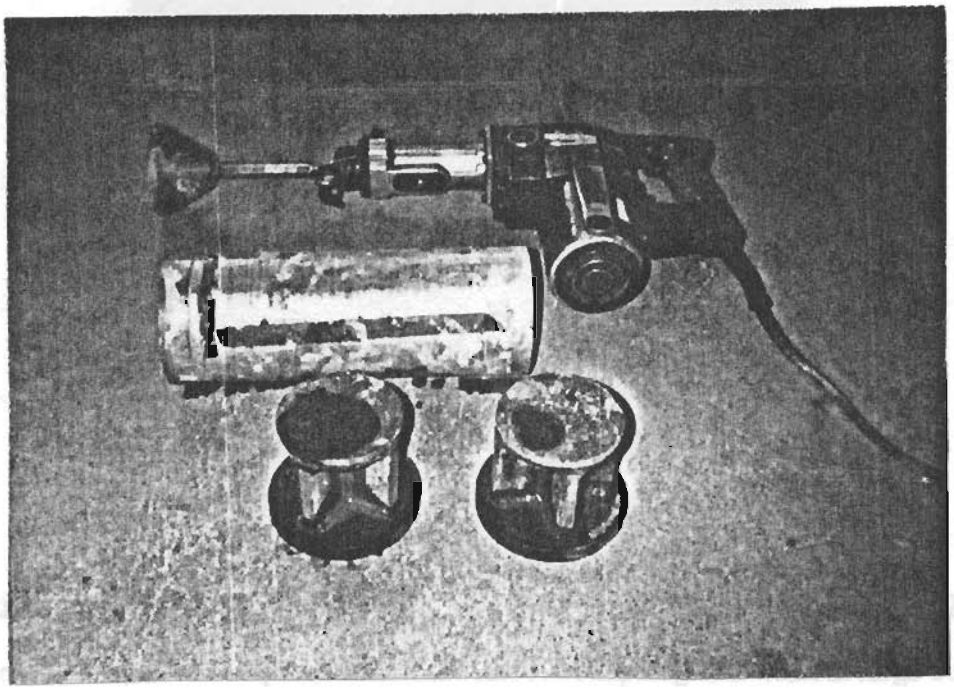


Plate 6.1 Vibrating hammer and mould for the preparation of frost heave test specimens

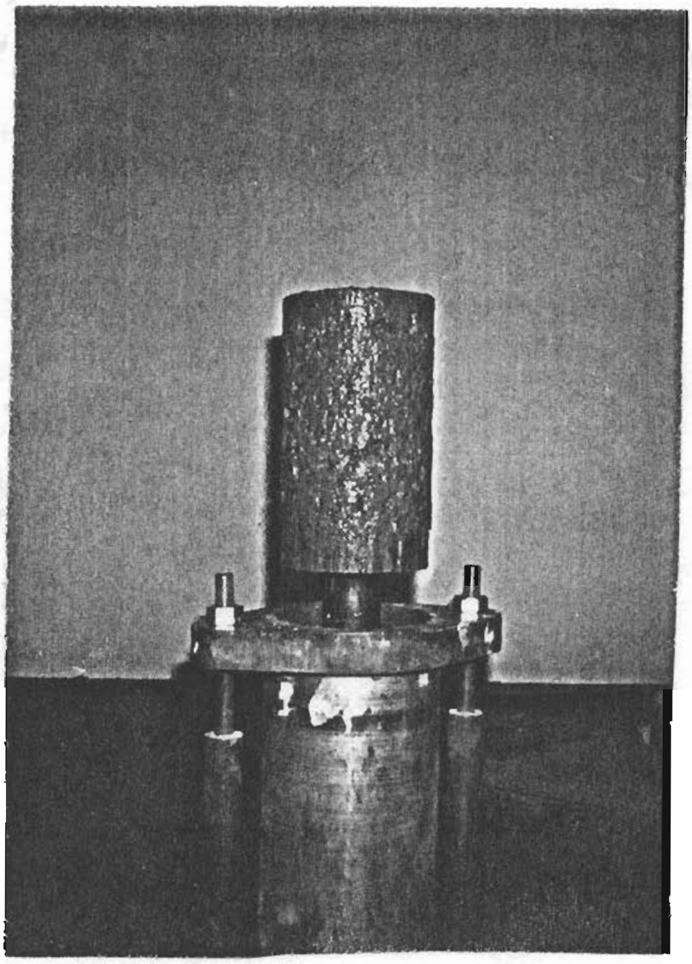


Plate 6.2 Stable sample after extrusion

Both end plugs were removed and the sample was extruded, as illustrated in Plate 6.2. After extrusion the sample was required to stand without collapsing and without large quantities of the aggregate falling away. A waxed sheet of paper was wrapped around the specimen and was secured with adhesive tape leaving a 50mm length of paper protruding above the top of the specimen. A 95mm diameter Tufnol disc of 5mm thickness, with a 10mm diameter recess located centrally in one side, was placed on top of the specimen with the recess uppermost. The specimen was then placed on a 102mm diameter porous ceramic disc of 13mm thickness with a pore size of 110 µm. The sample and the discs were placed in a copper carrier. The specimen attachments were weighed before sample preparation and were weighed again with the specimen so that the actual density achieved after compaction could be determined. The discs and carrier are shown in Plate 6.3.

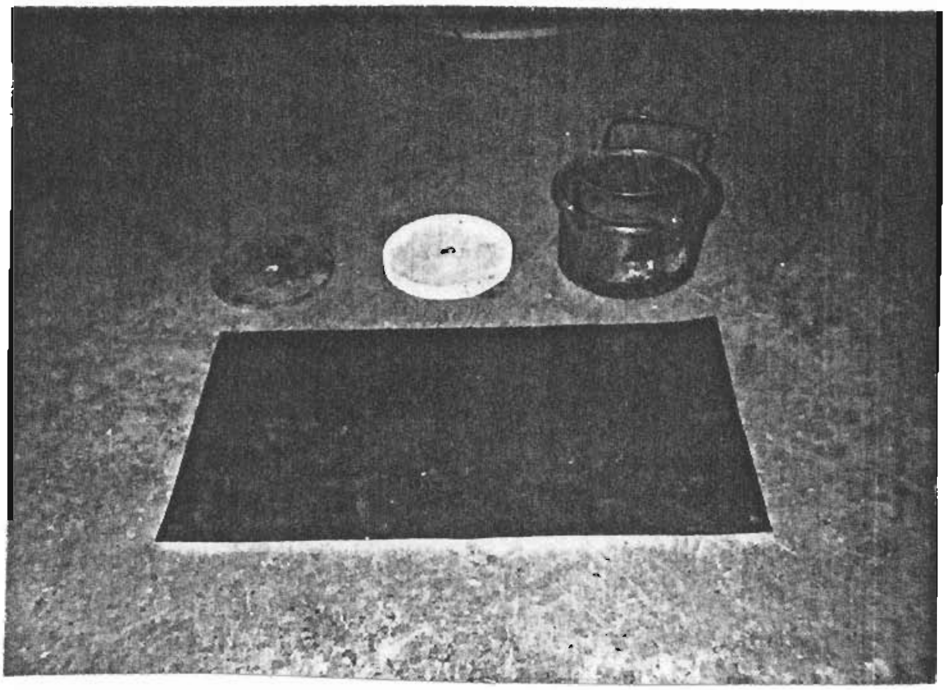


Plate 6.3 Discs and specimen carrier used to support the frost heave specimens

6.4.2 Test conditions

It was stated earlier that frost heave is dependent on the flow of water from below. However, it was decided to test the three aggregates at two other moisture contents as well as OMC, one below and one above OMC, to establish whether frost heave was also influenced by the moisture content at which the material was placed. To examine a wide range of moisture content, it was decided initially that tests should be carried out at 0.5 OMC and 1.5 OMC.

Three samples at each test condition were tested in accordance with Roe and Webster (1984). Table 6.1 includes the moisture content and density values at which the limestone specimens were tested.

TEST REF.No.	COMPACTION TARGET VALUES			MEASURED VALUES	
	ρ_b (kg/m^3)	MOISTURE CONTENT (%)	ρ_d (kg/m^3)	MOISTURE CONTENT (%)	ρ_d (kg/m^3)
L1	2361	1.75	2320	1.98 (0.56 OMC)	2074
L2	2401	3.5	2320	3.34 (0.95 OMC)	2316
L3	2442	5.25	2320	3.8 (1.08 OMC)	2071

Table 6.1 Test conditions of the limestone specimens

Note: ρ_b and ρ_d denote bulk density and dry density respectively

The target density and moisture content for L2 were $\rho_{d,peak}$ and OMC. Although samples L1 and L3 were prepared at different moisture contents, the target density remained at 2320kg/m^3 so that stable samples could be obtained. However, it was clear during compaction that this density could not be achieved for these two test conditions. Compaction of these samples was continued until an increase in compaction time did not change the volume of the material. When this stage had been reached the specimens were extruded. As they remained stable, it was decided to use these samples in the frost heave test. It can be seen in Table 6.1 that L3 had a moisture content much lower than the target value. This was due to a large quantity of water running from the sample during mixing.

Initially, the choice of test conditions for demolition debris was similar to that for limestone. The demolition debris had a water absorption of 8% which was much higher than the value of 0.45% for limestone and consequently the moisture contents of the samples for the frost heave tests on demolition debris were also higher. However, it was difficult to obtain stable samples at a moisture content of 0.5 OMC. The OMC for demolition debris was found to be 13% using the BS 5835 (1980) compaction test and consequently 0.5 OMC was lower than the water absorption value. Therefore there was not enough water present in these samples to bind the aggregate together. It was also impossible to obtain stable samples of the material at a moisture content of 1.5 OMC. To rectify this situation new target moisture contents were calculated as follows:-

$$\text{Low moisture content} = OMC - (OMC - W_a)/2 \qquad \ldots Eqn \quad 6.1$$

$$\text{High moisture content} = OMC + (OMC - W_a)/2 \qquad \ldots Eqn \quad 6.2$$

where W_a was the water absorption of the aggregate.

Stable samples could be obtained when these moisture contents were used. The test conditions of the demolition debris specimens are listed in Table 6.2.

TEST REF.No.	COMPACTION TARGET VALUES			MEASURED VALUES	
	ρ_b (kg/m^3)	MOISTURE CONTENT (%)	ρ_d (kg/m^3)	MOISTURE CONTENT (%)	ρ_d (kg/m^3)
D1	2010	10.5	1820	10.94 (0.84 OMC)	1815
D2	2060	13	1820	13 (OMC)	1824
D3	2090	15	1820	14.6 (1.12 OMC)	1802

Table 6.2 Test conditions of the demolition debris specimens

Crushed concrete had a water absorption value lower than 0.5 OMC so the same approach was adopted as that for limestone i.e. the target moisture content values were 0.5 OMC, OMC and 1.5 OMC. When the trial specimens were prepared it was concluded, after several attempts, that the target density of 2000kg/m^3 for C3 was too high. Therefore the target value for C3 was changed to the maximum density which could be obtained for this test condition in the trial samples. The moisture content and density values of the crushed concrete samples are listed in Table 6.3.

TEST REF.No.	COMPACTION TARGET VALUES			MEASURED VALUES	
	ρ_b (kg/m^3)	MOISTURE CONTENT (%)	ρ_d (kg/m^3)	MOISTURE CONTENT (%)	ρ_d (kg/m^3)
C1	2103	5.13	2000	6.3 (0.6 OMC)	1838
C2	2205	10.25	2000	8.3 (0.8 OMC)	2002
C3	2125	15.5	1840	13.2 (1.28 OMC)	1904

Table 6.3 Test conditions of the crushed concrete specimens

6.4.3 Setting up the self-refrigerating unit

The self-refrigerating unit (SRU), in which freezing was carried out, consisted of a large insulated box of internal dimensions 600mm x 600mm x 550mm. A wooden cradle for holding nine specimens was placed on supports above the water bath at the bottom of the chamber. Roe and Webster (1984) required that three specimens should be tested at each test condition, so for one test run in the SRU an aggregate at three moisture contents was tested. The SRU can be seen in detail in Figure 6.3 and the cradle is shown in Figure 6.4. The height of water in the water bath was maintained level with a constant level device (CLD) which was located at the side of the SRU. The test procedure by Roe and Webster (1984) states that the water should be maintained at a level so that the top of the porous disc under each specimen is not covered with water but damp i.e. the water level is about 1mm below the top of the porous disc.

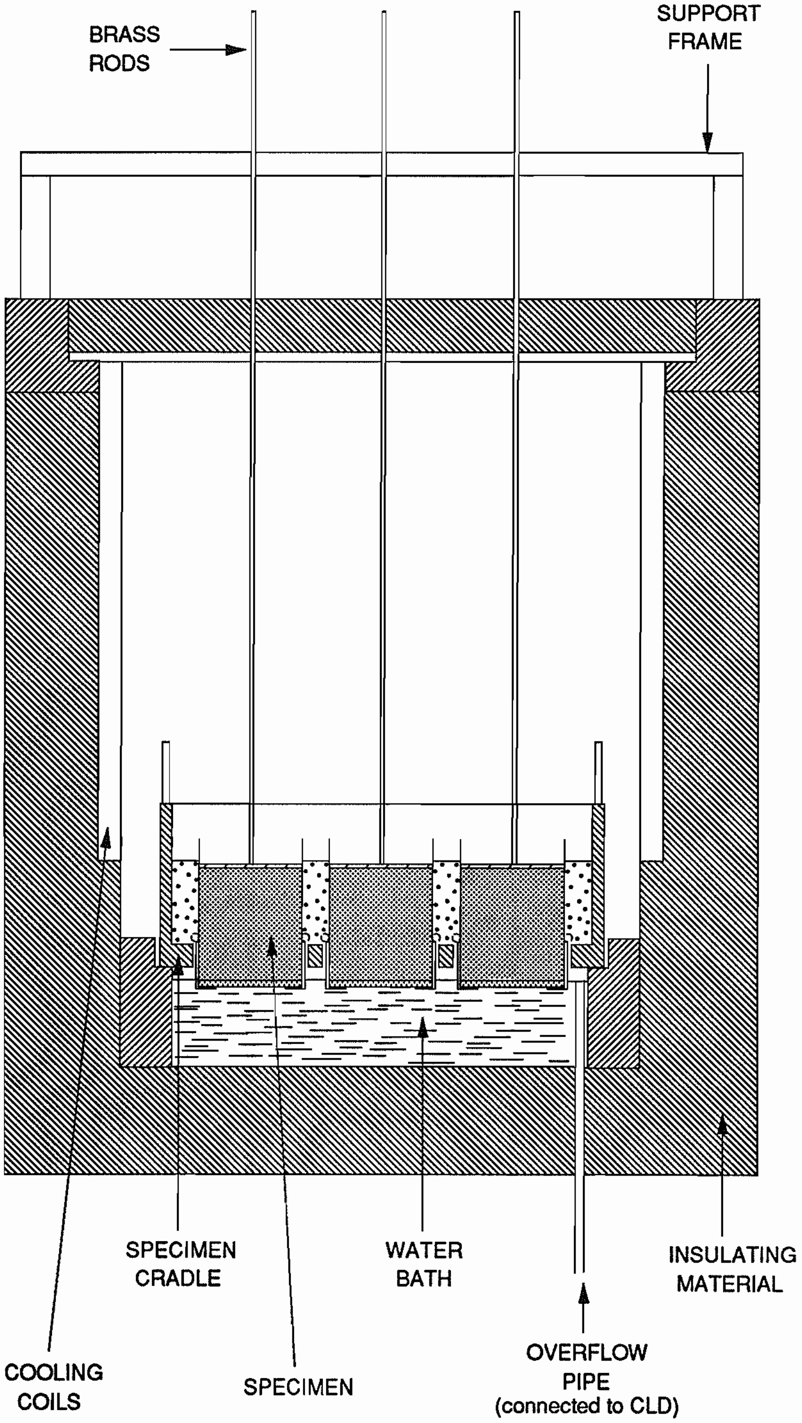


Figure 6.3 The self-refrigerating unit (after Roe and Webster, 1984)

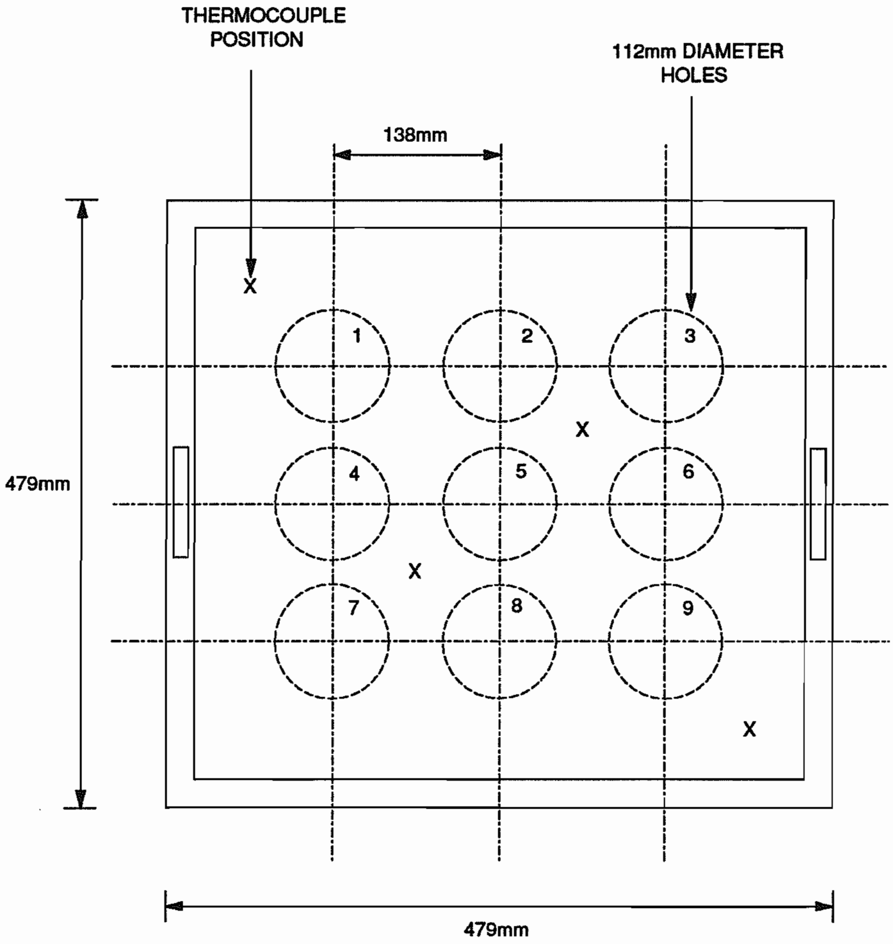


Figure 6.4 Specimen cradle (after Roe and Webster, 1984)

6.4.4 Placing the specimens in the SRU

Following the test procedure described by Roe and Webster (1984), each specimen was placed in a hole in the cradle. The holes were numbered in the sequence shown in Figure 6.4. The specimens to be placed in holes 1,3,5,7 and 9 had thermocouples placed between the carriers and the porous discs so that the temperature of the water could be monitored daily. Another thermocouple was placed under the Tufnol disc at the top of specimen 5 so that the junction of the thermocouple was in contact with the top of the specimen.

When the specimens were in position in the cradle and the thermocouples had been placed, the cradle was filled with a coarse sand to the same level as the tops of the specimens. Four more thermocouples were attached to wooden rods so that their junctions were 50mm from the bottom of the rods. These thermocouples were then placed in the sand to correspond with the positions marked X in Figure 6.4. The lid of the SRU was then closed.

To monitor the heave of the specimens, brass rods were passed through a datum frame on top of the SRU (Plate 6.4) and through the lid of the SRU until the ends were located in the central recesses of the Tufnol discs on top of the specimens. Small pieces of cotton wool were placed loosely in the gaps between the rods and the lid to prevent ice forming around the rods.

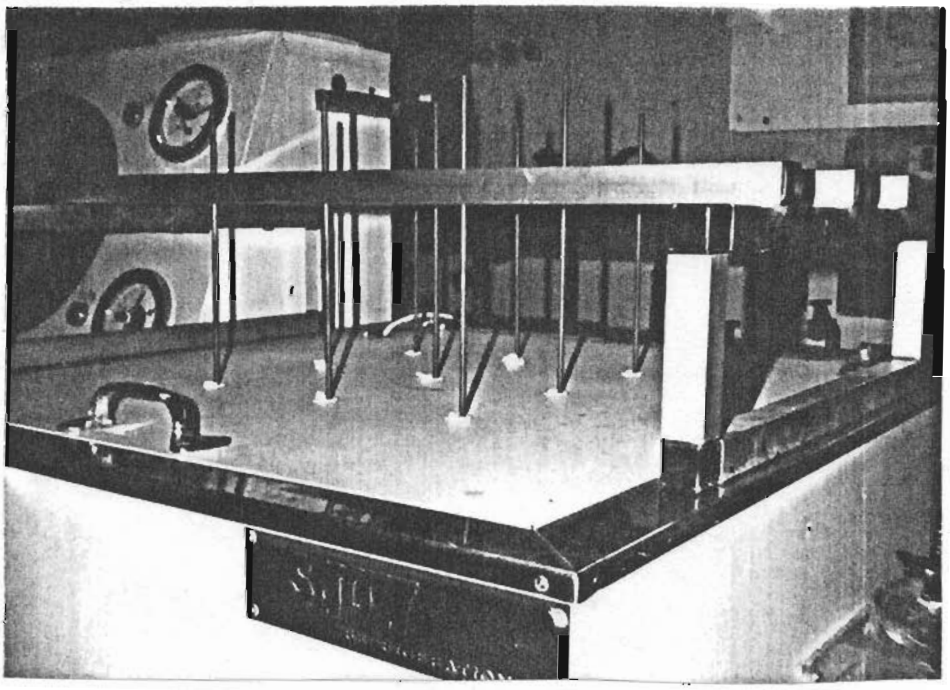


Plate 6.4 Brass rods protruding from the SRU

The SRU was then left at room temperature for 115 hours so that the specimens could equilibrate. When this period had elapsed, the distances between the tops of the brass rods and the top of the lid were measured to the nearest 0.5mm and the temperature of each thermocouple was recorded. The controls on the SRU for maintaining the correct water and air temperatures were switched on. The water was kept at a temperature of between 3^0C and 4.5^0C and the air temperature was maintained in the range -16^0C to -18^0C. The temperatures were continually monitored during the test using a chart recorder. If at any time the temperatures were found to be outside the limits listed above, the test would have had to be abandoned. Frost heave was recorded every 24 hours and at 96 hours the last set of readings was taken.

6.5 Classification of materials

To determine whether a material in a particular test condition is frost susceptible, the mean frost heave of three specimens should be calculated (Roe and Webster, 1984).

a) If the mean frost heave is less than 9mm, the material is classed as not frost susceptible.

b) If the mean is greater than 15mm, the material is classified as frost susceptible.

c) However, if the mean heave is in the range 9.1mm to 14.9mm, the material shall be regarded as 'not proven' (Roe and Webster, 1984).

If a material is classified in the last category, samples must be sent to two other laboratories for further testing. If the overall mean frost heave determined by the three laboratories is less than 12mm the material can be classified as not frost susceptible.

6.6 Results

The results for all tests are listed in Table 6.4. Frost heave is caused by ice lenses forming in the material, one of which is illustrated in Plate 6.5. L2, D2 and C2 were the samples closest to OMC and $\rho_{d,peak}$. The 3.5mm frost heave of the L2 samples was very low and was well below the 9mm limit. Limestone was classified therefore as not frost susceptible. However, the mean frost heave of the demolition debris (D2) samples was 12.3mm which indicated that demolition debris was in the 'not proven' range.

The moisture content of crushed concrete (C2) was 0.8 OMC which was much lower than expected. To obtain an indication of the likely frost heave at OMC, the frost heave results of the tests conducted on crushed concrete were plotted against moisture content. There appeared to be a direct relationship between initial moisture content and frost heave for crushed concrete which is shown in Figure 6.5. By interpolation of the results on the plot, a frost heave of 18mm was obtained at OMC (10%) which implied that crushed concrete would be classified as frost susceptible.

TEST REF. No.	MOISTURE CONTENT	SAMPLE 1 HEAVE (mm)	SAMPLE 2 HEAVE (mm)	SAMPLE 3 HEAVE (mm)	MEAN FROST HEAVE (mm)	STANDARD DEVIATION (mm)
L1	0.56 OMC	5.5	7	9	7.2	1.43
L2	0.95 OMC	5	2.5	3	3.5	1.08
L3	1.08 OMC	4	3.5	3	3.5	0.41
D1	0.84 OMC	12	12.5	13	12.5	0.41
D2	OMC	12	12	13	12.3	0.47
D3	1.12 OMC	10.5	10.5	11	10.7	0.48
C1	0.6 OMC	4	3.5	3.5	3.7	0.236
C2	0.8 OMC	10	10	13	11	1.414
C3	1.28 OMC	30	30	33	31	1.414

Table 6.4 Frost heave results for all materials

Demolition debris should have been tested at two other laboratories to satisfy the requirements of Roe and Webster (1984) but restrictions on finance for this research did not allow further testing. Enquiries were made of the suppliers of the aggregates to determine if frost heave tests had been carried out by them on the materials. Foster Yeoman Ltd. (1989) had carried out frost heave tests on limestone but Hughes & Salvidge Ltd. (1989) had not conducted tests on demolition debris. Fitzpatrick & Sons Ltd. (1989) had carried out frost heave tests on crushed concrete but not on the material supplied for this research.

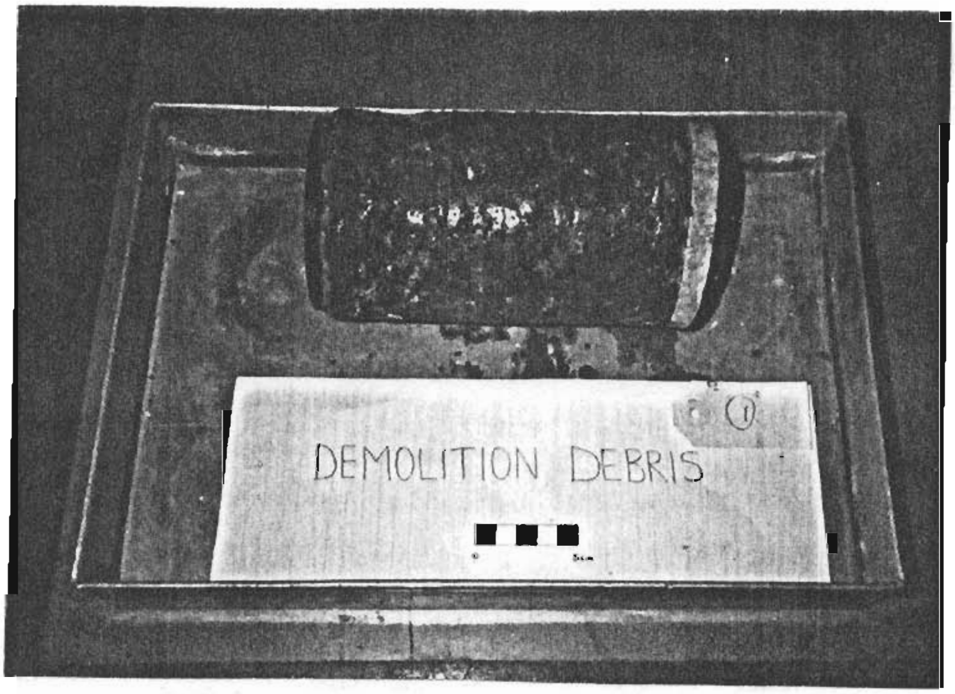


Plate 6.5 Ice lens formed in demolition debris frost heave specimen

The values listed in Table 6.4 for limestone were lower than the 4mm-6mm range of results obtained by Foster Yeoman Ltd. (1989). The samples tested by the supplier had particle gradings which fell almost centrally in the Type 1 grading envelope but the particle gradings of the samples for this research fell towards the fine side of the Type 1 grading limits. The particle gradings of limestone can be seen in Figure 6.6.

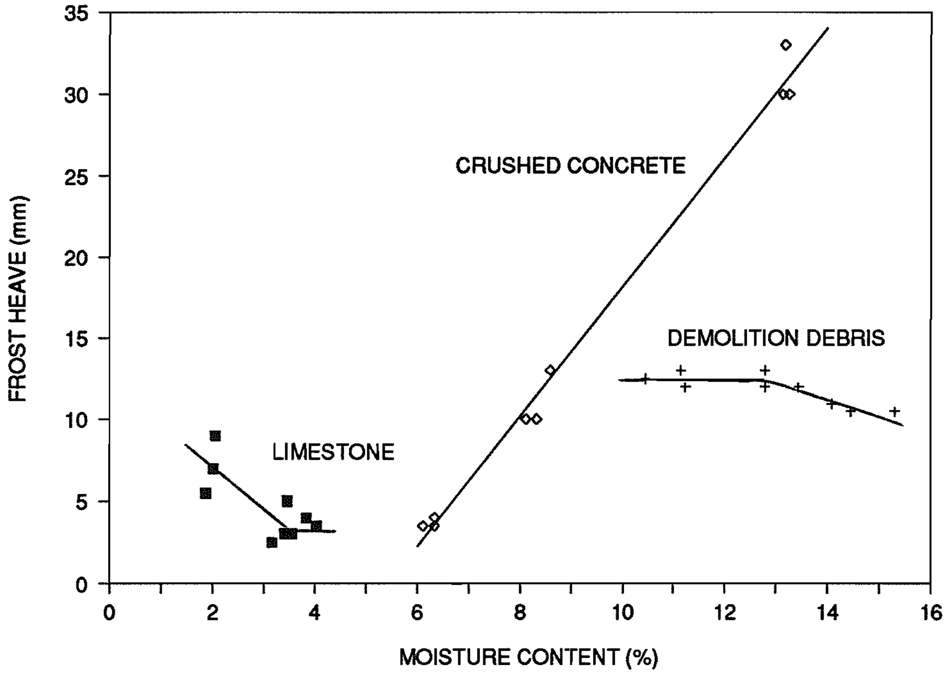


Figure 6.5 Influence of initial moisture content on frost heave

Normally high frost heave is associated with large quantities of fines in the material (Jones and Hurt, 1980). The proportion of limestone passing the 75μm sieve was 8% which is close to the fine side of the Type 1 grading envelope whereas the fines content of the suppliers' samples was 5%. This is the opposite of what would be expected when examining the frost heave results.

The limestone used in this research was coated with a red powder-like substance which the suppliers deduced was most likely to be a mixture of clay and silt. Clay, as a mass, is not likely to be susceptible to frost but it is doubtful if, in very small quantities, it would reduce the heave of limestone. Silty soils are likely to heave due to a moderate permeability and an ability to retain a high proportion of unfrozen water (Croney and Jacobs, 1967). The quantity of silt in the limestone samples was likely to be very small and should not have affected the heave of the limestone samples significantly. The presence of clay and silt, however, may have had a binding effect on the limestone when water was added which may have stabilised the samples and consequently reduced frost heave.

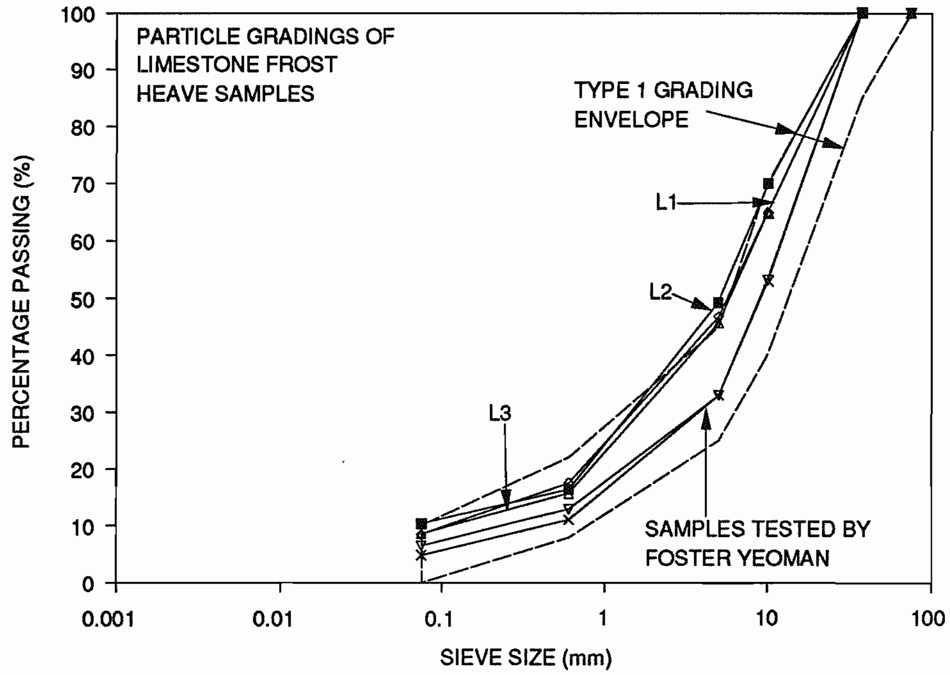


Figure 6.6 Particle gradings of limestone

Although Fitzpatrick & Sons Ltd. (1989) had not conducted frost heave tests on the batch of crushed concrete used in this research, tests had been conducted on crushed concrete from two other sites in 1985 and 1986. The frost heave results for these materials were 14.7mm and 21.2mm at OMC and $\rho_{d,peak}$. The value of 18mm which was noted earlier is within this range and would suggest that the high value obtained in this research was representative and not due to an unsatisfactory test procedure. Particle gradings of the supplier's material were not available.

The relationship between frost heave and moisture content was shown in Figure 6.5. A direct linear relationship between frost heave and the initial moisture content was apparent for crushed concrete. At a high moisture content, the frost heave of crushed concrete was excessive in comparison with that of the other aggregates. High frost heave is associated with the flow of large quantities of water into the material from below. The C3 samples were in a saturated condition before the frost heave test commenced and it does not seem likely that they would have been capable of taking in any more water. Therefore, in this case, it would appear that the voids in the compacted aggregate were not large enough to accommodate the expansion of water which was already contained in them. Frost heave might be dependent, not only on the flow of water into the material, but also on its initial moisture content.

To determine the increase in height of a saturated crushed concrete specimen (C3), due to the expansion of the initial moisture content, the following calculation was performed.

Using a moisture content of 13.2% and a ρ_d value of 1904kg/m^3 (see Table 6.3)

ρ_b = 2155kg/m^3

The volume of the specimen = $1.24 \times 10^{-3} m^3$

Therefore the mass of the specimen = 2.673kg

and the mass of solids = 2.36kg

The total mass of water in the specimen = 0.313kg

It was considered that the expansion of water in the voids between the particles was more likely to cause heave of the specimens than the water contained within the aggregate particles. In Chapter 3, the water absorption of crushed concrete was found to be 3.76%.

Therefore, the mass of free water in the specimen = 0.224kg

By using 1000kg/m^3 as the density of water and this mass of 0.224kg

the volume of free water in the specimen = $2.24 \times 10^{-4} m^3$

The total height of the frost heave specimen = 0.152m

The height corresponding to the volume of free water = 0.0274m

The volume of water increases by 9% on freezing (Neville, 1973).

After freezing the volume of free water would be = $2.44 \times 10^{-4} m^3$

and its height within the specimen would increase be = 0.0298m

Therefore the increase in height of the specimen on freezing = 2.4mm

A frost heave of 31mm was recorded for C3 and therefore the increase in height of the specimen due to expansion of the initial free water does not appear to be significant. If the pores within the particles were not capable of containing the expansion of the absorbed water on freezing, some water might be pushed out of the pores into the voids between the particles. This would increase the quantity of free water and consequently the height of the specimen on freezing would be increased. If the total moisture content of the specimen was assumed to be free water then the increase in height of the specimen, when frozen, would be 3.45mm. This assumption leads to an over estimate because some part of the total moisture content would remain within the crushed concrete particles. However, the increase in height of the specimen is still too small to be significant in a sample which exhibits a frost heave of 31mm.

When a similar calculation was conducted for C1 it was found that the increase in height of the specimen due to the expansion of the initial free water content was 0.64mm and a value of 1.24mm was determined for C2 which was the sample prepared close to OMC and $\rho_{d,peak}$. It cannot be concluded therefore that the initial moisture content had a major influence on the frost heave of crushed concrete, although it did have some effect. Further tests would need to be carried out to determine other contributing factors to the apparent high frost heave.

The particle gradings of the crushed concrete samples are shown in Figure 6.7. The C3 samples had large proportions of particles passing the 75µm sieve and this may have contributed to the high frost heave exhibited. C2, which had the smallest quantity of particles passing the 75µm sieve, achieved a heave of 11mm compared with a heave of 3.7mm exhibited by C1 whose fines content was slightly higher. There does not appear to be a direct relationship between frost heave and the quantity of fines in the samples because the variation in fines content of C1, C2 and C3 was very small.

Moisture content did not appear to influence the frost heave of the other materials directly but it is apparent that as the moisture content increased, the frost heave decreased slightly (Figure 6.5). It can be seen, at a moisture content of OMC or greater, that the frost heave of limestone remained unchanged. A decrease in frost heave was exhibited for demolition debris as the moisture content increased. Unfortunately, the lowest frost heave of 10.7mm was still above the 9mm limit. The particle grading of demolition debris falls to the fine side of the Type 1 grading envelope as can be seen in Figure 6.8. It would be interesting to determine whether samples containing coarser gradings of the three aggregates would exhibit less frost heave.

Standard deviations for the frost heave data are listed in Table 6.4. It is interesting to note that the results for demolition debris were the most consistent. It was expected that the frost heave of samples containing various constituents would vary more than that of more uniform materials. It is difficult enough to prepare identical samples with regard to particle size without the additional problem of various constituents in the aggregate. The results for limestone and crushed concrete were not very consistent, considering the uniformity of the aggregates. The particle grading of these well graded materials encompassed a wide range of particle size. It was difficult to obtain similar samples in a 102mm diameter mould when the material contained particles up to 37.5mm in size.

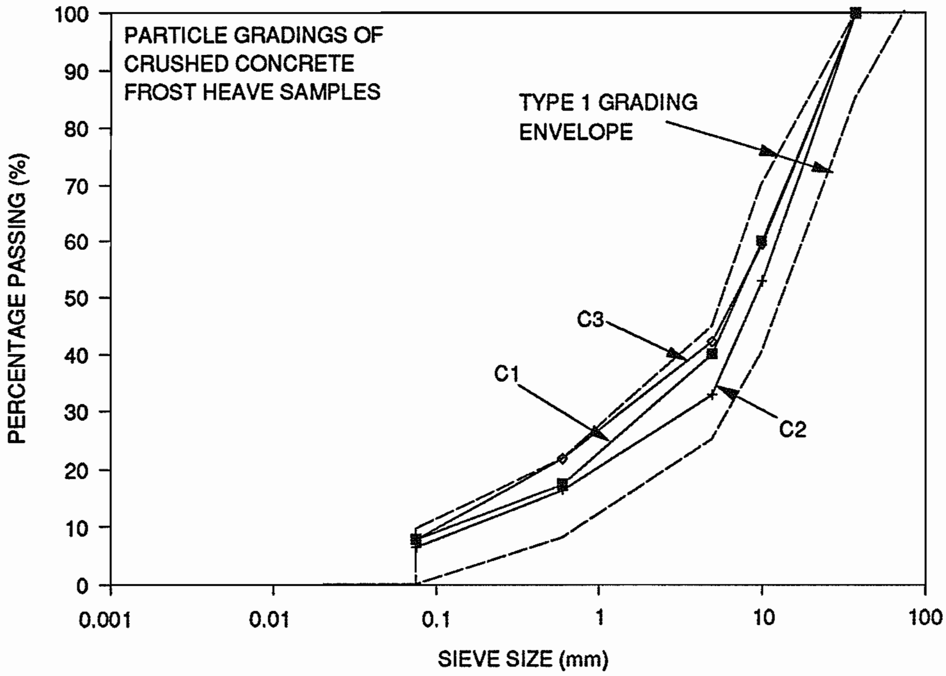


Figure 6.7 Particle gradings of crushed concrete

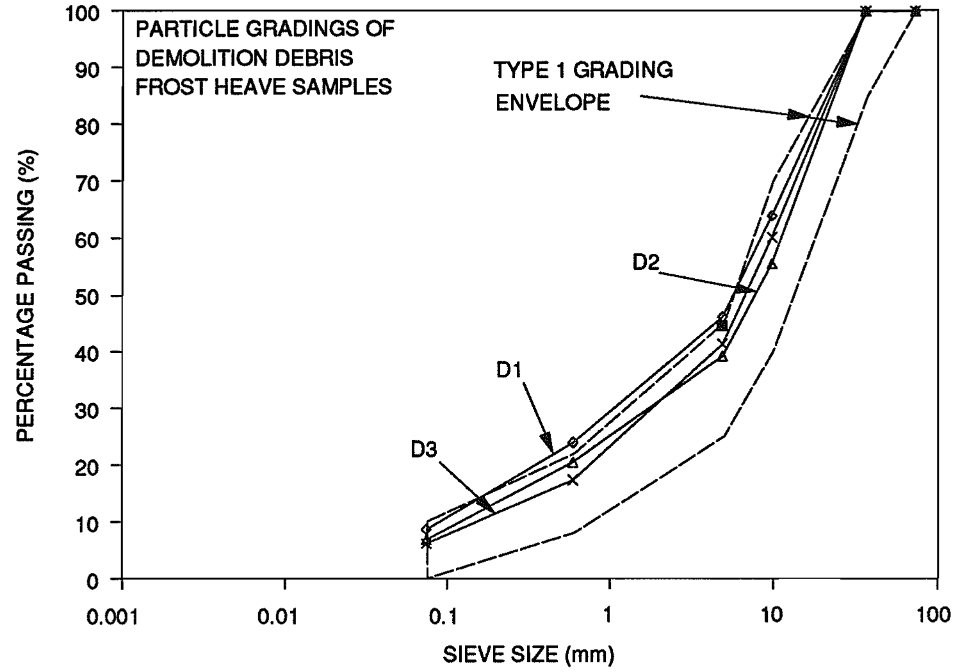


Figure 6.8 Particle gradings of demolition debris

6.7 Discussion

At a moisture content below OMC, crushed concrete had quite low frost heave results. On the basis of the results for crushed concrete presented earlier, severe problems might be caused on site if water moved laterally into a layer of the material or if a perched water table existed. Predicting the movement of water into a sub-base after construction is virtually impossible. The data suggest that crushed concrete should not be used as a sub-base material until other factors contributing to its apparent susceptibility to frost have been determined.

The results of the frost heave tests presented earlier were both disappointing and confusing. The demolition debris was made up of various constituents but was less susceptible to frost than crushed concrete which was a cleaner and more uniform material. The results for demolition debris were also more consistent than the frost heave results of the other materials.

It was surprising that no direct relationship between moisture content and frost heave existed for demolition debris, similar to that which was noticed for crushed concrete. It appears that water absorption did not play a major part in the frost heave process because demolition debris exhibited lower frost heave than crushed concrete but its water absorption was much higher. It was disappointing that the results of the tests did not pronounce demolition debris either frost susceptible or not frost susceptible. However, it has been found by Jones (1989) that several natural aggregates also fall into the 'not proven' category where further testing is required at other laboratories.

Croney and Jacobs (1967) found that the addition of cement to aggregates reduced frost heave. It was found by Sweere (1989) in a field trial that crushed concrete and demolition debris, used as sub-base materials in road pavements, exhibited better resistance to rutting three months after construction than when the material was first placed. This led Sweere (1989) to believe that recycled aggregates had a self-cementing effect. It may be that this binding effect would also reduce frost heave.

In general, crushed concrete and demolition debris both contain large quantities of cement. However, the cement might act better as a stabiliser if it was not attached to the particles. If crushed concrete was agitated in a concrete mixer, for example, or if it was placed in a system where it could be brushed using wire brushes, most of the cement should fall from the particles. The self-cementing effect of the recycled materials might not be very obvious with regard to the Roe and Webster (1984) test but it might be of considerable significance in the field. It would be interesting to conduct a field trial to try to determine the influence of the self-cementing effect of recycled materials on frost heave.

For both natural and recycled aggregates the main point for discussion, with regard to the frost susceptibility of sub-bases, is the construction of a layer which needs to be both stiff and permeable. The Department of Transport is soon to issue an Advice Note on the measurement of permeability. In the Specification for Highway Works (1986) there is no requirement concerning the permeability of sub-base materials but at the Symposium on Unbound Aggregates in Roads (1989) it was concluded that more emphasis should be put on permeability when deciding the grading and type of material. A permeable sub-base would have the advantage of low capillary rise and consequently frost-heave would be reduced. However, the main object of using a sub-base layer in a road pavement is to provide a platform for construction traffic and therefore it should be a stiff, closely packed layer. It is evident therefore that the functions of a sub-base are in direct conflict with each other. More use should be made of open packed material in a capping layer below the sub-base to provide sufficient drainage.

6.8 Conclusions

(i) On the basis of results presented earlier, limestone would be classed as not frost susceptible but it appeared that crushed concrete was highly susceptible to frost. The frost heave of demolition debris was in the inconclusive range but its results were the most consistent. Further testing at other laboratories would be required to confirm its susceptibility to frost.

(ii) Although the frost heave of crushed concrete appeared to be directly influenced by the initial moisture content of the specimens, the increase in volume of the material due to the expansion of this water on freezing was calculated and found to be relatively insignificant. The apparent dependence of frost heave on the initial moisture content was not noticed for tests conducted on limestone and demolition debris and further testing would be required to determine other contributing factors to frost heave.

(iii) At the Symposium on Unbound Aggregates in Roads (1989), it was concluded that testing of sub-base materials should become more site orientated. It is likely that recycled materials would be found to be less frost susceptible if they were examined on site some time after compaction, due to the self-cementing effect noticed by Sweere (1989). The condition of a sample in a 102mm diameter mould is not likely to be very representative of site conditions particularly when particles up to 37% of the diameter of the mould are included.

(iv) A capping layer should be constructed under the sub-base to act as a drainage layer because it is considered that the requirements of a sub-base layer to be both stiff and permeable are too demanding. This was also concluded at the Symposium on Unbound Aggregates in Roads (1989).

CHAPTER 7

RECYCLED AGGREGATE CONCRETE

7.1 Introduction

In the future it may be useful to find new sources of aggregate for the production of concrete due to the increase in demand for and decrease in supply of natural aggregate. Increasing numbers of concrete buildings are being demolished and the difficulty of disposing of the rubble has prompted an interest in the possibility of using crushed concrete as aggregate in new concrete (Nixon, 1978).

7.1.1 Mix design

In principle, the mix design of recycled aggregate concrete is not different from that of conventional concrete and the same mix design procedures can be used. In practice, slight modifications are required. Hansen (1985) concluded that for the DoE (Department of Environment) mix design, written by Teychenné, Franklin and Erntroy in 1982, the following modifications would be necessary when using recycled aggregate. This design method will be referred to hereafter as the DoE (1982) mix design.

a) When designing a concrete mix using recycled aggregate of variable quality, a higher standard deviation should be employed in order to determine a target mean strength on the basis of a required characteristic strength.

b) When coarse recycled aggregate is used with natural sand, it may be assumed at the design stage that the free water/cement ratio required for a certain compressive strength will be the same for recycled aggregate concrete as for conventional concrete. If trial mixes show that the compressive strength is lower than required, an adjustment of the water/cement ratio should be made.

c) For a recycled aggregate mix to achieve the same slump, the free water content will need to be approximately 10 litres/m^3 higher than for conventional concrete.

d) If the free water content of a recycled aggregate concrete is increased, the cement content will also need to be higher to maintain the same water/cement ratio.

e) Trial mixes should be made to obtain the required workability and the most suitable water/cement ratio.

7.1.2 Workability

Ravindrarajah (1985) found when recycled aggregate was used as the coarse fraction and natural sand as the fines in a concrete mix that an increase in free water of 8% was needed to achieve the same workability as that of natural aggregate concrete. Hansen and Narud (1983) reported needing a 5% higher free water content than for control mixes. Similarly, Mulheron and O'Mahony (1988) found when crushed concrete aggregate was used as the coarse fraction that the mixes were slightly harsher and less workable than the conventional aggregate mixes.

Frondistou-Yannas (1977) and Buck (1976) both reported that there appeared to be little difference in workability when recycled aggregate was used as the coarse fraction with a natural sand but Buck (1976) noticed that the slump was lower if recycled aggregate was used for both the coarse and fine fractions. Hansen and Marga (1988) agreed with this and stated that an increase in free water content of 14% was required when the total aggregate content in a mix consisted of recycled aggregate. An unusual result was reported by Yamato et al (1988) who found that workability was increased considerably when recycled aggregate was used. It is thought that this result was specific to his research and the type of aggregate used.

7.1.3 Strength of concrete

Hansen (1985) stated that the compressive, tensile and flexural strength of recycled aggregate concrete could be equal to or higher than that of conventional concrete, if the same or a lower water/cement ratio was used. In practice, however, the strength of recycled aggregate concrete is often found to be lower. Frondistou-Yannas (1977) reported a 4%-14% drop in compressive strength whereas Kemi and Nakagawa (1978), Ravindrarajah (1985), Yamato et al (1988), Mulheron and O'Mahony (1988), Nishibayashi and Yamura (1988) and Kasai (1985) all reported a reduction of between 14% and 32%. Nixon (1978) found in a series of tests that the compressive strength of the original concrete, from which the recycled aggregate was obtained, did not appear to affect the compressive strength of the recycled aggregate concrete very much. Kashino and Takahashi (1988) noted when less than 30% of natural aggregate in a concrete mix was replaced by recycled aggregate that there appeared to be no change in compressive strength.

7.1.4 Young's modulus

In the research reports mentioned in Section 7.1.3, values of Young's modulus were also quoted. All noted a reduction of 15%-40% in the Young's modulus of recycled aggregate concrete which was attributed to the large amount of weak, old mortar attached to the aggregate. Kasai (1985) reported a 10%-20% reduction in Young's modulus for concrete made with coarse recycled aggregate and natural sand. The lower Young's modulus of recycled aggregate concrete in general is explained by Frondistou-Yannas (1977). Recycled aggregate normally has a lower modulus than natural aggregate. As the modulus of concrete is dependent on the modulus of the aggregate present, the lower modulus of recycled aggregate concrete is not surprising.

7.1.5 Shrinkage and creep

Helmuth and Turk (1967) reported that the drying shrinkage of cement paste increases linearly with its porosity. Therefore if the porosity of the paste could be reduced, e.g. by decreasing the water/cement ratio, the shrinkage of cement paste would also be reduced. Mulheron (1986) noted an increase in shrinkage of 58%-95% more than control concrete depending on whether crushed concrete or demolition debris was used in the recycled aggregate concrete; demolition debris giving the worst result. Shrinkage of concrete also depends on the Young's modulus of the aggregate present and it is likely that the low modulus of the mortar in the recycled materials would cause an increase in shrinkage.

Mulheron (1986) found a 40%-100% increase in creep for concrete made using coarse recycled aggregate and natural sand. Ravindrarajah (1985) and Hansen (1985) also observed higher creep than for conventional aggregate concrete. Nishibayashi and Yamura (1988) also found the creep of recycled aggregate concrete made with coarse recycled aggregate and natural sand to be 50% higher than that of conventional aggregate concrete. The presence of aggregate in concrete restrains the volume change of the cement paste and therefore reduces creep. However, if the aggregate in concrete has a low modulus, e.g. recycled aggregate, then its ability to reduce creep is lower.

7.1.6 Impurities

Mulheron (1986) found the levels of chloride in natural and recycled aggregate to be below the point of detection. If recycled aggregate was to be produced from the crushing of a bridge deck or a concrete carriageway from a road or an airport, it is expected that the level of soluble chloride might be excessive and consequently the aggregate produced would not be suitable for aggregate in concrete. Chloride has little significant influence on the properties of plain concrete but in reinforced concrete the presence of chlorides initiates corrosion of embedded steel (BRE, 1980) and (BRE, 1982).

Other impurities, known to cause problems, are timber and vegetable matter from soil. The level of contamination in any recycled aggregate depends on the source of the aggregate and the method of production and control. Wood can be removed in flotation operations which are usually only employed by the larger fixed-site recycling plants. Organic matter is considered to be a harmful impurity in aggregate intended for use in concrete (Collis and Fox, 1985). Humus and oil, for example, can retard or even prevent the hydration of cement when present even in small quantities (Sherwood and Roeder, 1965).

Kemi and Nakagawa (1978) tested concrete containing increasing levels of paint, asphalt, gypsum, wood, soil and plaster. The compressive strength of the most heavily contaminated concrete was 85% that of the control concrete specimens. The Building Contractors Society of Japan (1981) determined the percentage of six contaminants which, when added to crushed concrete to be used as aggregate in new concrete, would cause no more than a 15% reduction in the compressive strength. The maximum percentages by volume were found to be 7% plaster, 5% soil, 4% wood, 3% gypsum, 2% asphalt and 0.2% paint. The Dutch standard (CUR, 1986) for recycled materials to be used as aggregates in concrete includes stricter limits on contamination. The quantity of asphalt is limited to 1% and the amount of wood present should be less than 0.6% (see Section 2.3.2).

Graf (1973) and Gaede (1957) decided that the allowable level of soluble sulphate in recycled aggregate concrete should be maintained between 0.5% and 1%. When present in sufficient quantity, sulphate in aggregate reacts with cement compounds if the aggregate is used in concrete manufacture. This results in excessive expansion and ultimately the deterioration of hardened concrete in wet or damp conditions (Lea, 1970). The British Standard for aggregate to be used in concrete, BS 882 (1983), does not specify sulphate limits for natural aggregates.

7.2 An examination of recycled aggregate concrete

Slabs of reinforced concrete produced in a laboratory, ranging in size from 800mm x 800mm x 150mm to 1200mm x 1200mm x 150mm were crushed in a single jaw crusher operation. When the slabs were too large to be fed directly into the crusher they were first broken up by a demolition ball. The 5mm-10mm fraction was sieved out in the laboratory and this was used as the coarse aggregate fraction for a series of recycled aggregate concrete mixes. Natural sand was used as the fine aggregate in all mixes, in view of previous work by Hansen (1985).

7.2.1 Mix design of recycled aggregate concrete

The mix for the concrete slabs was originally designed by other research workers for non-destructive testing in another project. Thames valley gravel was used as the coarse aggregate fraction and the strength of the concrete was 60N/mm^2. A decision was taken by the research workers to use a mix with a high sand content. The first set of mixes for this research, set A, was based on the same design to allow a direct comparison of control and recycled concretes. Three control and three recycled mixes were made with different water/cement ratios. In set B, the same recycled aggregate was used but the mix design was based on the DoE method (1982) and again three control and three recycled mixes were made. In all cases, the water content of the recycled aggregate mixes was increased to allow for the higher water absorption of the aggregate and the free water content was increased further by 8% following the recommendation by Ravindrarajah (1985). The cement content was also increased to maintain the original water/cement ratio. The mix proportions are listed in Table 7.1. The lower specific gravity of recycled aggregate was taken into account in other research by Hansen (1985) and Ravindrarajah (1985) but for the mix designs presented here it was decided that the same mass of aggregate would be used for all mixes. All aggregate was oven-dried before use but not pre-soaked before mixing.

Mix No.	CEMENT	FINE AGGREG	COARSE AGGREG	TOTAL WATER	FREE W/C RATIO	SLUMP (mm)
	Mass (kg) of constituents per m^3					
<u>SET A</u>						
C1	493	1004	670	212	0.36	10
R1	532	1004	670	255	0.35	50
C2	493	1004	670	233	0.4	115
R2	532	1004	670	229	0.3	10
C3	493	1004	670	246	0.45	190
R3	532	1004	670	279	0.4	190
<u>SET B</u>						
C4	580	744	839	236	0.35	35
R4	626	744	839	293	0.35	170
C5	580	744	839	265	0.4	190
R5	626	744	839	261	0.3	40
C6	580	744	839	293	0.45	235
R6	626	744	839	325	0.4	215

Note: C = control, R = recycled

Table 7.1 Mix quantities and slump

The following specimens from each mix were cast for various tests; six 100mm cubes for compressive strength tests at 7 days and 28 days, one cylinder of diameter 150mm and length 300mm for a Young's modulus test and tensile splitting strength test, and two 100mm x 100mm x 200mm specimens for shrinkage and creep tests.

For comparison purposes, some control and some recycled mixes in each set were made to have similar workability. The corresponding pairs can be seen in Table 7.1. In the following presentation of results, the properties of concrete mixes with similar water/cement ratios are also compared.

7.2.2 Workability of concrete mixes

The slump of the fresh concrete was measured in accordance with BS 1881: part 102 (1983). In Figure 7.1, it can be seen that there were similar relationships between slump and water/cement ratio for both the recycled and control mixes. The slump values for all mixes are listed in Table 7.1. The workability of the recycled aggregate mixes was higher than that of the control mixes. This is explained by Neville (1973). If aggregate is not pre-soaked before mixing, it becomes coated with cement paste which prevents ingress of water for saturation of the aggregate. It is likely if the aggregate does not absorb a quantity of water equivalent to its water absorption before mixing that it might draw water from the cement paste later and affect the bond between the aggregate and the paste.

The recycled aggregate used in this research was produced in a jaw crusher where the lowest jaw setting was 70mm-85mm. This material may have been less angular than the aggregate used by Ravindrarajah (1985) which was produced in a jaw crusher at a setting of 20mm. If the aggregate was less angular, the need for using less water than suggested by Ravindrarajah (1985) could be explained. Trial mixes could not be carried out because there was a limited supply of recycled aggregate for the work.

7.2.3 Compressive strength results

Compressive strength tests were carried out in accordance with BS 1881: part 116 (1983) and the results are summarised along with density of the concrete in Table 7.2. All concretes achieved high strength within 7 days and by 28 days the average compressive strength in both sets was similar. The tensile splitting strengths of the concretes in set B appeared to be slightly lower than those in set A but the Young's moduli of the concretes in both sets were similar.

Mix No.	FREE W/C RATIO	COMPRESSIVE STRENGTH (N/mm^2)		TENSILE STRENGTH (N/mm^2)	DENSITY (kg/m^3)	YOUNG'S MODULUS (kN/mm^2)
		7 DAYS	28 DAYS			
SET A						
C1	0.36	57	65	5.27	2424	30.2
R1	0.35	54.5	63.5	5.03	2345	30.1
C2	0.4	53	58	4.96	2377	26.5
R2	0.3	61.5	69.5	5.1	2378	35.6
C3	0.45	47	54.5	4.81	2363	25.8
R3	0.4	49	58	4.67	2317	28.5
SET B						
C4	0.35	58.5	62.5	4.6	2414	29.2
R4	0.35	54.5	65.5	4.3	2319	34.9
C5	0.4	50	57.5	4.6	2360	32.7
R5	0.3	65	73	4.45	2359	32.8
C6	0.45	42.5	53	4.2	2378	26.3
R6	0.4	44.5	57.5	4.3	2300	26.5

Table 7.2 Properties of hardened concrete

In Figure 7.2, the increase in compressive strength with age is shown for set A and a similar relationship for set B is illustrated in Figure 7.3. The rates of increase in strength of C2 and R3 were similar to those of C5 and R6 where these four mixes had the same water/cement ratio of 0.4. The recycled aggregate concretes had lower strengths at 7 days but by 28 days they had attained the same strength as the controls. It is apparent in Figure 7.4 that, as expected, the higher the water/cement ratio, the lower the strength appeared to be for the mixes.

In Figure 7.5, control and recycled mixes of the same slump were compared and it was found that the compressive strength of the recycled mixes was higher in most cases by 6%-17%. This was due to the lower water/cement ratios which were used in the recycled aggregate concrete mixes to achieve similar workability to that of the natural aggregate concretes.

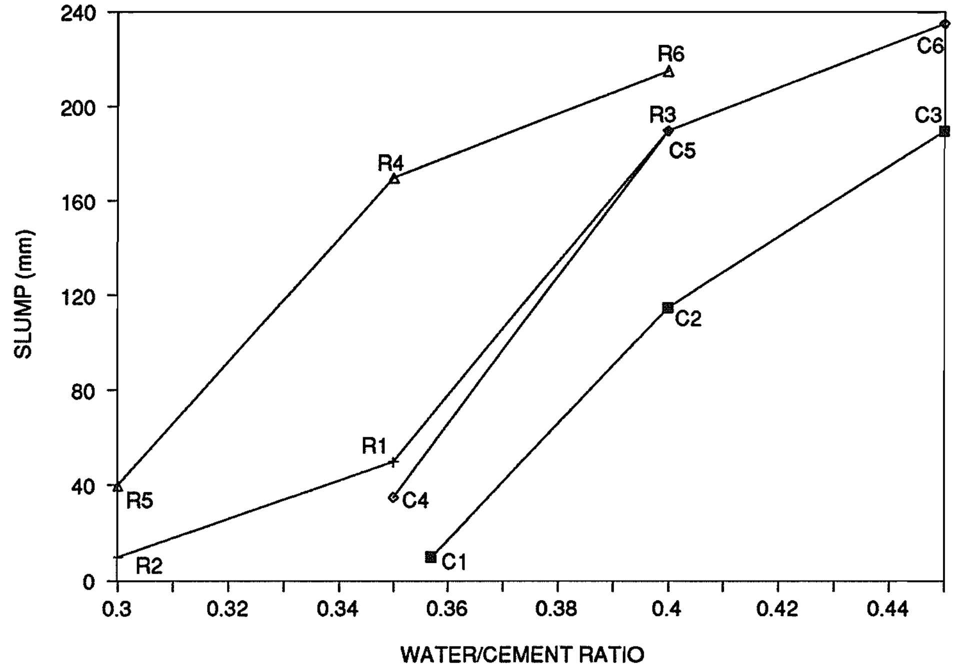


Figure 7.1 Influence of water/cement ratio on slump

If, however, compressive strength is compared on the basis of similar water/cement ratio, as in Figure 7.4, the results can be perceived in another way. In this approach, the control mixes of set A were stronger by 0%-3.7% but in set B the recycled aggregate concrete specimens were stronger than the controls by 0%-5%. This showed that, by adding 8% more water and cement, the original objective of producing recycled aggregate concrete with a compressive strength comparable with that of conventional aggregate concrete was achieved. The tensile splitting strength of the concretes is best compared when examining mixes of similar water/cement ratio in Table 7.2. The recycled aggregate concretes had

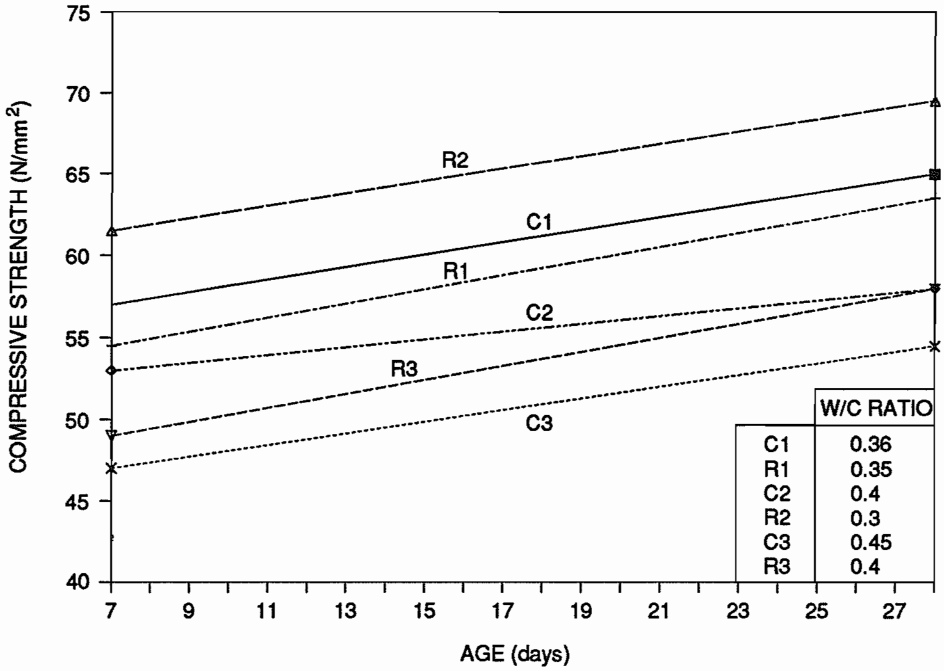


Figure 7.2 Strength development for set A

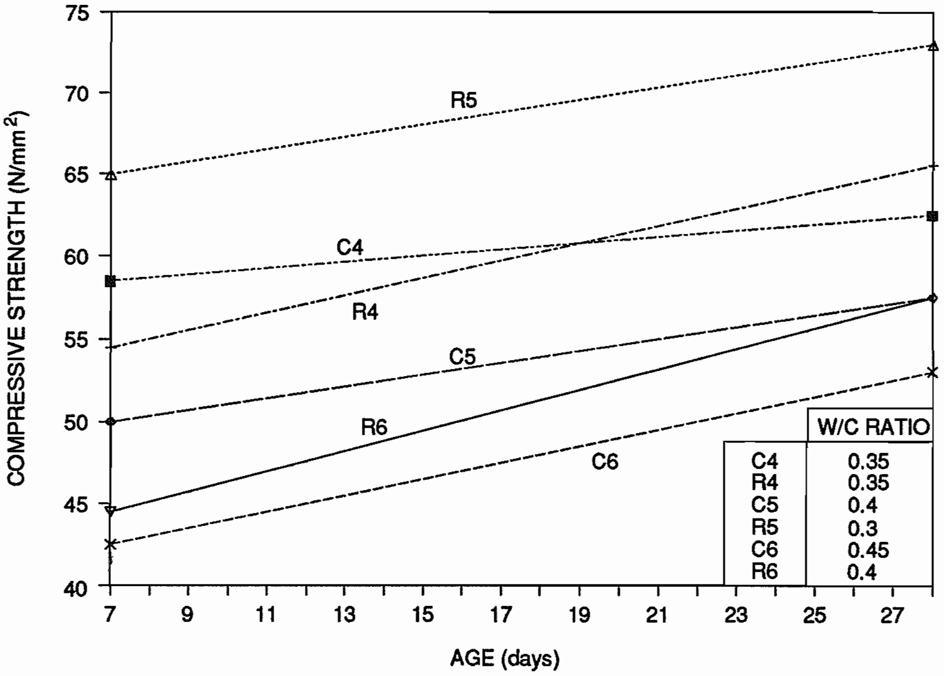


Figure 7.3 Strength development for set B

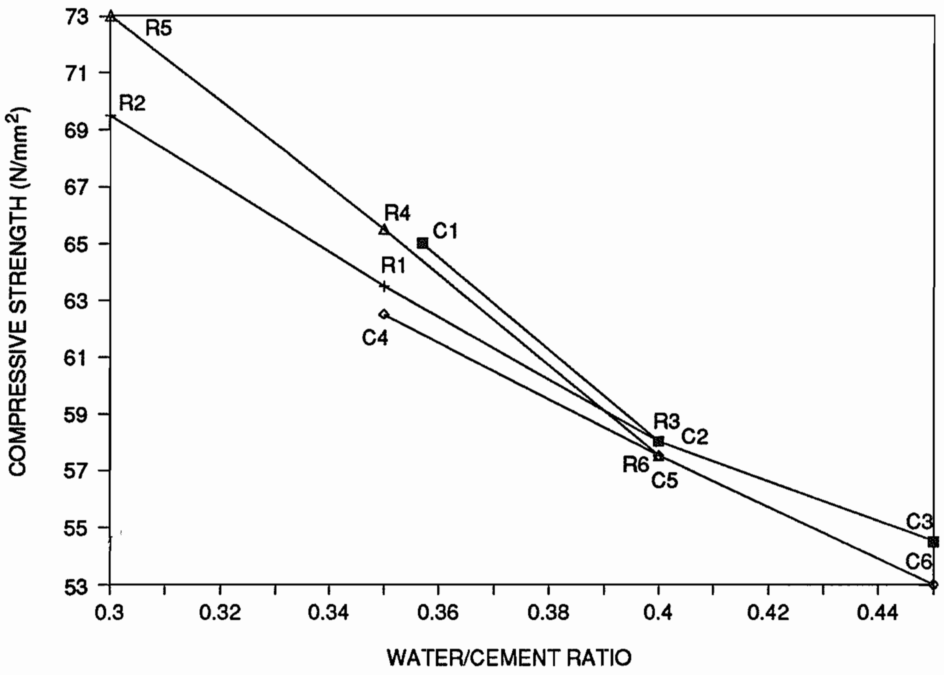
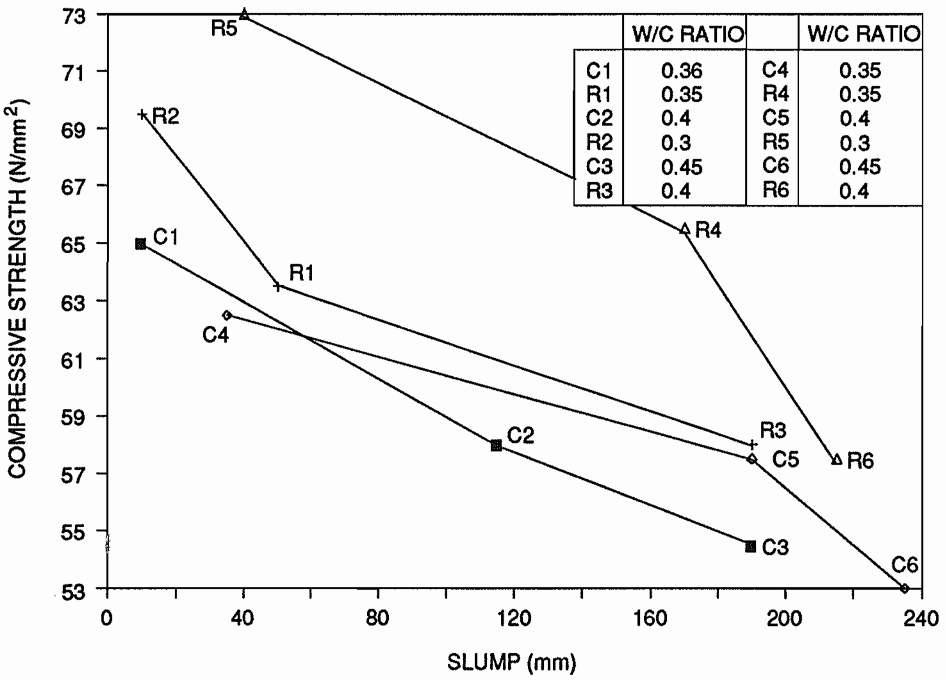


Figure 7.4 Influence of water/cement ratio on compressive strength



	W/C RATIO		W/C RATIO
C1	0.36	C4	0.35
R1	0.35	R4	0.35
C2	0.4	C5	0.4
R2	0.3	R5	0.3
C3	0.45	C6	0.45
R3	0.4	R6	0.4

Figure 7.5 Relationship between compressive strength and slump

tensile splitting strengths which were lower by 4.5%-7%.

The density of recycled aggregate concrete was about 2%-4% lower than that of control concrete. The old mortar present in the recycled aggregate particles is likely to have contributed to the lower density of the concrete.

7.2.4 Young's modulus results

Young's modulus tests were carried out in accordance with BS 1881: part 121 (1983) using a 200 mm Demec strain gauge to monitor strains in the concrete. The Young's modulus results were listed in Table 7.2. Figure 7.6 shows the relationship between Young's modulus and compressive strength. Trends in the results appear to be similar for recycled and control mixes from each set. On the basis of equal compressive strength, the Young's moduli of conventional concretes in set A were 2.3%-8% lower than those of the recycled aggregate concretes. The large quantity of sand in the mixes of set A may have contributed to the improved Young's modulus of the recycled aggregate concretes. When examining the results for set B, however, it appears that at lower compressive strengths the moduli of recycled aggregate concretes were 80% those of the controls but at strengths of 61-62 N/mm^2 the moduli of the control concretes were 92%-100% those of the recycled aggregate concretes.

From these results it can be seen that in most cases the recycled aggregate concretes had higher Young's moduli than the controls and this conclusion disagrees with the results of Ravindrarajah (1985) for similar tests. This contradiction is most likely due to the different recycled aggregates used and the difference in mix design, particularly for set A.

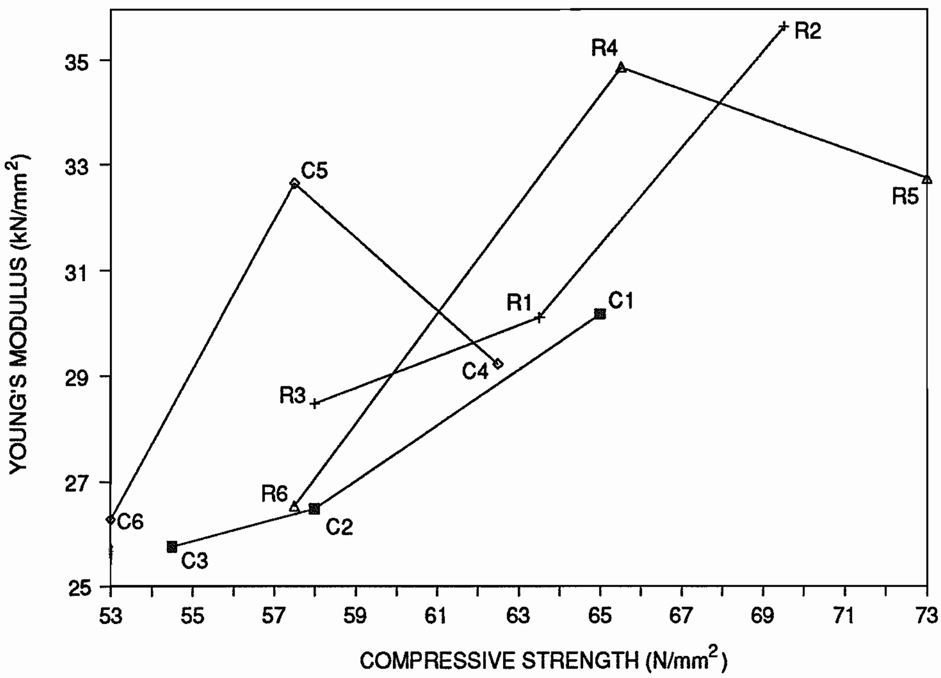


Figure 7.6 Relationship between Young's modulus and compressive strength

7.2.5 Shrinkage

Apparent deformations were monitored from the day after casting. For the first 28 days, the specimens were in a curing tank and all concrete specimens expanded considerably, especially the recycled aggregate concrete due to the high water absorption of the aggregate caused by old mortar coating the particles. The aggregates were not pre-soaked before casting and this is thought to be another reason for the high expansion of the concretes in the curing tank.

When the specimens were removed from the curing tank they were coated with two coats of bitumen paint to provide a seal so that constant humidity would be maintained to some extent. Shrinkage was monitored on all four sides of the specimens using 100mm Demec gauges. The results for set A and set B are plotted in Figures 7.7 and 7.9 respectively.

Mixes C1 and C4, both with a water/cement ratio of 0.35, achieved the lowest shrinkage whereas the recycled aggregate concrete with the highest water/cement ratio of 0.4 in set A, i.e. R3, exhibited high shrinkage. However, R6 which also had a water/cement ratio of 0.4 exhibited shrinkage which was similar to that of other concretes in set B. There appeared to be little difference in shrinkage when the two sets of mixes were compared except for C1 which achieved a shrinkage of 56% that of the other concretes in set A and also exhibited less shrinkage than the concretes in set B. This was encouraging because, in the mix design for set A, a higher sand content was included and therefore higher shrinkage was expected.

In general, if a concrete mix contains a large quantity of sand there is more surface area of aggregate to be coated by cement which allows more cement to be hydrated. The larger the amount of hydrated cement paste in concrete, the higher the shrinkage is expected to be. Restraint to shrinkage is normally provided by the aggregate contained in the concrete and Neville (1973) stated that the presence of unhydrated cement in concrete would also contribute to a reduction in shrinkage.

The recycled aggregate concretes in this study contained 8% more cement than the natural aggregate concretes as well as the extra cement in the old mortar surrounding the recycled aggregate particles. There was likely to be more potential for shrinkage in the recycled aggregate concrete due to this extra cement. It is clear from the results that recycled aggregate did not provide the same restraint to shrinkage as conventional aggregate but the difference appeared to be quite small in most cases. The recycled aggregate was of lower modulus than the natural aggregate and therefore its ability to restrain shrinkage was lower.

In Figures 7.8 and 7.10, it can be seen that the weight change of the specimens appeared to be dependent on temperature up to an age of 100 days but that the temperature had little influence as the concrete grew older. It can be concluded therefore that two coats of bitumen

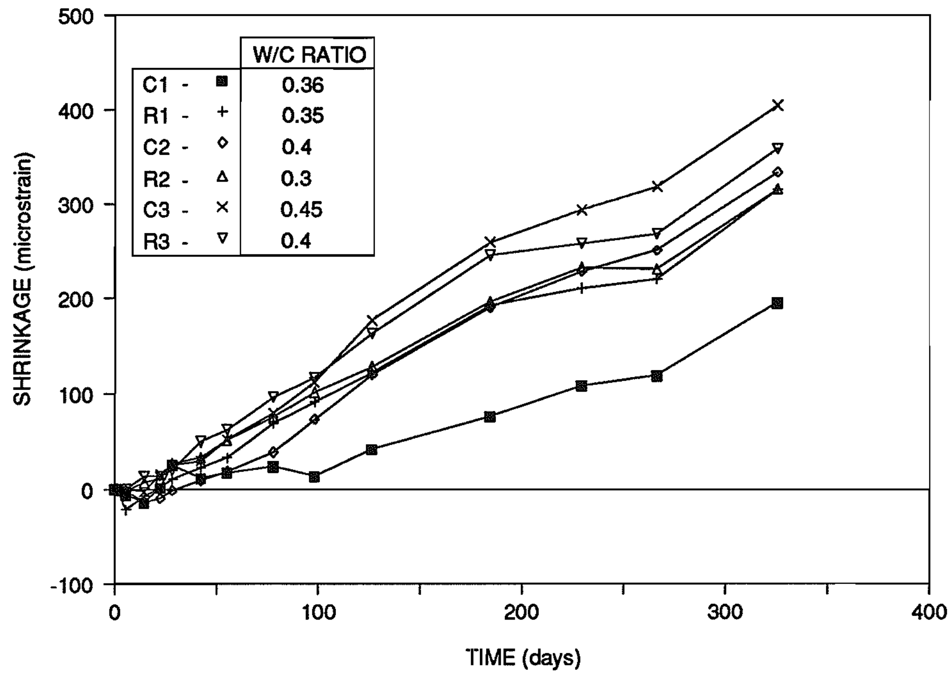


Figure 7.7 Shrinkage of the concrete specimens in set A

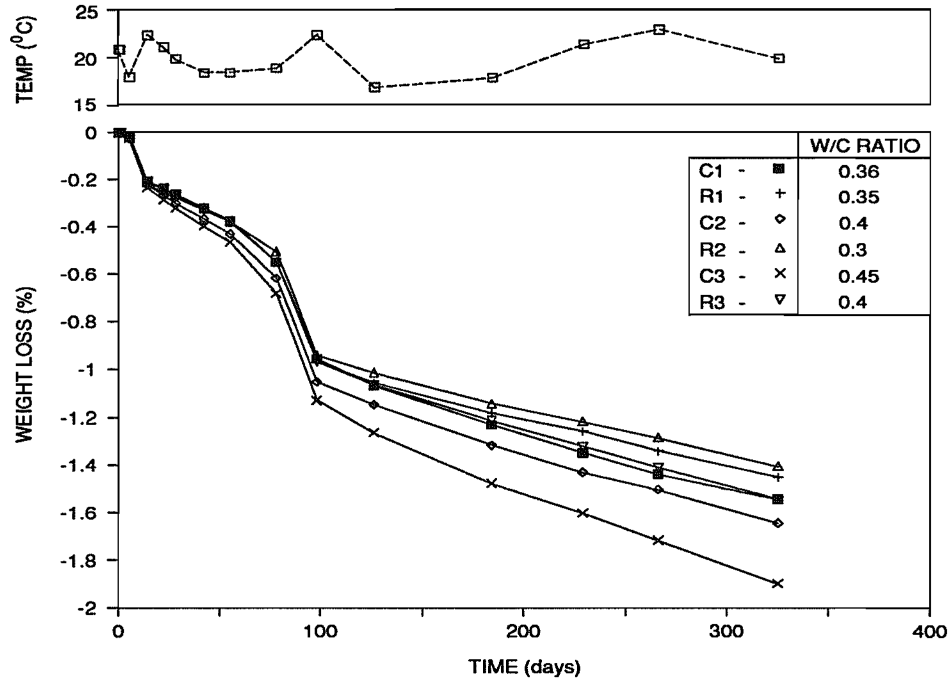


Figure 7.8 Weight change of the concrete specimens in set A

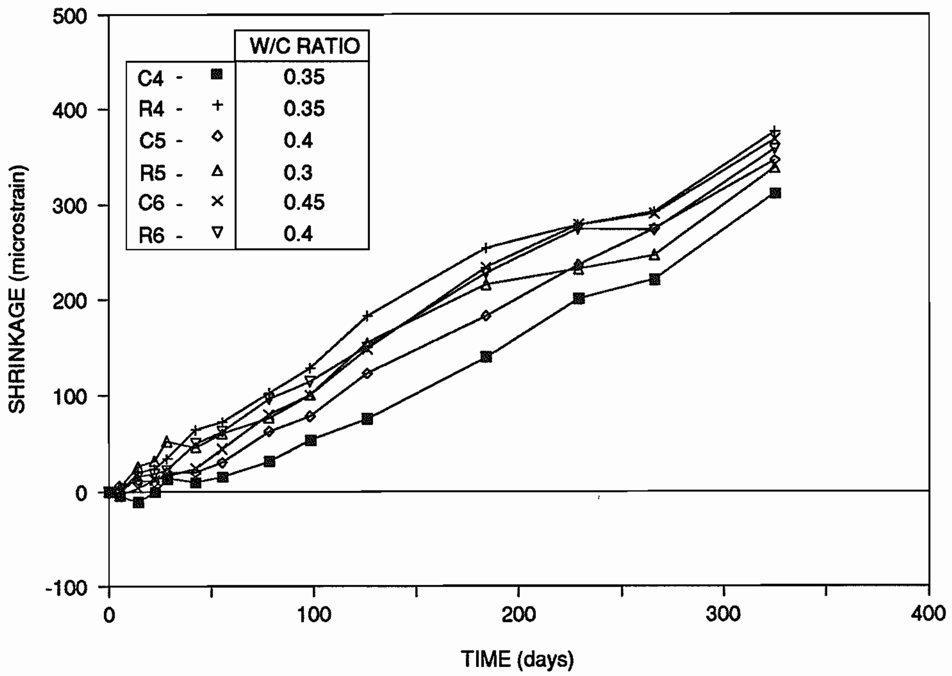


Figure 7.9 Shrinkage of the concrete specimens in set B

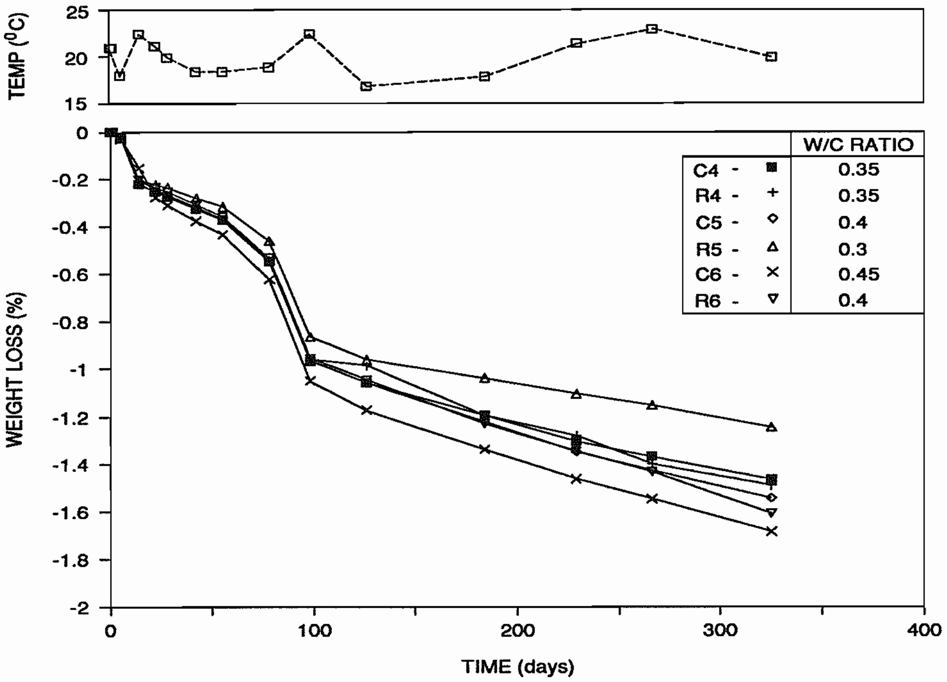


Figure 7.10 Weight change of the concrete specimens in set B

paint were not sufficient for complete protection from environmental conditions. Although the weight loss (defined by Neville, 1973) was relatively small, the loss of water would have caused some drying shrinkage.

7.2.6 Creep

Creep tests were performed on 100mm x 100mm x 200mm specimens stacked in pairs in a conventional creep rig at the University of Surrey. The creep apparatus had seven loading bays in all but only six were in use for this test. An overall view of the rig is provided in Plate 7.1 and a more detailed drawing of an individual loading bay is shown in Figure 7.11.

The specimens were painted with two coats of bitumen paint but the top and bottom faces were left untouched. These faces were then smoothed with a metal file and sandpaper to provide flat surfaces so that high stress concentrations would be avoided when the specimens were loaded. Each pair of specimens, when placed in position in the rig, was separated by a flat steel plate of 5mm thickness.

The stress in the system was maintained using a Greer-Mercier hydraulic accumulator and was applied to each pair of specimens through a flexible diaphragm at the base of each bay. Readings could not be taken on all four sides due to the structure of the creep apparatus so the specimens were orientated to allow access to three sides. Trial loadings were carried out at low stresses to check that the specimens were not loaded eccentrically. Creep was monitored using a 100mm Demec gauge. The stress on the specimens was maintained at 14.7N/mm^2 subjecting them to stress/strength ratios between 0.2 and 0.28. Although comparisons would normally be made for similar stress/strength ratios, only some of the concretes could be compared on that basis.

Creep for both sets of mixes is plotted in Figures 7.12 and 7.13. The recycled aggregate concretes with the highest water/cement ratios exhibited the highest creep because creep, like shrinkage, is dependent on the amount of hydrated cement paste in the concrete and on the modulus of the aggregate. R5 which had a low water/cement ratio appeared to creep very little. The low water/cement ratio of this mix would have produced a smaller quantity of cement paste and consequently it would be expected that creep would be low. However, creep is also dependent on the strength of concrete but because strength and water/cement ratio are related it is difficult to conclude which factor has the greatest effect on creep (Neville, 1973).

Plate 7.1 Concrete specimens in the creep apparatus

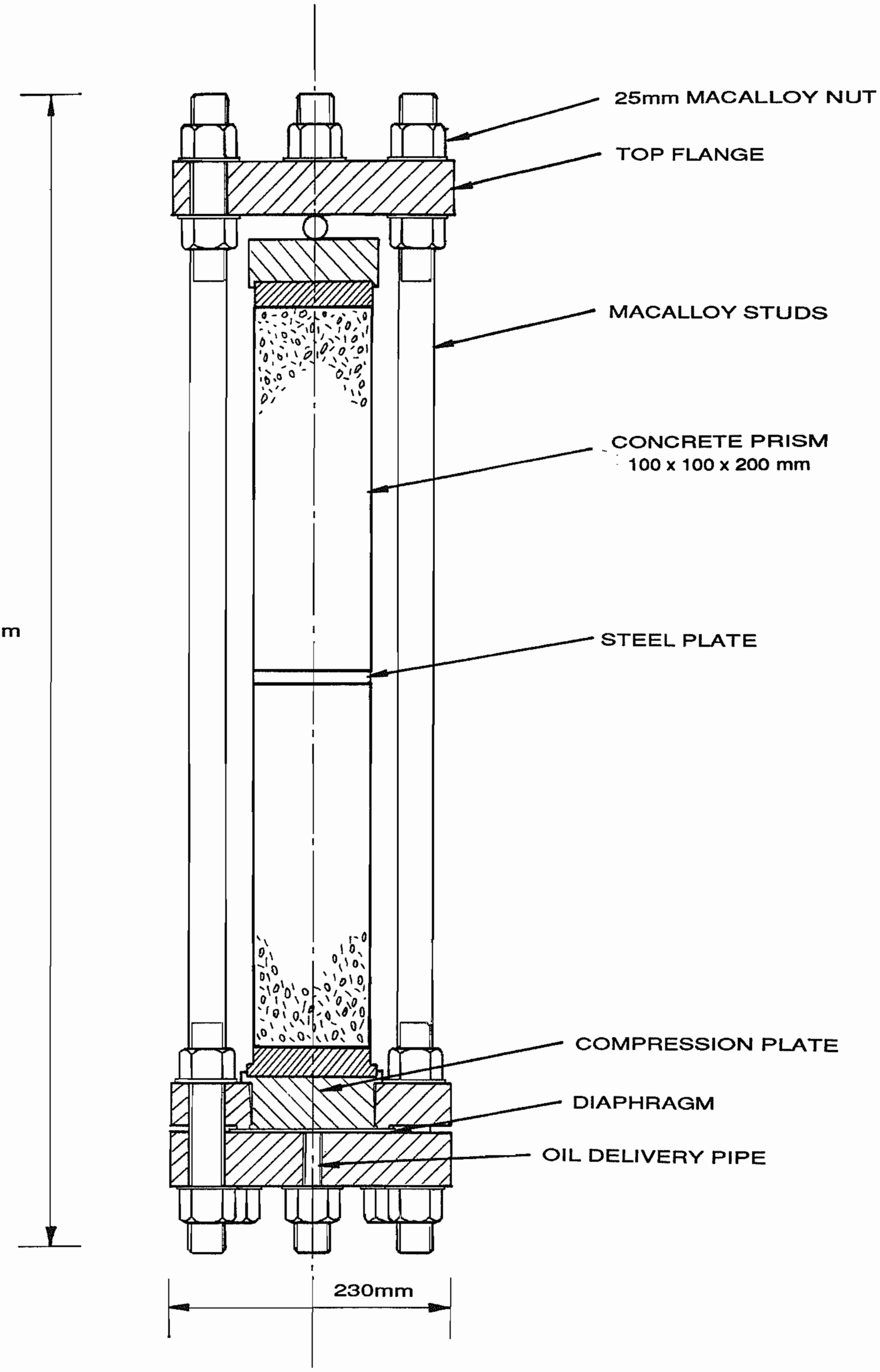


Figure 7.11 Details of a loading bay in the creep apparatus (after Edgington, 1969)

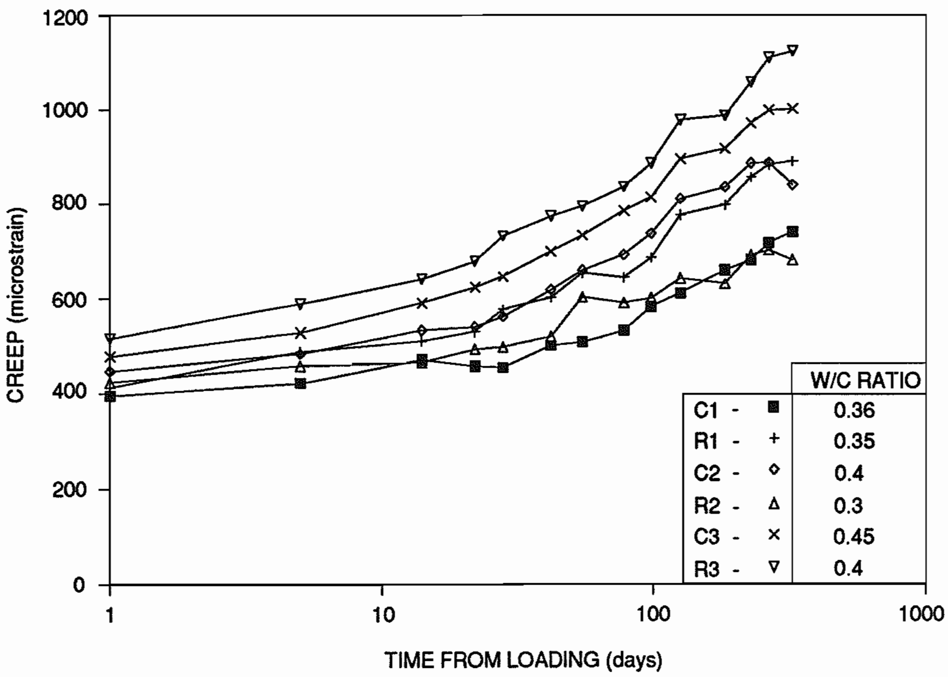


Figure 7.12 Creep of the concrete specimens in set A

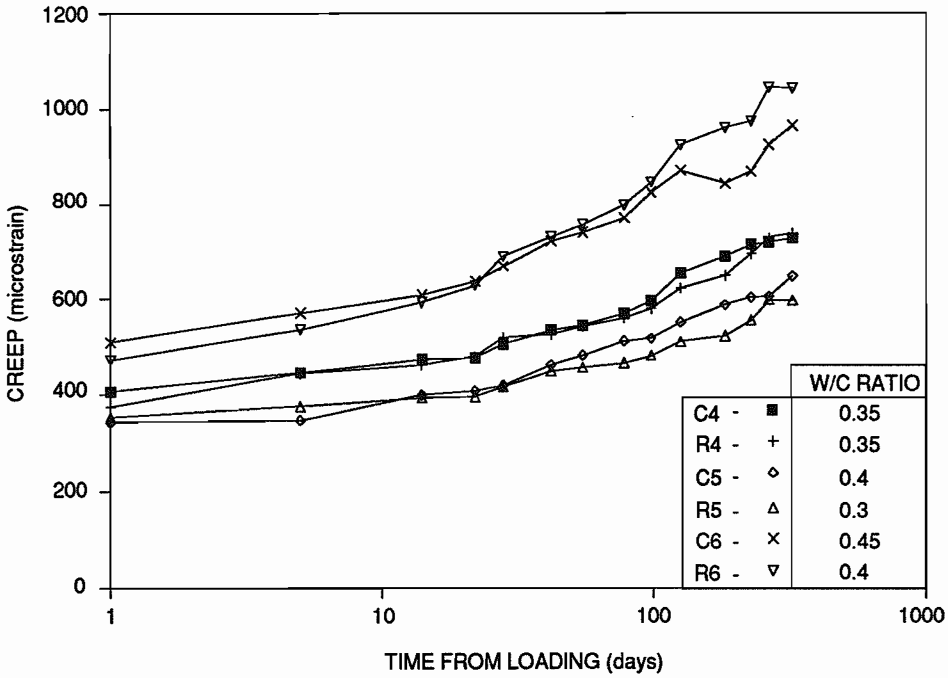


Figure 7.13 Creep of the concrete specimens in set B

7.3 Discussion

At the mix design stage, two decisions were taken which made this work different from other research in the field of recycled aggregate for use as aggregate in concrete. First, the specific gravity of the recycled aggregate was not taken into account so that each mix had the same mass of aggregate. It was considered that it would be easier to observe the effect of recycled aggregate on the properties of concrete by adopting this method. Secondly, the aggregates were not pre-soaked before mixing so that an accurate estimate of the water content in the mixes could be made. It is likely if recycled aggregate was to be used extensively for aggregate in concrete that its specific gravity would be taken into account at the mix design stage. Although the aggregate was not pre-soaked before mixing the results of the tests on hardened concrete appeared to be better than those of other researchers. It is suggested, however, due to the complex nature of concrete that it would be safer to pre-soak aggregates so that no movement of water would take place into the aggregate after mixing. Pre-soaking of aggregate could not be omitted from the procedure of concrete manufacture without further research.

When recycled aggregate concrete and conventional concrete were compared on the basis of strength and Young's modulus, it was noted earlier in this chapter that recycled aggregate concrete performed very well. This may be due to the following reasons.

(i) The recycled aggregate was clean and the original and new mixes were both made in a laboratory under controlled conditions.

(ii) It was considered that the porosity of the recycled aggregate would have had a large influence on the Young's modulus of the concrete. However, the angular particles of the recycled aggregate, compared with the smooth particles in the control mixes, provided greater particle interlock which apparently counteracted the effect of porosity. The high compressive strength of the recycled aggregate concretes could also be attributed to the angular particles because the mechanical bond in concrete is dependent on the surface shape and texture of the aggregate.

The rate of gain in strength of the recycled aggregate concretes appeared to be very consistent and it is clear from Figures 7.2 and 7.3 that a prediction of the 28 day strength of the recycled aggregate concretes would be more accurate than for the controls.

When Figures 7.7 and 7.8 were examined it was be observed that after 100 days there was almost a linear relationship between shrinkage and weight loss which may mean that as the concrete aged the water loss from the sealed specimens was largely responsible for shrinkage. A similar relationship is apparent in Figures 7.9 and 7.10 for set B. It can be seen from Figure 7.14 that the slopes of the lines are similar and that the recycled aggregate concretes exhibited the highest shrinkage whereas, as expected, the control mix with a water/cement ratio of 0.45 exhibited the highest weight loss. Similar relationships exist for the second set of mixes.

It is apparent from Figure 7.15 that the water/cement ratio and the type of aggregate did not have a major effect on shrinkage. The relationship between shrinkage and water/cement ratio was very similar for concretes made with both types of aggregate. This is encouraging because the problems expected with the use of recycled aggregate concrete in the civil engineering industry are mainly associated with shrinkage and creep.

It can be seen in Figure 7.16 that there appears to be a linear relationship between creep and stress/strength ratio for each type of concrete. Both recycled and natural aggregate concretes exhibited an increase in creep as the stress/strength ratio increased but at a stress/strength ratio of 0.26 the control concretes achieved only 65% of the creep exhibited by the recycled

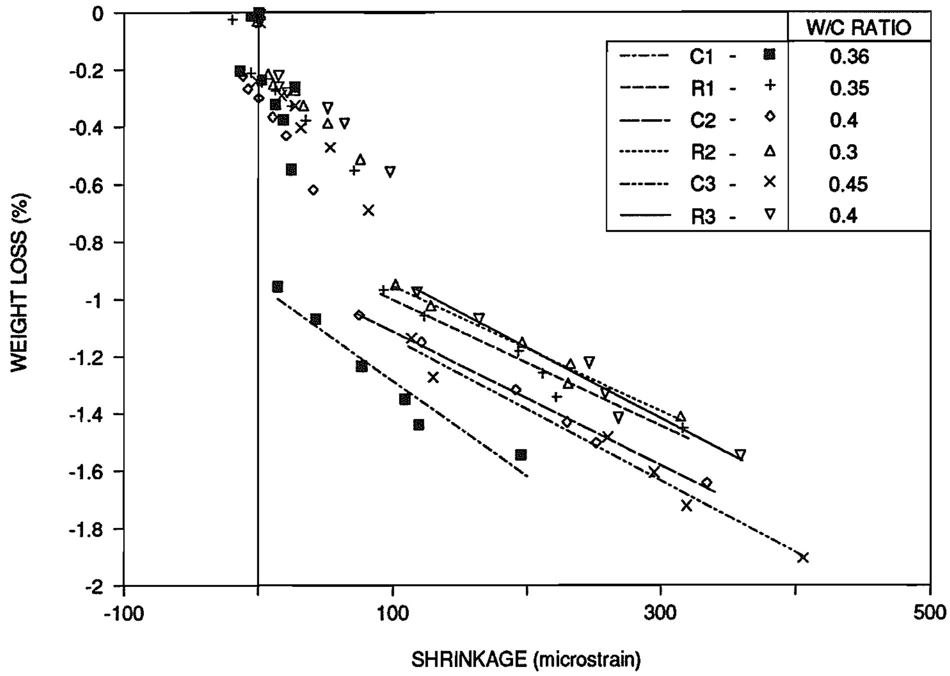


Figure 7.14 Relationship between weight loss and shrinkage for set A

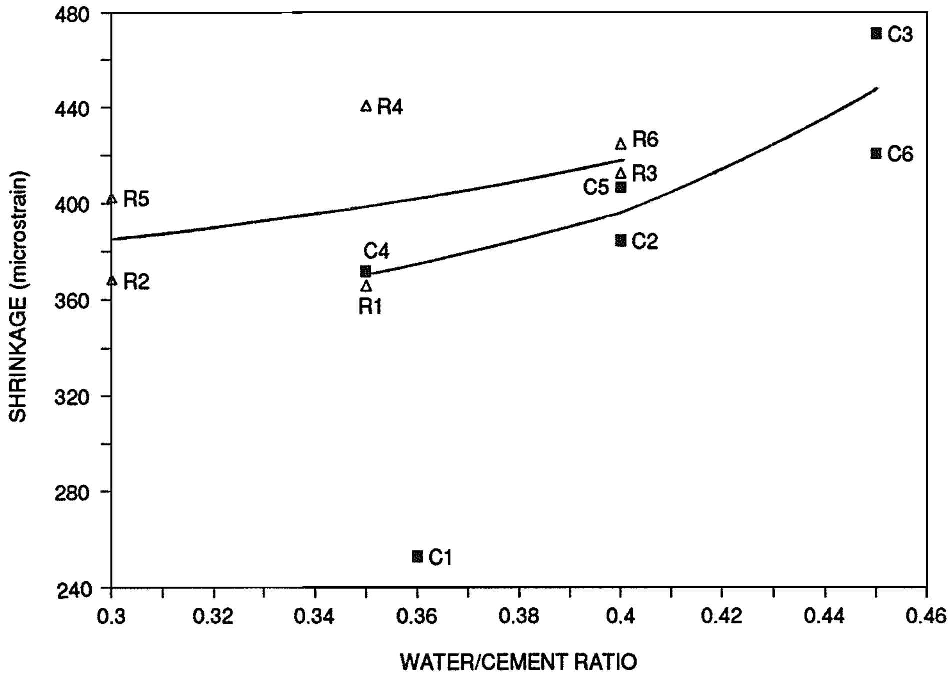


Figure 7.15 Relationship between shrinkage and water/cement ratio

aggregate concretes. As the strength of concrete is dependent on the water/cement ratio, it follows that the concretes with the highest stress/strength ratios also have the highest water/cement ratios. Therefore the influence of water/cement ratio on creep follows the same trend as the influence of stress/strength ratio, as can be seen in Figure 7.17.

7.4 Conclusions

(i) The 8% increase in free water in the recycled aggregate mixes, which was suggested by Ravindrarajah (1985), was too high to achieve a workability similar to that of the control concretes. It is likely that the water allowed in the mixes for water absorption of the aggregate was not completely used because the aggregates were not pre-soaked before mixing. This would imply that the free water was higher and consequently the workability would have been improved.

(ii) At early ages, the recycled aggregate concretes had lower compressive strengths but by 28 days had achieved the strengths of corresponding controls at more consistent rates.

(iii) The difference in the compressive strength of recycled aggregate concrete and conventional concrete was insignificant and the large differences found by other researchers, mentioned in Section 7.1.3, were not observed. In this study, the conventional aggregate consisted of smooth particles whereas the recycled aggregate particles were very angular. The better particle interlock in the recycled aggregate concretes would have contributed to the high strength.

(iv) In most cases, the recycled aggregate concretes achieved the same or higher Young's moduli. This also disagrees with the findings of other researchers but may be explained by the same reasons mentioned in Conclusion (iii).

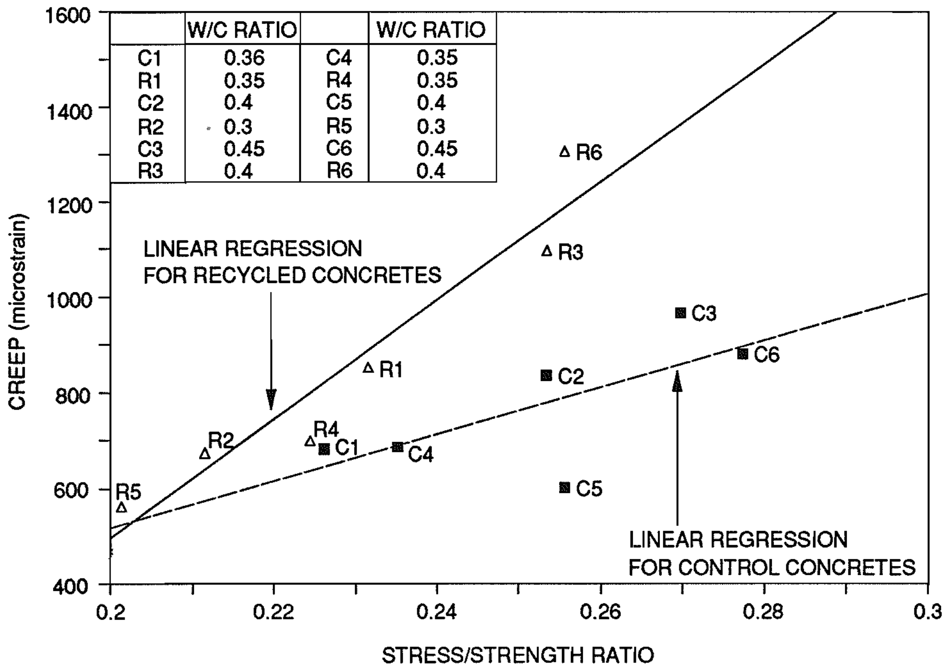


Figure 7.16 Relationship between creep and stress/strength ratio

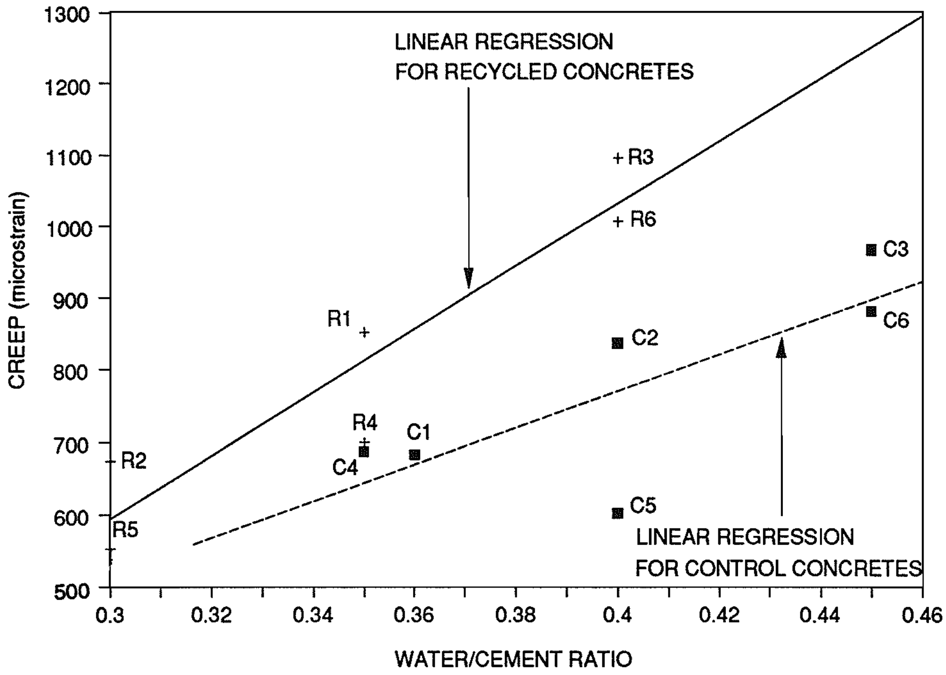


Figure 7.17 Relationship between creep and water/cement ratio

(v) The control concrete with the lowest water/cement ratio achieved the least shrinkage whereas both types of concrete made with high water/cement ratios exhibited high shrinkage. It was evident that shrinkage, to a large extent, was not very sensitive to water/cement ratio or the type of aggregate used.

(vi) Greater creep was exhibited in the recycled aggregate concrete particularly when a high water/cement ratio was used. This was attributed to the lower modulus of the crushed concrete aggregate and the larger quantity of hydrated cement paste in the recycled aggregate concrete. At a low water/cement ratio there appeared to be little difference in creep or shrinkage when the two types of concrete were compared.

(vii) The recycled aggregate in this research was very clean and the work was carried out in laboratory conditions. The results from this chapter could not be used as a direct guide for the manufacture of recycled aggregate concrete in industry unless the aggregate had been processed carefully and was clean. Small quantities of contaminants could hinder the hydration of cement and produce concrete which was not durable. If, in the future, recycled aggregate was to be considered for use as structural concrete, then the aggregate would need to be tested to examine chloride and sulphate contamination.

(viii) As Hansen (1985) suggested, trial mixes should be conducted where possible before making recycled aggregate concrete. It appears from the results of this research that better properties might be achieved if recycled aggregate was not pre-soaked before mixing and if low water/cement ratios were used. However, the data from this study is limited and further research would be required to confirm these conclusions.

CHAPTER 8

DISCUSSION

This chapter is divided into two parts. In Section 8.1, data from previous chapters are brought together to show some overall trends for the behaviour of recycled aggregates and recommendations are made on the placement of these materials, particularly for use in road sub-base layers. In Section 8.2, a more general view is taken on the use of recycled materials in civil engineering. This section includes suggestions for altering material which does not comply with existing specifications and some ideas are discussed concerning ways in which the civil engineering industry may come to terms with and use recycling of construction waste to its advantage.

8.1 Global review of test results

In Section 3.7, the results of density tests conducted on limestone and demolition debris in the Portsmouth field trial were presented. A series of laboratory density tests were also carried out, the results of which were presented in Section 4.1.3. It is interesting to observe whether the laboratory compaction test described in BS 5835 (1980) can reproduce site densities, as Pike and Acott (1975) suggested it would.

The limestone used on site had a specific gravity of 2.61 whereas the specific gravity of the limestone aggregate tested in the laboratory was 2.69. Therefore a comparison on the basis of dry density was not possible. However, when the results were presented in the form of V_s (proportion of volume occupied by solids), the particle packing on site could be compared with that achieved in the laboratory. The maximum V_s obtained on site was 96.5% and the minimum was 82% with an average for all tests of 90.3%. The peak dry density of 2320kg/m^3 which was obtained in the laboratory converts to a V_s of 86%. It is apparent therefore that V_s on site was higher which suggests that site compaction was more effective.

A similar comparison can be made for the results of density determination on demolition debris. It can be seen in Section 3.7 that the maximum V_s obtained on site in the upper layer was 85.3% and the minimum was 70.9% with an average of 79.9% for all density tests. The V_s corresponding to the maximum density in the laboratory was 73.4%. Therefore the difference between the peak V_s obtained in the laboratory and the average density obtained on site was 6.5% whereas the difference for limestone was 4.3%. It can be concluded therefore that site compaction was again more effective than that produced in the BS 5835 compaction test (1980).

One of the objects of this research was to establish the best condition in which to place recycled material and limestone so that the best performance with regard to compaction, shear strength, bearing capacity and frost heave could be achieved. Figures 8.1, 8.3 and 8.5 show the laboratory results of ϕ_{ds}, CBR and frost heave plotted against V_s for limestone, demolition debris and crushed concrete. Figures 8.2, 8.4 and 8.6 display the same data plotted against free water content. The results presented in these figures have already been presented in Chapters 4, 5 and 6. As might be expected, ϕ_{ds} increases as V_s increases (Figure 8.1). No relationship between ϕ_{ds} and free water content could be observed because the tests were conducted at similar moisture contents. CBR was found to be dependent on V_s although the relationship was not as clearly defined as that for ϕ_{ds} due to large scatter in the results, which can be seen in Figure 8.3. In Section 4.2.3, it was found that although CBR was primarily a function of V_s, there also existed some influence of free water content on CBR. When the moisture content was increased the CBR was greatly reduced. This can be seen in Figure 8.4 but the trend is not as definite as the dependence of CBR on V_s

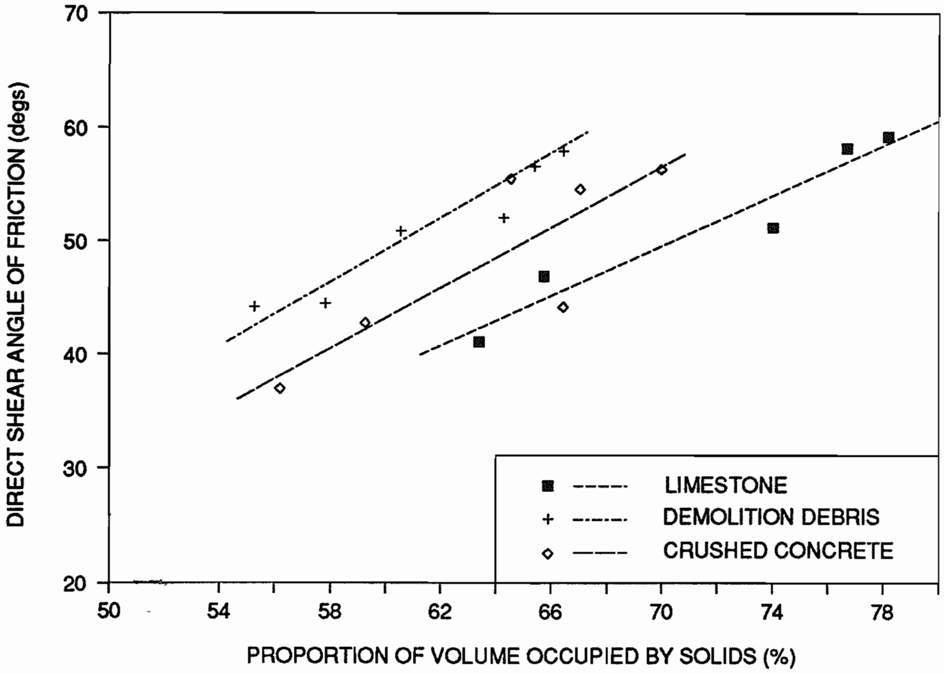


Figure 8.1 Influence of the proportion of volume occupied by solids on the direct shear angle of friction

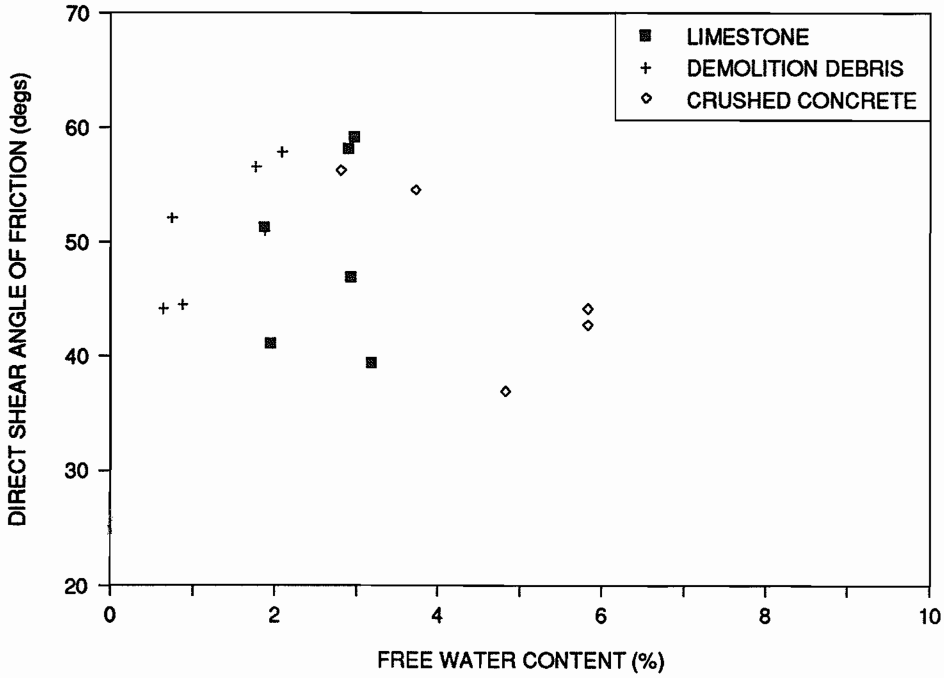


Figure 8.2 Influence of free water content on the direct shear angle of friction

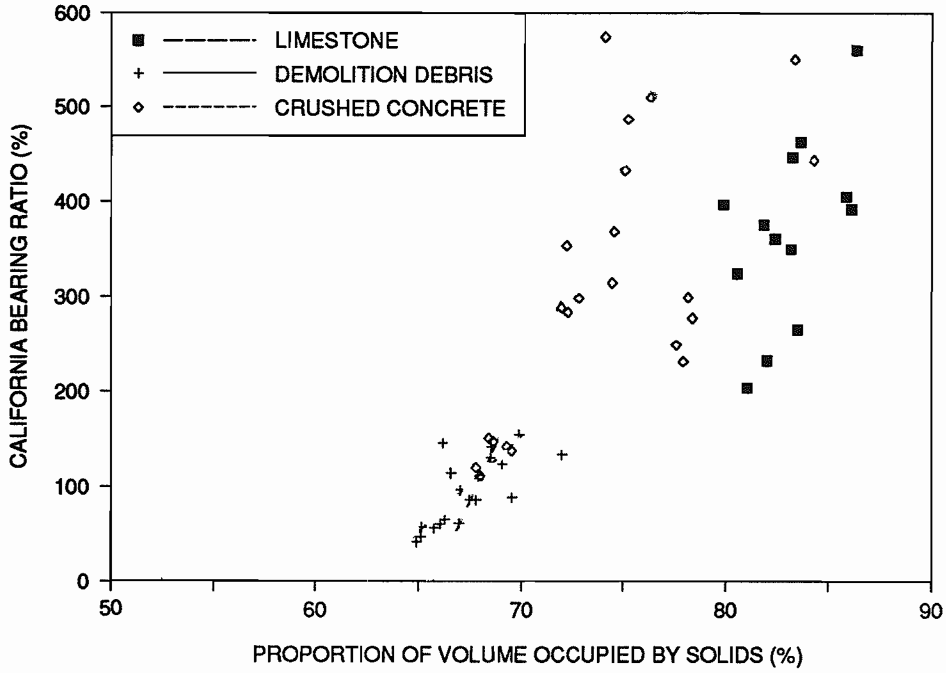


Figure 8.3 Influence of the proportion of volume occupied by solids on CBR

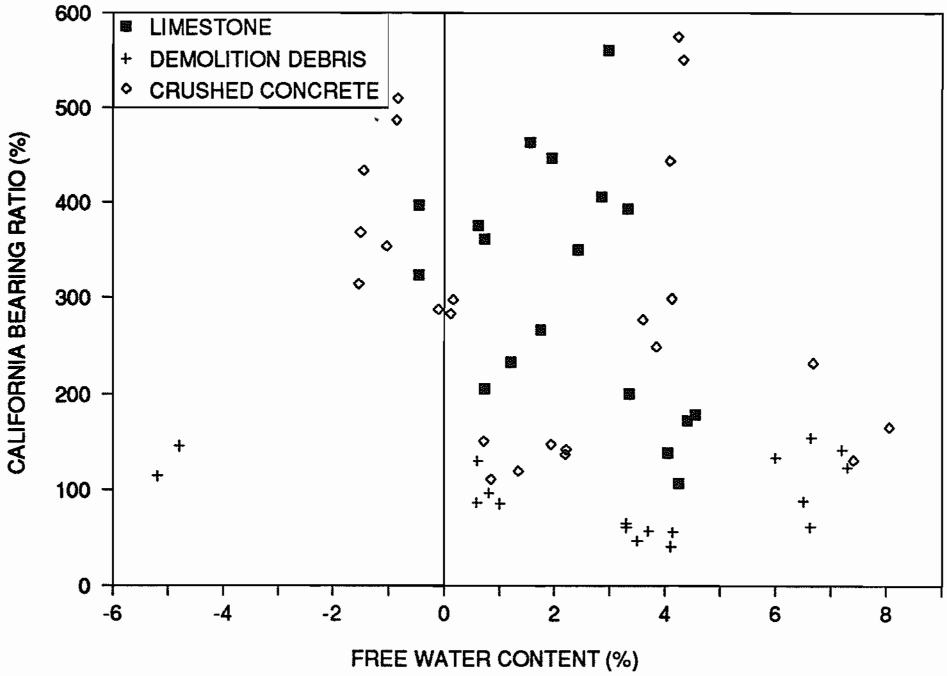


Figure 8.4 Influence of free water content on CBR

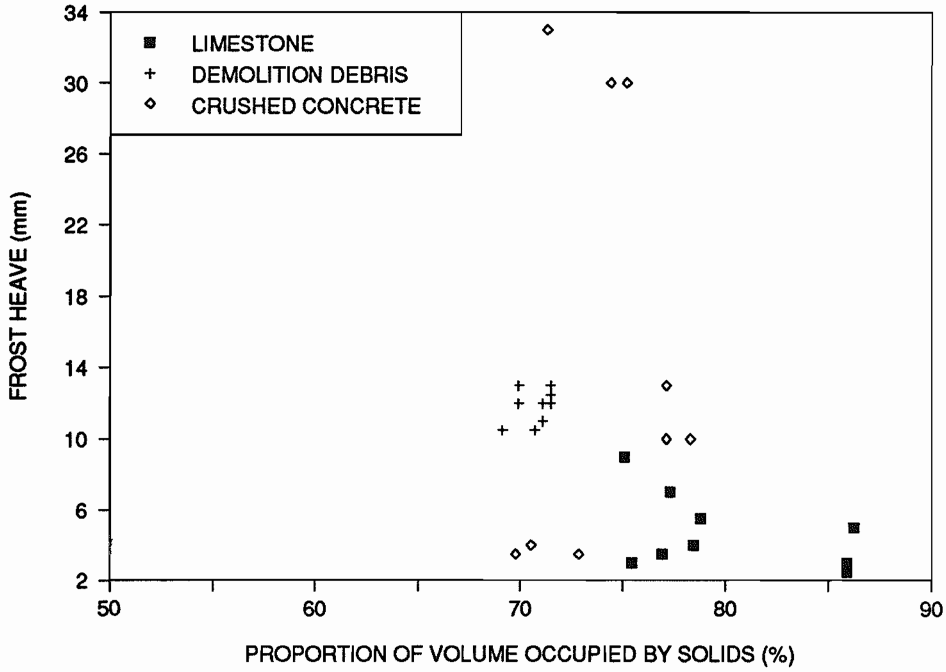


Figure 8.5 Influence of the proportion of volume occupied by solids on frost heave

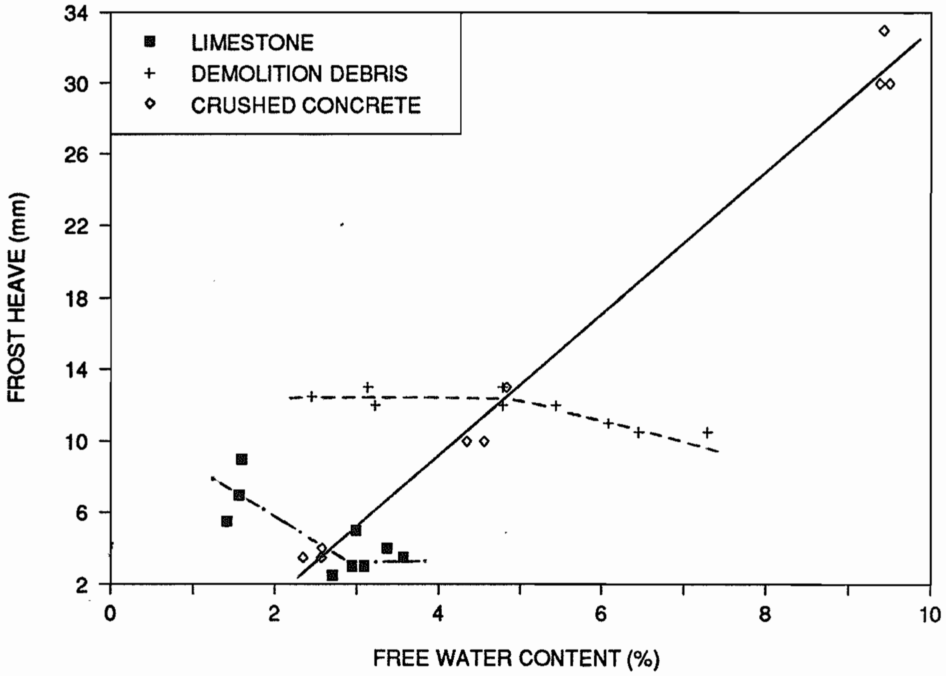


Figure 8.6 Influence of free water content on frost heave

shown in Figure 8.3. Definite trends did not exist for the frost susceptibility results shown in Figures 8.5 and 8.6 except for the apparent dependence of the frost susceptibility of crushed concrete on free water content. In most cases the frost heave samples for each material were compacted at a similar density and therefore a relationship between frost heave and V_s could not be observed.

The values of V_s and free water content which produced the best and worst performance of the materials were noted from Figures 8.1 to 8.6 and are listed in Tables 8.1 and 8.2. The peak V_s and optimum free water content, obtained from compaction tests on the materials, are also listed in these tables.

PARAMETER	V_s (%)					
	LIMESTONE		DEMOLITION DEBRIS		CRUSHED CONCRETE	
	Best	Worst	Best	Worst	Best	Worst
ϕ_{ds}	78 (59^0)	63 (41^0)	66 (57^0)	55 (44^0)	70 (56^0)	56 (37^0)
CBR	86 (550%)	80 (100%)	70 (155%)	65 (41%)	75 (566%)	67 (111%)
Frost heave	86 (3mm)	75 (6mm)	69 (10mm)	71 (13mm)	70 (3.5mm)	75 (30mm)
Compaction	86	80	73	66	77.5	66

Table 8.1 The influence of V_s on the performance of the materials

Some conclusions can be made from the data in these tables. For limestone to achieve the highest CBR and lowest frost heave values, a V_s of 86% was required. However, the highest V_s attained in the shear box tests was 78%. If shear box tests were conducted on samples at a V_s of 86% it is likely that ϕ_{ds} would have been higher because of the direct relationship between V_s and ϕ_{ds} and it is suggested therefore that limestone should be placed at a V_s of

86%. The peak density which was achieved in the compaction tests also corresponded to a V_s of 86%. It can be seen in Table 8.1, when limestone was compacted at a V_s of 80% or less, that the material was in its worst condition with respect to ϕ_{ds}, CBR and frost heave.

PARAMETER	FREE WATER CONTENT (%)					
	LIMESTONE		DEMOLITION DEBRIS		CRUSHED CONCRETE	
	Best	Worst	Best	Worst	Best	Worst
ϕ_{ds}	2.6^* (59^0)	2.6^* (39^0)	1.3^* (56^0)	1.3^* (44^0)	4.5^* (56^0)	4.5^* (37^0)
CBR	2.3 (550%)	4 (100%)	7 (155%)	4 (41%)	4 (566%)	1 (111%)
Frost heave	3 (3mm)	1.5 (6mm)	7 (10mm)	3 (13mm)	2.5 (3.5mm)	9.5 (30mm)
Compaction	3.1	2.4	5	1.1	5.3	2.5

Table 8.2 The influence of free water content on the performance of the materials

* For each material all shear box tests were conducted at a similar moisture content.

In Table 8.2, it is apparent that limestone performed best at a free water content of 2%-3%. However, the free water content which corresponded to the lowest density in the compaction tests was 2.4%. The highest frost heave was achieved at a free water content of 1.5% but the lowest CBR of 100% was obtained at a relatively high free water content of 4%.

The V_s at which demolition debris achieved its best performance with regard to CBR and frost heave was 69%-70% which is slightly higher than the value of 66% which was obtained in the shear box tests. These data suggest that demolition debris could be placed at a V_s of 69%-70% which is 4% lower than the peak V_s obtained in the compaction tests. The lowest V_s obtained in the compaction tests was 66% and when V_s was below this value the performance of the material deteriorated with respect to ϕ_{ds} and CBR. Surprisingly, the highest frost heave was observed when V_s was increased to 71%.

It can be seen in Table 8.2 that there was a large difference between the free water content of 7% required to achieve the highest CBR and lowest frost heave and the value of 1.3% which was obtained in the shear box tests. It was considered that the latter value was too low and that demolition debris should be placed at a free water content closer to that achieved at peak density in the compaction tests i.e. 5%-7%. Results quoted in Table 8.2 for demolition debris confirmed the observation made on the limestone data that material performance deteriorated with decreasing free water content.

The V_s at which crushed concrete achieved low frost heave and high shear strength was 70% with the highest CBR value attained at a V_s of 75%. It appears that a V_s in the range of 75%-77.5% would be the most suitable to achieve high values of ϕ_{ds}, CBR and density but a frost heave of 30mm was observed at a V_s of 75%. This excessive heave was more likely to be caused by the high moisture content of the samples and not by the level of particle packing. The lowest CBR and ϕ_{ds} values were obtained at a V_s of less than 67% which was close to the V_s corresponding to the lowest density achieved in the compaction tests.

The data suggest that crushed concrete should be placed at a free water content of 4%-4.5% which is slightly lower than the free water content of 5.3% obtained at peak density in the compaction tests. The initial moisture content of the crushed concrete samples appeared to have considerable influence on frost heave and at a free water content of 4% the frost heave of crushed concrete was 10mm. The behaviour of crushed concrete with regard to frost heave needs to be studied further to investigate the apparent dependency of frost heave on the initial moisture content and the benefits from setting action with time.

The above interpretation of the data in Tables 8.1 and 8.2 is made on the basis of the best and worst performance of the materials exhibited in laboratory tests. The behaviour of frost heave, however, was not investigated fully and the free water content did not appear to have a large influence on frost heave except for the apparent dependence of frost heave on the initial moisture content of crushed concrete.

It was noted earlier that higher V_s values were achieved on site than in the laboratory at lower moisture contents. In general, CBR and ϕ_{ds} increase as V_s increases and therefore the material should be compacted to the maximum V_s level attainable for a particular compactive effort at the moisture content required to achieve this maximum V_s. It was stated in Section 6.7 that to prevent frost heave damage to granular layers, a high permeability is desirable. However, if high values of CBR and ϕ_{ds} are to be achieved, a dense stiff layer would be required. This again reinforces the need for open packed capping layers to be used which would prevent capillary rise and therefore reduce frost heave.

8.2 Recycling in civil engineering

Considering the amount of repairs carried out on British roads every year, the design and construction of road pavements using natural aggregate in the unbound layers could be improved. Although road pavement deterioration cannot always be attributed to a weakened sub-base, it has been found, even when natural aggregate is used, that a sub-base can be damaged by frost heave or can be subjected to excessive deformations by construction traffic at an early age. The data from this research suggest that a road constructed using recycled aggregate in the sub-base layer would not be unlike a structure containing a natural aggregate sub-base. This observation could only be confirmed by conducting field trials using accelerated traffic loading or by placing recycled aggregate in the sub-base of a lower class road and monitoring its performance under normal traffic loading. It would be unhealthy to try to remove all scepticism of recycled aggregates until extensive research of this type has been conducted.

In recent years, demolition contractors have been forced to recycle construction waste due to high dumping costs, particularly in large cities. Some contractors have purchased basic mobile crushing and screening plant to recycle construction waste and use the recycled product either on the same site or locally. This is worthwhile but in a sense demolition contractors have been forced to bypass a step in what might be referred to as the ideal progression of recycling in the construction industry. The setting up of recycling plants by individual contractors, without some standard guidance on the production of high quality recycled material, could result in the production of many recycled products of variable content and quality.

The most efficient way of promoting recycling in the British civil engineering industry would have been to produce a standard to which recycled aggregates should comply. This might have had the effect of reducing the number of mobile recycling plants being installed. Alternatively, it could have encouraged demolition contractors to unite their efforts by setting up sophisticated, stationary recycling plants around large cities to which all construction waste could be brought for recycling. These recycling plants would be capable of recycling all grades of contaminated material with the guarantee of producing a good quality aggregate. The producer of recycled aggregate at a recycling plant should ensure that the aggregate complies with any existing specification for the purpose for which the aggregate is required. Random testing should also be undertaken at the site where the recycled aggregate is used.

A fixed site recycling plant would serve both as a dumping site and a source of aggregate. In some cases, however, demolition contractors might prefer to use mobile crushing plant for particular demolition contracts. Both stationary and mobile recycling plants are likely to have important roles to play in the demolition industry.

Three situations are likely to arise for a demolition contractor.

(i) The rubble from a structure under demolition is not to be reused as a high quality aggregate on the same site. If the rubble is to be used as capping material or to bring up levels on the site, then the demolition contractor would be justified in using a mobile recycling plant.

(ii) A structure is demolished and the rubble is needed for reuse on the same site but in the form of a high quality aggregate. The client could only be guaranteed of a high quality product if the rubble was recycled in a plant with good cleaning systems, particularly if the structure was made up of several types of material. The demolition contractor in this case has three options.

  a) A sophisticated recycling plant could be brought on to the site. Normally the expense incurred would not be justified unless the structure was very large and took a considerable length of time to demolish.

  b) A mobile crushing plant could be used and the demolition contractor could aim to produce a recycled product which would pass the required specifications although his success in this objective would not be guaranteed.

  c) The rubble could be sent to a large fixed site recycling plant which would guarantee the quality of the recycled product.

(iii) The third situation involves the recycling of relatively clean rubble e.g. a concrete slab in a road pavement. Fitzpatrick & Sons Ltd. (1989) proved that a Type 1 graded sub-base material could be produced using mobile crushing plant on a road

> maintenance contract and Somerset County Council (1989) used crushed concrete, obtained from the break up of the concrete slab of the M5 motorway near Taunton, as Type 1 sub-base material.

It can be seen that there is a place for both types of recycling operation in the civil engineering industry. The most efficient system would be a combination of both types of plant to be made available i.e. to have fixed site recycling plants placed around major cities and also mobile crushing plants for recycling on particular sites.

The installation of any type of recycling plant is likely to cause environmental problems locally. Proper controls on dust and noise should be organised at the initial stages of design and installation of the plant. Old quarry sites are ideal locations for recycling plants. Although the location of a quarry would have been governed by the mineral reserves present there would also have been some consideration given to its proximity to residential areas. If sufficient excavation had taken place the floor level of the quarry would be below ground level. Therefore, noise barriers for the recycling plant would already be in existence.

Dust could be controlled by the use of precipitators where dust from the crushers is cycloned off into an enclosed unit. Stockpiles of aggregate should also be sprinkled with water when the weather is hot and dry. Discarding or reusing fines and dust is a problem which has been encountered by producers of crushed rock and in recycling plants in Europe although some crusher fines have been used in the manufacture of low quality bricks (Mulheron, 1990).

8.3 Possible uses of recycled materials

In Section 2.3.1, some possible uses of recycled materials were listed and in this section some conclusions are made on the ability of the recycled aggregates examined in this study to pass the compliance tests. As stated previously, both crushed concrete and demolition debris could be used as road sub-base material but the frost susceptibility of any recycled material to be used in a particular job would need to be determined. However, as specified in Clauses 803 and 804 of the Specification for Highway Works (1986), the use of demolition debris is not allowed for use as sub-base material. From the results of tests conducted on demolition debris in this research it is likely that it could be considered for use as sub-base material.

If the sulphate content of recycled aggregate was found to be below 1.9g of $S0_3$ per litre then it could be used as the aggregate in lower grade, cement bound material for use in roads, as defined in Clauses 1035, 1036 and 1037 of the Specification for Highway Works (1986).

The particle grading of material to be used as pipe bedding is different to the well graded aggregate required for sub-base. Depending on the grade of pipe bedding required, the particle size of the material should be in the ranges of 20mm-5mm, 14mm-5mm or 40mm-5mm (Clause 503 of the Specification for Highway Works, 1986). To obtain these gradings, some screening would be necessary to remove large particles and fines. The recycled material would also have to be tested to ensure that its sulphate content was less than 1.9g/litre. Material to be used as backfilling to pipe bays should not have more than 3% of its particles passing the 0.075mm sieve and the maximum particle size should be 20mm (Clause 512). These limits on grading are more restrictive than those for sub-base aggregate.

Clause 505 of the Specification for Highway Works (1986) requires material for use as backfilling of trenches to be non-plastic and to have a 10% fines value greater than 50kN. These requirements are similar to those for material to be used as backfilling to earth retaining structures (Clause 513). It can be seen in Sections 3.4 and 3.5 that both crushed concrete and demolition debris complied with these requirements.

There are four grades of fill to structures listed in Clause 610 of the Specification for Highway Works (1986). From the results of the tests conducted on crushed concrete and demolition debris it can be concluded that the recycled materials would comply with the requirements for all four grades. If organic matter in the recycled materials was found to be below 2% by mass and the sulphate content was less than 1% by mass then the materials could also be used as aggregate for cement stabilised capping, as defined in Clause 614.

In Chapter 3, it was evident that a Type 1 certified limestone did not comply fully with the Type 1 sub-base requirements. In general, the only test which is performed on a conventional aggregate to be used as sub-base material is a particle grading test. If recycled material is to be used for a particular job, it usually must undergo all tests listed in the Specification for Highway Works (1986). One of the conclusions of this study is that conventional aggregate is as likely to fail the compliance tests as recycled aggregate. For consistency and the fair treatment of secondary materials in construction, it is suggested that all aggregates should undergo the same series of tests.

If a recycled aggregate fails the compliance tests, then there are several ways in which its performance may be improved. Altering the particle grading at the recycling plant should not be difficult if screens are available and if the jaw crushers are set at the correct setting to produce the required grading. The susceptibility of recycled materials to frost might be found to be lower if frost heave tests were conducted three months after placement of the aggregates rather than immediately after compaction. If further research proved that frost susceptibility did not improve with time, the addition of cement to recycled aggregate could reduce frost heave and improve its bearing capacity and shear strength. The partial replacement of recycled material with natural aggregate is likely to improve its performance but this partly defeats the purpose of recycling i.e. slowing down, as much as possible, the depletion of natural aggregate. If a sample of recycled material appears to be plastic then it is more than likely to have been contaminated with clay fines. This would have occurred if the fine material had not been removed before crushing. Again, in this case the recycling process would need to be altered if an acceptable material was to be obtained.

There is likely to be some scope for using recycled aggregate as aggregate in new concrete and extensive research has been conducted in this area. If precautions were taken e.g. increasing the cement content to achieve adequate strength or increasing the size of a concrete member to allow for shrinkage, then recycled aggregate concrete could compare well with conventional aggregate concrete. It might be more difficult to convince clients to use recycled aggregate in concrete than it is to persuade them to use recycled aggregate as sub-base and fill material. It is unlikely that there would be a large demand for recycled aggregate for use in structural concrete although some use might be made of crushed concrete as aggregate in lean concrete.

8.4 Estimates of errors

Estimates of the accuracy of the test data, particularly of the density and CBR results presented in Chapter 4, were not made. This creates difficulty when results are compared. An estimate of the errors likely to arise could have been made if several other tests were conducted but this would have taken considerable time. However, the consequence of errors in the measurement of specific gravity can be examined. In Figure 4.4, for example, the percentage air voids lines were calculated using a specific gravity of 2.56. If the specific gravity was within 0.5 of this value and the 0% air voids line was calculated using a specific gravity of 2.61, then all data points on the figure would fall to the left of the 0% air voids line. This 19% error in specific gravity would be the minimum error, if the dry density measurements were assumed to be accurate.

CHAPTER 9

CONCLUDING REMARKS

In this chapter, the main conclusions of the research on recycled aggregates are presented. Some conclusions were made earlier at the end of each chapter which were more specific to the chapter subjects. Ideas for further research in the field of recycled materials and in aggregates in general are also discussed with some recommendations for the production of a standard for recycled materials in Britain.

9.1 Conclusions

In Chapter 2, it became clear that Britain was well behind other countries in its attitude to recycling of construction waste. The construction industries of other countries have accepted recycling as a useful, alternative source of aggregate and have written standards to which recycled aggregates should comply. These standards mainly include allowable limits of contamination in the recycled materials.

Britain is relatively rich in natural aggregate reserves and therefore the civil engineering industry did not seriously consider the recycling of construction waste until dumping became expensive and an increased awareness of the environment became evident in recent years. In comparison, the Netherlands has poor reserves of natural aggregate and has developed sophisticated recycling plant which incorporates several sorting and cleaning techniques to produce high quality recycled aggregate for use in construction. This confirms that the technology and equipment exist to produce reusable aggregate from demolition waste.

At the Symposium on Unbound Aggregates in Roads (1989), it was decided that the plasticity test described in BS 1377 (1975) was not satisfactory for the determination of the plasticity of aggregates and that ideally a new test should be designed for this purpose. However, it was also recognised that the development of such a test might be difficult.

Some of the standard compliance tests involve testing aggregate in cylindrical moulds. The CBR test is conducted on material less than 20mm in particle size which is compacted in a 152mm diameter mould. Aggregate to be tested in the 150mm diameter mould of the BS 5835 (1980) compaction test is required to contain particles less than 37.5mm in size whereas in the frost heave test, particles up to 37.5mm in size are tested in a 102mm diameter mould. The three tests are conducted on aggregate which is to be used for the same purpose but the laboratory tests are conducted on different gradings. These gradings may also be unlike the grading of aggregate to be placed on site. This approach to aggregate testing is inconsistent and Dawson and Jones (1989), in the conclusions of the Symposium on Unbound Aggregates in Roads, stated that some effort should be made to standardise the grading and the ratio of the mould diameter to the maximum particle size.

In Chapter 8, it was found that limestone should be placed at peak dry density and optimum moisture content, as defined in the BS 5835 compaction test (1980), to achieve high values of CBR, ϕ_{ds} and frost heave. The results from tests on demolition debris suggest that it could perform well when placed at a density slightly below peak and at a moisture content below optimum. Crushed concrete should perform well if it is placed at peak density and optimum moisture content or slightly below.

Although the recycled materials could be described as lightweight aggregates, their shear strength was found to be similar to that of limestone. The shear box test method, which was developed by Earland and Pike (1985) to examine aggregate for use as sub-base material, was conducted on limestone and the recycled materials. From the results of these tests it was concluded that demolition debris and crushed concrete would be classified in the medium strength category and limestone in the highest strength category.

The critical state angles of friction of the recycled materials and of limestone were in the range of 40^0 - 42^0 and were considerably higher than the 33^0 value which was obtained for Leighton Buzzard sand. The value of Q, defined by Bolton (1986) as a constant depending on the mineralogy and compressibility of a material, was found for the aggregates to be greater than 10. This contradicted the suggestion by Bolton (1986) that materials with particle grains softer than quartz or felspar would have Q values less than 10. The results of this research suggest that Q may be dependent on the particle grading, the ratio of the size of the box to the maximum particle size in the sample or the angularity of the particles. The accuracy of the Cornforth (1973) loose heap test for the determination of the critical state angle of friction was found to be lower for well graded aggregates than for uniform, small grained sands.

The results from this research confirm that the requirement of a sub-base to be both a stiff and permeable layer is too demanding. More use should be made of a free draining capping layer which would reduce the risk of frost heave occurring in the sub-base. This conclusion was also noted at the Symposium on Unbound Aggregates in Roads (1989).

No definite conclusion could be made on the frost susceptibility of demolition debris because further testing, as required by Roe and Webster (1984), could not be conducted at other laboratories. Crushed concrete was found to be frost susceptible. The data suggest that in most cases the frost heave results of recycled materials would fall in the inconclusive or the frost susceptible ranges. However, the self-cementing ability of recycled aggregates may mean that the susceptibility of these materials to frost would decrease with time. Problems concerning frost susceptibility are also evident for many natural aggregates (Jones, 1989) and are not specific to recycled materials.

It can be concluded for the recycled aggregate concrete, examined in Chapter 7, that an increase in free water of 8% was too high to achieve a workability similar to that of natural aggregate concrete. There appeared to be little difference in the compressive strength and the Young's modulus of recycled aggregate concrete when compared to conventional concrete. The shrinkage and creep data presented in Chapter 7 suggest that a low water/cement ratio should be used in recycled aggregate concrete.

The use of mobile and stationary recycling plants was discussed in Chapter 8 and it was concluded that both would be useful, depending on the construction rubble produced during demolition and the quality of material required for reuse. The installation of a recycling plant is liable to cause objections from residents in its immediate vicinity but the environmental problems are likely to be no worse than those caused by the producers of crushed rock.

The possible uses of recycled material were also discussed in Chapter 8 where it was noted that recycled aggregate could be used as trench fill, pipe bedding, fill to structures, capping material, sub-base material and cement bound material in roads. The compliance tests for these uses are different but it is likely that recycled material could be made to conform by varying the grading, removing fines or by adding cement in some cases.

Less discrimination would exist against recycled aggregates if all aggregates were subjected to the same series of compliance tests. Some effort is required to help recycled aggregates gain the same respected status as natural aggregates and to ensure that all potential aggregates are treated fairly.

9.2 Suggestions for a standard on recycled materials in Britain

This research study was mainly involved with the physical properties of recycled materials and their ability to perform as construction aggregates. The investigation did not include a full examination of the contaminants in the materials. If a standard for use in Britain is produced, it is suggested that the contaminant levels listed in the Dutch standard, Centre Row (1988), should be used until further research has been conducted in Britain.

In the Specification for Highway Works (1986), compliance tests and limits for different uses of aggregate are listed. Recycled aggregate would be required to satisfy the requirements in this specification. It is suggested that a standard for recycled material should concentrate more on the sources from which construction rubble could be recycled, methods of production and quality control in the recycling process. Some recommendations might also be made on the placement of material on site to avoid segregation and to ensure the achievement of consistent density.

In the concluding remarks of the Symposium on Unbound Aggregates in Roads, compiled by Dawson and Jones (1989), it was stated that the classification of aggregates should be based on an internationally accepted set of tests. An international set of limits for these tests would not be possible due to mineral variation in different countries and climatic influences. Some countries would require frost heave specifications whereas others might need restrictions on the placement of wet aggregate in hot weather. The method of production and quality control of recycled materials could be quite similar internationally but the limits on contamination and rubble source might need to be altered due to the various aggregates and construction materials used in different countries.

Dawson and Jones (1989) concluded that testing of aggregates should take place at the location of aggregate production, on site during placement and that the final construction should also be checked to ensure that the aggregate has achieved design expectations. This echoes the recommendations made earlier for recycled aggregate production. It was suggested that extensive testing should take place at recycling plants and that random testing should also be conducted at the locations where the aggregate is used.

The most effective approach to aggregate assessment would be a combination of laboratory and site testing. Ideally, testing on site should extend to a site trial which could be conducted on a small area before the road construction commenced, using materials and conditions similar to those expected to be employed in the main construction. If the Specification for Highway Works (1986) included the requirement of a site trial for sub-base material, it is likely that a rapidly gained respect would be achieved for recycled materials on the basis of their ability to self-cement with time. The inclusion of site trials would increase the cost of road construction. However, highway engineers have wished for some time to move more towards site testing (Symposium on Unbound Aggregates in Roads, 1989) because it is considered that the results of laboratory tests do not relate to the actual demands required of an aggregate on site.

9.3 Further research

During the course of this research it became clear that there was much scope for further research into, not only the properties of recycled aggregates, but also aggregates in general. In this section, the ideas for further work can be divided into two groups. First, the areas for possible research on aggregates are examined and secondly some ideas are presented for research which would be considered necessary for the rapid progression of recycled material to a respected status as aggregate for construction.

In Chapter 5, an analysis using Bolton's dilatancy index (1986) was conducted on data from shear box tests on limestone, demolition debris and crushed concrete. Values of Q (a constant, depending on material type) were obtained from this analysis for the three aggregates. Bolton (1986) analysed tests on uniformly graded sands, for which he concluded that Q was 10. It was found in Chapter 5 that the Q value corresponding to each of the aggregates was greater than 10. This contradicts the conclusion made by Bolton (1986) that Q depends on mineralogy and compressibility and that Q for materials containing grains which are softer than quartz or felspar would be less than 10. The wet, well graded aggregates with particle sizes ranging from 37.5mm to less than 0.075mm were quite different to the small grained, uniform sands used by Bolton (1986) for his analysis. To confirm the findings of this research an intermediate step would be necessary i.e. the analysis of data from a series of tests on aggregates in a dry condition at various gradings so that the influence of particle grading on Q could be determined.

It would also be interesting to conduct a similar analysis on test data obtained from samples containing various maximum particle sizes to determine whether the ratio of the dimensions of the shear box to the maximum particle size affects Q. Another possible influence on Q may be the angularity of the particles. A further investigation could be carried out on rounded gravel and crushed rock particles to establish any dependency of Q on angularity.

In Chapter 6, a series of frost susceptibility tests was conducted on the limestone and recycled aggregates. An attempt was made to measure the influence of initial moisture content on the frost heave of the samples but the data for crushed concrete were the only results to show any definite trend. Calculations using these data suggested that the initial moisture content was not significant with respect to frost heave. However, the data were limited and it would be interesting to determine whether the initial moisture content of frost heave test specimens and of unbound road pavement layers affects the total frost heave exhibited. It may be easier to accomplish this when the design and construction of a new frost heave apparatus at Nottingham University have been completed (Baba, 1990). In this apparatus, it will be possible to measure the frost heave, the penetration of frost heave and water intake of aggregate samples during a test. The water intake therefore could be related to frost heave. Instrumentation to measure thaw weakening is also under development at Nottingham University for use in the proposed frost heave apparatus (Baba, 1990).

Demolition debris should have been tested at two other laboratories to fulfil the requirements of Roe and Webster (1984) because its frost heave was found to be in the inconclusive range. However, the aggregates could only be tested at one laboratory due to financial restraints on the research contract. It would also be useful to confirm, by further testing, the high frost heave results measured on the saturated crushed concrete specimens.

Another possible research project would be the determination of the effect of impurities on frost susceptibility. This could be accomplished by conducting frost heave tests on samples of demolition debris or crushed concrete containing varying quantities of brick, gypsum, soil and wood. The effect of these contaminants on the plasticity, shear strength, CBR and compaction of recycled aggregates would be another interesting research subject. Several investigations have been conducted on the effect of contamination in recycled aggregate for use as aggregate in concrete (Hansen, 1985) but very little has been reported on the contamination of recycled aggregate for sub-base.

It was suggested by Croney and Jacobs (1967) that stabilisation of an aggregate by the addition of cement helps to improve its resistance to frost heave. Further research could be conducted on stabilised samples of recycled aggregate to determine the quantity of cement needed to reduce the frost heave to less than 9mm. Sweere (1989) noted the ability of recycled aggregate to self-cement after some time under load. It would be interesting to determine whether this ability would also reduce frost heave. Cores of material would need to be extracted from a sub-base layer, which had undergone 2-3 months traffic loading or some form of accelerated loading, to evaluate the influence of this binding effect.

One final suggestion for further research on frost heave would be a study of the heave of aggregate obtained from crushing lean concrete and structural concrete to determine the effect of mortar in the aggregate on frost heave. The higher cement content in structural concrete might be sufficient to stabilise recycled aggregate made from it and therefore reduce frost heave.

The work in this research consisted of an examination of the physical properties of recycled aggregate without a chemical analysis. It appears that the next step should be an examination of the chemical properties. The quantity of sulphates, chlorides and other contaminants such as alkali-silica reacted concrete in the material would need to be known, particularly if it was to be placed near reinforcement.

The CBR test which was conducted on the aggregates is considered to be a crude stiffness test at large strains. It would be interesting to measure the elastic stiffness of compacted recycled materials. Sweere and Galjaard (1989) stated that stiffness is more important than shear strength in a sub-base layer because when the stiffness is increased the tensile strains in the asphalt layer become smaller. Consequently the life expectancy of the road pavement is increased (Sweere, 1989). A repeated load triaxial test has been developed at Delft University (Sweere, Penning and Vos, 1987) and another has been constructed at Nottingham University (Brown, O'Reilly and Pappin, 1989) to test the stiffness of aggregates for use in the unbound layers of road pavements. This test involves cycling the confining and deviator stresses, on a sample of 300mm in height and 150mm in diameter, to represent approximately the effects of repeated loading in a road pavement. For elastic behaviour, the results from the triaxial apparatus were found to be representative of those obtained in the field (Brown, O'Reilly and Pappin, 1989).

It would be interesting to conduct this triaxial test on the recycled materials used in this research to compare their performance with results of similar tests on limestone. Sweere, Penning and Vos (1987) conducted triaxial tests on recycled aggregate and found that the resilient modulus compared well with that of a conventional aggregate. Due to the complexity of the instrumentation and set up of the triaxial test, Sweere, Penning and Vos (1987) concluded that the test could not be used for routine testing of aggregates.

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