Samenvatting
Blijvende vormverandering van de ongebonden fundering in een flexibele verharding kan tot aanzienlijk onderhoud leiden. Om de mate van blijvende vormverandering te voorspellen zijn er in principe 2 opties mogelijk. De eerste is om aan de hand van modellen de blijvende vormverandering te voorspellen op basis van de in de ongebonden lagen aanwezige spanningen, de materiaalkarakteristieken, het vochtgehalte, de verdichtingsgraad en het aantal lastwisselingen. De tweede methode is veel eenvoudiger. In dat geval wordt de verharding zodanig gedimensioneerd dat de spanningen in de ongebonden lagen zo laag blijven dat niet voor blijvende vormverandering behoefte te worden gevreesd. Natuurlijk is dat spanningsniveau afhankelijk van de materiaaleigenschappen, verdichtingsgraad etc. In deze bijdrage wordt de ontwikkeling van zo’n spanningscriterium beschreven. In het laboratorium zijn een groot aantal triaxiaalproeven met een monotone danwel herhaalde belasting uitgevoerd op menggranulaten die varieerden in samenstelling, verdichtingsgraad en gradering. De mengverhouding beton- : metselwerk-granulaat is een belangrijke factor maar nog belangrijker is de verdichtingsgraad. Afhankelijk van deze parameters kan een bij een bepaalde steunspanning een vertikale spanning worden aangebracht die 0.45 a 0.61 maal de bezwijkspanning bij dat steunspanningsniveau is. Uit het onderzoek kwam ook naar voren dat de grootheden die het elastische vervormingsgedrag en het faalgedrag bepalen, voorspeld kunnen worden op basis van gradering, hoekigheid van het granulaat, samenstelling en verdichtingsgraad. Deze bevinding is belangrijk omdat daarmee wordt aangetoond dat receptspecificaties nog steeds zeer zinvol zijn en het niet altijd noodzakelijk is deze recept specificaties te vervangen door zgn functionele specificaties!

Trefwoorden
Triaxiaalproef, menggranulaten, faalgedrag, elastisch en blijvend vervormingsgedrag.
1. Introduction

Knowledge of the strength and deformation characteristics of unbound base materials is very important in order to allow proper pavement designs to be made. Common practice however is to safeguard the resistance to permanent deformation of unbound base and sub-base materials by means of specifications for the CBR, the gradation and the degree of compaction. Such a procedure is of course a very crude one since important factors like the stress conditions in the sub-base and base and the resistance to shear of the materials used, expressed e.g. by means of the cohesion and the angle of internal friction, are not taken into account. Therefore it is highly recommended to determine the resistance to shear and permanent deformation by means of monotonic and repeated load triaxial testing. Such tests however are relatively time consuming and therefore a need has been developed for simple transfer functions relating permanent deformation to material characteristics, stress conditions etc.

In the Netherlands, unbound base course materials are produced on a large scale by recycling concrete and masonry coming from demolishing old buildings and structures. Mixtures of 50% crushed concrete and 50% masonry (mass percentage) have been used successfully in road construction. During the last decades a lot of research has been done at the Delft University of Technology on the characterization of these mixtures by means of repeated load and monotonic triaxial tests (1, 2). The data base developed in that way has become sufficiently large to establish the transfer functions as mentioned above. This paper describes the development of such functions.

2. Test Program

Materials tested

Figure 1 shows the gradations of the base course mixtures of crushed concrete and crushed masonry that were tested. Other variables involved in the research were the degree of compaction and the ratio crushed concrete to crushed masonry. Test results obtained on the unbound base course mixtures were reported earlier (3).

Tests performed

On all materials sieve analyses were performed as well as modified Proctor and CBR tests. Furthermore monotonic (vertical strain rate 0.33%/s) and repeated load triaxial tests were performed for the determination of the strength, resilient and permanent deformation characteristics. For testing of the unbound granular base materials a large tri-axial test set-up was used (figure 2). The test specimens had a diameter of 300 mm and a height of 600 mm. The large diameter, and consequently large height were needed to obey the particle size – specimen ratio’s which are commonly set for triaxial tests (specimen diameter to max. grain size ratio of 5 – 7, specimen height to specimen diameter ratio of 2). Downscaling the gradation would allow the use of smaller specimens. Work done in this field however showed that downscaling is still "tricky
business” since most of the time the downscaled specimens showed to have different characteristics than the full scale specimens. As shown in figure 2, confinement was realized by means of vacuum, this implies that tests at various moisture contents were not

Figure 1: Gradation of the unbound base course materials (UL = gradation according to upper limit of the Dutch specifications; LL = gradation according to lower limit, FL = gradation according to Fuller; AL = average gradation; UN = uniform gradation; CO = continuous gradation).

Figure 2: Tri-axial test set-up for unbound granular base materials.
possible. The moisture content during the tests was close to the optimum moisture content as determined by means of the proctor test. The specimens were compacted using a large vibratory table to various degrees of compaction related to maximum modified Proctor density. Depending on the type of material and the composition of the mixtures of crushed concrete and crushed masonry, maximum densities between 1600 and 1800 kg/m$^3$ were obtained. For further details on the materials, specimen preparation, test protocols etc., the reader is referred to references (2) and (3).

3. Test results

Examples of the test results are given in figures 3, 4 and 5. Figure 3 shows for a number of unbound granular base course materials the dependency of the cohesion and angle of internal friction in relation to the particle grading. Figure 4 shows an example of the stress dependency of the resilient modulus of a particular base course material as a function of the degree of compaction (DOC) while figure 5 shows the development of the permanent deformation of a particular material as a function of both grading and DOC. Figure 5 indicates that the permanent deformation at 1 million load repetitions increases relatively slow with increasing applied vertical stress. However with a further increase in vertical stress, the samples seem to fail rather abruptly. This is indicated by the fact that 10% permanent deformation is obtained after only 50000 load repetitions.

4. Strength and deformation models

Strength

The (shear) strength of unbound base materials is dependent of the state of stress. It is common practice to describe the stress dependent shear strength by means of the Mohr-Coulomb criterion. This is described by means of equation 1.

$$\sigma_{1,f} = \frac{(1 + \sin \phi) \cdot \sigma_{\text{conf}} + 2c \cdot \cos \phi}{1 - \sin \phi} \quad \text{and} \quad \sigma_{d,f} = \sigma_{1,f} - \sigma_{\text{conf}}$$

Where:

- $c$ cohesion [kPa]
- $\phi$ angle of internal friction [gr.]
- $\sigma_{\text{conf}}$ confining pressure [kPa]
- $\sigma_{1,f}$ first principal stress at failure [kPa]
- $\sigma_{d,f}$ deviator stress at failure [kPa]

The relative stress ratio "$\sigma_{d}/\sigma_{1,f}$" indicates how close the material is to failure. At failure it reaches the value 1. Similarly, failure is reached when the deviator stress ratio $\sigma_d / \sigma_{d,f}$ reaches a value of 1.
moisture content is the average of the MC before and after the test, as specimens were tested immediately after preparation.

Figure 3: C and $\phi$ for gradings UL, CO, UN, AL, LL and FL, composed of 65% crushed concrete and 35% crushed masonry, crushed with a jaw crusher (J) and compacted to a degree of compaction of 100% from “virgin” specimens (0 days; these materials show some self cementation).

It should be mentioned that the $c$ and $\phi$ values shown in figure 3 were obtained on specimens that were used to determine the relationship between the stress conditions and the resilient modulus or that were used in repeated load triaxial tests to determine the resistance to permanent deformation (only specimens were used where the amount of permanent deformation was limited to 2% or less). Partly this was done to economize on the number of specimens that needed to be built but also to take into account the fact that in reality base course materials will always be subjected to some stress history before the asphalt layers are placed on top of them. Furthermore a comparison of the $c$ and $\phi$ values obtained on fresh samples with those that were obtained using the results from tests which had encountered some stress history, showed that the cohesion and angle of internal friction were hardly influenced by the stress history.
**Resilient modulus**

As shown in figure 4, the resilient modulus of unbound granular materials strongly depends on the state of stress. Equation 2 has been used to describe the stress dependency of the resilient modulus for unbound base and subbase materials (4).

\[ M_r = k_1 \left( \frac{\theta}{\sigma_0} \right)^{k_2} \]  

(2)

Where:

- \( M_r \) resilient modulus [MPa]
- \( \theta \) sum of the principal stresses [kPa]
- \( k_1 \) model parameter [MPa]
- \( k_2 \) model parameters [-]
- \( \sigma_0 \) reference stress of 1 [kPa]

**Permanent deformation**

Equation 3 (5) has been used to describe the stress dependent nature of the permanent deformation behavior. The equation has been used and validated for numbers of load repetitions larger than \( 10^6 \) (2, 3, 4, 5).

\[ \varepsilon_{perm} = A \left( \frac{N}{1000} \right)^b \left[ \exp \left( \frac{N.D}{1000} \right) - 1 \right] + C \left( \frac{N.D}{1000} \right) \]  

(3)

Where:

- \( \varepsilon_{perm} \) permanent strain [%]
- \( \sigma_{1,f} \) relative stress [-]
- \( N \) number of load repetitions [-]
- \( A \) model parameter [%]
- \( a_1 \) model parameter [%]
- \( a_2, b_1, b_2 \) model parameters [-]
- \( c_1 \) model parameter [%]
- \( c_2, d_1, d_2 \) model parameters [-]
Figure 4: $M_r$ - $\theta$ relations for UL-J-65 as a function of DOC (intended values: 97%, 100%, 103% and 105%) as determined 4 days after preparation of the samples.

Figure 5: $\sigma_1$-levels (for $\sigma_3=12$ kPa) at which $\varepsilon_p=1\%$, 5\% and 10\% at $N=10^6$, $10^6$ and $5 \times 10^4$ for gradations UL, AL and LL-65-J at DOC=97\%, 100\%, 103\% and 105\%.
3. **Models to predict the resistance to permanent deformation of unbound base and sub-base materials**

The question with respect to the prediction of the resistance to permanent deformation of unbound materials is whether one really wants to predict the development of permanent deformation with respect to the number of load repetitions or whether it is sufficient to set a threshold level for the stresses to be allowed. In a design system such as the South-African approach to mechanistic design (6), a choice is made for the latter. Also the results of the triaxial tests as performed at the Delft University indicate that setting a threshold to the allowable stresses could be an effective way to limit permanent deformation in unbound materials. Evidence for this is given in figure 6; this figure shows that the amount of permanent deformation can indeed be limited by allowing a certain ratio of applied vertical stress ($\sigma_1$) at a particular confining stress level over vertical stress at failure ($\sigma_{1,f}$) at that same level of confinement. This ratio depends of course on the amount of deformation to be allowed and the material characteristics. It should be noted that in order to limit the amount of testing, all repeated load permanent deformation tests were performed at a constant confining stress level of 12 kPa. This confinement level was selected because extensive finite element analyses using stress dependent resilient information for the base courses as determined in (2) as input, showed this stress level to be a reasonable value for base courses in pavement structures as used in the Netherlands. It should be noted that in the Netherlands the thickness of the asphalt surfacing is between 100 mm and 300 mm, and has a stiffness between the 5000 and 7000 MPa. The base thickness usually is between 150 and 300 mm (2).

In order to validate the approach of limiting the permanent deformation by setting a threshold value for the allowable stress level, finite element analyses were made on a large number of pavement structures; these structures are shown in figure 7. As can be

![Figure 6: Ratios of applied vertical stress over vertical stress at failure for mixtures of crushed concrete and masonry with different gradations and compacted to different degrees of compaction. All tests were performed at a confining stress of 12 kPa.](image)
seen in figure 7, the unbound granular base and sand subbase were modeled as stress dependent materials. Also Poisson’s ratio was taken as stress dependent. From these analyses relationships could be developed predicting the tensile strain at the bottom of the asphalt layer as well as the stress conditions in the unbound base course. An example of the results obtained is shown in figure 8. From the calculated stresses and the material characteristics, the development of permanent deformation in the unbound base could be calculated and design charts were developed. An example of such a chart is given in figure 9. The use of the charts is explained by means of an example. A 250 mm thick base course made of a mixture of 65% crushed concrete and 35% with gradation LL (LL-65%) is covered with a 40 mm thick asphalt layer that has a stiffness (Smix) of 3750 MPa. The pavement rests on a subgrade with a stiffness of 50 MPa and is loaded with an axle load (L) of 57.5 kN. Figure 8 shows that the $\sigma_d / \sigma_{df}$ ratio equals 0.671 in case the degree of compaction (DOC) is 103% and 0.626 when it is 100%. Figure 9 shows that the number of load repetitions to a rut depth (RD_SE) of 20 mm is $10^6$ in case the base course has a degree of compaction of 103% while this number is only 40,000 in case the degree of compaction is 100%.

Figure 8 shows that the degree of compaction of the base material does not really influence the stresses in the unbound base. Figure 9 however shows that the degree of compaction has an enormous effect on the permanent deformation development and that good quality compaction really pays off in terms of resistance to permanent deformation. Since all trucks don’t drive in exactly the same wheel path, also the effect of so called lateral wander was
evaluated. The figure however shows that lateral traffic wander has only a limited influence on the development of permanent deformation in the unbound base.

For the material with gradation LL, 35% crushed masonry and 65% crushed concrete the following transfer functions for limiting the permanent deformation were developed.

For a degree of compaction of 103 %: \( \log N = 0.331 - 32.029 \log \frac{\sigma_d}{\sigma_{d,f}} \)
For a degree of compaction of 100 %: \( \log N = 2.301 - 10.667 \log \frac{\sigma_d}{\sigma_{d,f}} \)

Where: \( N = \) number of load repetitions to 20 mm permanent deformation.

From these relationships one can derive that in order to allow \( 10^6 \) load repetitions, a deviator stress ratio of 0.61 can be allowed for a base with a degree of compaction of 103 % while this value is only 0.45 in case of a base compacted to 100 % relative density. From figure 9 one can derive that in case of a base with a relative degree of compaction of 103 %, approximately 40 mm of asphalt is needed to meet the stress ratio requirement. In case of a base with a relative degree of compaction of 100 %, approximately 80 mm would be needed. This result clearly shows the importance of achieving good compaction. Furthermore the relationships together with figure 7 show that permanent deformation of the unbound base is not really a problem if the asphalt top layer has a thickness of 150 mm and higher. In those cases the deviator stress ratio is so low that only minor deformations are to be expected.

![Figure 8: Example of a design chart to predict the tensile strain at the bottom of the asphalt layer (left vertical axis) and the deviator stress ratio in the base course (right vertical axis) for a particular gradation of a crushed concrete (65%) / crushed masonry (35%) mixture compacted to a degree of compaction of 97%, 100% and 103%.](image-url)
4. **Models to predict material characteristics**

In the previous section it has been shown that permanent deformation in unbound granular base courses can be limited or even overcome by setting threshold values for either the $\sigma_1 / \sigma_{1,f}$ or $\sigma_d / \sigma_{d,f}$ ratio. For practical reasons it would be very useful if the cohesion and angle of internal friction, could be predicted from parameters like gradation, angularity of the particles, degree of compaction etc. Therefore an analysis was performed on the triaxial data to find out whether such relations could be developed. The results of this analysis \((7, 8)\) are presented in this chapter. First of all however some background information on the development of the models will be given. Although parameters like e.g. cohesion and angle of internal friction are important parameters, they are not the most important ones. The most important parameter is the first principal stress the material can take at a given confinement level, degree of compaction, gradation etc. Another reason why relations to estimate the cohesion and angle of internal friction were not developed is that if equations with a good fit are developed to predict the cohesion and angle of internal friction separately, the estimate of the first principal stress at failure at a given confinement level by using the estimated $c$ and $\varphi$ could still deviate significantly from the real stress at failure. The approach therefore was to predict the vertical stress at failure given parameters like confinement stress, gradation etc. The equation that produced the lowest sums of squares of the differences between predicted and actual stress at failure value was selected. For the equations to predict the various parameters controlling the resilient modulus, a similar approach was adopted.

### Degree of compaction

Work done by van Niekerk \((2)\) has shown that the degree of compaction has a very strong influence on the mechanical characteristics of sands and unbound granular base materials. In order to take this parameter into account in the model development a so called compaction quality indicator ($qc$) has been defined. Equation 4 shows how $qc$ is calculated.

$$
qc = qc_1 \cdot \exp \left( - \left( \frac{rot}{DOC} \right)^{qc_2} \right)
$$

$$
rot = 101\% \cdot \ln(qc_1) \left( \frac{1}{1^{qc_2}} \right)
$$

Where:

- $qc$: compaction quality indicator [-]
- $DOC$: degree of compaction [%]
- $qc_1$: model parameter = 1.5 [-]
- $qc_2$: model parameter = 8 [-]
- $rot$: Point of rotation [%] (degree of compaction where $qc = qc_1/e = 0.552 [-]$)

Equation 4 gives $qc = 1$ at a degree of compaction of 101%. This is specified in the Dutch Standards for the compaction of unbound granular base course materials \((9)\). It was decided that the maximum value $qc$ could take is 1.5. This maximum value is called $qc_1$. Figure 10 is a graphical representation of equation 4.
Figure 9: Relationship between the $\sigma_d / \sigma_{df}$ ratio, the amount of lateral wander, the degree of compaction and the number of load repetitions to a permanent deformation in the base course of 20 mm.

**Strength**

To predict the strength characteristic of unbound granular base materials in relation to their gradation, compaction quality index and ratio amount of crushed masonry, the following equations were developed.

\[
c = c_6 \cdot qg \cdot qp \cdot qc^{c_7} \\
\phi = \phi_4 + \phi_5 \cdot qc \cdot qg
\]

Where:

- $c_6$ model parameter = 134.506 [kPa]
- $c_7$ model parameter = 2.2495 [-]
- $\phi_4$ model parameter = 30.27 [degr.]
- $\phi_5$ model parameter = 18.86 [degr.]
- $qp$: (0.4*percentage masonry+1.0*percentage concrete rubble)/100 [-]
- $qg$: grading quality [-],
  (UL=1 / FL=1 / CO=0.9 / AL=0.89 / LL=0.75 / UN=0.63, see also figure 1)
The fit between the first principal stress as predicted by means of equations 5 and 1 at failure and the measured one is given in figure 11 for two values of the confining stress being 0 kPa and 200 kPa.

Figure 10: Compaction quality indicator (qc) in relation to the degree of compaction (DOC).

Figure 11: Comparison between measured and predicted strength values for unbound granular base materials.
5. **Conclusions**

Based on the work presented in this paper, the following conclusions can be drawn.

1. For a given confining stress, permanent deformation in granular bases can be limited and controlled by setting threshold values for the ratio between vertical stress as applied and vertical stress at failure.
2. For the base course materials investigated, which were all made of mixtures of recycled crushed concrete and recycled crushed masonry, this ratio depends on the type of material, the gradation, the degree of compaction and the number of load repetitions to be allowed.
3. By means of the equations presented in this paper, it is possible to predict the first principal stress at failure at a given confinement level with a fairly high degree of accuracy for the unbound granular base materials tested in this research.
4. The relationships developed show a high dependency on material characteristics like gradation, angularity of the particles and degree of compaction.
5. The relationships indicate that performance related specifications for the materials as investigated can still be based on well known parameters which can easily be assessed in a short period of time and at low costs.
6. The relationships allow a quick analysis of the consequences (in terms of pavement life and costs) of using materials that are not completely in line with the specifications.

6. **References**

