Fracture and fatigue behaviour of a high strength limestone concrete as compared to gravel concrete

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In the Netherlands river gravel as traditional coarse aggregate for concrete is increasingly being replaced by other types of (natural) aggregate. After some more cracking was encountered during pile driving with limestone concrete as compared to gravel concrete, a comparative investigation into a number of material parameters was performed. For approximately equal compressive strength (85 to 90 MPa) differences in Young's modulus, fracture energy and brittleness were found.
With respect to the application of limestone concrete in slender concrete bridge girders, the fatigue behaviour was also studied. No significant differences were found between both concrete types, while the *S-N* curves as previously determined for concrete with normal strength appeared to apply also reasonably well to the investigated more brittle high strength concrete types.
Attention is also paid to the influence of a moisture gradient on the material properties as determined in fracture tests.

Keywords: **Limestone aggregate, high strength concrete, fracture mechanics, fatigue, moisture gradient**

1 Introduction

In the Netherlands river gravel is the type of coarse aggregate that has mainly been used in concrete in the past. Nowadays, for especially environmental reasons, the production of river gravel is being more and more limited and other types of coarse aggregate enter the scene.

It is known that the replacement of river gravel in concrete by another type of aggregate affects its material properties. However, when river gravel is replaced by a broken dense natural stone, in general the material properties of the concrete are not very strongly influenced. For that reason, apart from pointing at the influence of the type of aggregate on properties related to deformations, most design codes do not explicitly distinguish between different types of normal-weight aggregate in concrete (see among others EUROCODE No. 2 (1991)).

HERON, Vol. 40, No. 2 (1995) ISSN 0046-7316

Despite the fact that concrete material properties are not strongly affected by the type of normal-weight aggregate, there still may be some unexpected behaviour in certain circumstances.
An important example is the increase of damage (cracking) during pile driving with limestone concrete piles compared to traditional concrete piles. Because of these experiences a research programme in which a number of material properties of limestone concrete and gravel concrete were compared, was performed at TNO Building and Construction Research. In the investigation not only the traditional properties, like strength and stiffness, were determined. Attention was also paid to the softening behaviour of both types of concrete. The applied specimens were taken from existing foundation piles with an average compressive strength of about 85 to 90 MPa, which means that the concrete can be classified as a high strength concrete.

In the Netherlands limestone aggregate is also being used in relatively slender prefabricated concrete bridge girders. Since limestone concrete behaves more brittle than gravel concrete (Petersson (1981), Elices et al. (1992a)) and previous research (Van Leeuwen et al. (1979), CUR (1993)) showed a possibly increased sensitivity to fatigue loading for more brittle materials, it was decided also to compare the fatigue behaviour of both types of concrete. By using specimens from the same concrete piles the fatigue tests also contribute to the knowledge of the fatigue behaviour of high strength concrete.

2 Background information

2.1 *General*

Fracture mechanics of concrete is still a relatively young theory that most structural engineers are not yet familiar with. A short introduction will be given, followed by specific properties of limestone concrete as found in the literature. Characteristic results of fatigue behaviour of concrete are briefly reviewed. Finally, because it is assumed that a moisture gradient caused by the chosen curing conditions, influenced some test results significantly, the governing phenomenon is explained.

2.2 *Fracture mechanics of concrete*

Fracture mechanics of concrete, as first introduced by Hillerborg et al. (1976), gives the explanation for many phenomena, like bending strength and size-effect. New in the fracture mechanics approach for concrete as compared to the classical strength approach is the existence of a transition zone in between elastic material behaviour and the occurrence of a crack that can no longer transfer stress. Evidence for the existence of this zone can be given by the result of a deformation-controlled uniaxial tensile test (see Figure 1). Beyond the deformation at which the tensile strength is reached, stress does not directly fall back to zero, but gradually decreases with increasing deformation. This post-peak behaviour is the result of a local deterioration (also called "Fictitious crack") of the material and can be modelled by a stress-crack opening relation (see Hordijk (1991)).

The area under the stress-deformation relation in Figure 1 is the fracture energy, G_F (in J/m^2), and represents the energy required to create one unit area of crack. In order to define the brittleness of concrete, the parameter characteristic length l_{ch} (in mm) is used. Besides fracture energy also the

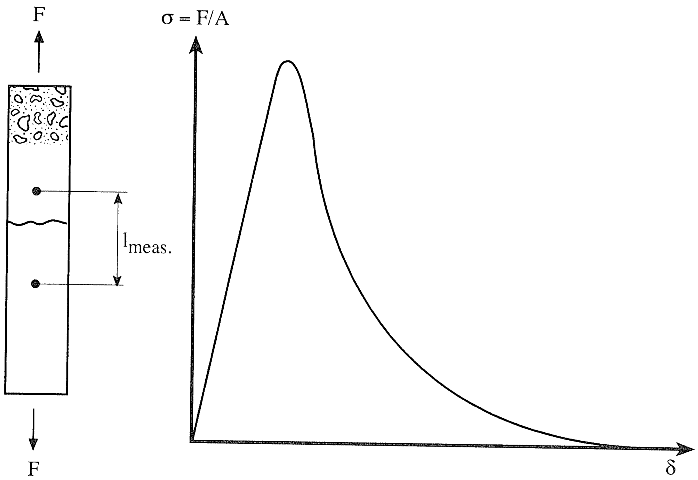


Fig. 1. Schematic representation of the behaviour of concrete in a deformation-controlled uniaxial tensile test.

parameters tensile strength and Young's modulus determine l_{ch}:

$$l_{ch} = EG_F/f_t^2 \qquad (1)$$

The smaller the value of l_{ch} is, the more brittle the concrete is.

2.3 *Limestone concrete*

Limestone is a term that is applied to a large and petrographically diverse group of sedimentary rocks (Smith et al. (1993)). As a result the properties of limestone aggregates may vary in a large range. The limestone aggregates that are usually applied for the production of concrete are of the hard, dense type.

The modulus of elasticity of dense limestone aggregates is larger than of quartzitic aggregates. In the CEB-FIP Model Code 90 (1993), for the ratio between the stiffness of concrete made with these aggregates a factor of 1.2 is given. For the bond strength between aggregate and mortar in literature a high value for limestone aggregate compared to other aggregates is reported (Hsu et al. (1963)). The explanation given for this result is that there is a chemical reaction between limestone aggregate and cement paste.

A comprehensive investigation into the impact strength of concrete is performed by Dahms (1968). In that investigation the impact strength is defined by the number of blows with a defined small amount of energy that a concrete specimen can endure before failing. Compared to concrete with broken quartzite aggregate a smaller number of blows could be resisted by concrete with basalt or limestone, respectively. The significant larger Young's modulus in combination with a significant larger Poisson's ratio for basalt and limestone aggregates are mentioned as possible cause for this result. However, it must be mentioned that in the same investigation concrete with river gravel (which is also predominantly quartzite) could resist an even smaller number of blows as the concrete with basalt or limestone. As possible explanation for this result the round and smooth surface of gravel is mentioned. So it can be concluded that significant differences in aggregate properties between limestone aggregate and gravel may have contributed to some different behaviour of concrete piles. A sound explanation, however, can not be given by the results found by Dahms (1968).

Elices et al. (1992a) report about an example of pile damage during driving and the analysis of it by using Fracture Mechanics concepts. From two almost similar types of concrete piles, one type of pile behaved well, while the other occasionally showed brittle behaviour. The main difference between these piles was the applied type of aggregate. Since classical approaches could not explain the observed behaviour, the fracture mechanics parameters fracture energy and brittleness (characteristic length) were determined. It was found that concrete of the piles that behaved well (natural rounded siliceous aggregates) was less brittle (higher values for G_F and for l_{ch}) than the other one (dolomitic aggregates).

2.4 *Fatigue behaviour of concrete*

After the introduction of concrete offshore platforms in the seventies interest into the fatigue behaviour of concrete increased considerably. In the Netherlands an extensive research programme was performed (CUR 1983, 1984, 1988, 1993). The strength of the applied concrete varied roughly between 30 MPa and 50 MPa. In compression a tendency for a shorter fatigue life in case of a higher compressive strength was reported by Van Leeuwen et al. (1979) and CUR (1993).

Fatigue behaviour of three different types of concrete (two normal-weight concretes and one light-weight concrete) with a cube compressive strength in the range of 70 to 85 MPa was investigated by Petkovic (1991). No significant differences were found between the Wöhler-curves for the investigated concrete types.

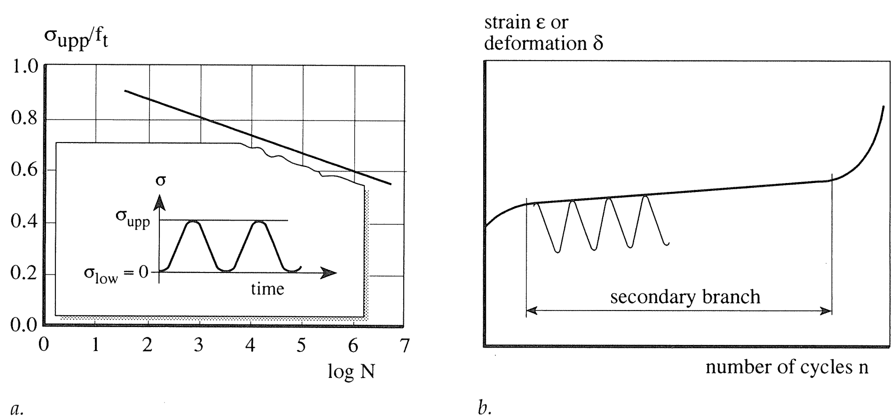


Fig. 2. Schematic representation of results that are found in fatigue tests on concrete: S-N diagram or Wöhler-diagram (a) and cyclic creep curve (b).

As far as typical results of fatigue tests are concerned, two relations are plotted in Figure 2. For the relative upper stress level of the applied load cycles as function of the logarithm of the number of cycles to failure, N, a more or less lineair relation is found (Figure 2a). Such a diagram is known as Wöhler-diagram or S-N diagram. When in a fatigue test on concrete the deformation at the upper stress level as function of number of cylces is plotted, then a curve as shown in Figure 2b will be found. The curve, known as cyclic creep curve, has a middle part in which the increase of deformation per load cycle is constant. There appears to be a strong relation between the slope of this secondary branche and the number of cylces to failure.

2.5 Moisture gradient

When a material property has to be determined in a fracture test, several influences may cause that a value different from the true one is found. For concrete, besides external factors, like incorrect boundary conditions, also the internal condition plays an important role. Due to the fact that a specimen dries out, eigenstresses are introduced in the specimen. The dry outer area of the cross-section wants to shorten, which is prevented by the wet inner area. In this case tensile stresses at the outside make equilibrium with compressive stresses in the middle of the specimen. Of course, when the drying period is long enough, the central part of the specimen will also dry, resulting in a constant moisture condition and no eigenstresses.

In a fracture test, like a uniaxial tensile or compressive test, it is intended to introduce a uniform stress distribution over the cross-section of the specimen. In that case the strength of the material is found by the applied maximum load divided by the cross-sectional area. The influence of eigenstresses will now be discussed by means of a tensile test (see Hordijk (1991)). When loading the specimen, in which eigenstresses are present, the outer area first reaches the tensile strength. With the softening behaviour of concrete, as shown in Figure 1, it means that by further loading the stresses in that area decrease while those in the inner area still increase. As a result, the maximum load found in the experiment will be smaller than the true value for the tensile strength times the cross-sectional area. In 5.2 some more attention will be paid to this phenomenon.

3 Experimental programme

3.1 General

The experimental programme consisted of two parts. In the first part a number of material properties under quasi-static loading conditions were determined (Hordijk (1993)), while in the second part the behaviour under fatigue loading was investigated (Hordijk (1994)). With regard to the difference in loading conditions in bridge girders under traffic loading and in concrete piles during pile driving, two different types of fatigue loading were applied in the experiments. These were, respectively, repeated compressive loading with compressive fracture ("compressive fatigue") and alternating tensile compressive loading with tensile fracture ("tensile fatigue").

3.2 *Quasi-static tests*

Compressive tests on prisms (250 mm × 100 mm × 100 mm) were used to determine the compressive strength and Young's modulus. For the tensile strength splitting tests on 100 mm cubes were applied. In order to determine fracture energy deformation-controlled four-point bending tests on notched specimens have been used (see Figure 3). All types of test were performed in the multiple of six.

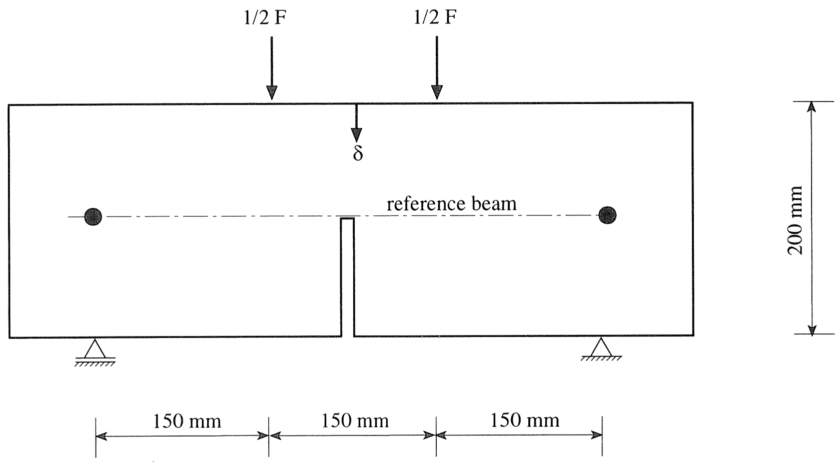


Fig. 3. Schematic view of the bending tests.

All the limestone concrete specimens were sawn out of one concrete pile with a cross-section of 450 mm × 450 mm. Similarly, all specimens with gravel concrete were also sawn out of one concrete pile. Before testing the end faces of the specimens were polished in order to ensure flatness and correct angles between sides. The age at testing of both applied concrete piles was about two months. With respect to the mix composition of the concrete piles the following information was supplied by the manufacturer. For both the limestone concrete and the gravel concrete the water-cement ratio was 0.4, the cement content (Portland C) was 380 kg/m^3 and the coarse aggregate/sand ratio (by volume) was 60/40. Furthermore a superplasticiser was used. The maximum aggregate size for the gravel concrete was 32 mm, while it was 20 mm for the limestone concrete.

For the curing conditions of the cubes and prims it was chosen to follow a Dutch Recommendation for the determination of compressive strength of existing concrete structures (CUR 1990) and to cure the specimens for 48 hours under water. As becomes clear when the results are presented, this curing procedure significantly affected the test results for the prisms. Therefore two additional 100 mm cubes were sawn for each concrete type. These cubes were prior to testing stored in the laboratory (±50% RH and ±20°C).

In the four-point bending tests the deflection was measured with LVDTs (linear variable differential transducers) at the front and rear side of the specimen. The average signal was used as control parameter, while the chosen deflection rate was 0.04 mm per minute.

3.3 *Fatigue tests*

For the fatigue tests specimens with the dimensions 250 mm × 100 mm × 100 mm were sawn out of the same concrete piles as used in the quasi-static tests. In order to apply the load in the compressive fatigue tests as centric as possible great care was given to obtain a correct shape of the prisms. Therefore in these tests all six sides of the specimens were polished. After preparing the specimens they were stored in the laboratory (±50% RH and ±20°C) till testing.

Both for the compressive fatigue tests and the tensile fatigue tests a 1000 kN loading equipment was used. In the compressive tests the specimen was placed on a fixed lower platen, while the load was applied on top of it via a ball bearing and an upper platen. In the tensile fatigue tests the specimens were glued between fixed upper and lower platen. For the connections between, for instance, the actuator and load cell prestressed swivel heads were applied. In order to prevent the specimens in the tensile fatigue tests from failing at the glue surface, two saw cuts with a depth of 10 mm reduced the middle cross-section to 100 mm × 80 mm. This procedure is often applied in tensile tests on concrete (Hordijk 1991) and previous research (CUR 1988) showed that fatigue results are not influenced by these saw cuts.

In the experiments the deformation of the specimens over the total length of 250 mm was measured with four LVDT's. By placing the LVDT's at the four sides the average signal represents the average deformation of the specimen, while separate readings were used to verify whether the load is applied centric. Since the increase in deformation with the number of cycles is very small it was necessary to measure micrometres accurately. Because, furthermore, a number of the fatigue tests last for a long period, in the trial tests great attention was paid to the influence temperature changes on the deformation measurements. It appeared that by keeping the conditions in the large laboratory hall as constant as possible, temperature influences could be prevented.

A data-acquisition system was specially developed to record the data. In order to limit the amount of data, two criteria, $\Delta\delta$ and Δn can be set before testing (see Figure 4). The readings are registered when the deformation of one of the four LVDTs increased more than $\Delta\delta$ or when after the last registration more than Δn cycles passed. Furthermore it is possible to determine a complete load-deformation relation for a cycle.

In order to determine the reference strength first three specimens were loaded till failure in a load-controlled manner. The applied loading rate in the compression tests was 15 kN/s (=1.5 MPa/s) and in the tensile test 0.5 kN/s (= 0.0625 MPa/s). In accordance with previous fatigue experiments (CUR 1988,1993) the applied frequency was 6 Hz. In order to prevent the first loading to be an impact loading, the specimen was gradually loaded till the required average load level before starting the experiment. Then the fatigue test was started with a zero amplitude and subsequently the amplitude was quickly increased manually. In practice this means that between 20 to 80 cycles had passed before the correct upper and lower load level was reached. This has been taken into account in the analyses of the test results.

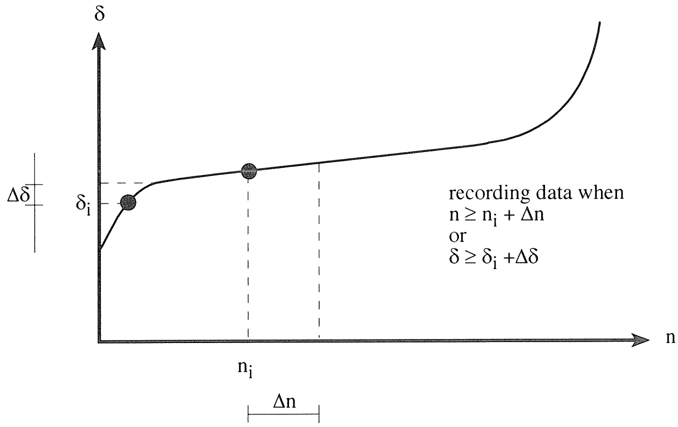


Fig. 4. Two criteria for recording data in the fatigue tests.

In order to determine a *S-N* relation three different upper stress levels were applied in the fatigue tests, while the lower stress level was kept constant. In the repeated compressive tests the applied relative upper stress level was, respectively, 0.80, 0.70 and 0.65. For the lower stress level a value near zero was chosen. Except for some first tests in which the lower stress level was $0.025f_{cm}$ (f_{cm} is the average compressive strength found in the three quasi-static tests) for most specimens a value of $0.05f_{cm}$ was taken. Since the effect of this small difference in lower stress level will be much smaller than the scatter, no distinction will be made between these tests in the analyses. The upper stress level in the tensile-compressive fatigue tests was, respectively 0.75, 0.60 and 0.45 times the static tensile strength (f_{tm}). Note that in the compressive fatigue tests the highest compressive stress is denoted as upper stress level, while it is in the tensile fatigue tests the highest tensile stress. With regard to the stresses in a concrete pile during hammering, for the lower stress level in the tensile fatigue experiments the absolute value equal to approximately five times the tensile stress at the upper load level was chosen. However, a constant value, equal to about $0.30f_{cm}$, was taken for the three different upper stress levels.

4 Results

4.1 *Quasi-static tests*

From the compression tests on prisms, cube splitting tests and four-point bending tests, the results, as presented in Table 1, were obtained.

Strength values

If for the ratio between the compressive strength found in the prisms and the cube compressive strength a factor of 0.85 is assumed, then the obtained strength yielded a cube compressive strength of approximately 73 MPa. It was known from other tests on similar concrete piles that the real cube compressive strength was significantly higher. Since the applied curing condition was held responsible for the low value for each concrete, two additional compressive tests on cubes, not stored in water, were performed. These tests yielded on average a compressive strength of 85 MPa and 91 MPa for respectively the gravel concrete and the limestone concrete. As far as the tensile splitting tests are concerned, it is not known by now if, and to what extent, these results were influenced by a moisture gradient. Despite all this, when it is assumed that the phenomenon of a moisture gradient works out for both concrete types in a similar way, the compressive and tensile splitting strength was approximately equal for both concrete types. In 5.2 the phenomenon of a stress gradient will further be discussed.

Table 1. Experimental results found in the quasi-static tests (Hordijk 1993).

	river gravel concrete A	limestone concrete B	ratio B/A
density (kg/m^3)	2379 (0.7)[1]	2428 (0.5)	1.02
compressive strength[2] (MPa)	61.6[2] (4.8)	62.8[2] (2.2)	1.02
cube splitting strength (MPa)	4.98 (10.4)	5.17 (9.9)	1.04
Young's modulus (MPa)	37300 (3.6)	47000 (6.8)	1.26
Fracture energy (J/m^2)	135	85	0.63
characteristic length (mm)	203	149	0.73

[1] Coefficient of variation between brackets.

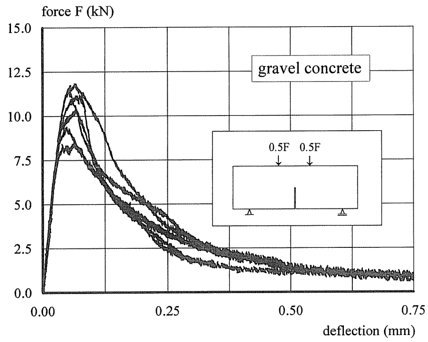
[2] The obtained compressive strength is measured on prisms (250 mm × 100 mm × 100 mm) and almost for sure to small due to the influence of eigenstresses resulting from a moisture gradient.

Young's modulus

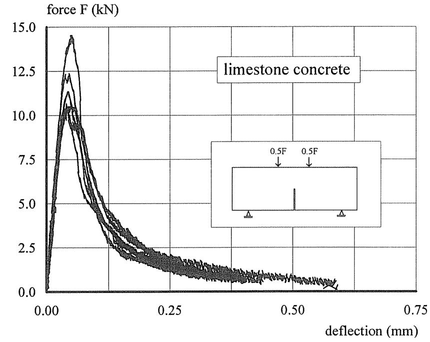
The modulus of elasticity of the investigated limestone concrete is on average 26% larger than that of the river gravel concrete.

Fracture energy and characteristic length

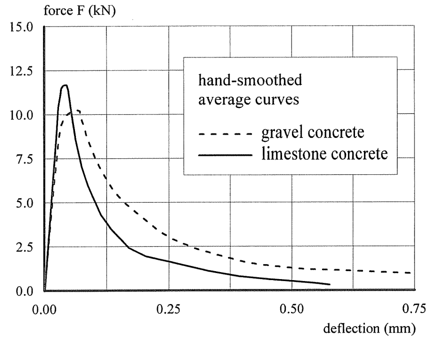
Load-deflection relations as found in the deformation-controlled four-point bending tests on notched specimens are shown in Figures 5a and 5b for, respectively, gravel concrete and limestone concrete. There appears to be a significant difference between both types of concrete, which also can be seen from Figure 5c were hand-smoothed average curves are compared. The descending branch for the limestone concrete is situated below that of the gravel concrete. As a result of the low force in the tail of the descending branch for limestone concrete it was not possible to continue the tests beyond a deflection of about 0.6 mm. At that point the self-weight of the beam and the loading platens together was almost equal to the load that still could be taken up by the beam. In that case the servo-control system was no longer capable to control the deformation and the tests were stopped.



a.



b.



c.

Fig. 5. Load-deformation relations from four-point bending tests on gravel concrete (a), limestone concrete (b) and hand-smoothed average curves (c).

As far as determination of the real value for the fracture energy is concerned, it is known that it can be accompanied with several errors (Elices et al. 1992b). In this investigation, however, it was mainly the objective to compare both types of concrete. In order to determine the fracture energy the total energy supplied to the specimen must be divided by the fractured area. For determining the fracture energy all curves have been extrapolated till a deformation of 0.75 mm, while furthermore account was given to the fact that the location of load application does not coincide with the location where the deflection was measured (Hordijk 1993). By this procedure a value of 135 J/m^2 and 85 J/m^2, was determined for respectively the gravel concrete and limestone concrete. By using the average values in table 1 for the tensile (splitting) strength and the Young's modulus a characteristic length l_{ch} ($l_{ch} = EG_F/f_t^2$) of respectively 203 mm and 149 mm was calculated. So, according to the definition in the fracture mechanics theory, the investigated limestone concrete behaved more brittle than the gravel concrete.

4.2 *Compressive fatigue tests*

The three stress-deformation relations for both concrete types obtained in the quasi-static reference tests, are given in Figure 6. The compressive strength was 78.2 MPa (v.c. = 2.9%) for the gravel concrete and 73.1 MPa (v.c. = 3.2%) for the limestone concrete.

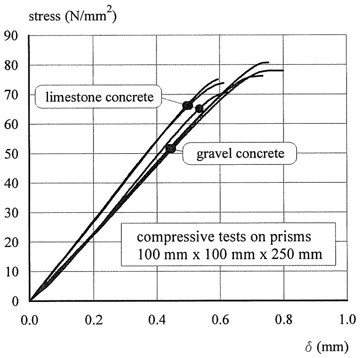


Fig. 6. Stress-deformation relations for compression; Reference tests for compression fatigue.

In the compressive fatigue tests three experiments on gravel concrete showed a divergent result. Though it could not clearly be proved, it is expected that a moisture gradient may have also played a role in this case (Hordijk (1994)). Anyhow there was enough reason to exclude these results from the analyses.

The results of the fatigue tests for both concrete types, presented in a Wöhler-diagram, are given in Figure 7. From the separate data-points as well as from the linear regression curves it can be seen that there is no significant difference between both types of concrete.

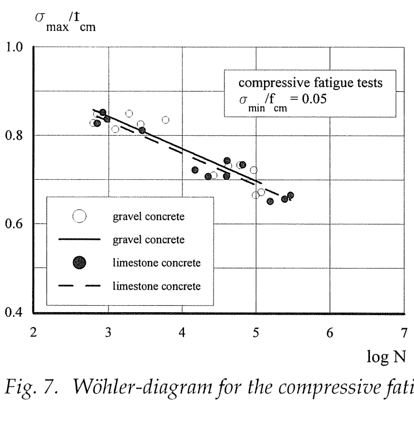


Fig. 7. Wöhler-diagram for the compressive fatigue tests.

A characteristic result showing the development of the deformation at the upper stress level with the number of cycles is shown in Figure 8. Such a curve is called cyclic creep curve. From previous research (CUR (1988)) it is known that there is an approximately linear relation between the increase of the strain in the secondary branch (secondary creep rate) and the number of cycles to failure *N*, when both plotted on a logarithmic scale. In Figure 9 it can be seen that the fatigue experiments in this investigation showed a similar relation, while it furthermore is equal for limestone concrete and gravel concrete.

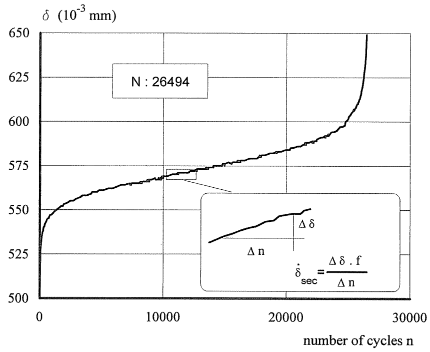


Fig. 8. Example of the relation between deformation and number of cycles to failure.

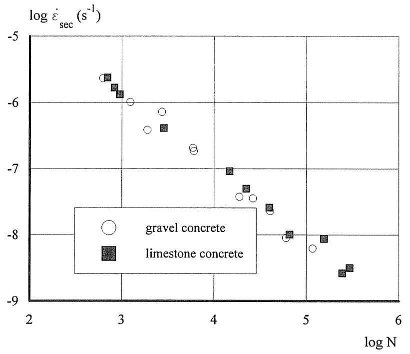
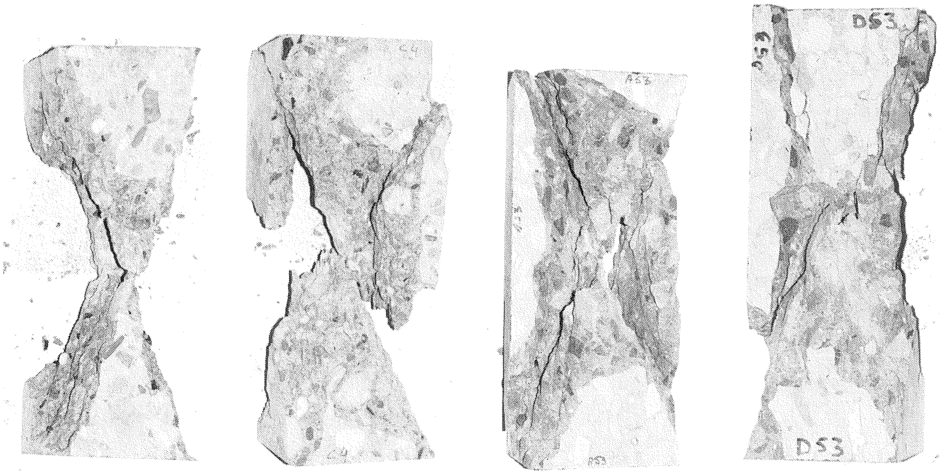


Fig. 9. Relation between secondary creep rate and number of cycles to failure in compression fatigue tests.

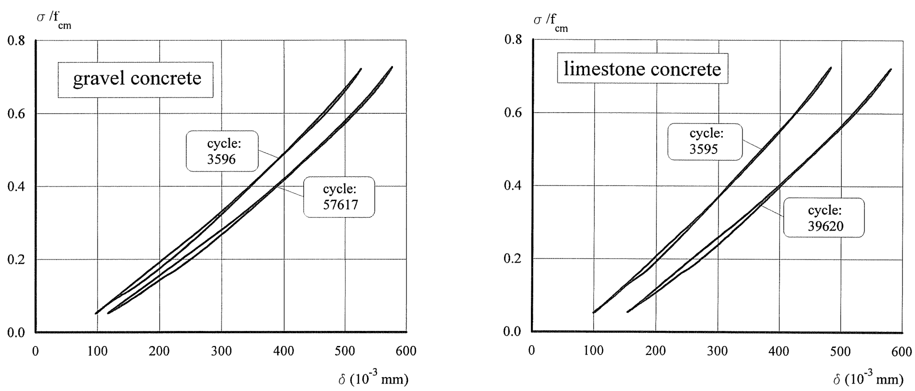
Failure of the specimens occurred in an explosive way. Therefore a protection jacket was placed around the specimen in order to prevent pieces of concrete to be swept away. From the concrete pieces that remained after failure the typical shape as usually found in compressive failure could be distinguished in most tests (Figure 10). It was not possible to observe a general difference between the failure patterns in both concrete types. Cracks ran through the aggregates for both concrete types.



a. *b.* *c.* *d.*

Fig. 10. Photo's of rest pieces of the concrete specimens after failing in concrete compression (a,b: gravel concrete; c,d: limestone concrete).

In order to illustrate the measured load-deformation relation in load cycles two typical loops are shown in Figures 11a and 11b for respectively gravel concrete and limestone concrete. The plotted loops belong to respectively a load cycle in the beginning and at the end of the fatigue life. As can be seen, the ascending and descending part of the loops almost coincide for both concrete types.



a. *b.*

Fig. 11 Measured load-deformation relations for load cycles.

4.3 *Tensile fatigue tests*

Contrary to the presentation of the results of the compressive fatigue tests in 4.2 the results of the tensile fatigue tests will be directly shown in relation to previous obtained results. The data-points for limestone concrete and gravel concrete in the Wöhler-diagram (see Figure 12) display a large scatter. The solid line in Figure 12 represents the average relation obtained with 184 experiments on gravel concrete with a cube compressive strength of about 45 MPa (CUR (1988)). The relation can be described by:

$$\log N = 8.94 - 7.68\ \sigma_{max}/f_{tm} - 0.37\ \sigma_{min}/f_{cm} \tag{2}$$

and belongs to alternating tensile-compressive fatigue loading with tensile fracture, dry concrete and a frequency of 6 Hz, which corresponds with the circumstances in the present investigation. A large scatter in tensile fatigue tests was also found in previous investigations as can be seen by the 90% confidence limit ($\log N \pm 1.36$), plotted in Figure 12 by the dashed lines.

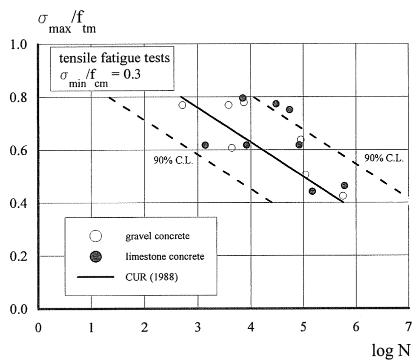


Fig. 12. Wöhler-diagram for the tensile fatigue tests and results as previously found for normal strength concrete.

Typical results for the cyclic creep curves are presented in Figure 13. In CUR (1988) the relation between the logarithm of the strain rate in the secondary branche ($\log\dot{\varepsilon}_{sec}$) en the logarithm of the number of cycles to failure ($\log N$) was determined to be equal to

$$\log N = -3.25 - 0.89\ \log\dot{\varepsilon}_{sec} \quad (\log\dot{\varepsilon}_{sec} \text{ in } 1/s) \tag{3}$$

with a 90% confidence limit equal to $\log(N) \pm 0.40$. In Hordijk (1991) it is shown that the increase in strain in a tensile fatigue test is mainly due to a very local increase in deformation. As a result a comparison of the cyclic strain rate with previous results will only be possible if the same measuring length is applied. In order to eliminate this problem it is better to rewrite equation 3 in a relation between log N and log $\dot{\delta}_{sec}$. This is also more in agreement with the physical behaviour. Since the measuring length on which equation 3 is based, was equal to 300 mm, the new relation becomes:

$$\log N = -1.05 - 0.89 \ \log \dot{\delta}_{sec} \qquad (\log \dot{\delta}_{sec} \text{ in mm/s}) \tag{4}$$

The data-points that have been found in the present investigation are together with relation 4 plotted in Figure 14. In general, it can be seen that the results found in this investigation on high strength concrete fit rather well in those found previously for normal strength concrete.

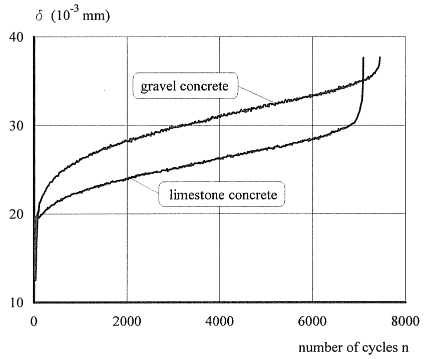


Fig. 13. Examples for the development of deformation with number of cycles in tensile fatigue tests.

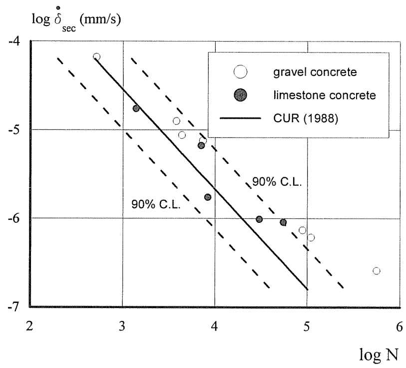


Fig. 14. Relation between deformation rate in the secondary branch and number of cycles to failure.

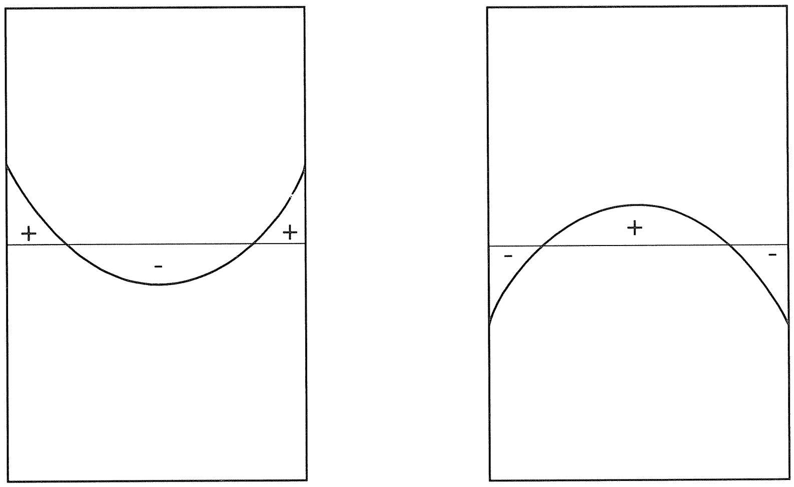
5 Discussion

5.1 *General*

In Chapter 4 experimental results have already partly been compared with results found in the literature. In this chapter several aspects of the obtained results will be further discussed. Results obtained for the investigated high strength concrete will be compared with results for normal strength concrete as previously found. As its the author's opinion that the phenomenon of a moisture gradient does not get enough attention in many research projects, some attention will be paid to it.

5.2 *Moisture gradient*

For a number of experimental results, as found in the described investigation, it is believed that a moisture gradient has had an undesired significant influence. In 2.5 the phenomenon is explained for a drying specimen. In that case tensile eigenstresses in the outer part of a cross-section are in equilibrium with the compressive stresses in the inner part of the cross-section (see Figure 15a). When a specimen is taken from an existing structure and stored under water for two days, an opposite stress distribution occurs. The outer area wants to swell, which is prevented by the dry inner area, causing compressive stresses (see Figure 15b).



eigenstresses due to drying

a.

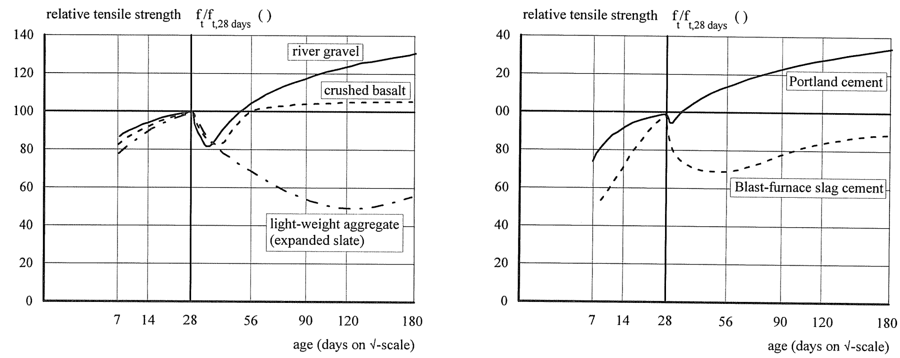
eigenstresses due to placement of a dry specimen under water

b.

Fig. 15. Schematic representation of eigenstresses due to differential drying (a) and wetting of an already dried specimen (b).

According to the literature the tensile splitting strength is less affected by drying then the uniaxial tensile and flexural strength (see Hordijk (1991)). The explanation for it may be that the biaxial compressive region under load application in a tensile splitting test coincides with the region of tensile eigenstresses due to differential shrinkage. For a dry specimen that is placed under water, where the tensile eigenstresses are in the inner part, this would mean that the tensile splitting strength is more affected. As stated before, it is not known to what extent the results obtained in this investigation suffered the consequences of a moisture gradient.

So far, the phenomenon has mainly been discussed qualitatively. Though it can not be proved, it is believed that in the compression tests on prisms that were placed under water for two days, the obtained strength is at least 10 MPa too low. No results were found in the literature for compression tests on prisms that were cured in a similar way. For drying specimens compression tests on cubes by Bonzel (1970) or prisms and cylinders by Fouré (1985) did not show the influence of a moisture gradient. Both publications clearly show the phenomenon for tensile tests. Bonzel (1970) cured specimens for 28 days under water and subsequently dried them in the air. The development of strength was determined by testing specimens at several ages. Results for three types of aggregate are plotted in Figure 16a. As can be seen, a strength reduction for concrete with natural aggregate up to 20% was found, while for a light-weight aggregate concrete the reduction can even be 50%. The explanation for the fact that light-weight concrete is much more sensitive for the phenomenon is that the porous aggregates act as a moisture buffer, resulting in a more pronounced differential shrinkage. Bonzel (1970) also found an influence of the type of cement (see Figure 16b).



a. *b.*

Fig. 16. Influence of the curing condition on the tensile strength as found in a unixial tensile test for different types of aggregate (a) and different types of cement (b) (Bonzel (1970)).

The results presented from literature (Figure 16) clearly demonstrate the importance of the phenomenon of differential drying shrinkage for strength tests. In many investigations great care is paid to accurately measuring the force. It may be obvious now that by choosing incorrect curing conditions the error can be much greater. The phenomenon for drying specimens and tensile tests has been investigated thoroughly (Bonzel (1970), Fouré (1985)). Further research on the influence of it on other strength tests and for dry specimens that are placed under water for some time, is required. Furthermore, it may be clear that the phenomenon is not only active in laboratory experiments. Eigenstresses due to differential shrinkage will also be active in real structures and can have consequences for the capacity of it. Further research on this aspect, where advanced numerical tools can be very helpful, is required.

5.3 *Fracture mechanics parameters of limestone concrete*

The earlier observed relative brittle behaviour of limestone concrete as compared to gravel concrete was also found in this investigation. The results showed great similarity with those obtained by Elices et al. (1992a) for concrete with gravel and dolomitic aggregates (also a type of limestone aggregate). Though Petersson (1981) also observed a more brittle behaviour for concrete with crushed limestone, for a number of properties different results were found. Besides a strong reduction in fracture energy, Petersson found an increase in tensile strength and a decrease in Young's modulus, which suggests that the investigated type of limestone was not of the hard dense type.

Though differences are found for fracture mechanics parameters of limestone concrete as compared to gravel concrete, this does not mean that a distinction has to be made between these concrete types for all structural analyses. Experiences with limestone concrete for many years in many countries are the evidence for this. Nevertheless, the difference in brittleness in combination with the increased stiffness (for a blow with a certain energy the stresses in the concrete pile are higher for a higher stiffness) may very well be the explanation for the observed behaviour of limestone concrete piles during hammering under certain circumstances. In that respect it must be realized that concrete piles have to endure very severe loading during hammering. A small change in material behaviour may cause fracture for one type of concrete while for the traditional concrete type no problems were encountered.

5.4 *Fatigue behaviour of high strength concrete*

For the tensile fatigue behaviour the results for the investigated high strength concrete are in 4.3 already compared with those previously found for normal strength concrete. Taking into account the fact that tensile fatigue tests are accompanied with a large scatter, it can be concluded that no significant differences were found between both types of concrete (see Figure 12). Furthermore, the Wöhler-curve for normal strength concrete (CUR 1988) appeared to apply reasonably well for the investigated high strength concrete.

In previous compressive fatigue tests in the Netherlands (CUR (1983), CUR (1993)) most specimens were cured and tested under water. For a limited number of tests on dry specimens (cube compressive strength is 45 MPa; 6 Hz) the following Wöhler-curve was determined:

$$\log N = [-17.0/(1-R)^{0.5}]\log(\sigma_{max}/f_{cm}) + 2.0 \tag{5}$$

with $R = \sigma_{min} / \sigma_{max}$

The Wöhler-diagram in Figure 17 shows that the data-points for the investigated high strength concrete do not differ significantly from this relation. In Figure 18 the Wöhler-curve for the high strength concrete, determined by linear regression on all the data-points (limestone concrete and gravel concrete) is compared with results for normal strength concrete as found in an extensive investigation by Holmen (1979) and results for high strength concrete as found by Petkovic (1991).

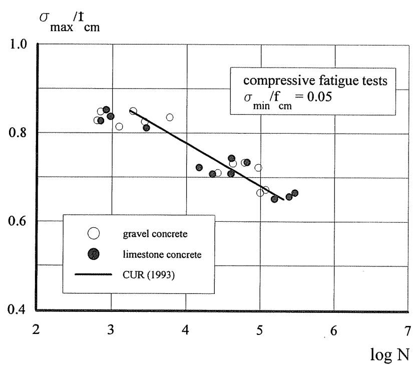
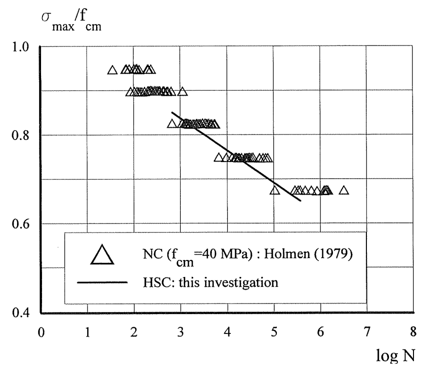
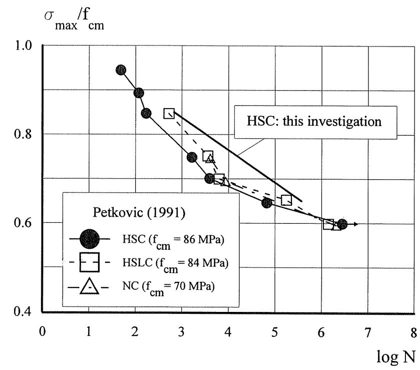


Fig. 17. Wöhler-diagram (compression) for the investigated high strength concrete compared with relation 5 for concrete with a cube compressive strength of 45 MPa.



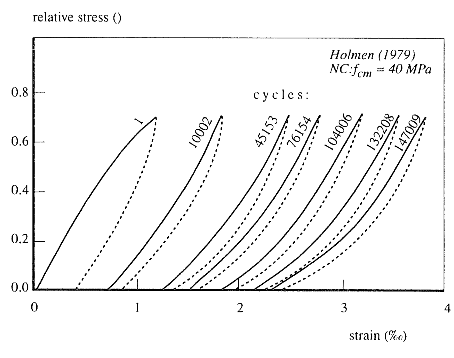
a.



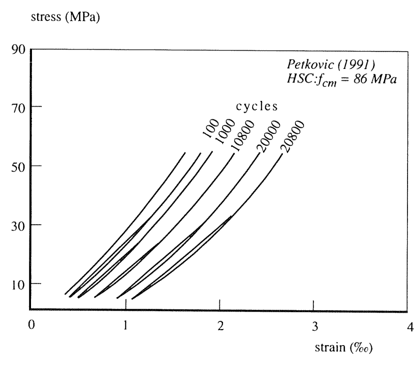
b.

Fig. 18. Comparison of the average Wöhler-curve (compression) for the investigated high strength concrete types with results for normal strength concrete, 40 MPa (Holmen 1979) (a) and high strength concrete found by Petkovic (1991) (b).

As far as the development of strain with the number of cycles is concerned, Petkovic (1991) reports the secondary part of the curve (see Figure 13) to be less steep and longer for high strength concrete. Also for the stress-strain relation during a load-cycle a difference between high strength and normal strength concrete is found (see Figure 19). As can be seen, the load-deformation relation for high strength concrete as found by Petkovic (1991) shows good agreement with those found in the present investigation (compare Figures 11 and 19b).



a.



b.

Fig. 19. Stress-strain relations for load cycles as reported for normal strength concrete by Holmen (1979) (a) and high strength concrete by Petkovic (1991)(b).

6 Concluding remarks

From the investigation into the behaviour of a high strength limestone concrete as compared to the behaviour of gravel concrete with a approximately similar strength, the following conclusions can be drawn:

- The limestone concrete behaves more brittle according to the definition in the fracture mechanics approach than the gravel concrete. The increased brittleness for limestone concrete in combination with a higher stiffness may be the explanation for an observed increase in cracking of concrete piles during hammering. Also the larger Poisson's ratio for limestone aggregate as reported by Dahms (1969) may have contributed.
- Curing conditions of specimens for strength tests can have a great influence on the strength that will be found. A moisture gradient due to drying of a wet specimen or due to wetting of a dry specimen when it is placed under water, causes eigenstresses in the specimen which reduces the capacity of the specimen, resulting in a virtual lower strength. Further research on the phenomenon of a moisture gradient for laboratory tests as well as for real structures is recommended.
- No significant differences were found between the fatigue behaviour of the gravel concrete and the limestone concrete investigated.
- For compression fatigue as well as for tension fatigue it appeared that Wöhler-diagrams as previously found for normal strength concrete apply also reasonably well for the investigated high strength concrete types.

Acknowledgement

The support given by B.V. LODEWIKUS VOORGESPANNEN BETON is greatly acknowledged.

Notations

A	cross-sectional area
$C.L.$	confidence limit
E	Young's modulus
F	force
f	frequency
f_t	tensile strength
$f_{t,28\ days}$	tensile strength at 28 days
f_{tm}	mean tensile strength
f_{cm}	mean compressive strength
G_F	fracture energy
HSC	high strength concrete
HSLC	high strength light-weight concrete
l_{ch}	characteristic length
$l_{meas.}$	measuring length

n	number of cycles
N	Number of cycles to failure
NC	normal-weight concrete
$v.c.$	coefficient of variation
σ	stress
σ_{min}	minimum stress
σ_{max}	maximum stress
σ_{upp}	upper stress level
σ_{low}	lower stress level
ε	strain
$\dot{\varepsilon}_{sec}$	cyclic strain rate in secondary branch
δ	deformation or deflection
$\dot{\delta}_{sec}$	cyclic deformation rate in secondary branch
$\delta_{av.}$	average deformation

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