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Poland
Research on building structures and building physics

Proceedings of an interuniversity research seminar

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Proceedings of the second interuniversity seminar on research on building structures and building physics, held at Eindhoven, The Netherlands.

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ABOUT THE COVER PAGE:

INTERMEZZO

The name of this contribution is “Intermezzo” because it deals with something between structural engineering and art.

On the occasion of the faculty’s 25th anniversary which took place last September, we designed an installation and called it: a “monumental temporary festivity contraption”, supervised by the Dutch artist ANTOON VERSTEEGDE.

This installation was ment as a gift from our faculty to the entire community of the university on the occasion of our lustrum.

We designed, and with “we” I mean in the first place our commission for special activities Mr. Wim de Hoop, Mr. Peter Bergman and myself, together with six students: Wolfgang Dobber, Martin Heylman, Didier Hendriks, Paul van Hoek, Richard Schwarzenberg and Marco Welhuis. This team together with Antoon Versteegde, made the design and all the preparations. In the second place I will name the 240 first-year students who built the installation in one day. The design aims were as follows: the installation should be built with bamboo sticks, elastics and rope for the connections and only built by manpower, no machines at all. The installation should cover an area of four acres and reach into the sky as high as possible. Eventually our tower was “only” 30 metres high.

In the film we saw that the design has three parts:

(1) THE TOWER: inspired by the geometry of the towers built by the Russian engineer V.G. Suchov, who lived from 1853 to 1939.

Our design was forty metres high. Imagine, only bamboosticks, elastics and rope for the connections and no machines at all to raise the tower.

(2) A large circle of twenty-five “flagtrees”; for each year of our existence one tree, with each ten flags, up to ten metres high.

(3) Four hundred and fifty so-called “spectators”. Small figures only three metres high and all facing the tower.

One can say:
- the tower symbolizes the faculty’s future;
- the “flagtrees” the 25th anniversary and
- the spectators our future engineers or just spectators.

For this all we used: 6.000 bamboo sticks
15.000 elastics
2.000 m. rope
1.600 m. tape
and 250 flags

Eindhoven, november 1992
ir. J.F.G. Janssen
Preface.

Having a seminar for the second time between two universities almost establishes a tradition. This second inter university seminar "research on building structures and building physics" was held at Eindhoven university from 18th till 20th November 1992. It was a continuation of the seminar entitled: "new ways in building structures and building physics" which was held at Wroclaw in November 1990. During the two years between these two seminars a number of contacts between the two universities took place. Worth mentioning are the working visits to Eindhoven of Mr Stys and Szechinski, the visit of Polish students last September, and Dutch assistant professors to Wroclaw.

The seminar was lively frequented, mostly more than 45 people participated. In five sessions 24 papers were presented. The main topics of these sessions concerned numerical and experimental research on concrete, material behaviour of concrete and masonry, restoration and maintenance, research on steel structures etc.

The papers presented during the seminar are included in these proceedings in the same order as they were presented. Also a number of papers related to the subjects as mentioned above are printed, without being presented.

A word of thanks to the chairmen of the sessions, the speakers and the writers of the papers.

Not only in seminar sessions the work of EUT was presented. Visits to laboratories and the building site of the Alco Dome completed the technical part of the Polish visit.

In visits to the PSV football stadium, Rotterdam harbour and a building with a structural design by D3BN at Rotterdam we tried to show something more about the way we build in the Netherlands, and being there a visit to the windmills of Kinderdijk in the sundown is almost a must. Enjoying music in our new music centre combines experiencing a new building and relaxation on the tunes of Mozart, Britten and Haydn.

Having a tradition also means looking into the future. One of the outcomes of the meeting about this future was that new projects should be formulated on new topics, so the strong points of both universities can be used in an optimum way. Appointments have been made to have contacts on an interhuman basis in the beginning of next year.

The covering page shows the festivity contraption made on the occasion of the 25th anniversary of our department.

This contraption can be the symbol of such new projects and many fruitful seminars to come.

mrs. M. Alblas
ir. J.F.G. Janssen
ir. H.L. Schellen
ir. A.Th. Vermeltfoort
3 December 1992
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Numerical analysis of concrete behaviour
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RETURN OF THE LINEAR STRESS METHOD
IN REINFORCED CONCRETE CALCULATION THEORY

1. Introduction
With a team of researchers, we are doing experimental and theoretical work on the foundations of reinforced concrete strain calculations, taking into account rheology and concrete cracking. The research has resulted in 12 doctoral theses. Our results belong in the 21st century. We are in a position to express a view on the direction in which the currently recognized theory of concrete is developing. In our opinion the current trends in the development of concrete theory are incorrect. At present a nonlinear model of concrete and concrete elements is adopted. We predict a return to the linear theory of concrete elements. The return will accommodate recent progress in the theory of cracks and concrete rheology. This view may be surprising to many, as the previously developed linear theory of concrete has been largely abandoned. The nonlinear theory, resisted at first, is now gaining wider and wider currency and is accepted in international standards. We anticipate a large degree of psychological resistance to abandoning the current trend to develop the nonlinear theory and focusing on the linear model of concrete.

2. Basic assumptions of the linear model of concrete
The behaviour of concrete strain in the process of loading is observed during the imposed course of strains and the simultaneous
measurement of the currently acting force. If strain is imposed with a constant velocity and unloading proceeds in the same way, we observe a linear strain-load relationship during unloading and reloading. Some strain remains, which we shall refer to as residual strain. In the one-dimensional problem a superposition of elastic concrete strain and nonelastic strain can be observed. In the case of elastic strain the modulus of elasticity does not depend on the load level. Residual strain is dependent on load level and the dependence is nonlinear. Loading with imposed strain was applied in the working load range, at critical load and at the post-critical state. Fig. 1 presents the dependence of displacement in the middle of the span of a simply supported beam during multiple loading and unloading in a wide range of loads at the post-critical state. Let us consider the special case of unloading and reloading in the post-critical range. One has to keep in mind the residual deflection, which depends on the loading history. The beam displacements, elastic. Residual deflection is superimposed on the elastic displacement of the beam. The current, nonlinear theory of concrete elements assumes a nonlinear stress distribution in the compression zone of the concrete at the critical and post-critical states. It follows from the above experiment that during unloading, we obtain a linear change of stress in the compression zone of the concrete. A nonlinear stress distribution may be locked up in the form of a self-balanced stress system in the section under consideration. This self-balanced system will be called self-stress. Self-stress does not take part in the balancing of external load. At the limit state, a linear concrete stress distribution can be assumed for the balancing of external load, so a nonlinear distribution of compression in concrete is assumed incorrectly. The limit state is
obtained by the yielding of reinforcement, the amount of which reinforcement has to be assumed to be less than when failure occurs due to concrete failure before the reinforcement yields.

The above considerations should suffice as a justification for the need to use a linear theory of concrete and a demonstration of the obvious inconsistence of the current nonlinear theory with experimental results. Below we shall present some considerations which - if the linear theory of concrete is assumed - will enable us to explain certain familiar phenomena which have so far been explained differently, usually incorrectly.

3. Experimental determination of self-stress in a reinforced concrete beam

Let us consider the case of a reinforced concrete beam in which strain in the reinforcement is measured. Before the critical state is reached, that is before the yielding of the concrete, strain in the reinforcement can be identified with stress. When the beam is unloaded, at the pre-critical state some strain remains in the beam, that is self-stress remains. Tension in the reinforcement must be self-balanced by self-stress in the concrete. Fig. 2.

A very important practical conclusion follows from the above observation: at the critical state of a beam under flexure, external load is balanced only by a force corresponding to a strength called the design strength of reinforcement, which is equal to the characteristic strength less self-stress in the reinforcement caused by nonelastic strain in the concrete occurring under load. In the early days of the nonlinear theory of concrete, the characteristic strengths of the reinforcement and of the compressed concrete were used for the critical state. As a result
of experimental work, inconsistencies were concealed in the design strengths of the concrete and of the reinforcement. It follows from the above discussion that at the limit state, concrete has a linear distribution, whence the arm of internal forces under flexure. The notion of the design strength of concrete in the current sense is not needed. Self-stress due to nonelastic concrete strain explains the observed strain and the limit state better.

4. Self-stress in the concrete beam and the cracking moment
Let us consider the case of concrete beam loaded in such a way as to be unloaded before it cracks. We shall demonstrate that in this process of loading, self-stress occurs in the beam due to nonelastic concrete strain. In an unloaded beam, residual strain could arise as shown in Fig. 3, larger in the tension zone than in the compression zone. We shall regard residual strain as given, probable: in a particular problem dependent on humidity conditions and the loading history. Residual strain cannot develop freely in a given section: it is constrained by the compatibility condition. To satisfy the compatibility condition, additional elastic strain must develop, which gives rise to self-stress. Self-stresses meet the condition of self-balancing, that is the sum of the stresses in a section is zero and the moment is zero. As a result we obtain a self-stress distribution as in Fig. 3. Self-stress is superimposed on stresses from external loads. At the pre-cracking state the total stress distribution is as shown in Fig. 4. Stresses from external load are linear; superimposed on them is self-stress due to nonlinear concrete creep. Self-stress decreases tensile stresses in the bottom fibres of the beam. There used to be a view that the tensile strength of concrete under flexure is greater than the
strength of concrete under tension. There are also other interpretations assuming a rectangular stress distribution in the tension zone of the concrete, which cannot be assumed in the balancing of external load as the self-stress component does not take part in the balancing of the external moment. The adoption of the linear theory explains the magnitude of the cracking moment in a consistent manner.

5. Self-stress in a cracked section due to nonlinear concrete creep
For a cracked section we assume a probable residual strain distribution as in Fig. 5. Residual strain appears only in the compression zone of the concrete. The compatibility condition gives rise to elastic strain which causes self-stress satisfying the condition of self-balancing. In the reinforcement, self-stress appears, which was observed experimentally in chapter 3. Residual stresses in the reinforcement can be calculated in accordance with the assumed residual strain distribution. Residual stresses in reinforcement are taken into account in determining the design strength of reinforcement.

6. Self-stress in the prestressed beam
As a result of the transfer from the nonservice state of a prestressed beam to the service state, residual concrete strain can develop for instance linearly as in Fig. 6. Residual strain at transfer from the nonservice state to the service state does not satisfy the compatibility condition. Additional elastic strain occurs causing self-stress. Self-stress is superimposed on the service state stress as in Fig. 6c and on the nonservice state stress as in Fig. 6d. Due to concrete creep in a no-crack design
beam, tension in the extreme fibres has to be taken into consideration. Otherwise the prestressed beam may only become a partially prestressed beam.

7. Linear model for concrete calculations

Beams and other reinforced concrete structural elements undergo linear strain under load. Due to cracking in the tension zone and to concrete creep, additional strain appears, which is superimposed on the elastic effects. For the purposes of strain calculation, a model of defective elastic structure is adopted, defects appearing in the equation as generalized load. Generally the differential equation of a given element can be presented in the following form:

\[ \mathcal{L} (\phi) = p^w + \mathcal{D} \]

where \( \mathcal{L} \) is a differential operator, the same as in the elastic theory.

- \( \phi \) - resolvent function
- \( p^w \) - external load
- \( \mathcal{D} \) - operator describing distortions.

Cracks are the cause of the discontinuity of strain. Solutions are being sought in the form of generalized functions.

Numerous examples point to the adequacy of the calculations to the experimental results. The adopted linear model of concrete justifies the assumption of superposition of effects; the calculations are unequivocal.
Research on building structures and building physics.

Literature


Fig. 1 Load-deflection relationship under forced deflection of a simply supported beam.

Rys. 1 - Zależność obciążenie - ugięcie podczas wymuszonej strzałki ugięcia swooadnie podpartej belki
Fig. 2 Reinforcement strain in a simply supported beam during loading and unloading under forced deflection.

Rys. 2 - Odkształcenia zbrojenia w belce swobodnie podpartej podc obciążen i odciążen przy wymuszonej strzałce ugięcia
Fig. 3 Pre-cracking self-stress in a beam: $\varepsilon^*$ - assumed residual strain, $\varepsilon^* = \lambda + Bz$, total strain satisfies the assumed condition of a plane section; $\sigma^*$ - residual stress called self-stress.
Fig. 4. Total stress distribution at the pre-cracking state beam.
Fig. 5 Self-stress in a cracked beam section: $\varepsilon^o$ - assumed residual strain, $\varepsilon^t = A + Bz$, total strain satisfies the assumed condition of a plane section; $\varepsilon^o_r$ - residual strain in the reinforcement; $\sigma_b = \sigma^o + \sigma$ - total stress in a loaded beam is the sum of self-stress and linear stresses balancing external forces.
Fig. 6 Self-stress in a prestressed beam caused by concrete creep:
a) $\epsilon^N$ - assumed residual strain at transfer from nonservice to service, b) $\sigma_b$ - self-stress in concrete caused by transfer from nonservice to service, c) total stress at the service state, d) total stress at the nonservice state.
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Fig. 1 - Load-deflection relationship under imposed deflection for a simply supported beam.

Fig. 2 - Reinforcement strain in a simply supported beam during loading and unloading under imposed deflection.

Fig. 3 - Pre-cracking self-stress in a beam: \( \varepsilon^p \) - assumed residual strain, \( \varepsilon^T = \Lambda + \Delta z \), total strain satisfying the assumed condition of a plane section; \( G^R \) - residual stress called self-stress.

Fig. 4 - Total stress distribution at the pre-cracking state beam.

Fig. 5 - Self-stress in a cracked beam section: \( \varepsilon^p \) - assumed residual strain, \( \varepsilon^T = \Lambda + \Delta z \), total strain satisfying the assumed condition of a plane section; \( \varepsilon^r \) - residual strain in the reinforcement; \( G_h = G^E + G \) - total stress in a loaded beam is the sum of self-stress and linear stresses balancing external forces.

Fig. 6 - Self-stress in a prestressed beam caused by concrete creep: a/ \( \varepsilon^p \) - assumed residual strain at transfer from nonservice to service state, b/ \( G_h \) - self-stress in concrete caused by transfer from nonservice to service state, c/ total stress at the service state, d/ total stress at the nonservice state.
Research on building structures and building physics.

NONLINEAR ANALYSIS OF RC CRACKED SLABS AND PANELS WITH

BOUNDARY ELEMENT METHOD

by

Maciej Minch

1. Introduction

The real behaviour of reinforced concrete structure differs widely from the results of a linear elastic computation. At the beginning of loading RC construction shows nonlinear deformations because of nonlinear concrete stress-strain relation. As an example of such behaviour the characteristic load-midpoint deflection curve (P-w) of RC rectangular slab is demonstrated in Fig. 1.

![Figure 1: Characteristic load-midpoint deflection curve of RC slab.](image)

Even under working loads $P_w$ and obviously up to failure loads $P_{fr}^{IV}$, the curve is extremely nonlinear which is additionally caused by nonlinear stress-strain relation of reinforcement bars and discontinuity of planar structure. The first change of the stiffness is caused by the beginning of cracking for cracking load $P_{fr}^I$. Construction becomes heterogeneous as a result of cracks and bond-slip between concrete and reinforcement. The crack treated as a defect causes the zoning of the structure region. Each of the zones is connected with another one by means of reinforcement bars appearing in the cracks. So, the edges of the cracks are not free from tension at the points of connections and simultaneously the general vector of displacements has a jump equal to the opening of the crack. Additionally, the forces of the broken adhesion between steel bars and concrete as well as some other transverse forces appear in cracks areas. Under greater loading the further propagation of cracks is shown up to second reduction of the stiffness, when the reinforcement begins to yield $P_{fr}^{II}$. This finally leads to the failure load $P_{fr}^{III}$ associated with the crushing of concrete. For load $P_{fr}^{IV}$ when the slab reaches limiting membrane state. These forces as discussed above, can cause a fundamental redistribution of stresses in the cracked construction compared with the homogeneous one. This thesis
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has been proved by a number of experimental tests for RC panels and slabs.

As a result of stiffness reduction caused by cracking, the values of bending moments are lower compared with those ones obtained from the linear elastic theory. Thus the reinforcement in cracked zones could be divided in the better possible manner. Additionally as a result of bending moments which are decreasing in cracked zones RC slab can be more loaded before reaching limited strains (point E) or local failures (point F).

A typical characteristic disloading and reloading line has been shown in Fig. 1. The separation of elastic w and plastic deflection w respectively, has been assumed according to small strains theory. Such assumption makes computation for this process much easier.

The purpose of this paper is to present some aspects and main problems of theory and computations of planar RC structure. This paper presents the application of nonlinear boundary element method (BEM) to the analysis of cracked RC slabs and panels. It was assumed that computations are valid for working loads because most of the constructions are loaded within the limits (i.e. limits between points A and C). In this loading area the linear theory of elasticity as well as yield line theory appears to be not valid. The mathematical theory of defects, i.e. impulse functions and generalized function calculations as well as the theory of cracks, RC theory and numerical methods, have been used to obtain the final solution. The results of numerical examples are presented below.

2. Material properties

For the computation of planar structure the behaviour of concrete should be considered in the biaxial domain. The concrete properties are influenced by many different factors. Therefore the biaxial stress-strain relation and the failure criterion of concrete depends on the results of the tests that are performed to obtain these relations. The biaxial tests of Kupfer [9] for short time loading and proportionally increasing load proved to be the most reliable. Different authors have used these test results to develop analytical formulation of the failure and deformation behaviour of the concrete. Link [12] developed an incremental formulation for the tangent stiffness of the concrete on the basis of Kupfer tests. The stresses are normalized in terms of the uniaxial cylinder strength, therefore the formulation can be used for different grades of concrete. The failure criterion cannot be used as plasticity condition, because it describes a boundary for the maximum stresses and does not allow any statements about the plastic deformations. The concrete physical law of Link [12] was used in computation of planar structure within presented method.

The stress-strain relation of steel bars was taken as a well known elasto-plastic relation from uniaxial tests.

3. Bond-slip characteristic

After cracking of concrete, the tensile forces in the cracked area are transmitted by bond to the reinforcement consisting of steel bars. Along the segments of broken adhesion the steel bars co-operates with the concrete trough the tangential stresses distributed on the perimeter of the bar. The slip is defined as a relative displacement between reinforcement bars and surrounding
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cracks. The increment of tensile stresses in the steel bars were approximated by the third-degree curve. Hence, the tangential stresses and bond-slip relationships, as representation of the stiffness of the bond have been found to be in the agreement to tests of Dörr [6], (i.e. the second-degree distribution along the segment $l_f$, where $l_f$ means distance between cracks).

4. Constitutive law for crack width

Physical law of the crack opening was taken from: the beam analogy given by Borcz [3], general assumption of cracks theory and the equilibrium conditions in the crack. This can be written in the following incremental form:

$$
\dot{r}(s) = \dot{r}_p(s) + \dot{r}_e(s) \mathbf{F}(\bar{N}, \bar{M}) ,
$$

where $\mathbf{F}$ means nonlinear function of normal tensile $\bar{N}$ and normal bending moment $\bar{M}$ to the crack line $s$. Note, that the vector $\dot{r}$ (i.e. modified Burgers vector depending additionally on inner forces in the crack) is divided on elastic $\dot{r}_e$ and plastic $\dot{r}_p$ parts respectively (see deflection analogy Fig.1). Such formulated model allows to assume any arbitrary form of physical laws of cracks width.

5. Mathematical model of crack

It was assumed that the steady crack $s$ exists in the region of the structure $\Omega$. The crack is connected with local coordinate system $(s, n)$. From the theory of defects and fundamental solution given by Gefland [8], we can obtain discontinuity of Burgers' vector which is mathematical defined as difference of the limits of general displacement functions on the both crack edges (crack is treated as a binary boundary) $[f]_s = \lim_{f} - \lim_{f}$, with following properties:

$$
[u]_s = - \frac{\partial}{\partial s} (s, \bar{N}) ,
$$

$$
[\partial u / \partial n]_s = - \frac{\partial}{\partial s} (s, \bar{M}) ,
$$

$$
[\mathbf{X}(\bar{u}, \bar{w})]_s = 0 ,
$$

where $\mathbf{X} = \bar{n} \cdot \bar{c} \hat{\nabla}$ is the differential operator which means the normal derivative to the crack line $s$, $\bar{n}$ denotes vector of direction cosines and $\bar{c}$ means elastic constants tensor.

The first and second property (Eq.2) and (Eq.3) give discontinuity of displacement vector $u$ and derivative of deflection $w$ on the curve $s$ respectively (displacement of the crack for panels and opening angle for slabs respectively). Whereas the third property (Eq.4) ensures continuity of tension vector $\bar{N}$ ($\bar{N} = \bar{N} \hat{d} \theta$ - where $\bar{N}$ means the thickness of the structure). The tension continuity on the edges of the cracks is ensured by reinforcement bars which appear in the cracks. Because of finite number of steel bars existing in the cracks, the fulfillment of the tension conditions Eq.4 takes place only in the discrete way for the points in which reinforcement occurs. Outside the reinforcement points, on the remaining edge segments of the cracks, the boundary conditions should be equal to the conditions corresponding to the free edges should be fulfilled. Note, the nonlinear condition Eq.4 causes conjugation of plane and bending state in analogical way as for large deflection.

In order to satisfy the conditions Eq.2 and Eq.3 the modeling on the crack edges by dipole normal forces for panels and dipole normal moments for slabs was assumed respectively, (see Minch [13] and
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6. Modeling by Boundary Element Method

Deformation behaviour depends on the history of the loading as well as nonlinearity of material properties. Hence, equations and definitions of the boundary element method in rate form according to the Bubbia [4] formulations were assumed. According to small strains theory, total strain rate for inelastic problem can be divided into an elastic \( \varepsilon^e \) and inelastic \( \varepsilon^i \) part of total strain rate tensor respectively (see analogical separation for deflection on Fig.1). Herein, the inelastic strains mean any kind of strain field which can be considered as initial strains, i.e. plastic or viscoplastic strain rate, creep strain rate, thermal strain rate and strain rate due to other causes. So, we can write now the equations of considered problem in terms of nonlinear BEM formulations for fictitious tractions vector \( \mathbf{p} \) and body forces \( \mathbf{h} \), finally leading to initial stresses \( \sigma^i \):}

\[
\mathbf{H}(\mathbf{u}, \mathbf{w})^T \mathbf{A} \mathbf{p} = \mathbf{R}^i \mathbf{p} + \mathbf{F} \mathbf{x} + \mathbf{Q}(\mathbf{u})
\]

(5)

where matrices \( \mathbf{H} \) and \( \mathbf{A} \) are the same as for elastic analysis, matrix \( \mathbf{B} \) is due to the inelastic stress integral. Matrix \( \mathbf{F} \) refers to the fundamental function cause by forcing traction with vector \( \mathbf{x} \), i.e. modeling density of crack opening for panel and slab respectively. Matrix \( \mathbf{Q} \) describes bond, bond-slip relations and other displacements due to aggregate interlock and dowel action of reinforcement in the crack additionally related to displacement \( \mathbf{u} \).

7. Incremental computations

The results correctness depends on the choice of right type boundary elements and a careful discretization of the structure. The influence of the above on the problem to be studied cannot be neglected. The appropriate simulation of the load-carrying behaviour of RC structure is more important then the accuracy of the numerical calculations. The question what kind of numerical methods or boundary elements should be preferable chosen cannot be answered satisfactorily.

Equation 5 must be solved numerically with iterative and incremental techniques. Iteration results due the fact that the right side of Eq.5. depends directly on function \( \mathbf{u} \). In addition function \( \mathbf{u} \) depends indirectly on physical law (Eq.1) The incremental computation is caused by rate form of Eq.5. For iteration and incremental computations the modified Newton-Raphson method was applied.

8. Numerical examples

Simply supported square slabs DA1, DA2 tested by Absi and Brandt [1]

The all edges simply supported slabs DA1 and DA2 with thickness 10 cm, tested under constant load by Absi and Brandt [1], are numerically analyzed in order to check the performance of the computing procedure. The slab DA1 was reinforced bottom by bars \( \Phi 8 \) mm every 8 cm parallel to the edges. The slab DA2 was reinforced bottom with the same bars as slab DA1, but parallel to the diagonal lines. In Fig.2 the experimentally received load-midpoint deflection relation of the slab is compared to the results of BEM numerical calculations. Fig.3 shows experimentally and numerically obtained results of concrete and reinforcement strains for DA1 slab.
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Fig. 2. Comparison of calculated load-midpoint deflection relations with the test result of slabs DA1 and DA2 [1].

Fig. 3. Comparison of calculated load-midpoint concrete and reinforcement strains relations with the test results of slab DA1 [1].

Panel WT3 tested by Leonhardt and Walther [10]

The result of simply supported square panel WT3, tested by Leonhardt and Walther [10], were taken to check BEM solution for plain stresses. The panel was reinforced horizontally in different way for bottom and top part. The bottom zone (R6) had 2 Ø 8 mm bars each 6
cm fixed in 4 rows, the top zone and vertical bars (R2) were 2 × 5 mm each 26 cm. In Fig. 4 the experimentally received load-midspan deflection relation of the panel is compared to the results of BEM and other authors (Bukozturk [5], Flögl [7] and Lewiński [11]) FEM numerical calculations. Fig. 5 shows experimentally and numerically obtained results of BEM and FEM by Lewiński [11] for reinforcement strains for panel.

9. Conclusions

The numerical results obtained for the problems of slabs and panels indicated that the presented method is capable to predict sufficiently and satisfactorily response of reinforced concrete planar structure. More numerical results and experimental data is still needed. More realistic description of aggregate interlocking and dowel action is also necessary.

The examples presented in paper show the advantages of using FEM in inelastic problems in preference to FEM. Considerable reductions in the data, required to run a problem, can be achieved for the same degree of accuracy.

References

Fig. 4. Comparison of calculated load-midspan deflection relations with the test result of panel WT3 [10].

Fig. 5. Comparison of calculated load-midspan reinforcement strains relations with the test results of panel WT3 [10].
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A NEW MODEL FOR BOND SLIP AND CRACK WIDTH CALCULATIONS
OF TENSION MEMBERS

1. INTRODUCTION

The bond between concrete and steel is of fundamental importance to many aspects of reinforced concrete behaviour. The spacing and width of cracks, internal distributions of stresses along the steel after cracking, tension stiffening between primary cracks, relative slip between steel and concrete strongly depend on bond. The most important factors influencing bond characteristics are: the type of steel bars, concrete cover, confining effects, concrete strength and the type of load.

The bond phenomenon has been studied for decades \cite{1}, \cite{2}. Now specimens with a very short bond length are used. They allow to establish the relationships between the local slip and the bond stress for every type of load. Several analytical models have been developed on that base but little attention has been given to distributions of stresses along a tension member \cite{3}, \cite{4}. Still there are some important problems which are not solved or their solution is very doubtful:

a) the length of bond transfer,

b) boundary conditions for slip and strains,

c) concrete stress distribution across a tension member,

d) the rise and development of internal and secondary cracks.

This paper shows a new model for calculations which deals with those problems (fig.1). All model parameters are clearly connected with experimental results.
2. MAIN ASSUMPTIONS AND GOVERNING EQUATIONS.

When a reinforced concrete member is subjected to a sufficiently high tension force, primary cracks form and a relative bar to concrete slip occurs. The compatibility and equilibrium equations are:

\[ \varepsilon_a(x) - \varepsilon_b(x) = \frac{d\Delta}{dx} \]  

\[ \frac{d\delta_a(x)}{dx} = -\frac{4\tau(x)}{d} \]  

\[ N = \sigma_a F_a = \delta_a(x) F_a + \delta_b(x) F_b \]

where \( \varepsilon_a, \sigma_a, E_a \) and \( \varepsilon_b, \sigma_b, E_b \) - strain, stress and elastic moduli for steel and concrete respectively,

\( \Delta \) - slip between steel and concrete,

\( \tau \) - bond stress

\( d \) - bar diameter
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There are four unknowns in equations (1) - (3), so another one must be establish. It can has the following forms: \( \tau = \tau (\Delta) \), \( \tau = \tau (x) \) or \( \tau = \tau (\Delta, x) \).

In this paper two functions were chosen. The first one is the bond stress just after cracking. At that moment, the length of bond transfer and location of secondary cracks are determined. The bond function has the next form:

\[ \tau = k(a-x) \sqrt{x} \]  \hspace{1cm} (4)

where \( a, k \) constans explained later.

Substituting (4) to (2) gives

\[ \sigma_a(x) = \sigma_o - \frac{4k}{d} \left[ \frac{2}{3} a x^{1.5} - \frac{2}{5} x^{2.5} \right] \]  \hspace{1cm} (5)

and substituting (5) to (3) yields to

\[ \sigma_b(x) = \frac{4k\mu}{d} \left[ \frac{2}{3} a x^{1.5} - \frac{2}{5} x^{2.5} \right] \]  \hspace{1cm} (6)

where \( \sigma_o = \frac{4(1+\eta \mu)}{E \ell} R_{tr}, \ \eta = E_a/E_b, \ \mu = F_a/F_b \)

\( R_{tr} \) - tensile concrete strength

The compatibility condition for \( \sigma_o \) at \( x = a \) gives

\[ k a^{2.5} = \frac{15 R_{tr} \cdot a}{8 \ell \mu} \]  \hspace{1cm} (7)

From (4) one can obtain

\[ \tau_{max} = \frac{2 k a^{1.5}}{3^{1.5}} \]  \hspace{1cm} (8)

so from (7) and (8) the bond length equals

\[ a = \frac{5 R_{tr} \cdot \tau}{4 \cdot \sqrt{3} \ell \mu \cdot \tau_{max}} \]  \hspace{1cm} (9)

Substituting (5) and (1) the local slip for \( \tau_{max} \) is

\[ \Delta = \frac{4}{E_a} \int_{a/3}^{a} \left[ \sigma_a(x) - \sigma_b(x) \right] dx = \left[ \frac{3}{7} - \frac{3 \sqrt{3}}{9} \right] \frac{\sigma_o a}{E_a} \]  \hspace{1cm} (10)

In the pull-out test, when a force equals \( P = \sigma_o F_a = (4+\eta \mu)R_{tr}F_b \) the local slip \( \Delta \) can be measured, so from (10):

\[ a = \frac{E_a}{\sigma_o} \frac{\Delta}{\frac{3}{7} - \frac{3 \sqrt{3}}{9}}^{-1} \]  \hspace{1cm} (11)

Using experimental value \( \Delta \) and equations (9) and (11) it is possible to calculate \( \tau_{max} \) and \( k \).
3. CRACKING DEVELOPMENT

3.1. Just after primary cracks occur

Just after cracking not only internal cracks but also a "potential" secondary crack appear. It is caused by uniform tensile stress in concrete ($\sigma_b$) and additional strains in concrete surrounding a steel bar which depend on bond forces.

\[ u(x,y) = \frac{d}{4E_b} \int_{0}^{a} \tau(x) \left[ \frac{2(1-v_b)}{(y^2 + \alpha^2)^{0.5}} + \frac{\alpha^2}{(y^2 + \alpha^2)^{1.5}} \right] d\alpha \]  

(12)

\[ \delta \varepsilon_b(x) = \frac{du(x,y)}{dx} \]  

(13)

where $\Delta_b$ - Poisson's ratio for concrete

This model can explain the nonuniform strain distribution in concrete cross-section much better than arbitrary chosen $\psi$ [5].

Location of secondary cracks can be found from a condition:

\[ \sigma_b(x) + \delta \varepsilon_b(x) \cdot E_b = R_{\tau} \]  

(14)

The change of steel stress distribution caused by secondary crack is shown at fig. 3. The values of $x_1$ and $\sigma_{\delta}$ are computed from equilibrium conditions for bond and steel stresses respectively.

Using Tepfers's model [6], it is possible to check if that crack remains inside or reaches concrete face.

3.2. Cracking development and the influence of loading history

Internal and secondary cracks occur just after primary cracks and change a type of bond. It strongly depends on a kind of a bar.
For deformed bars the next function is used:

\[ \tau(x) = g \cdot x \cdot \sigma_a(x) \]  

(15)

where \( g \) - factor taken from pull-out test

Using equations (1) - (3) and (15) one can obtain

\[ \sigma_a(x) = \sigma_o \cdot \exp(-g \frac{x^2}{\tau}) \], \( \sigma_o = P/F_a \)  

(16)

\[ \sigma_b(x) = \sigma_o \mu \left( 1 - \exp(-g \frac{x^2}{\tau}) \right) \]  

(17)

The maximum value of bond stress is given by (from 15):

\[ \tau_{\text{max}} = \tau \left( \sqrt{\frac{d}{4g}} \right) = \sigma_o \sqrt{\frac{\tau \cdot g}{2 \cdot e}} \]  

(18)

When load increases and \( \tau_{\text{max}} \) remains constant, a "\( g \)" value decreases, so a bond peak value moves away from a primary crack. At the same time the steel stress distribution becomes more uniform (fig.3)

\[ \alpha_f = 2 \left[ \Delta(0) - u(0,r) - \sum \alpha_i \right] \]  

(19)

where \( \Delta(0) - \sum \alpha_i \) is simply a slip of steel bar calculated for \( x \) from 0 to \( x_1 \).

Fig.3. Steel and bond stress distribution along the bar
In some cases a free shrinkage at the same distance must be taken into consideration, so additional width of crack equals \( \delta a_t = 2E_{sh} \alpha \). The tension member under more complex loading history may be treated in a similar way. The main governing parameters of the model - \( \alpha, k \) and \( g \) can be obtained from experimental load- or slip-control tests, which were carried also for cyclic and reversed loads\(^4\).

4. NUMERICAL EXAMPLE

Since there is no such an experimental work in which both specimens of a very short bond length and for tension cracking analysis are checked simultaneously, the numerical example shows only the main ideas.

Material properties:

- \( b=h=0,075m \), \( R_f=2,0MPa \), \( E=210 \) GPa, \( n=7,0 \), \( d=0,012m \), \( \mu=0,02 \).
- It was assumed that at cracking level ( \( \sigma_o = 114 \) MPa) \( \tau_{max}/R_f=2 \).
- The bond length equals \( \alpha=0,1083m \). The first secondary crack appears at \( x_0=0,08m \). It reaches \( e=0,008m \) from centroid of a bar.
- Up to \( \sigma_o=149\)MPa \( \tau_{max} \) is constant but a peak value moves away from \( x=0,036m \) to \( x=0,068m \). At \( \sigma_o=182\)MPa another secondary crack rises at \( x=0,068m \) and it reaches the concrete surface.
- The ultimate crack spacing is \( 0,068m \), but for secondary cracks it equals \( 0,081m \) (fig.4).

![Graph](image_url)

**Fig.4.** Steel stresses along the bar.
5. CONCLUSIONS

The presented model shows the movement of a peak bond stress, an increase of the steel stress between the primary cracks, rising and development of secondary cracks such phenomena as a nonuniform concrete strain distribution in cross-section or the width of secondary cracks can be explain much better. Experimental values \( k, \alpha \) and \( g \) are easy to calculate from pull-out test. Using this method it is possible to evaluate both the width of crack and the tension stiffening.

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NUMERICAL ANALYSIS OF CONCRETE SILO SHELL

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Introduction

Due to complexity of concrete behavior in various stress stages the development of rational and sufficiently general model of concrete structures is a complicated task. This development needs accurate description of materials, structures and also well developed computational methods. Those requirements could meet computer methods.

The paper presents a solution of a problem of an analysis of structural concrete shells based on finite element method, which is applied to the analysis of a silo shell.

A model of a structure

Constitutive relation in an element of a shell are based on the theory of thin, layered, elastic, homogeneous and orthotropic shells [1]. The following assumptions are made:
- displacements of a structure are small.
- strain distribution in a section according to Kirchhoff-Love hypothesis.
- on the boundary surfaces of each layer the conditions of continuity of displacements and stress are fulfilled.

An important feature of such a computational model is the interface of axial forces and bending moments which is the consequence of different mechanical properties of concrete layers and non-symmetrical distribution of reinforcement in an element of a shell. The model of smeared cracks is applied. The composite material-stiffness matrix is formed by superposition of component material-stiffness matrices as follows [2]:

$$[D] = [D]_{c} + \sum_{i}^{n} [D]_{i}$$  \hspace{1cm} (1)

where: \(n\) is the number of reinforcing directions.
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\[ [D]_c \] is the stiffness matrix of concrete layers. 
\[ [D]_i \] is the stiffness matrix of reinforcement in direction \( i \).

The elements are divided into a constant number of layers of the same thickness and different properties depending on stress (strain) rate and mode of cracking. The reinforcement is also treated as a smeared layers.

The state of stress (strain) is analyzed separately for each layer of concrete and steel and then the material-stiffness matrix is formed.

Models of materials

The constitutive relations for concrete are based on Kupfer, Gerstle [4] and Cedolin et al. [5] ideas. Octahedral stress-strain relationships are applied to calculate secant shear and bulk modulus.

The biaxial failure function is applied according to Kupfer and Gerstle [4] tests (fig.1). Steel is assumed to be an elastic-perfectly plastic material.

The mechanics of cracked reinforced concrete is described by following parameters: compressive strength of concrete, shear stiffness due to aggregate interlock and dowel action, tension stiffening.

The reduction of compressive strength of cracked in one direction concrete is concerned as a function of lateral tensile strain according to Collins and Vecchio ideas [5] (fig.2). The shear stiffness of cracked reinforced concrete due to aggregate interlock is also a function of lateral strain as shown in fig.3. The shear stiffness due to dowel action is concerned as a function of crack direction and properties of reinforcing bars according to Shirai and Sato ideas [6] (fig.4). The tension stiffening effect is implemented according to Eurocode 2 [7].
The cracking is the basic feature of the concrete behavior. The cracks are assumed to appear in the directions perpendicular to the principal tensile strains, in each concrete layer separately.

Application of the model

The presented model was applied to the analysis of prestressed silo shell. The structure was filled with sugar. Three modes of emptying were taken into account: the central discharge and the eccentric discharge with the eccentrics of 4.15m and 9.30m. For that modes of emptying the loadings were applied to the structure according to the Polish Standard [8]. Prestressing was distributed so as the silo shell behave as the partially prestressed, which means in that case that for static loading no cracking is allowed. The silo diameter was 20m, its high 50m and the wall thickness 0.23m. The calculations in the case of the most dangerous loading
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were conducted in two stages - the elastic and nonelastic ones. Significant differences were observed between the stages both for stresses and displacements (particularly in the region where patch load due to eccentric discharge was applied, fig. 5 and fig. 6). In figure 5 deformed mesh and distribution of radial displacements is presented in the elastic stage. The figure 6 presents radial displacements of the silo shell when the nonlinear model was applied.

Conclusions

Computer methods are becoming an advanced tool for design of concrete structures. Its great potential lies in the ability to work with developing constitutive laws. Obviously they are an approximation of the reality but this approximation can be controlled. A very important advantage of such methods is their application to the cases where analytical models cannot or can hardly be used.

References


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WAVE CONCEPTION OF FLEXING
OF THE REINFORCED CONCRETE BEAMS

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Eindhoven – October 1992

Summary. Tests show that in loaded reinforced concrete beams take place a wide range of sophisticated processes. To such processes belongs for example, changeability of the modulus of elasticity along the beam length. This changeability depends mainly on the level of the concrete inner work state, on which have influence level of the beam load and other factors as for instance cracks. In the main part of the theories, concerned on the beams work, those problems are generally omitted and the constant values of stiffness are assumed. This paper presents a try to build a conception taking into consideration all mentioned changes. This conception is named "The Conception of Wave Stiffness Distribution". Model considering upper mentioned remarks was build and tested. Results are presented in paper.

1. Distribution of stiffness on the beam length

Analysis of the chosen cross section of the reinforced concrete beam, shows that during the load increase, two cases of work of such cross-section may occur. In the first case, chosen cross-section may work, during the all time of load application, in the noncracked state. In the second case, crack may occur in chosen cross-section, in moment when tensile stresses overtake the tensile strength of concrete. In this case cross section will work up to the moment of damage in cracked state. Work of both cross-sections is up to the specified load level the same, and differentiates after crossing some load level, so called cracking load.

It is possible to estimate that cracks, it means cracked cross-sections, occur in some distance on the beam length and that there is the finite quantity of cracks. Cracked cross-sections in that situation, are taking only small part of the beam length. Most of cross-sections are first type what means that the most part of the beam work in noncracked state.

Stiffness of the chosen cross-section of the beam depends on following factors:
- changeability of the modulus of elasticity of concrete $E_c$,
- changeability of modulus of inertia of the cross-section $I_z$.

Hence we consider that stiffness of the first case sections will decrease due to the changes of the modulus of the elasticity, during all loading process up to damage of the beam. This process we describe using continuously decreasing function as shows Fig.1.

Consider second case sections, their stiffness at the beginning will slowly fall down due to the load increase, as it was in the first case. Change becomes when the crack develops. Stiffness jump down that moment. After the crack stabilize, stiffness still decrease but little faster than in the first case sections. This process shows Fig.2.

There exist many factors changing described process. One of the most important will be time factor. Stiffness of cross-section will decrease caused by creep and shrinkage of concrete.
Simultaneously stiffness will increase caused by aging of concrete. Those influences will mix together giving a picture as shows Fig. 3.

Stiffness of the second case sections will decrease slowly up to moment when the crack develops. Further stiffness jump down as it was upper mentioned. Simultaneously shrinkage and creep will increase and aging will decrease speed of this process. Result is as shows Fig. 4.
Additionally it is important to remember about influence of other factors as for example about influence of shape and type of embedded reinforcement.

Described process we may spread on the whole beam. It means that we introduce parameter describing position of section in element.

Consider decreasing of stiffness due to the load increase, stiffness of the loaded beam in the points where minimum inner force develops differ from points where inner forces are maximal. Additionally this effect will mix with the stiffness change which occur due to the crack develop.

As a conclusion on this stage, we may consider that in heavy loaded reinforced beam in the same moment when some sections are close to the damage, most of them sill work in phase I. This process is shown in Fig.5

Fig.5 Distribution of stiffness on the beam length during the loading process: a) I phase, b) first crack development, c) II phase, d) III phase
2. Basic assumptions of the wave conception of stiffness distribution

Analysing processes described in section 1, we may assume that during the beam loading period stiffness of the beam is changing caused by following reasons:

- changes of the material (concrete) properties due to the load and time increase,
- changes of the cross section geometry caused by crack development.

Generally we assume stiffness as function of the 3 main parameters:
- loads,
- position of the cross section,
- time.

It may be formally written as follows:

\[ B = B(p, \xi, t). \]  

Expression (1) may be simplified introducing coefficient of the level of the load increase, which may be expressed as follows:

\[ \frac{M}{M_r} = m_r, \]  

where \( M \) is the loading moment and \( M_r \) is the cracking moment.

Values of the \( m_r \) must change in following interval:

\[ 0 \leq m = \frac{M}{M_r} \leq \frac{M_n}{M_n}. \]  

When \( m_r = 1 \) first crack develops. Values of \( \frac{M}{M_r} \) are from interval 4 - 10, dependent on quality of concrete and many other factors.

Hence stiffness may be expressed in following form:

\[ B = B(m_r, t). \]  

Charts of the function fulfilling upper mentioned conditions are shown in Fig.6.

Tracing the stiffness function on the upper shown figures, shows that in first phase of beam work stiffness decrease continuously and for most part of cross sections those changes are continued in the same manner up to the damage of the beam. In those cross sections cracks will not develop. Only in several points bean cracks occur and only in those places stiffness will jump down.

Those lead to the conclusion that the stiffness function may be treated as a sum of two functions. One of those functions will exist in the whole period of the beam work and second will arise in the moment the first crack develop. Values of second function will exist only in the cracks environment.

Such stiffness function for the immediate loads may be assumed formally as follows:

\[ B = f(m_r, t - \tau = 0) + h(m_r, t - \tau = 0), \]  

where \( m_r = \frac{M}{M_r} \) - that is relation between loading and cracking moments.

\( t - \tau = 0 \) - load starting time.

In general case function \( f \) describe distribution of stiffness along the beam, before and after first crack develop. Second function arise in the cracking moment in the cracks environment.

Knowledge of the upper functions will lead to the seek solution of problem.
The function \( f \) was in present paper assumed in the shape (6):

\[
f(m_r, t - \tau = 0) = \frac{B_{I_0} - B_{I_b}}{1 + am_r(B_{I_0} - B_{I_b})} + B_{I_b},
\]

where \( B_{I_0} \) is the stiffness of the nonloaded cross section in the moment of the load application and \( B_{I_b} \) is the stiffness of the cross section just before cracking in the moment of the load application.

\( \alpha \) is the coefficient of the stiffness level during the cracking process and it may be calculated from the assumption that for \( m_r = 1 \) (cracking) function \( f = B_{I_1} \), where \( B_{I_b} < B_{I_1} < B_{I_0} \).

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Fig. 6 Distribution of stiffness on the beam:
- a) due to the stage of strain,
- b) due to the position of the cross section,
- c) due to the load stage
For upper assumptions coefficient $\alpha$ takes following values:

$$\alpha = \frac{B_{1\omega} - B_{11}}{(B_{1\omega} - B_{1b})(B_{11} - B_{1b})}. \quad (7)$$

For phase $1 f(m_r, t - \tau = 0) = B_0$.

Hence function (6) take a following shape:

$$f(m_r, t - \tau = 0) = \frac{B_{1\omega} - B_{1b}}{1 + \frac{B_{1\omega} - B_{1b}}{m_r}} + B_{1b}. \quad (8)$$

Chart of the upper function shows Fig.7.

![Chart of the upper function](image)

Fig.7 Chart of the function $f(m_r, t - \tau = 0)$

Function $h$ was assumed in the following shape:

$$h(m_r, t - \tau = 0) = -(B_{1b} - B_{11})^{\frac{1.1m_r^4}{2.1+m_r^2}} g(m_r, t - \tau = 0). \quad (9)$$

where $B_{11}$ is the minimal stiffness of the cracked cross-section,

$g(m_r, t - \tau = 0)$ is function describing influence of cracks on the stiffness.

Generally function $g$ take a shape as follows:

$$g(m_r, t - \tau = 0) = \left(\cos \frac{2\pi m_r}{2} - 1\right)^4. \quad (10)$$

In the special case function $g$ may take constant value $\beta$. $\beta$ will describe the average level of stiffness on the crack area considering work of cracked and noncracked cross sections.

Chart of the function $h$ is shown in Fig.8.

In general case of beam working under the immediate loads in the full range of load, distribution of stiffness considering upper given remarks will be described using formula (5) and will look as shows Fig.9.
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For example, according to assumed rules, stiffness function for simply supported beam, working in the I phase, will look as follows (11):

\[ B^I(\zeta, p_r, t - \tau = 0) = \frac{B_{f1} - B_{f3}}{1 + (B_{f1} - B_{f3}) \zeta (1 - \zeta)} + B_{f2}, \]

(11)

\( \alpha = 1 \) in this case,

and \( \zeta = \frac{p}{p_r}, p_r = \frac{P}{P_c} \),

where \( p_r \) is the cracking load.

---

Fig. 8 Chart of the function \( g(m_r, t - \tau = 0) \)

Fig. 9 Distribution of stiffness on the beam length in the full range of work
Stiffness function described in formula (11) is shown in Fig. 10

For the phase II function of stiffness distribution, assumed according to the proposed rules, for the simple supported reinforced concrete beam, will look as follows:

\[ B(\zeta, p_r, t - \tau = 0) = B^I(\zeta, p_r) - B_{II}(\zeta, p_r) \left[ \frac{\sin(2\pi r \zeta)}{2} \right] 4 \frac{1 + \zeta p_r (1 - \zeta)}{2 + \zeta p_r (1 - \zeta)}. \]  

Value \( n \) in formula (12) describe quantity of cracks on the beam length and may be calculated from shear conditions.

\[ n = \frac{l}{l_r}, \quad \text{where} \ l_r = 0.4 \frac{d}{\mu}. \]

This function (12) shows Fig. 11.
3. Deflections

Description of the function of the stiffness distribution, made in previous section allows us to determine deflections of the reinforced concrete beam. Those deflections calculation will consider all previous assumptions what means changes of stiffness on the beam length.

Using the curvature equation in shape (13):

$$\rho = \frac{1}{r} = \frac{d^2y}{du^2} = -\frac{M(u)}{B(u)}. \quad (13)$$

Stiffness $B(u)$ is given through expression (5). Inserting (5) to equation (13) we receive:

$$\rho = \frac{d^2y}{du^2} = -\frac{M(u)}{f(u)+h(u)}. \quad (14)$$

Hence further

$$\rho = \frac{E_f R(u) - E_f R(u)}{1 + B_f R(u) - B_f R(u)} + B_f R(u) - [B_f R(u) - B_f R(u)] \rho(u) \frac{1}{1 + n} \quad (15)$$

Equation (15) describe the beam curvature in both phases of work.

Through integration eq (15) and foundation constants from the boundary conditions, we receive expressions for displacements (deflections) of the beam. Formally they may be written as follows:

$$\theta = \frac{dy}{du} = -\int \frac{M(u)}{f(u)+h(u)} du + C_1, \quad (16)$$

$$y = -\int \int \frac{M(u)}{f(u)+h(u)} dudu + C_1 u + C_2. \quad (17)$$

Integration of the equation (17) may be made using finite differences method in the case when the boundary conditions are given on the both ends of the beam. For other cases Runge-Cutta method should be applied.
4 Example

As for example deflections of the single supported beam shown in Fig.12 were calculated. All example calculations were made using written computer program which scheme shows Fig.13. For the comparison FEM model of upper shown beam were build and tested considering crack influence.

All FEM calculations were done using DIANA FEM system. Results of the FEM calculations are shown in Fig.14.

![Fig.13 Calculating program scheme](image1)

![Fig.14 Calculated deflections of the beam using FEM method](image2)
5 Conclusions

In paper the process of flexing of the reinforced concrete elements was analysed. The conception of wave stiffness distribution based on results received by Szechinski in paper [1] was proposed.

This conception assume, that the stiffness of the reinforced concrete cross section is changing due to the load and position change. Results of calculations of the example beam, using proposed model, are shown in Fig.15.

Those results were compared with results of calculations of The FEM model build using the DIANA system [2].

Two FEM models were considered.

First analysed model was linear in full range of load. In second model influence of cracks was considered.

Charts in Fig.15 show that results received from calculations of the linear model are much lower then proposed in this paper. Results received from calculations of the second model show that influence of cracks is valuable.

Still proposed results are higher than calculated using FEM nonlinear models. It is possible that results of FEM calculations will increase to the level of received in paper results, when other phenomena, as for example plasticity of concrete in compressed zone, will be considered.

From the other side parameters of the proposed model should be checked using the laboratory results.

As a conclusion it possible to estate that the proposed conception is suitable, to the very detailed analysis of work of the reinforced concrete flexed elements. Especially in cases when knowledge of displacements may decide about usefulness of the working structure.
References

EXPERIMENTAL INVESTIGATION OF THE DYNAMICALLY LOADED FLOOR - SLAB SYSTEM

1. INTRODUCTION

The phenomenon of large relative displacements between prestressed-concrete floor slabs and steel supporting girders was observed in August 1991 during the assembling phase of complex building - Nieuwbouw Laboratory DRS. After putting floor-slabs over ground floor a whole section of them moved up to 0.08m. An extensive experimental program, aiming at the explanation of the above mentioned phenomenon, has been performed at BRO TU Eindhoven. In the assembling phase, when uncontrolled displacements took place, the main loading acting on a structure was dead load of concrete slabs supported by relatively feeble steel skeleton. As the phenomena entailing displacements of a structure shrinkage and creep of concrete, and strains due to thermal expansion are often reported. After detailed examination they were excluded as a possible reasons for slabs displacements. An as eventual source for unpredicted loadings, the dynamic factors have been assumed. They are mainly connected with wind, ground vibrations generated by traffic and vibrations generated by miscellaneous sources (crane, compressor, etc.). The efforts connected with the explanation of possible reasons for slab displacement have been concentrated on three interrelated problems:

- experimental analysis of the kinematic conditions at the supports of prestressed-concrete slabs,
- measurement of vibrations coming from different sources located in the vicinity of construction,
- numerical analysis of the structure with special interest paid to data from experiments.

2. KINEMATIC CONDITIONS AT THE SUPPORT OF PRESTRESSED-CONCRETE SLAB

The main goal of the investigations was the friction phenomenon at the supports. Taking into account the existence of rubber pads between concrete slab and steel beam, it was considered necessary to evaluate the friction conditions among three different materials adhering to each other. The real conditions were simulated in the laboratory. Concrete slab was supported on steel beams through rubber pad or steel roller 30 mm in diameter. Lateral force was put to concrete slab by means of hydraulic actuator with programmable displacement. Maximum displacement range in hydraulic actuator was 90 mm and it was also the maximum shift of the slab at the supports. Reactions at the supports were calculated for initial conditions for displacement $\Delta = 0$. Lateral force $F$ was transferred on slab through steel frontal plate. Load cell was included in testing arrangement. Values of $F$ and $\Delta$ were monitored on X-Y plotter and simultaneously were measured automatically at the rate of 50 measurement/minute. Loading was transferred to concrete slab in terms of controlled displacement activated by the hydraulic actuator. Three different displacement histories were applied in research program (two constant rate and one pulsating displacement). Experimental results were the basis for evaluation of the dynamic and static friction coefficients for various loading conditions and different materials.
3. MECHANISM OF FRICTION AT THE SUPPORTS OF THE REAL STRUCTURE

The characteristic decrease of friction just at the beginning of slab displacement was observed in experiments. That phenomenon is independent of displacement rate (Fig.1).

Fig.1. Decrease of friction due to the presence of a glue layer for the monotonic (a), and pulsating (b) loading condition.
The range of the dynamic friction depression depends on the width of rubber band. For narrower band the range of 'friction depression' is smaller than for broader one (Fig. 2).

**Fig. 2. 'Friction depression' for the rubber band of 20 and 40 mm in width**

It seems, that when the movement between steel and rubber commences, glue on the rubber acts as a 'greasing agent' between the planes of friction. Glue is formed into thin 'rollers', which facilitate friction. Making use of that empirical observation, the mechanism of friction was divided into three phases presented schematically in Fig. 3.

**Fig. 3. Mechanism of friction at the support of concrete slab**

The global value of static friction coefficient for real - structure conditions, at the support of concrete slab may be assumed as 0.55 - 0.60. When the phenomenon of 'glue rollers' formation occurs, the values of dynamic friction coefficient may be even three times lower and attain the range 0.19 - 0.36. This last finding is particularly important for dynamic analysis of the structure.
4. EXPERIMENTAL ANALYSIS OF THE TRAFFIC INDUCED VIBRATION

The instant values of acceleration were recorded in four measurement points at the vicinity of DSM lab. Signal from accelerometers was continuously recorded on digital recorder. At the same time signal was monitored on oscilloscope with the printer. Detailed Fourier analysis of recorded signals was performed thereafter on digital analyzer. The overall time of data recording at four points was about five hours. The vibrations were generated by heavy trucks passing by at the distance 50 m from the lab building (smooth asphalt road) and by highway traffic. A highway was located on embankment 5 m in height, 130 m from the lab. The maximal values of acceleration amplitudes were in accordance with the direction of Rayleigh waves propagation and reached 0.0055 m/s for frequency span 19 - 20 Hz. Regarding frequency span more closely, one may deduce, that almost all maxima are the multiples of some basic frequency 3.1 Hz. The basic value 3.1 Hz equals approximately the double value of eigenfrequency for the structure \( f = 1.55 \) Hz.

5. EXPERIMENTAL ANALYSIS OF THE CRANE INDUCED VIBRATIONS

Site measurements of the crane induced vibrations were performed during different phases of crane operations. The crane was assembled in the neighborhood of one - storey building with stiff ground floor and continuous foundation on ground piles. The total mass of the crane was about 130 tons. The distance from the laboratory building was 30 m. The accelerometer was glued to a concrete block (0.10 x 0.20 x 0.30 m) embedded 0.10 m into the ground, and fixed to the uncovered crest of a pile foundation. The ground and foundation crest accelerations were monitored during different phases of crane operation. The records of acceleration are this time more regular than for traffic induced vibrations. For they may be often considered as periodic vibrations, it was reasonable to acquire values of amplitudes and frequencies from the direct measurements. Some records of accelerations, which may be considered as a representative, are shown in Fig.4.

![Fig. 4. Representative recordings of accelerations for various phases of crane operation](image-url)
The values of amplitude oscillate in very wide range, from 0.0370 to 0.225 m/s². They are 10 - 10000 times higher than the vibrations coming from road traffic. The frequencies of about 1 Hz, which are close to the eigenfrequencies of the structure may be spotted in many recordings, however the majority of vibrations spectrum is located in the frequency span 70 - 110 Hz. Some additional informations were provided by Fourier analysis of accelerations. There were some very strong peaks of acceleration amplitudes for frequencies of about 70 - 100 Hz. They reach values of 0.15 - 0.4 m/s², which are extremely high in comparison with the previously recorded amplitudes.

6. CONCLUSIONS

Numerical calculations were performed for loading histories created on the basis of experimentally recorded accelerations. The structure was modeled according to STRUC computational system recommendations. The maximal values of accelerations for each of the dynamic loadings differ between one another of two order of magnitude. The least absolute values of accelerations and displacement were obtained for traffic induced vibrations. Their magnitude is thereabouts 0.019 m/s² and 0.0087 m, while for crane induced vibrations they are 2.13 m/s² and 0.031 m respectively. These values pertains to the lateral accelerations and displacement of the supports of prestressed - concrete slabs.

From kinematic point of view, the movement of slab can commence only in the moment, when lateral force (or acceleration) exceeds the value of friction force at the support.

Experiments performed on kinematic conditions at the support, revealed the existence of characteristic 'depression' of friction coefficient. This phenomenon is connected with the glue roller formation in contact layer between steel girder and rubber band. It commences after 0.002 - 0.005 m displacement of concrete slab. In the moment of placing a concrete slab on supports, small movement of rubber band can be forcedly by crane or slab-sling. It leads directly to the situation, when real friction coefficient is very low (about 0.2), so the minimal acceleration necessary for the initiation of slab movement is thereabouts 1.9 m/s².

According to the site-measurement and numerical calculations, the traffic induced vibrations cannot be considered as a source of accelerations strong enough to move on slabs.

The only source of vibrations, having energy to increase the values of accelerations up to the limit of 2.0 m/s², are crane-induced vibrations. Dynamic calculations demonstrated, that even 30 second impulses are able to develop the lateral accelerations of supports with amplitude above 1.0 m/s².

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THE LONGWAVE RADIATION AROUND BUILDINGS

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1. INTRODUCTION

Until approximately twenty five years ago, the net longwave radiation exchange between a building and its thermal environment were considered minimal in comparison with the solar radiation absorbed by the building surfaces. More recent studies comparing the predicted heat flow through elements with measured values illustrate that significant errors may be introduced into the calculation of heat flow through building elements if the longwave radiation exchange at the outside surface is neglected. Recently, interest has also grown in the use of radiative heat losses of the buildings at night and is focused on the atmospheres's behavior as heat sink during the night for producing thermal radiative cooling. It is indicated that about 20% of the heat losses during the heating period originate in the radiative energy exchange between the building envelope and the thermal environment.

Of course, the longwave atmospheric radiation flux at the earth's surface is a very important factor for the energy balance and its quantitative knowledge is of fundamental importance for any meteorological, agrometeorological, hydrological and agricultural problems.

Several different procedures have been developed over the years for calculating intensity of the longwave radiation from the atmosphere at the ground. Since no network of the observation stations exists for this flux, estimates must be made from routinely collected meteorological data, like the air temperature, the water pressure, the dewpoint of air, cloud cover factor etc.

Various empirical models describing intensity of the longwave radiation have been developed only for horizontal surfaces (at the ground) and this limitation excludes pitched roofs and walls. The most frequently cited models (Berdahl and Fromberg, 1982; Idso and Jackson, 1969; Swinbank, 1963; Unsworth and Monteith, 1971) were empirically verified only for clear sky conditions. Cole (1976) proposed a simple empirical model for calculation intensity of the longwave radiation incident upon surfaces inclined at any angle, from horizontal to vertical. This model was modified numerically and adopted to meteorological conditions in the South-West region in Poland (51°N).

When considering the longwave radiative interaction between building envelopes and the thermal environment two main problems may be distinguished. The former is how we can calculate intensity of the incoming longwave radiation from the sky at different angles. And the latter is what is the influence of the the thermal radiation of the environment on the radiation balance and subsequently on heat balance of the building envelopes.

This paper presents the comparison of the measurements obtained for surfaces inclined at any angle to the horizontal with the modified Cole's model and indicates possibilities of it's applications.
The accepted in this paper model of the longwave radiation incident upon the building envelopes is essentially based on the one submitted by Cole (1976) and was adopted to weather conditions in the South-West region in Poland. In this model the intensity of the thermal radiation coming from the outside environment depends on the value of the air temperature, cloud cover factor, the inclination angle displayed by the elevated plane, from 0° (horizontal plane) to 90° (vertical plane), and on values of the measurement-determined empirical coefficients.

The longwave radiation $R(\alpha)$ incident upon a surface inclined at an angle $\alpha$ to the horizontal is a combination of atmospheric $R_A(\alpha)$ and ground radiation $R_G(\alpha)$, (Cole, 1976):

$$R(\alpha) = R_A(\alpha) + R_G(\alpha)$$

(1)

Where: $R_A(\alpha)$ - the atmospheric radiation upon a surface inclined at an $\alpha$ angle, $Wm^{-2}$

$R_G(\alpha)$ - the ground radiation upon a surface inclined at an $\alpha$ angle, $Wm^{-2}$,

The atmospheric component is given by:

$$R_A(\alpha) = R_A k_1 + b k_2 \delta T_a^4$$

(2)

Where: $R_A$ - the atmospheric radiation incident upon horizontal surface, $Wm^{-2}$,

$k_1, k_2$ - the coefficients of slope angle between 0° and 90°,

$b$ - the coefficient representing weather conditions,

$\delta$ - Stefan-Boltzmann’s constant ($5.67 \times 10^{-8}$ $W m^{-2} K^{-4}$),

$T_a$ - the air temperature, K.

The radiation received from the ground is given by:

$$R_G(\alpha) = R_G \sin^2(\alpha/2)$$

(3)

where $R_G$ is the ground radiation in $Wm^{-2}$.

Cole (1976) published values of all coefficients for different angles of inclination. For the purpose of this study values of $R_A$ and $R_G$ was numerically modified. Using equations (2) and (3) with the modified $R_G$ coefficients the values of longwave radiation were computed for four angles of inclination and for two seasons. These were compared and correlated with measurements that were carried out on a real building in Wroclaw (51°), in a few days cycles in winter and summer. Exemplary results are shown in Figure 1 and 2. Figure 1 presents obtained data for winter conditions when the range of values of longwave radiation was from 180 to 340 $Wm^{-2}$ at the air temperature varied from -12.0 to 0.5 °C. Figure 2 presents the comparison for summer conditions; the range of values was from 200 to 450 $Wm^{-2}$ at the air temperatures varied from 10.0 to 24.5 °C.
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Fig. 1. Comparison of the observed and simulated downward longwave radiation, based on the modified Cole's model, incident upon surfaces inclined at 0° (horizontal), 30°, 60° and 90° (vertical), in winter in Wroclaw (51°N).

Fig. 2. Comparison of the observed and simulated downward longwave radiation, based on the modified Cole's model, incident upon surfaces inclined at 0° (horizontal), 30°, 60° and 90° (vertical), in summer in Wroclaw (51°N).

3. DISCUSSION AND CONCLUSIONS

The longwave radiative heat exchange between the outside surface of the building envelopes and the thermal environment may have a significant effect on the heat gains and losses from buildings. From the point of view of radiative heat losses, low-sloped roofs are exposed to much more unfavourable atmospheric radiation than walls. In the conditions of a clear sky at night the level or low-sloped roof, not overshadowed by other buildings or natural features, receives the smallest part of the longwave sky-line irradiance whereby its heat emission is compensated in the least degree. At this time, the radiative heat losses from this roof are the greatest, particularly that the atmosphere in the wavelength region from 8 to 14 μm is almost totally pervious to the longwave radiation (atmospheric window) and the maximum of the low-sloped roof outside surface radiation occur in the wavelength from 10 to 12 μm. As a result of these radiative heat losses the temperature of the...
outside surface of the roof may drop below the ambient air temperature (radiative cooling). In the case of external walls, the effect of the radiative heat exchange between them and the thermal environment is much more smaller since the heat losses by radiation are compensated by comparable gains of radiant energy originating from the atmosphere, the ground and any surrounding buildings.

The measured and calculated values of the longwave radiation agreed for all angles of inclination and weather conditions. The best statistical correlation was found for measurements obtained under a clouded sky (Figure 1). The measurements showed that the longwave radiation incident upon inclined building envelopes is almost independent of the inclination angle for overcast sky. However, as could be expected, for clear sky conditions the thermal radiation from the atmosphere is smaller for the vertical direction than for the horizontal direction. As a consequence the outside temperature of a horizontal roof drops below the temperature of walls of building in a clear night. This relationship can be important for various designs of radiative cooling systems. Predictably, the highest radiation intensities for clear skies were measured for vertical surfaces.

Figure 3 compares the modified Cole's model and the present data, with models by Idso and Jackson (1963), Roach (1955), Swinbank (1963), and Unsworth and Monteith (1971) calculated for horizontal surfaces at a range of air temperatures. For clear sky conditions the highest correlation was found for the model by Idso and Jackson (1969). For winter, the agreement is not so good, particularly in reference to the most frequently cited Swinbank's model. The differences between the two models were discussed in detail by Nowak (1989).

Fig. 3. Comparison of the longwave radiation incident on horizontal surfaces in winter and summer conditions, for clear and overcast skies, calculated from the modified Cole's model (circles) and other models (lines).

For an overcast sky, the modified Cole's model predicts values in much closer agreement to the models by Unsworth and Monteith (1971), Roach (1955) and the theoretical relationship for the black body model. The set of available measurements for an overcast sky was, however, limited to a narrow range of air temperatures, around 0 °C, and further measurements would be helpful.

The modified Cole's model will permit, among others things, a description of full particulars of the radiative heat transfer between the outside surface of walls and roofs and their thermal environment and can be used for modeling of various
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components of the heat balance of building envelopes (Nowak, 1991). This model could also be used for preparation of the distribution maps of the atmospheric radiation at the ground for given area including the atmospheric pollution influence for large cities areas. The model could be very helpful to proper shape of the spectrally selective surfaces for heating and cooling applications (including building envelopes), particularly surfaces with infrared-selective emission for radiative cooling to low temperatures (Eriksson et al., 1984). Moreover, this model could also be used for much more accurate calculation of the heat exchange in non-typical buildings, for example in greenhouses (Silva and Rosa, 1987).

In conclusion, an encouraging agreement was found between the Cole's model adopted to the meteorological conditions in the South-West region in Poland and a set of measurements of the longwave radiation incident on a surface, for different angles of inclination. It will be important to extend this validation to a much wider range of geographical regions and different levels of air pollutions in particular.

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1. Introduction

Air pollution with water causes enormous damage to existing building facades, whether of brick, natural stone, render or reinforced concrete. Winds also carry pollutants into and of the country that local industry and automobiles do not produce. Air pollution ratings are based on 9 factors; 3 of which are the most important:

- sulfur dioxide emission,
- the degree of rain acidity,
- the amount of acid rains.

The causes of damage on facade building materials are of a complex nature and experience shows that in practically all cases, different mechanisms interact. The most important causes of damage are produced by the weathering vehicle "water".

2. Building damage caused by dampness in masonry

Water penetrates mineral wall materials by different
absorption mechanisms and along with aggressive noxious substances from the air, causes the greater part of all facade damages. Damage causes are complex and interdependent: an object specific damage analysis is the most important step to be taken before each restoration project.

The following simplified descriptions outline the causes of damage:

1. Rising moisture through horizontally and vertically penetrating water.

   Rising moisture from ground levels is absorbed by capillary action and vises, along with dissolved salts, into the dry masonry zones. Here the water evaporates and the salt remains. This process takes place in the majority of buildings constructed in Wrocław up to 1945, because of the lack of horizontal and vertical waterproofing.

2. Hygroscopic moisture

   Hygroscopicity denotes the property of salts to absorb and bind water from the surrounding air. The higher humidity and degree of salting the more moisture is absorbed by building material.

3. Condensation

   This is the conversion of air water vapour into liquid on or in the masonry. This case is very general in buildings constructed up to 1980 because the thermal insulation of external walls is very low ($k > 1.16 \text{ W/m}^2 \text{ K}$).

4. Driving rain and splash water.

5. Laterally penetrating moisture from drain, slope and pressure water.

3. **Facade cleaning**

The cleaning methods may be subdivided into the following groups:
- methods using water without chemical additives,
- methods using water with chemical cleaners,
- wet and dry chemical.

The methods using water without chemical additives may be differentiated on 3 main groups:
- pressureless sprinkling with cold and warm water, cleaning with jets of warm water at various pressures and temperatures (60 bar/60°C, 90 bar/60°C, 150 bar/60°C, 60 bar/90°C, 90 bar/90°C) and cleaning with steam jets (30 bar/150°C).

Obviously, in the individual cases a pressure and temperature and time of cleaning of various facade is evaluated.

Obviously, the pressure, temperature and time of cleaning of individual facade should be evaluated separately, Adamowski (1989).

The evaluation of the effects of different cleaning methods is based on the interpretation of physical and chemical investigations as well as on a subjective description of the microscopic and macroscopic change of state. This includes:
- water absorption capacity,
- capillary water absorption,
- water penetration according to Karsten,
- water-soluble and acid-soluble components,
- the ion concentration of salts,
- the surface colour,
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- the roughness.
The respective success of cleaning may be described relatively well, quickly and cheaply by three characteristics:
  - the capillary water absorption as a measure of the opening of the pores,
  - the colour as directly visible feature, and
  - change of roughness as an indicator of the loss substance.

4. Facade impregnation

According to the laws of capillarity of hydrophobic water repelling treatment of a mineral material is understood to mean a modification on the wetting angle in relation to the building material surface. The wetting angle takes on values that are $> 90^\circ$. This results in a so-called negative capillary rise and means that under atmospheric pressure, water will not be absorbed by the capillaries, but repelled. The hydrophobic water repelling treatment is achieved by an impregnation, i.e. a saturation of the building material with a water repelling agent. Since almost 35 years silicon-organic compounds are used as hydrophobing and consolidating agents for facade impregnation. There is evidence that on buildings the durability of hydrophobic treatments is limited to 10-15 years. But, in several cases the treatment of natural stone facades with silicon-organic agents reveals problems like limited durability or even subsequent damage some years after application. Silicon-organic chemicals do not prevent the swelling and shrinking due to changes in humidity, so that the stone material is still affected by stress-strain

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processes. In the case of insufficient penetration depth of the agent, two zones of highly different mechanical and hygric behaviour are joined by a sharp borderline. Periodically changing external conditions may lead to a contour scaling of the hydrophobic layer. The swelling due to humidity may be reduced by pre-treatment with amino-functional surfacants. They show strong interactions with mineral surfaces and therefore they can be used to improve particular properties of silicon conservation products. Aminoalkyl silanes work as primers for polysiloxanes and for SiO$_2$-gel on clay mineral basal planes as well as on quartz surfaces. SiO$_2$-gels, modified by elastomeric dialkyl siloxane, show a low modulus of elasticity and remarkably reduce formation of shrinking fissures.

5. Drying of building walls by microwave energy

Group of scientists formerly of the Technical University of Wroclaw has established a company named "Plazmatronika". This group worked out quite original and safe technology of drying humidified walls by using high power microwave energy. The microwave method has the advantage in shortening the period of drying humidified walls - from months to days. Additionally, fungies, moss and alga etc. which are usually in humidified walls are completely destroyed in the microwave field (2.45 GHz). Services in the drying of building walls are followed by durable protection against water by the hydrophobic horizontal barrier injection which is made inside the wall. This method is specially recommended for cellarless rooms against rising masonry moisture.

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MODELLING OF WIND LOADS ON BUILDINGS

Parameters which determine the response of structures to wind loads.
A sensitivity study.

ir. C.P.W. Geurts

ABSTRACT: The fluctuating wind loads on buildings can be expressed in time domain and in frequency domain. The description in frequency domain as a spectrum is used in this project. A sensitivity study is executed to the wind velocity, the damping ratio, the coherence, the natural frequencies of the structure and the roughness length of the terrain.

INTRODUCTION

In 1992 the new Dutch code on structural loads, NEN 6702, the "TGB 1990, belastingen en vervormingen", was presented. The working group wind loads of the Royal Society of Engineers in the Netherlands, KIVI, wrote a comment on the parts that covers wind loads, containing the request for the limits of the field of application. In order to determine these limits, the project, reported here, was started. The first findings were presented in a graduation report (ref. 3). In this paper the sensitivity study which was carried out using a finite element computer program is presented. A short introduction into the subject is given first. Thereafter the procedure of the program is explained, and finally the results are presented. A discussion of these results concludes this paper.

WIND LOADS

In calculating the wind loads on buildings, the wind velocity, the turbulence, the aerodynamic and dynamic admittance and the drag coefficient have to be determined. This means that several errors are introduced. The schedule of calculation is provided after a short description of the parameters.
average wind speed

For the determination of the wind loads on buildings, estimations of the wind velocity at meteorological stations are used. The velocities are hourly mean wind speeds, that occur once in the reference period. These are estimated in standard circumstances, according to the definition of the World Meteorological Organisation: "measurements of wind for synoptic purposes should refer to a height of 10 m in an unobstructed area" (ref. 7). This means a roughness length of 0.03 m. This wind velocity is called potential. For the description of wind velocities at other heights than 10 m the loglaw is used, formula 1.

\[ U(h) = \frac{u^* \ln \left( \frac{h-d}{z_0} \right)}{\kappa} \]  

(1)

where:
- \( U(h) \) mean wind speed at height \( h \) (m/s)
- \( u^* \) friction velocity (m/s)
- \( \kappa \) von Karman constant, it has the value 0.4
- \( z_0 \) roughness length (m)
- \( d \) displacement height (m)

\( z_0 \) and \( d \) describe the roughness of the terrain. More information on these parameters can be found in ref. 7. The mean wind speed over other terrain roughnesses will be described later.

fluctuations of wind speed, wind loads and response

The fluctuations of the wind velocity can be described in time domain or in frequency domain, see fig. 1. The description in time domain cannot be used to predict the loads in future, because of the stochastic properties of the wind. In this project the representation in frequency-domain in the form of a spectral analysis is used.

Several descriptions of the spectrum \( S_{uu} \) of the longitudinal wind velocity fluctuations, as shown on the right in fig. 1, are known. These are compared during the project.

The wind is not fully correlated in time and space. In order to account for the gustiness of the wind, the coherence is introduced. Davenport (ref. 2) suggested a non-dimensional exponential function for the description of the coherence. This function describes the spatial correlation between the wind velocity in two points. It is a function of height, width, depth or a function of the absolute distance between the two points.

\[ \text{coh}(n) = e^{\frac{-C.n.h}{\pi}} \]  

(2)

in which:

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Various ways in describing the coherence are available in literature. In every description the definitions of \( u, H, C_x \) and/or \( C_y \) are different. More information and a comparison between different descriptions can be found in ref. 3.

The fluctuating wind velocity leads to fluctuating wind loads in a point, according to formula 3.

\[
S_f(n) = \rho U_{\text{mean}}^2 A^2 C_D^2 S_u(n)
\]  

in which:

- \( S_f(n) \) autospectrum of wind loads, as a function of frequency
- \( \rho \) density of the air
- \( U_{\text{mean}} \) mean wind velocity
- \( A \) loaded area, for which the point is representative
- \( C_D \) drag coefficient
- \( S_u(n) \) spectrum of wind velocity fluctuations, as function of frequency
The relation between the loads in two points, 1 and 2, is given in formula 4.

\[ S_{FL2}(n) = \gamma^2 \sqrt{S_{F1}(n)S_{F2}(n)} \]  

(4)

in which:

- \( S_{FL2}(n) \) cross-correlation function
- \( \gamma^2(n,n) \) coherence function, a function of frequency and height
- \( S_{F1}(n), S_{F2}(n) \) auto-correlation function in point 1 and 2

The fluctuating response, in this study displacements are calculated, is found by integrating formula 4 over the total loaded area and by multiplying the result with the dynamic admittance function \( H^2(n) \). An example of this function is given in formula 5.

\[ H^2(n) = \frac{1}{16\pi^4 n_i^4 M_i^2 [(1-(\frac{n_i}{n})^2)^2 + 4(\frac{\xi}{n_i})^2]} \]  

(5)

- \( n_i \) natural frequency of the building in the \( i \)th natural mode (Hz)
- \( M_i \) mass in the \( i \)th natural mode (kg)
- \( \xi \) damping ratio (%) 

The schedule of the procedure, described above, is given in textframe 1.
NUMERICAL SIMULATION PROCEDURE

The procedure of textframe 1 is implemented in a computer program, called DYNRES, which was developed by TNO, Department of Buildings (ref. 1). This program is an annex to the finite element program DIANA. The purpose of DYNRES is to account for the incomplete spatial correlation of wind velocities and pressures between the loadingpoints. Until now, it is not possible to do this directly in a finite element program.

DYNRES, which was developed for calculating the response of traffic-light portals, has been extended for calculating the response of 3-D structures, such as buildings.

The program calculates the response of a structure, using the formulas of coherence and spectra. The structure has to be modelled with DIANA. The element-matrices and the eigenfrequencies and eigenmodes are calculated with DIANA. These results are used as input for DYNRES. The program also needs the following input:

- terrain roughness
- form of the spectrum and form of the coherence function
- drag coefficient and loaded area for every loadingpoint
- density of air
- orientation of the building, absolute.
- wind direction, absolute

INITIAL CONSIDERATIONS

Before proceeding to the sensitivity study, we must know what portion the dynamic part of the response has in the total response. For several case-studies calculations of the response are made. The average displacements of the top of the buildings and the maximum fluctuating displacement were estimated. In table I the results of these calculations are presented. In the two columns at the right, the relative parts of the static and fluctuating response as a percentage of the total displacement are listed.

It is seen from table I that about half of the maximum displacement of the top of the buildings, loaded by wind with high velocities, is due to the dynamic part.

In ref. 4 a sensitivity study is carried out by Littler for the parameters eigenfrequency, mass, damping ratio and wind velocity. A comparison is made between full-scale measurements, wind tunnel tests and calculations for a building in London. The results of this research are in good agreement with the results of the calculations in ref. 3. The measurements made by Littler concentrate on the determination of the acceleration. In ref. 3 only displacements were calculated.
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Table I: examples of calculations of the static and fluctuating part of the displacements on top of some buildings

<table>
<thead>
<tr>
<th></th>
<th>( u^* \text{(m/s)} )</th>
<th>( X_{\text{st}} )</th>
<th>( X_{\text{fl}} )</th>
<th>( X_{\text{fl}}/X_{\text{st}} )</th>
<th>%_{\text{st}}</th>
<th>%_{\text{fl}}</th>
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<td>flat Simiu 2 (fig. 2)</td>
<td>2.98</td>
<td>0.288</td>
<td>0.184</td>
<td>0.452</td>
<td>59.3</td>
<td>40.7</td>
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<tr>
<td>flat Delft zuid (fig. 3 &amp; 4)</td>
<td>2.82</td>
<td>0.188.10^1</td>
<td>0.153.10^1</td>
<td>0.342.10^1</td>
<td>55.3</td>
<td>44.7</td>
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<tr>
<td>Waseda University</td>
<td>2.2</td>
<td>5.48.10^3</td>
<td>5.104.10^3</td>
<td>1.058.10^3</td>
<td>51.8</td>
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<td>Commerce Court</td>
<td>2.4</td>
<td>3.006.10^1</td>
<td>2.652.10^1</td>
<td>5.658.10^1</td>
<td>53.1</td>
<td>46.9</td>
</tr>
<tr>
<td>190 meter Tower</td>
<td>2.8</td>
<td>3.339.10^2</td>
<td>3.381.10^2</td>
<td>6.72.10^2</td>
<td>49.7</td>
<td>50.3</td>
</tr>
<tr>
<td>Yokohama KN</td>
<td>2.03</td>
<td>6.040.10^3</td>
<td>5.372.10^3</td>
<td>1.141.10^2</td>
<td>52.9</td>
<td>47.1</td>
</tr>
<tr>
<td>Simiu 1</td>
<td>2.81</td>
<td>8.161.10^1</td>
<td>1.031</td>
<td>1.847</td>
<td>44.2</td>
<td>55.8</td>
</tr>
<tr>
<td>Simiu 3</td>
<td>2.81</td>
<td>1.385.10^1</td>
<td>1.444.10^1</td>
<td>2.829.10^1</td>
<td>49</td>
<td>51.0</td>
</tr>
<tr>
<td>Simiu 4</td>
<td>2.81</td>
<td>3.565.10^3</td>
<td>3.65.10^3</td>
<td>7.215.10^3</td>
<td>49.4</td>
<td>50.6</td>
</tr>
</tbody>
</table>

SENSITIVITY STUDY

In order to determine the accuracy of the Dutch code on wind loads a sensitivity study was carried out, varying the following parameters:
- expression for the spectra of wind velocity fluctuations \( S_{\text{ww}} \)
- expression for the coherence \( \text{Coh}(n) \)
- the exponential parameter in the expression for the coherence \( \text{C}_x \) or \( \text{C}_y \)
- friction velocity \( u^* \)
- damping ratio \( \zeta \)

damping, friction velocity, coherence

It is shown in fig. 2 that the influence of damping ratio and friction velocity is important. It is also important to determine \( \text{C}_x \) and \( \text{C}_y \) accurate.

From these results it can be concluded that the relation between \( u^* \) and \( \sigma_2^2 \) can be expressed in the form of a power law (formula 6).

\[
\frac{u^*}{u_2} = \left( \frac{\sigma_1^2}{\sigma_2^2} \right)^{\alpha}
\]  

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It is found, that $\alpha = 5.45$ suits the calculated results best. In ref. 4 a power-law constant $1/2 \alpha$ between 2.7 and 3.3, is found. This is of the same order of magnitude.

The relation between $\zeta$ and $\sigma^2$ can also be expressed in the form of a power-law, formula 7.

$$\frac{\zeta_1}{\zeta_2} = \left(\frac{\sigma_1^2}{\sigma_2^2}\right)^{\alpha}$$  \hspace{1cm} (7)

A value $\alpha = -1$ suits the results best. This is in accordance to the definition of the dynamic admittance function (formula 4).

The exponential coefficients $C_x$ and $C_y$ in the coherence function are determined out of tests, in which a wide range of coefficients is found. For the formulas a mean value is chosen. In fig. 2 only the sensitivity to $C_y$ is given. If the real coefficient is 50% of the assumption, then underestimations of the fluctuating displacement of 20% can be found, using the assumed value.
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Eigenfrequency
For an existing building, see tabel I, "Delft zuid", the structure was analysed and a calculation of the eigenfrequencies was made. Results of further calculations with DYNRES, using these design-state eigenfrequencies are compared with calculations with DYNRES, using the measured, ready-state, eigenfrequencies. The results for the spectrum $S_x$ of displacements is given in fig. 3. The variance of the displacements can be calculated out of this figure. It it found that the variance in design-state is 40% higher than the variance in ready-state.

![Graph showing the difference between design state and ready state.](image)

**fig. 3:** *Comparison of calculated response, using design-state and ready-state values*

Roughness length
The sensitivity to the roughness length on the variance of the displacement in the calculation of the response, due to the determination of $u^*(\text{rough terrain})$ out of the potential velocity, using formula 8, is given in fig. 4.

$$u^*_r = 0.4 \frac{u(60)}{\ln\left(\frac{60}{z_0}\right)}$$  \hspace{1cm} (8)

It is seen that for low values of roughness length the response is rather sensitive to changes in roughness. For $z_0 > 0.5$ m the sensitivity is small.

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**Fig. 4:** sensitivity of roughness length, using formula 7

In the Dutch code on wind loads, NEN 6702, values for the roughness length are awarded to three different areas in the Netherlands. In ref. 7 a review of measured values of roughness lengths is given. Comparing ref. 7 with NEN 6702 shows differences between code and measurements. The values of the code and the upper and lower limits from ref. 7 are given in table II, below.

**Table II:** values of roughness lengths (m) according to NEN 6702 and according to Wieringa (ref. 7)

<table>
<thead>
<tr>
<th>Description</th>
<th>Assumption</th>
<th>Upper Limit $^1$</th>
<th>Lower Limit $^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NEN 6702, area 1, no buildings</td>
<td>0.1</td>
<td>0.29</td>
<td>0.06</td>
</tr>
<tr>
<td>NEN 6702, area 2, no buildings</td>
<td>0.2</td>
<td>0.29</td>
<td>0.06</td>
</tr>
<tr>
<td>NEN 6702, area 3, no buildings</td>
<td>0.3</td>
<td>0.49</td>
<td>0.16</td>
</tr>
<tr>
<td>Coast of Holland (1 or 2, NEN 6702 $^2$)</td>
<td>0.1 or 0.2</td>
<td>Not of interest</td>
<td>&lt; 0.05</td>
</tr>
</tbody>
</table>

$^1$ from ref. 7

$^2$ the coastal area is not taken into consideration within the three classes of NEN 6702, but only in the fourth line of the table.

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In ref. 6 a classification table for the roughness length is given. The assumption is made that in determining the roughness length a maximum error of one classification-range is made. This means, that instead of the upper limit of 0.29 m also a value of 0.5 m can occur in area 2 and instead of \( z_0 = 0.49 \) meter a value of 1.0 meter can be realistic in area 3. In the following the values of tabel II are used.

**Effect of roughness length on both static and dynamic part of the displacements.**

The effect on the static part of the response of changes in roughness length is contrary to that on the dynamic part. When the roughness length increases, the wind experience more friction. This means, that the mean velocity will decrease and therefore the static part of the displacement will decrease. On the contrary, more turbulence is generated and therefore more fluctuations occur. This means that the dynamic part of the displacements will become more important. The maximum fluctuating displacement is proportional to the square-root of the variance of the displacements.

In table III the errors are listed following calculations with the upper and lower limits of table II, relative to the response calculated with the 'NEN-value'.

**Table III: influence of assumptions of roughness length for non-built-up area's**

for NEN 6702 on displacements

<table>
<thead>
<tr>
<th>area</th>
<th>influence on the displacement</th>
<th>static</th>
<th>dynamic</th>
<th>total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>upper limit</td>
<td>lower limit</td>
<td>upper limit</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td>-14%</td>
<td>+7.5%</td>
<td>+10%</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>-5%</td>
<td>+25%</td>
<td>+4%</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>-7%</td>
<td>+9%</td>
<td>+8%</td>
</tr>
<tr>
<td>coast</td>
<td></td>
<td>xx</td>
<td>+ &gt; 100%</td>
<td>xx</td>
</tr>
</tbody>
</table>

It is seen, that in the areas 1, 2, and 3 the errors, assuming that the limits in tabel II are correct, are very small. In the coastal area it is possible, that errors of the windpressure, and so of the displacements, occur of about 10 % and more. In the Dutch code on wind loads of 1974, NEN 3850, a coastal area was defined in which this effect was counted for. In NEN 6702 the coast is no longer a separate area. This means, that close to the sea special attention should be given to the determination of the wind loads, especially for high buildings.
REMARKS

It is known that the determination of the maximum hourly-mean wind velocity at a certain place in the Netherlands includes errors, because of the interpretation of findings at weather stations at other places. These errors can be in the order of 20%. An underestimation of 20% in the wind velocity leads to an underestimation of the response of about 40%.

The effect on the response of the calculated eigenfrequency is tested by comparison with the measured response of an existing building. This is done for only one building, so the results may not be representative for other buildings. Further research on this aspect should be done.

Table II and III give results for non-built-up areas. The analysis can also be carried out for built-up areas. NEN 6702 gives a roughness length of 0.7 m. This means that, especially in the coastal area, for cities near the sea, the influence on the response is even greater. More details are provided in ref. 3.

CONCLUSIONS

This study makes clear that for the determination of the fluctuating displacement several parameters are of importance. The most important of them are the wind velocity, the damping ratio, and those parameters, which determine the spatial coherence. A further study into the influence of the eigenfrequency has to be undertaken. Other research shows that the estimation of the first eigenfrequency is also of great interest.

It is seen in ref. 3 that the formulas describing the spectra and coherence are different in accuracy. This leads to uncertainties in the calculations. Further research of spectra and correlation of wind pressure on an object is necessary to understand this subject.

The dependence of the response on the roughness length is taken into consideration. A longer roughness length means a smaller average wind velocity, but larger fluctuations. This means, that the effects on the maximum displacements are opposite.

The Dutch code on wind loads does not define a coastal area. It is seen, that for high buildings in this region this can lead to underestimations of the wind loads of about 10 to 30%.

A selection of buildings in the design-stadium has to be made. The natural frequencies can be calculated from the structural properties. This has to be compared with an estimation of the natural frequencies. The calculated response has to be compared with the measured response.
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Abstract

In order to protect old leaded windows of historical churches against corrosion by acid rain, air pollution and vandalism, protective glazing is placed at the outer side of the original windows. An extra advantage is the smaller heat loss by transmission. Situating the protecting window at the outer side of the original window induces an extra possibility for surface condensation. Depending on whether the cavity is naturally ventilated with warm indoor church air, or cold outdoor air, surface condensation may occur on the outside protecting glazing or on the inside original glazing. This contribution describes a developed computer model to predict the probability of surface condensation on the original or protecting glazing. The model was validated by measurements at an experimental model in a so called hot-box and by in situ measurements in a church in Germany.
Introduction

From the beginning of this century a decline of the condition of the glass in lead windows has been ascertained. While most of the historical windows were not altered by centuries, the last decades most of the windows have been affected by corrosion (fig. 1). No intervention therefore leads to degradation and loss of valuable glass paintings. The cause of the degradation must be explained by the increasing pollution of the air, heating of the church and natural age.

Processes of corrosion are accelerated by the presence of water. Rain and condensation lead to chemical deterioration at the surface of the glazing. Therefore water, especially when polluted, is the natural enemy of valuable glass paintings. One of the most effective ways to protect the glazing is to place a transparent glazing at the outside of the original glazing: the original glazing is moved to the inside and the protecting glazing is located at its original place (fig. 2). In this way acid rain no longer is in contact with the original glazing. The additional advantage is protection against vandalism. Besides the heat loss by transmission decreases.

That is why this way of protecting valuable old glazing is practised all over the world. However, a lot of mistakes have been made by doing so: especially closed cavities between the glazing have lead to serious problems of condensation.

Therefore it is a well known practice to ventilate the cavity with air. However there are two opposite opinions of ways to do so: In some European countries (England e.g.) measurements of surface temperatures, relative humidities and condensation quantities lead to the preference of outside ventilation [Gib84], while elsewhere in Europe (Holland and Germany e.g.) inside ventilation was chosen to be a safe, well known practice [Jut84].

This is the reason why we chose for a more physical, modelling approach in this research.
The window system

The physical operation of a naturally ventilated cavity with outside openings can be described in the following way: relative cold outdoor air enters the cavity at the bottom openings of the outside glazing; it warms up, decreases in weight and rises inside the cavity. While the air temperature inside the cavity increases, relative humidity decreases: therefore the chance on surface condensation inside the cavity decreases. However, the entering of cold outside air cools down the original valuable glazing and increases the chance on surface condensation at the inside bottom of it (fig. 3a).

Ventilation of the cavity with warm indoor air seems to be more unfavourable: the (absolute) moist air enters the cavity at the top opening of the inside glazing and cools down at the cold outside glazing: its increasing density sinks it down and the relative humidity increases. Surface condensation on the outside glazing may be the result. The valuable inside glazing however remains warm and no surface condensation appears on it (fig. 3b).
If the two glazings were equal (and non valuable) the choice was simple: inside ventilation more rapidly leads to surface condensation than outside ventilation. The prevention of surface condensation however on the more valuable inside glazing makes it to a more difficult problem: when and how often does surface condensation on the inside glass occur and what is the effect of a moist climate in the cavity on the original glass?

Physical models

The problem to be solved was divided into two parts: description of a physical model to describe the micro-climate near the window system and in its cavity as a function of the in- and outdoor climate and description of a physical model to describe the climate in the whole church as a function of the outside climate and the use of the church. Coupling of the two models describes the interaction of window system and indoor church climate and therefore the response of both to the outdoor climate.

Physical model of the window system

The physical model to describe the naturally ventilated window system was taken from [Rei83]. While surface temperatures and cavity air temperature change along the height of the cavity the window was divided into small segments along the height of the window. For each segment a heat and mass balance was described.

In figure 4 one segment is represented.
We recognise the following symbols:

- \( q_1 \): heat flux at the inside glazing [W/m\(^2\)]
- \( q_e \): heat flux at the outside glazing [W/m\(^2\)]
- \( q_h \): heat flux connected with heating or cooling the air [W/m\(^2\)]
- \( q_r \): heat flux by radiation between the glazing [W/m\(^2\)]
- \( q_{c1} \): convective heat transfer in the cavity at the outside glazing [W/m\(^2\)]
- \( q_{c2} \): the same at the inside glazing [W/m\(^2\)]
- \( q_{a1} \): absorbed irradiance of the sun at the inside glazing [W/m\(^2\)]
- \( q_{a2} \): the same at the inside glazing [W/m\(^2\)]
- \( q_h \): longwave heat exchange at night [W/m\(^2\)]
- \( T_i \): inside air temperature [°C]
- \( T_e \): outside air temperature [°C]
- \( T_{s1} \): surface temperature of the outside glazing [°C]
- \( T_{s2} \): the same of the original inside glazing [°C]
- \( T_{m} \): air temperature at the middle of the cavity [°C]
- \( T_{b1} \): the same at the bottom of the segment [°C]
- \( T_{b2} \): the same at the top of the segment [°C]
- \( T_h \): temperature of the sky [°C]
- \( h \): height of the segment [m]
- \( d \): depth of the cavity [m]

For each segment the following heat balances can be described:

- **The heat balance for the whole segment:**
  
  \[ q_1 + q_{c1} + q_{c2} = q_e + q_h \]

- **The heat balance for the inside glazing:**
  
  \[ q_1 + q_{a2} = q_{c2} + q_r \]

- **The heat balance for the outside glazing:**
  
  \[ q_{c1} + q_e + q_{a1} = q_e + q_h \]

The heat exchange by radiation between the glass plates was linearized and the convective heat transfer in the cavity was described by an empirical relation, taken from [Hoe87].

To calculate the (mean) air velocity another equation was needed: the pressure drop in the cavity has to equal the pressure difference due to the density differences at the inlet and outlet.
The calculation process was iterative: an initial guess of the mean air velocity was followed by the calculation of air- and surface temperatures, pressure drop in the cavity and pressure difference at in- and outlet, and the velocity was changed until equilibrium was reached.

**Experimental validation of the window model**

The computer model was validated by stationary measurements at a laboratory model of the window system in a so called hot-box/cold-box and in situ in a real church in Keyenberg Germany.

**The hot box-cold box**

The hot-box/cold-box is a laboratory instrument, consisting of a relative warm- and cold climate chamber (figure 5). A model consisting of three window systems was placed between the chambers: inside, outside and mix ventilation. In the warm climate chamber the stationary indoor climate of a heated church was simulated (air temperature 16°C, relative humidity 60 %). In the cold climate chamber the air temperature was lowered until surface condensation was reached in the window systems.

![Diagram of hot-box/cold-box](image)

**Fig. 5: Hot-box/cold-box in front view**

The following physical quantities were measured:
- Air temperatures and relative humidities inside the climate chambers;
- surface and cavity air temperatures of the three systems;
- air velocities and relative air humidities inside the cavities;
- heat fluxes at the surfaces of the glass plates.

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Inside ventilation
Outside ventilation
Mixed ventilation

Fig. 6: Front view and sections of experimental set-up

a) Vertical section Λ-Λ'
b) Vertical section test wall
c) Horizontal section test wall

Fig. 7: Photo of test wall in hot-box/cold-box

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Results

Figure 10a gives the outside ventilation results of the surface temperatures as a function of the outdoor air temperature as they were measured and calculated in and for the hot-box/cold-box. At an outdoor temperature of below -1 °C surface condensation at the inside glazing was detected.

Figure 10b gives the results of the inside ventilation model. Already at outdoor temperatures of below 6 °C condensation was detected at the cavity side of the outside glazing. The temperature of the original inside glazing however remained above the dewpoint. Figures 10c and 10d give the corresponding temperatures of the cavity air. Figures 10e and 10f show the measured and calculated air velocities.

In situ measurement in a church

In a church in Keyenberg Germany measurements were carried out at two window systems (in- and outside ventilation) at two different orientations: north and south. Besides the measurements which were done in the laboratory model the outdoor climate and the irradiance of the sun were measured (fig. 8 and 9).
Results

Figure 11a shows the measured and calculated surface temperatures of an outside ventilated outside glazing. Figure 11b shows the corresponding surface temperatures of the original inside glazing. As was expected a large measurement error occurred measuring the surface temperature of the glazing which was shined by the sun. Due to practical reasons the PT100’s could not be masked for sun irradiance. As the goal of this research was surface condensation due to low surface temperatures, the measuring error at higher surface temperatures was accepted. Comparison of the measured and calculated surface temperatures showed differences up to 2 K; the standard deviation of the mean 15 minutes taken measurements was about 1 K.

Wind effects on the outside ventilation system were not modelled yet; dynamical pressure differences at in- and outlet may have a significant effect on air velocities in the cavity.

Fig. 11: 'in situ' results
a) Surface temperatures protective glazing at a height of 0.2 m (outside ventilation)

b) Surface temperatures leaded glazing at a height of 0.2 m (outside ventilation)
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Outside ventilation: height = 0.24 m

Inside ventilation: height = 0.24 m

Outside ventilation: $T_i = 15.7 \, ^\circ C \ T_e = -5.5 \, ^\circ C$

Inside ventilation: $T_i = 15.7 \, ^\circ C \ T_e = -5.5 \, ^\circ C$

---

Fig. 10: Results from experiments in hot-box/cold-box

a) Surface temperatures as a function of outside temperature (outside ventilation)

b) Surface temperatures as a function of outside temperature (inside ventilation)

c) Cavity air temperatures as a function of height (outside ventilation)

d) Cavity air temperatures as a function of height (inside ventilation)

e) Air velocity in cavity as a function of outside temperature (outside ventilation)

f) Air velocity in cavity as a function of outside temperature (inside ventilation)
Conclusion

The presented calculation model gives a reasonable prediction of the expected surface temperatures of the glazing and air velocities in the cavity of naturally ventilated window systems. It therefore can be used as an instrument to determine the choice of a naturally ventilated window system to protect valuable church windows.

Placing the outside glazing however could introduce an effect on the inside climate of the church: air infiltration at the original leak glass-in-lead window could change significantly by inside ventilated systems.

At this moment a thermal and hygrical model of the church as a whole system has been finished. A comparison with measured data of several churches over the period of several years takes place at the moment. Comparison with measured data of other researchers already showed promising results.

At the next meeting I hope to show you more of it!
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Energy Simulation in Building Design

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Abstract
Design decision support related to building energy consumption and/or indoor climate, should be based on an integral approach of environment, building, heating, ventilating and air-conditioning (HVAC) system and occupants. The tools to achieve this are now available in the form of computer simulation systems which treat the building and plant as an integrated, dynamic system. Although its potentials reach beyond the area of Computer Aided Building Design, the paper describes building and plant energy simulation simulation within the context of CABD, design decision support and design evaluation.
Currently, computer simulation is only used indirectly as a design decision support mechanism; ie its power is not delivered very efficiently to the design profession. Future research directions are indicated, aimed at providing a mechanism to overcome this problem by developing an "intelligent front end" which bridges the gap between sophisticated computer simulation tools and the design profession.

INTRODUCTION
The dynamic thermal interaction, under the influence of occupant behaviour and outdoor climate, between the building and its heating, ventilating and air conditioning (HVAC) system is still difficult to predict. In practice, this often results in non-optimal, malfunctioning, or even "wrong" building/system combinations. Other topics belonging to the same problem domain are (in no particular order): Sick Building Syndrome, Building Energy Management Systems, application of passive solar energy, HVAC system and control development and testing, integrated systems (eg floor heating, ice rinks, swimming pools), and unusual building/system combinations which may occur for instance when a historical building finds a new use (eg a church being converted into a multi-purpose centre) or in case of relatively new developments like atria.
The above mentioned problems and the need for an integral design approach of building, HVAC system and occupants, are becoming more and more important. Therefor a research project was initiated on development/enhancement of building performance evaluation tools which treat the building and plant as an integrated, dynamic system (Hensen 1991).
One of the techniques available, is modelling and simulation.† Currently the most powerful tool available

† Modelling is the art of developing a model which faithfully represents a complex system. Simulation is the process of using the model to analyze and predict the behaviour of the real system.
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for the analysis and design of complex systems, is computer simulation. Modelling and simulation have become indispensable engineering techniques in the fields of design (e.g., buildings, plant configurations, and on the component level) and operation (system control, understanding, and interaction). The main reasons for this are that these techniques offer vast advantages - over for example experimentation - with respect to:
- economy; in an increasing number of cases, simulation is faster, better and cheaper than experimentation,
- prediction; allows analysis of a (model of a) system which does not yet exist, and
- education; models are easily adapted, inexpensive to operate, able to simulate adverse conditions and may also serve as an aid in communication.

Of course simulation and experimentation are effectively complementary; experimentation to discover new unknown phenomena or for validation purposes, and simulation to understand interactions of the known components of a system.

In the current context, modelling and simulation is thus used for predictions in order to support design decisions on real world problems regarding buildings and the HVAC systems which service them. The building in question may be an existing structure, a proposed modification of an existing structure, or a new design.

Although the potentials of building and plant energy simulation reach beyond the area of Computer Aided Building Design, here we will describe simulation within the context of CABD, decision support and design evaluation.

THE CABD CONTEXT

Since the early 1960's, the use of computers in the field of building design - i.e. CAD (Computer Aided Design) which was only later specified to CABD (Computer Aided Building Design) - has been increasing steadily (Rooney and Steadman 1987). Although, according to Gero (1983), its potential has and is taking longer to realise than was first thought. This statement is still true, judging from comparison of Gero's predictions for the then immediate future of 1983 with the actual situation at present (see e.g. Ratford 1991).

Having mentioned this, CAD in the field of building design has received more and more attention, both from research and commercial communities. Due to economic factors, the draughting function has received the most - commercial - attention and is now becoming well established in building design practices.

The design process itself is very complicated, as may be concluded from the vast amount of work aimed at establishing models of the process of design. As Butera (1990) points out, the architectural design process may even be approached using principles from the so-called "deterministic chaos" theory. The complexity and the diversity of parameters to be taken into account leave large opportunities to chance in identifying the design optimum. The optimum may be regarded as a "strange attractor" in this context.

Due to its complexity, general software to aid in the design process, is much less developed and received much less attention than draughting and design process management tools. In recent years promising research activities have been - or are about to be - initiated aimed at relieving this deficiency. These studies often involve pluri-disciplinary research teams and employ very sophisticated research techniques (e.g. Dubois 1990, and Clarke and Duffy et al. 1991).

There is however one activity throughout the design process which has received much attention: building performance appraisal. Powerful, computer-based models were created to assess cost, performance and visual impact issues in design: from life-cycle cost estimates at the design proposal stage, through realistic visualisations of the design, to comprehensive evaluations of building energy and environmental performance. A demand for systems which possess both draughting and appraisal functions is however steadily growing. In response, appraisal programs were appended to draughting packages, thus creating what we may call early CABD systems.
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As elaborated by Clarke (1989), CABD, and the sub-systems which it comprises, are affected by continuous changes in: power and cost of hardware, quality of software, elegance and effectiveness of human-computer interfaces (HCI), and user interface management systems (UIMS), and in (computer aided) software engineering (CASE) methods with which a greater degree of sub-system integration is possible. Table 1 (Clarke 1989), which is self-explanatory, summarizes several of the important issues in this respect.

Table 1 Issues underlying CABD evolution (Clarke 1989)

<table>
<thead>
<tr>
<th>Issue</th>
<th>Time Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Immediate (now)</td>
</tr>
<tr>
<td>Technology</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Micros</td>
</tr>
<tr>
<td></td>
<td>• Drafting</td>
</tr>
<tr>
<td></td>
<td>• Early performance</td>
</tr>
<tr>
<td></td>
<td>• Prediction</td>
</tr>
<tr>
<td>Applications</td>
<td>• Drafting</td>
</tr>
<tr>
<td></td>
<td>• Information</td>
</tr>
<tr>
<td></td>
<td>• Technology</td>
</tr>
<tr>
<td></td>
<td>• Performance</td>
</tr>
<tr>
<td></td>
<td>• Prediction</td>
</tr>
<tr>
<td>Education and training</td>
<td>• Applications</td>
</tr>
<tr>
<td></td>
<td>• Exploration</td>
</tr>
<tr>
<td></td>
<td>• Hardware</td>
</tr>
<tr>
<td></td>
<td>• Familiarisation</td>
</tr>
<tr>
<td>Research</td>
<td>• Application</td>
</tr>
<tr>
<td></td>
<td>• Knowledge</td>
</tr>
<tr>
<td></td>
<td>• Validation</td>
</tr>
<tr>
<td></td>
<td>• Systems</td>
</tr>
<tr>
<td></td>
<td>evaluation</td>
</tr>
<tr>
<td>Impact</td>
<td>• Expensive &amp;</td>
</tr>
<tr>
<td></td>
<td>• time consuming</td>
</tr>
<tr>
<td></td>
<td>• Job shifts</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Net result</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

When this table is projected on the actual situation at present, it seems that we are already in the short-term or perhaps even mid-term columns as far as the technology is concerned. This is due to recent technological and economical developments: ie relatively inexpensive, high performance, graphics workstations, strong reduction of data storage costs, and emergence of early expert systems (see eg Mac Randal 1988). To further illustrate this: at the start of the present work (late 1986) a high-resolution, bit-mapped, graphics workstation, offering a performance of 1.5 Mips (million instructions per second) and 70 Mbyte data storage, was purchased for approximately 20 KECU (= £1 50000). Now, 5 years later, two new workstations have been ordered one of which is only half the price and offers a 15 Mips performance, and another which still costs 20 KECU but offers 28 Mips performance and 1 Gbyte of disk storage capacity, instead.

As another exemplification of fast developing technology consider the following quote from Hartman (1988), which in addition illustrates usage of worldwide networking as may be deducted from the
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With respect to applications, in 1992, we still seem to be in the "immediate" column, except perhaps for 3-D visualisation which appears to be the next commercial goal (ie following draughting). The same is true for education and training, where as yet, only few educational institutions offer in-depth postgraduate training.

Regardless of technological developments, C ABD will not become commonplace unless there is a high standard of user training. Although, as pointed out by Clarke (1989), it could be that ultimately user training becomes less important due to high level assistance by the computer. This does not imply however, that setting-up of education and training schemes is not of the utmost importance now.

With respect to research, up to now the majority of activity is directed towards proving the system and towards the acquisition of application knowledge. In the field of building energy simulation for example, a number of projects concerned with model validation have been or are being carried out. Currently there are indications that the building energy simulation research activity is broadening in its scope (see eg Augenbroe and Laret 1989, and Clarke and Maver 1991). More effort is being expended on human-orientated C ABD, through expert systems, HCI research and the like (eg Clarke and Rutherford et al. 1989). There is also a greater tendency to approach the problems underlying C ABD in a multi-disciplinary, inter-institutional manner (eg Clarke and Hirsch et al. 1986, Clarke and Irving et al. 1988, and Augenbroe and Winkelmann 1990). This is also reflected in the recent formation of building analysis clubs: International Building Performance Simulation Association (IBPSA) based in the United States, Building Environmental Performance Analysis Club (BEPAC) in the United Kingdom and Building Analysis Groups (BAG) in the Benelux.

C ABD is not a remedy for all difficulties. At worst it is an automation of much of the mechanics of design. At best it allows an evaluation of the relationships inherent in a given design hypothesis. At present, the application of C ABD is expensive, in terms of required human resource, and time consuming. In the near future the profession will probably experience a skills shortage. In the longer term, however, with further advances in technology, application knowledge and education and training, C ABD might bring important changes in the design process, involving de-skilling and the breakdown of professional boundaries. C ABD could well become the common denominator of all parties involved in the design process, through some future IBDS (Integrated Intelligent Building Design System). This will lower or even remove inter-professional barriers and improve the quality of the end product, the building.

As indicated, C ABD is an evolutionary process which is characterised by several strong interrelations between quite different issues. For example, the level of application of energy simulation is as much a function of education and training as it is of hardware and HCI. Of course, C ABD must also be regarded in the light of other technological and other developments which are taking place around us. That is, C ABD will certainly become integrated in the "office of the future" which might offer multi-media personal work environments, incorporating integrated CAD/CAE (Computer Aided Engineering) features. One important issue not yet mentioned yet is that of politics. There are trends towards requiring that specified conditions must be achieved during the operation of buildings as well as in the design of buildings (as in ASHRAE's 1989 standard on ventilation and air quality, and towards setting up "responsibility chains", ultimately making a design team liable for the performance of the end product (as implied in for instance ASHRAE's (1989) guideline for commissioning of HVAC systems). It could well be that if these
trends are followed and accepted by the building industry, such issues will become major catalysts in the evolution of CABD. Building energy simulation must be placed within this evolutionary context. Contemporary energy models are an important improvement compared with the traditional methods they replace. However, there are still several important developments which must be undertaken before valid, easy to use models can be delivered to the design profession.

BUILDING ENERGY SIMULATION

As indicated in the introduction, modelling and simulation have become popular engineering tools since they permit us to predict the behaviour of a system before the conditions we are interested in occur, and indeed even without the system actually existing. In fact, modelling and simulation are the only techniques available that allow us to analyze arbitrarily non-linear systems accurately and under varying experimental conditions.

Simulation is used in many areas of science and engineering. It is used in different senses to study a variety of systems which may be classified as: continuous vs discrete, deterministic vs stochastic, or dynamic vs steady-state. It may be clear that building energy simulation addresses very complicated, highly interacting, continuous, deterministic, dynamic systems.

Table 2 The evolution of building energy models (Clarke 1988)

<table>
<thead>
<tr>
<th>Generation</th>
<th>Handbook oriented</th>
<th>Indicative</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st generation</td>
<td>Analytical in formulation</td>
<td>Application limited</td>
</tr>
<tr>
<td></td>
<td>As simplified as possible</td>
<td>Difficult to use</td>
</tr>
<tr>
<td></td>
<td>Piece meal in approach</td>
<td></td>
</tr>
<tr>
<td>2nd generation</td>
<td>Dynamics important</td>
<td>Feedback loop</td>
</tr>
<tr>
<td></td>
<td>Still analytical</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Still piece meal</td>
<td>Increasing integrity</td>
</tr>
<tr>
<td></td>
<td>Suitable for low-order problems with time invariance</td>
<td>vis-a-vis the real world</td>
</tr>
<tr>
<td>3rd generation (current generation)</td>
<td>Field problem approach requiring numerical methods Integrated view of energy sub-system</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Suitable for high-order problems with time variation Heat and mass transfer considered Better user interface and partial CABD integration</td>
<td>Leading to Predictive</td>
</tr>
<tr>
<td>Next generation</td>
<td>Full CAID integration</td>
<td>Predictive</td>
</tr>
<tr>
<td></td>
<td>More advanced numerical methods</td>
<td>Generalized</td>
</tr>
<tr>
<td></td>
<td>Intelligent knowledge-based</td>
<td>Easy to use</td>
</tr>
<tr>
<td></td>
<td>Object-orientated software architecture</td>
<td></td>
</tr>
</tbody>
</table>

Building energy modelling and simulation is part of an evolutionary process in the field of building design tools. Table 2 (from Clarke 1988) summarizes one view of the evolution of these design tools, from the traditional via the present day simulation approach to the 4th generation tools by the late 90s. More information on the state of the art in building energy simulation can be found in reviews by Winkelmann (1988), and Wiltshire and Wright (1988).

Early work in current generation approaches, focussed on the relation between building design and energy consumption (eg Clarke 1977, Bruggen 1978), or between building design and thermal comfort (eg Lamers 1978). In this and in later work (eg Hoen 1987), the auxiliary system was still more or less regarded...
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as a given boundary condition instead of as a variable. These workers emphasized the building side of the overall problem domain, while others (e.g., McLean 1982, Murray 1984, Tang 1985, Lebrun 1988) focussed more on the plant side.

In the former approach the influence of the plant system is more or less neglected by over-simplification; estimation of energy consumption is based on some presumed, imposed indoor air temperature profile. In the latter approach the complex building energy flow paths are usually grossly simplified, and the building (or each building zone) is commonly regarded as just another plant component which in this case imposes a thermal load on the system.

Although justifiable at that time, it is now felt that neither approach is preferable for the majority of problems which are affected by the thermal interaction of building structure and auxiliary systems. We started from the principle that both building and plant have to be approached on equal levels of complexity and detail while taking into account all major fluid flow and heat transfer couplings.

For the present work, we started from an established building energy simulation environment and enhanced this on the plant simulation side of the overall problem domain: the ESPR (Environmental Systems Performance, Research version) energy simulation environment (Clarke et al. 1991), a system which is currently under development at various research centres throughout Europe among which the Universities of Strathclyde and Eindhoven.

Reporting this work, Hensen (1991) describes a "modular-simultaneous" technique for the simulation of combined heat and fluid flow in a building/plant context. The present performance of the system indicates that it is practical to solve the building/plant heat and mass flow network in detail. Moreover, the solution of complex building/plant/fluid flow networks in the transient state is now feasible on inexpensive computers. This enables an integral approach of the thermal interaction of building structure and heating and ventilating systems, and also provides the basis/power for design decision support in this area.

DESIGN SUPPORT VIA SIMULATION

In the field of building energy related issues, there is a certain tradition of using computer simulation for design support, involving the generation of knowledge which is subsequently transferred to the design profession. This kind of design support is thus based on knowledge transfer from "specialists" in a certain part of the overall problem domain, towards the design profession. With respect to the transfer process itself, there are may different approaches. At one end of the spectrum of possibilities one finds the so-called design-aids, while consultancy work for a specific design could be located at the other end of the spectrum. To demonstrate both these approaches by an example:

Design-Aids
This is a form of knowledge transfer in which the "specialists" try to generate generic knowledge which is supposed to be suitable for a range of buildings and which is usually aimed at being used by the designers. This kind of knowledge is commonly based on regression techniques applied to the results of multiple parametric runs of more powerful modelling systems. The results to emerge can often be reduced to simple relationships or presented in tabular or graphical form. Figure 1 is a typical example (from CEC 1986) showing summer overheating assessment graphs for medium-weight houses.

It is obvious however, that there are a number of drawbacks from such an approach, the most important ones being: (1) a particular aspect is regarded in an isolated manner, (2) this approach is only possible for a limited number of variables, and (3) the results are only valid for a certain combination of environment, building, installation, and occupancy pattern which is quite similar to the one used to generate the results.

Consultancy Work
In the current context this involves the generation of specific knowledge for a particular design by a specialist. Work we did on predicting air flow through proposed shopping arcades; i.e., the
Figure 1 Summer overheating assessment graphs for medium-weight houses (from CEC 1986)

Figure 2 Shopping arcade in the Heuvelgalerie in Eindhoven

The Heuvelgalerie involves an extensive shopping mall. This 4-level complex incorporates a 220 metre long shopping arcade, interspersed with atria and dome-shaped roofs, approximately 20,000 m² shops, a 8,600 m² concert hall, 3,000 m² restaurants, a 1,200 units car park, offices, and apartments.

It should be apparent that such a building is a highly complicated system. For instance, the manner
in which air will flow depends on the external pressures on entrances and domes, temperature differences inside and with respect to outdoors, and impulses by the ventilation system. For this building \textit{ESP} was used to make various predictions with respect to the indoor environment (Pernot and Hensen 1990).

![Figure 3 Predicted effect of proposed "wind sluice" to decrease the air velocities in the pedestrian entrance area; i.e. the passage connecting to the main square.](image)

As an example, consider Figure 3 which shows results with respect to the air velocities which may be expected in the pedestrian entrance area. For commercial reasons, the architects and the developers want the entrance areas to be as open as possible. From the first results it was clear however that the original design proposal (incorporating air curtains for the main entrance) would lead to unacceptably high air velocities. One of the main conclusions was that additional air flow restrictions were necessary. For this it was suggested to apply double sets of sliding doors (wind sluices) at the "cafe" and "west" entrances, and to incorporate extra sliding doors + side hung doors at the main entrance. In case of severe wind, it should be possible to further restrict the (open) cross-section of the main entrance.

These are merely two examples, more or less demonstrating both ends of the spectrum of knowledge transfer/design decision support possibilities in the area of building energy use. Obviously there are various intermediate approaches; these often take the form of some simplified calculation method. The envisaged user profile usually shifts from more designer-like towards more specialist-like as the method shifts from design-aids towards the full simulation model.

Since buildings are complex mechanisms, involving phenomena such as transient conduction and air movement, there is a growing realisation that traditional design tools cannot cope with this complexity.
Particularly at the earlier stages of the design process, there is a need for rapid feedback on the cost and performance consequences of alternative design scenarios. The present system of specialist consultants, while adequate for the detailed design and final specification phases, fails to provide this immediate 'ad hoc' advice.

CONCLUSIONS AND FUTURE DIRECTIONS

It may be apparent that while development of sophisticated building performance evaluation tools as indicated above will comprise a valuable addition to the building engineer's toolkit, they also create new problems deriving from the conflict between the necessity for the tools to be powerful, comprehensive and according to first thermodynamic law principles to adequately represent the real world complexity while also being simple, straightforward and intuitive to facilitate user interaction. Such problems are not restricted to novice users but they apply to experienced users as well (Van Nes 1991).

As Clarke (1991) points out, the conflict between power and ease of use is further exaggerated by the divergence of the conceptual outlook of the design orientated program users and the technically orientated program developers. And to complete the confusion, there is the subtly different terminology of the various engineering professions. One - very promising - way to tackle these problems, is by utilisation of Knowledge Based System (KBS) and Human-Computer Interaction (HCI) techniques to create an Intelligent Front End (IFE).

Using these techniques it is possible to construct a user interface which incorporates a significant level of knowledge in relation to building description - in the face of real world uncertainty and realistic performance assessment methodologies. Such a system would direct a user's line of enquiry, allowing 'What do you suggest?' and 'Why do you ask?' type responses. It would also be expert enough to devise an appropriate performance assessment methodology and to coordinate model operation against this.

Using an IFE, the powerful simulation core may be invoked much earlier in the design process, because it is readily available to the designer. Obviously specialist consultancy will still be necessary, but this can be limited to the more common questions / problems.

Figure 4 Current and future route of knowledge transfer / design decision support starting from simulation tools which treat the building and plant as an integrated, dynamic system.
This shift from the more traditional approach using design-aids and via specialist consultancy, towards future direct application of powerful simulation tools by the design profession, is indicated in Figure 4. This future kind of design decision support in the area of building energy and indoor climate thus derives its power from its simulation core and its ease of use from some intelligent interface. It is in this direction that we are currently orienting our research activities.

Acknowledgements
The author is deeply indebted to Professor Joe Clarke of the Energy Simulation Research Unit at the University of Strathclyde in Glasgow, who's support was essential for this work.

References
Research on building structures and building physics.


APPLICATION OF ELECTRORESISTANCE METHOD TO MOISTURE TESTING IN BRICK WALLS

1. Introduction

Moisture content in capillary-porous building materials influences significantly mechanical and thermal properties of the structure as well as its life. Being aware of moisture content in different elements of a structure is necessary both when building new structures (e.g., dampness of base during floor-laying) and when repairing houses devoid of vertical and horizontal damp courses. The application of non-destructive methods renders it possible to measure moisture content in various parts of a structure relatively quickly and to observe changes in moisture content in time. One of major problems here is humidity evaluation in structures built-up of elements of different porosity, e.g., in brick-and-mortar walls. Dielectric, microwave and neutron methods, successfully applied to moisture content evaluation in concrete [1,2], have much more limited application to measurements of humidity in walls because they indicate only mean humidity values inside or at the surface of the structure.

Separating the brick humidity values from mortar humidity values is in such a case very difficult, or even impossible. Therefore, due to its suitability for measurements in relatively small, limited spaces, it seems reasonable to apply electroresistance method to moisture content evaluation in brick structures.

2. Testing

The electroresistance method of moisture content evaluation is based on the dependence between electric conductivity in capillary-porous
material and the water content in the material. The dependence between electric resistance $R$ and dampness $U$ may generally be expressed by the function [3]:

$$ R = \frac{A}{U^n} \quad (1) $$

where:

- $A$ - constants dependent on the electrodes shape, chemical constitution, structure and mechanical condition of the examined medium.

The measuring technique consists in determining the electric resistance between electrodes placed directly in the examined material, or between electrodes surrounded by a material absorbing moisture from the medium under examination.

A sensor [fig.1] was constructed for moisture testing in brick structures. Its small dimensions /10 mm long, 10 mm wide, 2 mm thick/ make it possible to place it in a joint or in a $\phi=16$ mm hole drilled in brick (1). After filling the hole with mortar (2) and establishing sorptive balance between the mortar and the surrounding brick, it is possible to determine the amount of water in the brick. The electric resistance is measured in the space between the sensor electrodes (3) placed in PVC frame (4). Cu-Ko thermocouple connected to one of the electrodes (5) makes it possible to correct the electric resistance with regard to the real temperature of the examined medium. The electric resistance of mortar is dependent on its dampness which, in turn, is dependent on moisture content in the surrounding brick.

![Fig.1 The structure of the electro-resistance sensor.](image-url)
In order to determine the dependencies between the electric resistance of sensors and mortar humidity, the sensors were placed in 6 cylindrical samples (4 cm diameter, 8 cm high).

The indirect dependencies between the brick dampness and sensors’ electric resistance were determined on the basis of examining 3 brick halves, where 3 sensors were placed in drilled and filled with mortar holes.

Two types of mortar, i.e., 1:6 cement mortar, 1:1:6 cement-lime mortar and common brick of different bulk densities $\gamma = 1.73 \, G/cm^3$ and $1.84 \, G/cm^3$ were used for testing. Humidity by mass for mortar and brick was determined by the traditional gravimetric method.

The resistance was measured at the temperature 18-20°C by RLC bridge type U 902 where variable current of 1000 Hz and 9V was applied.

3. Testing results

The obtained correlations between humidity by mass $U_m$ for cement and cement-lime mortars and the electric resistance of sensors placed in mortars are shown in fig.2.

The results obtained during sensor scaling show that there exists a dependence between electric resistance and humidity in virtually the whole humidity values range for cement and cement-lime mortars, i.e., from the state of full saturation to the moisture in air-dry state. The difference in the course of the dependence $U_m = f(\log R)$, slight at the low humidity level, rises together with the rise of the saturation level of mortars, exceeding at the full saturation 4% of absolute humidity values. The reasons for this are to be found in chemical constitution as well as in different porosity structures of mortars. Hence, in order to apply electroresistance method to the evaluation of mortar humidity, e.g. in wall joints, a scaling curve established in accordance with a mortar type must be adopted. The scaling curves shown in fig.3 concern the dependence between humidity by mass for brick and electric resistance of sensors placed in holes drilled in bricks and surrounded by cement or cement-lime mortar.
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Fig. 2 Correlation between humidity by mass for mortars $U_m$ and electric resistance $\log R$

Fig. 3 Correlation between humidity by mass for bricks $U_m$ and electric resistance $\log R$ of the sensor placed in mortar.
The curves tracking depends both on bulk density values for bricks under examination as well as on the type of mortar surrounding the sensor. At low humidity the influence of mortar type is slight, which the dependence in fig. 2 proves. At high moisture content values, greater differences are mainly caused by different chemical constitution of mortars which is responsible for the type and concentration of ions in the liquid. As ion conductivity influences dominantly electric resistance at high saturation level, the differences in the chemical constitution of mortars are responsible for different electric resistance values when brick humidity remains the same. Disregarding this fact may result in an additional error in establishing the moisture content in brick of up to 2.5% absolute humidity value.

When bricks have different bulk densities and the same moisture content, higher electric resistance values of sensors placed in bricks of lower bulk density are observed. For bricks of \( \gamma = 1.73 \text{ G/cm}^3 \) and \( \gamma = 1.84 \text{ G/cm}^3 \) bulk densities the difference in humidity by mass at the same electric resistance value of mortar is 1-2% depending on the moisture content.

When evaluating moisture content in bricks by the electroresistance method it is necessary to determine scaling curves taking into consideration the type of bricks and mortar in which the sensor are placed.

4. Recapitulation

On the basis of electroresistance testing results, dependencies between the electric resistance of sensors placed in mortar and humidity by mass for mortar and brick were established. The high values of the correlation coefficient \( \eta > 0.97 \) and values of mean relative square deviation \( \nu_k < 12 \% \) allow practical application of the obtained relations to moisture content evaluation in mortars and bricks in brick walls.

When examining humidity in mortar directly by the electric resistance measurement, the mean absolute error in moisture content evaluation is \( +0.4\% \), while when determining indirectly moisture content in brick the mean error is \( +0.5\% \). Those error values will not be exceeded if appropriate scaling curves for sensors are applied, determined strictly for a particular type of mortar and brick.
The shape and dimensions of the constructed sensors render it possible to place them in joints, and the presented here methodology of measurements allows determining moisture content separately in brick or mortar at any place of a building structure.

BIBLIOGRAPHY


1. Einleitung

Die Warmbehandlung des Betons, insbesondere Niederdruckdampfverfahren wird in industrie-mäßiger Technik bei Herstellung von Fertigteilen allgemein verwendet. Das Verfahren ist sehr vorteilhaft, weil die im Beton stattfindenden physikalisch-chemischen Reaktionen wesentlich beschleunigt werden können. Negative Auswirkungen dieses Verfahrens bringen vor allem thermische Spannungen im erhärteten Beton mit, die durch große Temperaturgradienten hervorgerufen sind, sowie Spannungen, die durch ungleichmäßige Änderungen des Volumens einzelner Betonbestandteile verursacht werden. Diese Spannungen bewirken, daß im Gefüge des warmbehandelten Betons diverse strukturbezogene Fehler entstehen. Es handelt sich vor allem um Mikrorisse und erhöhte Porosität, welche Druck- und Ermüdungsfestigkeit sowie andere wichtige Eigenschaften des Betons weitgehend beeinträchtigen [1, 2, 3]. In vielen Artikeln, die sich auf diesen Themenkreis beziehen, wird behauptet, daß das Gefüge des warmbehandelten Betons, der unter diesen Umständen erhärtet, viel stärker mit Mikrorissen und anderen Fehlern übersät ist, als das des normal erhärteten Betons [1, 2, 4].

Diese Behauptung mag schon richtig sein, aber es fehlen diesbezüglich Daten — ausgenommen Untersuchungen zur Festigkeit oder Porosität — unterstützt durch unmittelbare Beobachtungen, welche die Rißbildung während der Warmbehandlung eindeutig beweisen könnten.

Die vorliegende Arbeit stellt sich zur Aufgabe, die Ergebnisse solcher Beobachtungen unter Anwendung der Schallemission zu präsentieren. Die notwendigen Untersuchungen wurden im Labor
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auf dem extra zu diesem Zweck errichteten Prüfstand durchgeführt.

2. Der Prüfstand und die Untersuchungen

Schema des Prüfstandes mit Bauelementen, wie im Bild 1 gezeigt.

Bild 1. Schema des Prüfstandes


Im Laufe der Untersuchungen wurden stets die Temperatur und akustische Signale, die aus dem warmbehandelten Beton herkamen als Impulssumme aufgenommen. Es wurde auch eine graphische Aufzeichnung der Impulse der Schallemission mittels Analogoszilloskop und eines Plotters ausgeführt.
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Tabelle 1
Zusammenstellung der die realisierten Zyklen der Betonwarmbehandlung kennzeichnenden Parameter

| Zyklus- | Zeitdauer der Phase [h] | Zeitdauer des Zyklus \( t_c \) [h] | Temperatur |
| bezeichnung | | | \( T_0 \) | \( T_{\text{max}} \) | \( T_k \) |
|-----------|-----------------|-------------------------------|------------|
| Zyklus 1  | \( \tau_1 \) 2,0 | \( \tau_2 \) 2,0 | \( \tau_3 \) 4,0 | \( \tau_4 \) 2,5 | 10,5 | 20 | 60 | 30 |
| Zyklus 2  | \( \tau_1 \) 2,0 | \( \tau_2 \) 2,0 | \( \tau_3 \) 3,0 | \( \tau_4 \) 1,5 | 0,5 | 20 | 80 | 30 |
| Zyklus 3  | \( \tau_1 \) 2,0 | \( \tau_2 \) 1,5 | \( \tau_3 \) 3,0 | \( \tau_4 \) 1,5 | 8,0 | 20 | 95 | 30 |

3. Ergebnisse der Untersuchungen und Analyse

Die erzielten Ergebnisse im Bereich der Messung der Impulse einer Schallemission während der Warmbehandlung des Betons werden im Bild 2 gezeigt.

Bild 2. Impulssumme einer Schallemission, aufgenommen während der Warmbehandlung des Betons, ausgeführt im Zyklus 1, 2 und 3.
Aus dem Bild 2 geht hervor, daß der Verlauf der Variabilität der Impulssumme einer Schallemission, die während der Warmbehandlung aufgenommen wurde, ähnlich für ausgeführte thermische Zyklen ist. Diese Ähnlichkeit drückt sich durch folgende Tatsachen aus:

- in der Phase der Vorerhärtung gibt es keine Schallemission,
- in der Phase der Anwärmung tritt eine deutliche Schallemission ein,
- in der Phase der isothermischen Erwärmung beobachtet man eine Stabilisierung der Zunahme der registrierten Impulssumme der Schallemission,
- in der Kühlsphase tritt erneut die Schallemission ein, jedoch viel schwächer als in der Anwärmungsphase.

Trotz dieser Ähnlichkeiten gibt es auch deutliche Unterschiede:

- erste Signalzahl der Schallemission tritt in der Anwärmungsphase ein; mit der Temperaturzunahme eilt das Vorkommen der Signale,
- die Impulssumme einer Schallemission, die während der Anwärmungsphase oder während der Kühlsphase aufgezeichnet wird, wird um so größer, je schneller die Zunahme der Temperatur des Betons oder aber auch die Senkung der Temperatur zustande kommt,
- die Gesamtsumme der Impulse einer Schallemission, die während der Warmbehandlung aufgezeichnet wird, wird um so größer die Temperatur der isothermischen Erwärmung ist.

Bemerkenswert war die Tatsache, daß schon in der Anwärmungsphase, besonders an deren Anfang eine große Anzahl von Impulsen der Schallemission zum Vorschein kam, d.h. zum Zeitpunkt, wo der Beton noch nicht ganz erstarrt war. Es gilt als unwahrscheinlich, daß Signalquelle für die Schallemission auf dieser Etappe Mikrorisse waren. Hier kann man vermuten, daß die Signalquelle vor allem durch Reiberscheinungen hervorgerufen werden, z.B. durch gegenseitige Erlanderung der Zuschlagkörner und Luftblasenbewegung. Örtliche Verlagerungen der Zuschlagkörner und Luftblasenbewegung sind in der Phase der Anwärmung des Betons möglich. Die Ursache hierfür sind vor allem Spannungen, die auf ungleichmäßige Änderungen des Volumens des erwärmten Wassers und der erwärmten Luft im Vergleich zu Änderungen des Volumens des

Schon beim ersten Augenblick, wenn man die aufgezeichneten Impulse betrachtet, sieht man, daß die Vermutungen richtig sind. Es wurde beobachtet, daß in der Anwärmungsphase Impulse überwiegen, die als Reiberscheinungen erklärt werden können, weil ihre Zeitdauer ziemlich lang ist, wie auch ihre Zunahmezeit und die kleine Amplitude.


Es ist anzunehmen, daß die qualitative statistische Bewertung der aufgezeichneten Signale der Schallemission eine viel breitere Auslegung der erzielten Ergebnisse erlaubt.

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Bild 5. Charakteristischer Impuls einer Schallemission, betrachtet als Folge der Mikrorissenbildung (Kühlungsphase).
4. Schlüpfolgerungen

1. Die durchgeführten Untersuchungen haben gezeigt, daß die Messung einer Schallemission während der Warmbehandlung des Betons möglich ist.

2. Es wurde bewiesen, daß die Summe der akustischen Impulse, die während der Warmbehandlung aufgezeichnet wurden, strikt von den Parametern dieser Behandlung abhängig ist. Diese Summe ist desto größer, je größer die Temperatur der isothermischen Erwärmung ist und je schneller die Temperatur des Betons in der Kühlungsphase sinkt.


Literatur


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1. Einleitung

Die bisherigen Projektierungsverfahren berücksichtigen nur die Berechnung des Temperaturflächenfeldes zur Auswahl der entsprechenden Lösungsvariante [1], [2]. Wie es nachgewiesen wurde, ist das mit großem Fehler belastet, weil der Einfluß des Temperaturraumfeldes als wesentlich betrachtet werden muß, was auch in der Anwendung der Hohlkammersteine in den Einzelschichten des Außenmauerwerks bestätigt wird.

Gleichzeitig werden die arbeitsaufwendigen Untersuchungen realisiert, die offensichtlich beabsichtigen, die Warmdämmparameter der Blocksteinwände für verschiedene Materialvarianten und unterschiedliche Hohlkammersteinsysteme zu bestimmen.


2. Berechnung der Temperaturraumfelder


Die den Temperaturwert bestimmende Gleichung, bei der man veränderliche Materialeigenschaften voraussetzt, hat im allgemeinen für räumlich inhomogenen Bereich folgende Form:


\[
\frac{1}{\Delta x^2} \left[ \psi_{i+\Delta,j,k} - \psi_{i,j,k} \right] = \frac{1}{\Delta y^2} \left[ \psi_{i+\Delta,j+\Delta,k} - \psi_{i,j,k} \right] + \frac{1}{\Delta z^2} \left[ \psi_{i,j+\Delta,k} - \psi_{i,j,k} \right]
\]

Legt man auf dem betrachteten räumlichen Bereich ein regelmäßiges Gitter \( \Delta x = \Delta y = \Delta z \) auf, wodurch auch die Lösungsgenauigkeit beeinflusst wird, und nimmt man die gegenseitige Abhängigkeit der Temperatur und des Wärmevermängkoeffizienten \( K \) zwischen den Raumgitterplätzen aus der Gleichung (1) an, so erhält man:

\[
\psi_{i,j,k} = \frac{K_{i+\Delta,j,k} \psi_{i+\Delta,j,k} + K_{i-\Delta,j,k} \psi_{i-\Delta,j,k} + K_{i,j+\Delta,k} \psi_{i,j+\Delta,k} + K_{i,j-\Delta,k} \psi_{i,j-\Delta,k} + K_{i+\Delta,j+\Delta,k} \psi_{i+\Delta,j+\Delta,k} + K_{i+\Delta,j-\Delta,k} \psi_{i+\Delta,j-\Delta,k} + K_{i,j+\Delta,k} \psi_{i,j+\Delta,k} + K_{i,j-\Delta,k} \psi_{i,j-\Delta,k}}{K_{i+\Delta} + K_{i-\Delta} + K_{j+\Delta} + K_{j-\Delta} + K_{k+\Delta} + K_{k-\Delta}}
\]

Die in der Literatur vorhandenen Informationen, die sich auf die numerische Lösung des Gleichungstyps (2) beziehen, lassen sich auf die Lösung der auch die Randbedingungen berücksichtigenden linearen Gleichungssysteme zurückführen.

Bei den numerischen Berechnungen können Berechnungsprozesse wesentlich vereinfacht werden, indem man die Angaben in Form einer Matrix der Wärmevermängkoeffizienten \( K \) zwischen den Raumgitterplätzen vorbereitet. Der Wärmevermängkoeffizient \( K \) zwischen den Gitterplätzen wird manchmal Wärmeübergangskoeffizient oder Wärmeübertragungskoeffizient genannt. Anders gesagt ist es reziproker Wert des Wärmevermängswiderstandes zwischen Gitterplätzen. Die erste Matrix A besteht aus den Koeffizienten \( K \), die im Bereich der Fläche von nacheinander folgenden
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If -t -k II j // I

Abb. 1. Aufbauschema der 2-Ziffer-Zeichengelbsozittmatrizen
und das Matrizenadresstes auf dem Raumgitterplatz

Für den Gitterplatz (i,j,k) des in Bereich mit den Dimensionen X, Y, Z aufgelegten Gitters bedeuten die Ausdrucke der Matrix [Aij]; mit der Dimension (X+1) [Y(Z+1)]:

- Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i,j-1,k); Aij,2j-1+p - Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i+1,j,k).

Der Index p hat für den gegebenen Gitterplatz den konstanten Wert (k-1). (Z+1).

Auch für den Gitterplatz (i,j,k) bedeuten die Ausdrücke der Matrix [Bij] mit der Dimension (X+1) [Y(Z+1)]:

- Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i,j,k+1); B_{i,j,k}^{1} - Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i-1,j,k); A_{i+1,j,k}^{1} - Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i,j+1,k).

Bi,k-1+p - Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i+1,j,k).

Bi,k-1+p - Warmedurchgangskoeffizient K zwischen den Gitterplätzen (i,j,k) und (i+1,j,k).
Flachquerschnitten in Gitterschrittabständen bestimmt werden. Die andere Matrix B enthält hintereinander folgende Koeffizienten, die die Plätze der benachbarten Fläche verbinden. Sie sind senkrecht zu denen, die zum Aufbau der Matrix A verwendet wurden. Die Matrizen A und B sind Leitwertmatrizen, die 2- oder 3-Kennziffer-Aufbauschemas haben.

3. Aufbau der Gleichungen und Leitwertmatrizen mit zwei Kennziffern


\[
q_{1,j,k} = (A_{i,2j-2+p} \cdot q_{i,j-1,k} + A_{i,2j-1+p} \cdot q_{i-1,j,k} + \\
A_{i,2j+p} \cdot q_{i,j+1,k} + A_{i+1,2j-1+p} \cdot i+1,j,k + \\
B_{i,k+r} \cdot q_{i,j,k+1} + B_{i,k-1+r} \cdot q_{i,j,k-1})/
\]

\[
(A_{i,2j-2+p} + A_{i,2j-1+p} + A_{i,2j+p} + A_{i+1,2j-1+p} + \\
B_{i,k+r} + B_{i,k-1+r})
\]
Der Index \( r \) hat für den gegebenen Gitterplatz den Wert \((j-1) \cdot (z+1)\).

4. Beispiel für Außenmauerwerk aus Schlackenbetonsteinen

Den "Warmedurchgangskoeffizientwert" für solch ein Mauerwerk mit dem Verfahren von der "zustatlichen Wand" experimentell zu bestimmen, ist nicht richtig, weil die Voraussetzungen des Verfahrens nicht erfüllt werden. Die isothermischen Flächen sind in diesem Fall nicht parallel zu der Wandoberfläche. Wenig geeignet ist auch das "Wärmekastenverfahren", denn es läßt nur die Bestimmung der Wärmemenge zu, die durch die Wand durchfliesst, ohne Punkte mit infolge der vorhandenen Wärmemuraumbrücken niedriger gewordener Temperatur zu ergeben.

Zur Beurteilung der Leichtbetonsteinwand (ohne Putz) wurde ein komplexes Verfahren gewählt. In der ersten Etappe wurde die räumliche Temperaturverteilung zur Auswahl der repräsentativen Punkte für experimentelle Untersuchungen berechnet. Der ausgewählte und sich wiederholende Wandabschnitt (Abb. 2) wurde durch den Raumgitter mit dem Schritt 0,05 x 0,05 x 0,05 m und 0,05 x 0,05 x 0,045 m geteilt.

Zur Bestimmung des Wärmedurchgangskoeffizienten zwischen den Gitterplätzen wurde der im Labor ermittelte Wärmeleitfähigkeitskoeffizient \( \lambda \), der durch die Messung nach der Bock'schen Methode mit dem Gerät der Firma "Neutron" erhalten wird, eingesetzt.

Die Matrix \( B \) wurde aus Wärmedurchgangskoeffizienten aufgebaut, die im Bereich der die Gitterplätze der inneren Wand auf der warmen Seite verbindenden Fläche und weiterhin in den Querschnitten C-C, B-B, A-A und in der Außenfläche auf der kalten Seite bestimmt wurden. Die Matrix \( A \) enthält alle Wärmedurchgangskoeffizienten \( K \) der Gitterplätze, die benachbarten Flächen nacheinander mit einzelnen zu den obengenannten orthogonalen Querschnitten verbinden.

Die Abb. 2 stellt die in der warmen Fläche der untersuchten Wand berechneten Temperaturverteilungen für Temperaturwerte \( t_e = -18^\circ C \) und \( t_i = 18^\circ C \) dar. Die Berechnungsstufe des Verfahrens erlaubte also die kühlsten Bereiche auf der Wandoberfläche, d.h. in den Fugen zwischen den Hohlkammersteinen, zu ermitteln.

Beim Untersuchungen in der Klimakammer wurden weiterhin die ermittelten kühlsten Bereiche durch die Messung mit dem Widerstandsthermometer zusätzlich testiert.

5. Schlußfolgerungen

1. Bei der Entwicklung der Hohlkammersteine für die Außenwände
oder der mit dem "Wärmedämmstoff" gefüllten Leichtbetonsteine ist es erforderlich, das Temperaturraumfeld vorzumodellieren, um in weiteren Untersuchungen effektivere Lösungen einzusetzen.

2. Die bisherige Modellierung der Hohlkammersteine mit einer Vereinfachung, die nur auf das Temperaturflächefeld begrenzt wurde, ist mit dem Fehler belastet, daß dessen räumlicher Charakter nicht berücksichtigt wird (siehe Temperaturdiagramme Abb. 2) und kann nur dann zugelassen werden, wenn von außen eine zusätzliche Wärmedämmsschicht vorgesehen wird, die das Temperaturfeld in der Hauptwand und besonders auf deren warmer Fläche ausgleicht. Ein Beispiel für schlechte Lösung, das dargestellte Schlußfolgerung bestätigt, ist die früher angewendete Lösung eines Hohlkammersteinmauerwerks mit "üblichem" Mortel, wobei das ungleichmäßige Temperaturfeld auf der warmen Wandoberfläche bei der Lösung mit dem einzelnen Hohlkammerstein die Ursache der häufigen Unterfrierungen vor allem in den Stoß- und Querfugen war. Im Mauerwerk aus Hohlkammersteinen sollen überall, wo es möglich ist, die sogenannten "warmen" Mortel (Sanierungsmörtel) verwendet werden.


Literatur:


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INFLUENCE OF CEMENT CHEMICAL COMPOSITION ON ITS RESPONSE TO SUPERPLASTICIZER ADDITION IN LIGHT OF RHEOLOGICAL RESEARCHES

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1. INTRODUCTION

Reaction of cement pastes, mortars and concrete mixes. on superplasticizers' admixtures indicates that water requirements of cement and cement pastes rheology are connected with their colloidal and chemical properties /1-3/.

The adsorption of superplasticizing admixtures upon the cement particles surface brings about fluidifying effect, generates high negative potential, and results in rheological behaviour which varies, depending on the chemical nature of admixture, its concentration, the cement composition and temperature of the fresh mix, mixing procedure and the time after water has been added /2-7/.

The important factors which effect the compatibility between a given cement and a water-reducing admixture with respect to cement are: the $C_3A$ content, alkali content / optimum, form of CaSO$_4$ /, free lime content and extent of areation /2,4,7/.

It is reasonable to assume that the principal products of early hydration of portland cement are formed by reaction of aluminate phase which is manifested by a change in the rheological properties of fresh cement pastes /1, 8-11/.

It is generally accepted that in the first hydration stage /1-2 minutes/ only a small quantity of $C_3A$ reacts and the product kind is affected by the liquid phase composition /SO$_4^{2-}$, OH$^-$/ /8,12,13/.

From among several hydration products which can be formed and their mixtures, the most important for rheological properties of cement are fine grained crystals of ettringite /8,9,14/. Variations in contents and reactivity of industrial cements, even within the same part coming from the same cement plant, result in the fact that the conditions to form only crystalline ettringite deviate more or less from optimal ones, which are manifested also by their rheological behaviour /15-17/.
The addition of ionic sulphonates / commercial superplasticizers type melamine -formaldehyde condensate - SMF, naphthalene -formaldehyde condensates -SNF and modified lignosulphonate HLS/ can upset the balance between soluble sulphates and aluminate contents and affect the morphology and order formation of hydration product by direct incorporation of organic constituent into the bulk of amorphous gel, and surface sorption into the more crystalline phases /2.3.5.12.19/. The semicolloidal gel particles /stabilized by incorporation of organic molecules, which tend to increase the volume of amorphous material/ have a higher solubility than the stable, crystalline hydrate phase, hence the initial prolonged supersaturation, influence the hydration kinetics, delay the nucleation and inhibit crystal growth of early hydrates /18,19/. This can explain why, for relatively small surface of cement grains in comparison to typical colloids, and for the fluidifying, the required amounts of additives are big, why their effect diminishes and why the maximum workability is obtained, when the time of superplasticizers addition /optimum addition time/ corresponds to the beginning of dormant period of cement hydration without admixture.

If, as it seems likely to be, the early hydration of cement is largely controlled by diffusion through protective coatings around cement grains, then it follows that the cements kinds, of which initial conditions create a denser, less permeable and more adhesive hydrate without admixture, will fluidify in a similar way independently of admixtures introduction time.

This paper illustrates how commercial cement can vary in its response to SNF superplasticizer. This sensitivity is related to the composition of cements.

2. EXPERIMENTAL

2.1. Materials
- Industrial portland cement clinkers, /partial chemical composition as given in Table I/ were ground to SSB = 300 or 350 ± 10 m²/kg. Irrespective of clinker's chemical and mineralogical composition, the cements from clinkers K₁ and K₇ were prepared with addition of natural CSH: cements from clinker K₈, using addition of natural dihydrate or
mixture CS with $\text{CS}_2\text{H}_{0.5}$ / 60 : 40 /, and from the remaining clinkers / Tab. I / natural $\text{CS}_2\text{H}_2$.
- superplasticizer type NSF, powder.
- deionised water.
- high grade purity boric acid / as hydration retarder /20.21/.

Table I. Composition of clinkers /percent by weight/

<table>
<thead>
<tr>
<th>Clinker</th>
<th>$C_3S$</th>
<th>$C_2S$</th>
<th>$C_3A$</th>
<th>$C_4AF$</th>
<th>$\text{Na}_2\text{O}$</th>
<th>$\text{K}_2\text{O}$</th>
<th>$\text{Na}_2\text{O}$</th>
<th>$\text{K}_2\text{O}$</th>
<th>$\text{SO}_3$</th>
<th>$\text{FCaO}$</th>
<th>DS</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1</td>
<td>67.0</td>
<td>8.0</td>
<td>3.5</td>
<td>15.0</td>
<td>0.28</td>
<td>0.22</td>
<td>0.42</td>
<td>0.26</td>
<td>1.29</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>K2</td>
<td>36.0</td>
<td>37.0</td>
<td>4.7</td>
<td>16.7</td>
<td>0.10</td>
<td>0.85</td>
<td>0.66</td>
<td>1.00</td>
<td>0.8</td>
<td>117</td>
<td></td>
</tr>
<tr>
<td>K3</td>
<td>57.0</td>
<td>23.0</td>
<td>8.5</td>
<td>8.0</td>
<td>0.10</td>
<td>1.00</td>
<td>0.76</td>
<td>0.90</td>
<td>1.0</td>
<td>92</td>
<td></td>
</tr>
<tr>
<td>K4</td>
<td>58.6</td>
<td>14.9</td>
<td>9.3</td>
<td>13.4</td>
<td>0.10</td>
<td>0.70</td>
<td>0.56</td>
<td>0.70</td>
<td>1.0</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>K5</td>
<td>51.8</td>
<td>25.5</td>
<td>11.0</td>
<td>7.9</td>
<td>0.10</td>
<td>1.20</td>
<td>0.89</td>
<td>0.50</td>
<td>1.7</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>K6</td>
<td>51.0</td>
<td>22.0</td>
<td>12.0</td>
<td>9.7</td>
<td>0.20</td>
<td>0.70</td>
<td>0.66</td>
<td>0.40</td>
<td>1.2</td>
<td>47</td>
<td></td>
</tr>
<tr>
<td>K7</td>
<td>60.0</td>
<td>12.0</td>
<td>13.5</td>
<td>7.0</td>
<td>0.11</td>
<td>1.06</td>
<td>0.81</td>
<td>0.38</td>
<td>2.23</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>K8</td>
<td>57.4</td>
<td>18.0</td>
<td>14.0</td>
<td>8.2</td>
<td>0.20</td>
<td>1.1</td>
<td>0.92</td>
<td>0.40</td>
<td>1.2</td>
<td>32</td>
<td></td>
</tr>
<tr>
<td>K9</td>
<td>64.7</td>
<td>8.5</td>
<td>14.9</td>
<td>7.9</td>
<td>0.20</td>
<td>0.70</td>
<td>0.66</td>
<td>0.50</td>
<td>2.6</td>
<td>59</td>
<td></td>
</tr>
<tr>
<td>K10</td>
<td>53.0</td>
<td>20.0</td>
<td>15.4</td>
<td>7.5</td>
<td>0.20</td>
<td>1.20</td>
<td>0.99</td>
<td>0.70</td>
<td>2.1</td>
<td>55</td>
<td></td>
</tr>
</tbody>
</table>

The pastes were made by mixing cement with water, or boric acid solution / 2.5 wt% for w/c =0.4: 3.3 % for w/c = 0.3: 1 wt% in relation to cement/ with and without NSF. Superplasticizer was added into the mix water or 3 minutes after water or acid solution were added. The water cement ratio w/c was of 0.4 without and of 0.3 with of NSF addition.
The condition for pastes preparation / mode of mixing, intensity, time, temperature about 20°C were the same for all samples examined. No special steps were taken to remove air from the mix.

2.2. Testing procedure

The rheological tests were performed with a rotating coaxial cylinder viscometer Rheotest RV-2, using device H. In order to reduce sliding, the surfaces of both cylinders have been corrugated. The shear cycle started 10 minutes after the first addition of water or water with admixtures. The time for complete increase - decrease cycle was 12-15 minutes. Influence of time on apparent viscosity of pastes was measured for 60 minutes.

3. RESULTS AND DISCUSSION

The C_{3}A content for the tested cements varied from 3.5 to 15.4 wt%, free CaO from 0.8 to 2.6 wt%, alkalies as Na_{2}O from 0.42 to 0.99 wt%, and sulphatization degree /DS/ of alkalies in clinkers from 33 to 117 % /Table 1/.

The flow curves for cement without addition SHF / Fig. 1.a./ show a great differences in not only position of curves but also in the width of antiloops. Taking into consideration the position of the flow curves, they come from the following sequences:

\[ C_7 \approx C_{10} \approx C_9 > C_5 > C_4 \approx C_6 \approx C_3 > C_2 \approx C_9 \approx C_2 \]

decrease of consistency

and for the width of antiloop, the sequence is:

\[ C_{10} \approx C_7 \approx C_8 \approx C_5 > C_3 >> C_6 \approx C_4 > C_9 \approx C_2 \]

decrease of width of antiloop

The rheological tests revealed that among the cement pastes without addition of SHF, the highest consistency and the widest antiloop demonstrate pastes prepared from cements containing larger quantities of alkalies, with lower degree of their sulphatation / C_{10}, C_8, C_5, C_7/ and a greater content of free lime / the IR analyses show, that a
considerable part of free CaO contained in cement C₉ had been bound into carbonates.

As the earlier researches indicate /11/ the optimal conditions for control of aluminate phase hydration by addition of CSH₂, appear only in pastes prepared from cement C₄. In the case of other investigated cements, as it results from Fig. 1, the availability of sulphates deviates less or more from the optimal ones.

Addition of 1 wt% of retarder /boric acid/, by the same w/c ratio reduces antytixotropy /antiloop/ in all cement pastes except in cements C₄ and C₇ /containing anhydrite as hydration regulator/, substantially decreases yield value and plastic viscosity. The smaller retarding influence of boric acid on cement C₈ C₅ C₉ and a negative one on cement containing anhydrite may be explained by too small quantity retarder for this cement. It influences on the rheological properties of the cement pastes as presented in Fig. 2. Preparation of C₈ with a
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A mixture of $\text{CSH}_{0.5}$ and $\text{CS} / 40:60$ considerably decreases consistency of the pastes without retarder and eliminates retarder's influence on rheological behaviour. /Fig. 3./.

$$\eta_{\text{app}} [\text{Pa}\cdot\text{s}^{-1}]$$

1. $\text{H}_2\text{O}$
2. $\text{H}_2\text{O} + \text{H}_3\text{BO}_3$
3. SNF into $\text{H}_2\text{O}$
4. SNF to the cement paste
5. $\text{H}_3\text{BO}_3 + \text{H}_2\text{O} + \text{SNF}$ to the cement paste

$W/C = 0.4$

$W/C = 0.3$

$\bar{\gamma}_0 = 48 \text{ s}^{-1}$

$C_3A$-wt.[%]

Fig. 2. Comparison of apparent viscosity of cement pastes with and without additives with consideration of on flow curves hysteresis

$$\gamma [\text{Pa} \times 10^3]$$

1. $\text{H}_2\text{O}$
2. $\text{H}_2\text{O} + \text{H}_3\text{BO}_3$ $W/C = 0.4$
3. SNF into $\text{H}_2\text{O}$ $W/C = 0.3$
4. SNF to the cement paste

$\text{IC-81-1}$
$\text{IC-81-2}$
$\text{IC-81-3}$
$\text{IC-81-4}$

Fig. 3. Effect of the sulphate kinds and the addition of retarders on flow curves
Response of cement pastes to the superplasticizer addition

As it can be seen from data presented in Fig. 4 investigated cements show great differences in their response to addition of 1 wt% of SNF and the influence of added superplasticizer on several cements depends on composition of cements uncomparably more if SNF is being added to water.

![Flow curves with addition of SNF](image)

Fig. 4. Flow curves with addition of SNF: a – to mix water, b – to cement pastes

Obtained results revealed that the cements, of which the consistency and antiloop are the least ones without addition of superplasticizer. /Fig. 1/ liquify similarly independently of the SNF addition mode /C_2, C_4/. The similar behaviour of the cements C_3, C_5 and C_8 /Fig. 2. and Fig. 4./ results from the fact that, in the presence of superplasticizer, the solubility of gypsum increases. The cements
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C10 C7 C9 and C5 / containing more free lime and C1. C9 with a great content of C3S / Table 1. / revealed that for the same quantity of SNF a fluidifying is similar to remaining cements only by addition of SNF to the paste with a delay / Fig.2 and Fig.4b /.

Addition of 1 wt% of SNF to all cements 3 minutes after water was added drastically improved liquifying of pastes, independently of the quantity and kind of sulphates added. The addition of retarder to the pastes mentioned above, reveals its influence on these cements, for which the action effect of additive depends strongly on its introduction mode / Fig.5 /.

1%SNF into water

Fig.5. Comparison of influences of SNF addition modes on the apparent viscosity as a function of time

From technological point of view it is difficult to mix the constituents of concrete with small quantity of water and superplasticizers to be added with delay. Usually the additive will be introduced to mix water and in order to obtain a better result, the quantity of additive will be increased as shown in Fig.5. Increasing of additive to a cement with higher content of CaO, improves its rheological parameters with preservation of their great variability in time. Addition SNF with delay, besides a good liquifying, drastically decreases the influence of time, and moreover, requires two times smaller quantity of the additive.
4. CONCLUSIONS

This study was carried out to elucidate influence of composition commercial portland cements on their response to SNF superplasticizers.

In the light of rheological studies the influence of SNF addition to cements depends on the contents of free lime, alite, and alkali and the kind of sulphates. For the cements of greater content of mentioned constituents / besides sulphates/ the effect of SNF action depends on its introduction mode.

The addition of SNF with delay practically eliminates influence of cement composition on liquifying effect.

The research results do not reveal, however, the influence of C₃A content and its reactivity on the effect of SNF superplasticizer addition irrespectively of the introduction mode.

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Research on building structures and building physics.

Masonry research
Research on building structures and building physics.
BOND STRENGTH OF MASONRY
Rob van der Pluijm

ABSTRACT

Bond strength is not a well defined property of masonry. Normally three types of bond strength can be distinguished:
- tensile bond strength,
- shear (and torsional) bond strength,
- flexural bond strength.
In this contribution the behaviour and strength of masonry in deformation controlled uniaxial tests will be summarized (Van der Pluijm, 1992). The flexural bond strength will be related to the tensile bond strength by means of a FEM calculation.
A shear test-setup to study the behaviour and strength of bed joints will be introduced. In a test rig based on the proposed test-setup 54 deformation controlled tests were carried out. The results of the tests will be discussed.

INTRODUCTION

To achieve a more fundamental insight in the behaviour of masonry, a combined analytical, experimental and numerical research program has been started in the Netherlands. The final goal of the research program is the provision of a fundamental basis for design rules and so making possible a modification and extrapolation of existing rules. Therefore deformation-controlled test are carried out in the experimental part of the program. In these deformation controlled test, the tests are controlled by deformations measured on the specimen.

Bond strength of masonry plays an important role in different kinds of structural parts. In cavity walls with no or small vertical loads, the bond strength is the main parameter to resist lateral loads and in shear walls it plays an important role for strength and stability. Normally three types of bond strength can be distinguished:
- tensile bond strength,
- shear (and torsional) bond strength,
- flexural bond strength.
There is made a distinction between the tensile and flexural bond strength although both properties are directly governed by tensile stresses. But the flexural strength is also influenced by geometrical properties. Therefore it must be considered as a structural parameter and not as a material parameter.

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USED MATERIALS

Three types of units have been used. Every type of unit was applied in combination with two types of mortar. In table 1 some specific figures concerning the used materials and the used combinations of units and mortar in the test-program are presented.

Table 1 Used Material

<table>
<thead>
<tr>
<th>Units</th>
<th>Compressive strength (dry) (NEN 2489, N 3836) (MPa)</th>
<th>Bulk Density (at preparation) (kg/m³)</th>
<th>Dimensions (mm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joosten (JG)</td>
<td>66</td>
<td>1994</td>
<td>204x98x50</td>
</tr>
<tr>
<td>(yellow wire cut brick)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vijf Eiken (VE)</td>
<td>33</td>
<td>1880</td>
<td>208x98x50</td>
</tr>
<tr>
<td>(red soft mud brick)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand Lime (CS)</td>
<td>35</td>
<td>1810</td>
<td>212x102x54</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Compressive strength (NEN 3835, 28 days) (MPa)</th>
<th>Cement:lime:sand volume ratio</th>
<th>weight ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>8.2</td>
<td>1:1:6</td>
<td>1:0.48:6.72</td>
</tr>
<tr>
<td>B</td>
<td>3.0</td>
<td>1:2:9</td>
<td>1:0.96:10.0</td>
</tr>
<tr>
<td>C</td>
<td>17.6</td>
<td>2:1:9</td>
<td>1:0.24:5.04</td>
</tr>
</tbody>
</table>

Combination of Units and Mortar

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>JG</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>VE</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>CS</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>CS</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

The Joosten brick is a wire cut brick with a low suction rate. The Vijf Eiken brick is a soft mud brick with a high suction rate. The values for the compressive strengths, mentioned in table 1, were determined according to the Dutch codes of practice NEN 2489, 3835, 3836. For the masonry units this is done by applying a compressive load on the units in the direction of the smallest dimension. The mortar compressive strength in the code is established on half prisms which are left over from flexural bending tests.

In this paper series of specimen will be referred by their code, e.g. a specimen consisting of Joosten bricks and mortar B will be referred as JG.B.

TENSILE TESTS

The deformation-controlled tensile tests were performed in a test rig at the Stevin Laboratory of the Delft University of Technology (Hordijk, 1991). The way the specimens were manufactured and the measurements were performed, is described in (Van der Pluijm and Vermellooort, 1991). Five types of specimen were used:
two kinds of masonry specimen (100x100x177 mm³ and 100x110x110 mm³), mortar prisms (40x40x160 mm³) and two kinds of unit specimen (prisms 60x52x150 mm³ and cylinders Ø74x45 mm³) (see Fig. 1). The dimensions of unit and mortar specimens were reduced with saw cuts in the middle cross-sectional area. The reduction was necessary to enforce the location of the crack where the deformation is measured. The magnitude of the reduction differed from specimen to specimen. Tests were carried out in multiple of three for most of the combinations.

Tests on mortar prisms will not be discussed, because the mortar prisms were not representative for the mortar in the joint. One of the reasons for this is the influence of the suction rate of units on the mortar in joints.

**Tensile Strength**

The tensile strength of the units is determined by dividing the maximum measured force by the cross-sectional area at the saw cuts. The CS unit was only tested in the long direction of the unit because it is supposed that this unit-type has an isotropic behaviour. The mean test results are given in table 2. Also the coefficient of variation (c.o.v.) is given but it is emphasized that its reliability based on a few individual test results is very small.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>f_{tu} (MPa)</th>
<th>c.o.v. (%)</th>
<th>Number of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>JG-prism</td>
<td>2.36</td>
<td>21</td>
<td>2</td>
</tr>
<tr>
<td>JG-cylinder</td>
<td>3.51</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>VE-prism</td>
<td>2.47</td>
<td>14</td>
<td>3</td>
</tr>
<tr>
<td>VE-cylinder</td>
<td>1.50</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>CS-prism</td>
<td>2.34</td>
<td>10</td>
<td>3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>f_{tb} (MPa)</th>
<th>c.o.v. (%)</th>
<th>Number of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>JG.B</td>
<td>0.30</td>
<td>24</td>
<td>3</td>
</tr>
<tr>
<td>JG.C</td>
<td>0.50</td>
<td>29</td>
<td>6</td>
</tr>
<tr>
<td>VE.B</td>
<td>0.22</td>
<td>45</td>
<td>3</td>
</tr>
<tr>
<td>VE.C</td>
<td>0.13</td>
<td>100</td>
<td>3</td>
</tr>
<tr>
<td>CS.B</td>
<td>0.29</td>
<td>34</td>
<td>4</td>
</tr>
<tr>
<td>CS.A</td>
<td>0.33</td>
<td>51</td>
<td>5</td>
</tr>
</tbody>
</table>

In most of the masonry specimens a crack formed at the bond surface between mortar and unit. Only one masonry specimen within the JG.C-series broke in the mortar joint. In the masonry specimens with two joints, 4 time's the upper joint and 5 time's the lower joint broke. The bond surface after fracture of masonry with CS-units was remarkable smooth. From the results in table 6, it can be observed that:

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- The Joosten brick is strongest in the direction perpendicular to the bed joint (JG-cylinder), while the Vijf Eiken brick is the strongest in the direction parallel with the bed joint (VE-prism). This result can be explained by the difference in direction of the layered structure of the bricks due to the fabrication process.
- The tensile bond strength of VE.B (0.22 MPa) is significant higher compared with that of VE.C (0.13 Mpa). This is remarkable because mortar C (2:1:9) is stronger than mortar B (1:2:9).
- An increasing mortar strength (B - A - C) has only a positive influence on the masonry bond strength for the Joosten brick (low suction rate). This is one of the reasons that it was concluded that a mortar prism cannot be used to determine the quality of the mortar in the joint.
- There is a great scatter in test results. Among other reasons this is caused by the influence of the effective bond surface. From close observation of the crack surface of the specimens, it became clear that the area where the mortar and unit were bonded together in the specimens differed from each other and was considerable smaller then the cross-sectional area. This phenomenon will be discussed later as the 'net bond area'.

Fracture Energy and Post-Peak Behaviour
The fracture energy $G_f$ of a crack is defined as the amount of work that is needed to create a stress free crack and is equal to the area under a $\sigma$-$\delta$ diagrams established in a deformation controlled test. In table 4 the calculated $G_f$ values for units and joints are presented. All masonry specimens with CS-units failed uncontrolled after the ultimate load was reached. Therefore no values for the fracture energy are given. The uncontrolled failure does not mean that the fracture energy is zero. It can be expected that the fracture energy of the bond surface for the CS-units is smaller than for the brick units, because in the first case the interlock will be less important during the formation of the very smooth crack plane.

<table>
<thead>
<tr>
<th>Unit type</th>
<th>$G_f$ (J/m²)</th>
<th>c.o.v. (%)</th>
<th>Number of Tests</th>
<th>$G_f$ (J/m²)</th>
<th>c.o.v. (%)</th>
<th>Number of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>JG-prism</td>
<td>117</td>
<td>-</td>
<td>1</td>
<td>12</td>
<td>64</td>
<td>3</td>
</tr>
<tr>
<td>JG-cylinder</td>
<td>128</td>
<td>3</td>
<td>3</td>
<td>7</td>
<td>53</td>
<td>6</td>
</tr>
<tr>
<td>VE-prism</td>
<td>61</td>
<td>24</td>
<td>3</td>
<td>8</td>
<td>66</td>
<td>4</td>
</tr>
<tr>
<td>VE-cylinder</td>
<td>73</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>43</td>
<td>3</td>
</tr>
<tr>
<td>CS-prism</td>
<td>67</td>
<td>14</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The fracture energy of the units is approximately ten times higher than that of the bond surface. As expected for the joints, the fracture energy is very low, which corresponds with a very brittle behaviour. The shape of the descending branch under mode I loading in plain concrete can be described with (Hordijk en Reinhardt, 1990):
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\[ \frac{\sigma}{f_{tu}} = \{ 1 + (c_1 \frac{W}{W_c})^3 \} e^{-c_2 \frac{W}{W_c}} - \frac{W}{W_c} (1 + c_1^3) e^{-c_2} \]

in which \( c_1 = 3, c_2 = 6.93 \) and

\[ w_c = 5.14 \frac{G_f}{f_{tu}} \]

From a comparison between test results and Eq. [1] it is concluded that these formulae can be used well for units and joints.

**Influence of Net Bond Surface**

By close observation of the cracked specimen, it became clear that the area where the joint and unit were bonded together, was smaller than the cross-sectional area of the specimen. For each of the masonry specimens the 'net bond surface' was determined by visual inspection of the crack-surface. An example is shown in Fig. 2.

![Fig. 2 Net bond surface of VE.B specimens](image)

When the net bond surface is taken into account, the scatter of the tensile bond strength and the fracture energy becomes smaller. In Table 5 the influence is shown. Although the coefficient of variation is not reliable, it gives a fairly good impression of the influence of the net bond surface.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Tensile Strength</th>
<th>Fracture Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_{th,esa}^{**} ) (MPa)</td>
<td>( f_{th,nba}^{**} ) (MPa)</td>
</tr>
<tr>
<td>VE.B</td>
<td>0.26 (45)*</td>
<td>0.56 (26)</td>
</tr>
<tr>
<td>VE.C</td>
<td>0.13 (43)</td>
<td>0.51 (51)</td>
</tr>
<tr>
<td>JG.B</td>
<td>0.30 (24)</td>
<td>0.86 (16)</td>
</tr>
<tr>
<td>JG.C</td>
<td>0.50 (29)</td>
<td>1.47 (20)</td>
</tr>
</tbody>
</table>

* Coefficient of variation between brackets

Earlier in this paper the high tensile strength of the VE.B compared with the VE.C was
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mentioned. There is no longer a significant difference between the two, when the strength values based on the net bond surface for these two are compared.

In Fig. 3 the fracture energy of the joint is plotted against the tensile strength. From the figure it can be observed that it is not possible to define a meaningful relation between the tensile strength and the fracture energy. The average bond surface of the specimen was 35% of the cross-sectional area. If the net bond surface is supposed to be square, it follows that the net bond surface of a wall will be 57% of the cross-sectional area. So the bond surface of the wall is approximately 1.7 times greater than that of the specimens. The same holds true for the fracture energy and the tensile strength of the wall; both based on the gross cross-sectional area. It is noted that a possible influence of perpends is totally neglected in this way. If it is assumed that the fracture energy is independent of the units, then the average fracture energy of the specimen is 7.4 J/m² and amount to 12.5 J/m² for the wall.

Concluding Remarks
With the performed tests, the fracture energy and post-peak behaviour of the units and mortar joints could be determined. It appeared that fracture energy of the bond surface is approximately ten times less than the fracture energy of the units. The tensile bond strength depends highly the unit. A stronger mortar did not result in a higher tensile strength when it was applied with VE and CS units. Therefore it is concluded that mortar prisms are only applicable for the determination of the mortar quality and that it is in principle not correct to use the mortar strength in formula for masonry strength.

A disadvantage of the small masonry prism is the influence of the edges on the net bond surface.

RATIO BETWEEN TENSILE STRENGTH AND FLEXURAL STRENGTH

The tensile bond strength and flexural strength (defined as $M_u/(1/6bh^2)$) are not equal. The difference is a well-known phenomenon. Anderson (Anderson, 1981) already proposed a difference in stiffness under compressive and tension in order to explain this phenomenon, because the position of the neutral axes was shifted out of the mid-plane towards the
compressive zone in his flexural bending tests on masonry specimens. In (Vermeltfoort and Van der Pluijm; 1991) it is shown that the Young’s-modulus of the mortar joints under compression (measured on specimens manufactured at the same time with the same materials and by the same mason as described here) can be two to three times higher than the value under tension. This difference can be explained by the existence of contact areas in the bond surface that are only able to transfer compression stresses. From fracture mechanics it is known, that the post-peak behaviour (softening) plays a very important role. Due to the softening is still possible to reach an equilibrium and a higher internal moment after the tensile strength is reached in the extreme fibre. For this behaviour, the shape of the descending branch is the most important parameter. In Eq.[1] and [2] the shape is determined by the fracture energy and tensile strength.

In (Van der Pluijm, 1992) it is shown that the post peak behaviour is the main cause for a difference between the tensile and flexural bond strength. The influence of the fracture energy on the ratio λ between flexural strength and the tensile bond strength will be quantified by means of a FE analysis.

FE analysis
In order to quantify the ratio λ a beam is modelled. In Fig. 4 the mesh of the beam is given. The beam is 600 mm long and 100 mm high. The height corresponds with the thickness of a leave of a cavity wall in the Netherlands. The beam is modelled with isoparametric quadrilateral plane stress elements. In the middle of the beam a discrete crack is modelled with interface elements. The interface elements represent a bed joint where the crack is supposed to occur. At first this possible crack behaves linear-elaistically. When the tensile bond strength perpendicular to the bed joint is reached, a descending branch according to Eq.[1] is followed. In the remaining part of the beam, linear behaviour is presumed.

Fig. 4 Element mesh, loads and boundary conditions for FE calculation.

In table 6 the parameters used in the FE analysis are presented. The non-linear FE analyses are made with the DIANA code of TNO Building and Construction Research. The Young’s-modulus of the beam is taken equal to that of the VE brick. This is correct for the part of the beam near the modelled joint, but for the rest of the beam this value is somewhat high because the influence of the joints on the overall stiffness is neglected in this way. The values in table 6 are based on previous tables.
Table 6 Used Parameter in the FE-analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus of beam (MPa)</td>
<td>6050</td>
</tr>
<tr>
<td>Young's modulus of interface (MPa)</td>
<td>2000</td>
</tr>
<tr>
<td>Tensile bond strength of interface (MPa)</td>
<td>0.3</td>
</tr>
<tr>
<td>Fracture Energy $G_f$ of interface (J/m$^2$)</td>
<td>12</td>
</tr>
</tbody>
</table>

In Fig. 5 the moment-deflection curve in the mid section of the beam and the found ratio $\lambda$ in the calculation is presented. The influence of the tensile post peak behaviour becomes smaller when the height of the beam increases, because the influence of the descending branch on the total internal moment of the cross section becomes smaller. This is why the flexural strength can not be regarded as a material parameter.

SHEAR TESTS

Test Set-up

Well known test set-ups to determine the shear bond strength of bed joints in masonry are the RILEM test and the Hofmann/Stöckl test. The Hofmann/Stöckl test results in a uniform shear stress distribution, but the normal stresses are still high (Stöckl et al., 1990). From a practical point of view, a disadvantage of the Hofmann/Stöckl test is the complex loading rig that is needed to apply the proposed load.

In order to determine the shear bond strength and friction as a function of the normal stress, the test set-up shown in Fig. 6a was developed. The basic idea of this set-up is that...
moments and shear forces can lead to a pure shear load in the middle of the specimen if the moment \( M = V \cdot d/2 \). By means of two steel moulds an axial load is transformed into the desired moments and shear forces. In the middle of the joint the bending moment is zero (see Fig. 6b). This test set-up results in a linear stress distribution as shown in Fig. 7. Of course the shear stresses are zero at the top and bottom of the specimen.

This is a disadvantage compared with the Hofmann/Stöckl test. It was concluded that this set-up leads to an acceptable stress distribution. The advantage of set-up is that the principle is simple; normal compressive forces can be applied easily and it can be used in normal compression test rigs. A disadvantage are the heavy steel parts that are used to change normal axial force into moments and shear forces. Although the calculated linear stress distribution here and by Stöckl et al. are useful, it must be emphasized that their meaning is limited, because in the tests, non-linear behaviour can have a great influence on the stress distribution.

**Test Program and Experience with Test Set-up**

The used materials are described in the beginning of this paper. For each combination of units and mortar three different levels of normal compression stress were applied. The tests were carried out in multiple of three. The stress levels were not equal for each type of specimen. The normal compression stress level varied between 0.1 MPa and 1.0 MPa. During the tests the problem occurred that the unit could fail in tension before shear failure of the joint occurred. Therefore the normal stress levels were adapted to the type of unit used. The possible failure of the units in tension is a disadvantage of the test set-up. Although the bending moment in the joint is practically zero, it increases to its maximum value over the thickness of the unit. It can be concluded that the test set-up is not suitable for the combinations of units with low tensile strength and high shear bond strength of the mortar joint. To avoid this problem, the set-up could be modified by placing the specimen in the middle of a beam with fixed ends that is tested in shear. By doing so the bending moment in the units are reduced.

**Analysis of test results**

In Fig. 8 a shear load displacement diagram is shown. In the figure the quantities that are discussed in this paper are shown and defined.

**Shear Strength**

In Fig. 9 the ultimate shear stress (Fig. 8 A) is plotted against the normal compression stress in the bed joint. The results are interpreted with the well known relation of Mohr-Coulomb (Eq. [3]).

\[
\tau = c - \sigma \cdot \tan(\varphi)
\]

For each material combination, the linear best fit lines are also presented. From Fig. 9 it can be observed that the cohesion \( c \) is dependent of the units and the mortar. It can also be

![Fig. 7 Shear (\( \tau \)) and normal (\( \sigma \)) stress distribution in the joint of the specimen.](image)
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seen that the angle of internal friction $\varphi$ is approximately the same for all the used material combinations. Only the material combination VE.C shows a remarkably high angle, but there are only results available for two normal compression stress levels. Therefore the best fit line is less reliable than the other one's that are based on three different stress levels. Furthermore the VE bricks have a recess in the bed face so it is possible that in combination with the strong and stiff mortar C (1:½:4½) a compression strut is formed in the joint that can explain the high angle.

Post Peak Friction Level
When the shear deformation increases after peak load is reached, the shear stress reduces until a horizontal friction level is reached (Fig. 8 B). In Fig. 10 the mean shear stress of the horizontal friction level is plotted against the mean normal compression stress. It can be seen that the relation between the shear strength and normal strength is independent of the used materials. The friction coefficient equals 0.75.

Fig. 9 Shear strength of bed joints as a function of the applied normal compression stress

Fig. 10 Friction level as a function of the applied normal compression stress
Mode II Fracture Energy and softening distance

In Fig. 11 and Fig. 12 the shear fracture energy $G_{II}$ (Fig. 8 C) and the 'non-linear' softening distance $v_{\text{nonlin}}$ (Fig. 8 D) are plotted against the normal compression stress.

From Fig. 11 it can be observed that the mode II fracture energy increases when the normal compression stress becomes greater. In Fig. 11 the linear best fit line is plotted for each type of unit. It can also be concluded that the mode II fracture energy is dependent on the units but not on the mortar. The suggested linear dependency is only given a first approximation of the increase of the mode II fracture energy. The same observations can be made for the nonlinear softening distance. From a comparison of the influences of $v_{\text{nonlin}}$ and $(\tau_u - \tau_{\text{fr}})$ on the increase of $G_{II}$ it became clear that the increase of $v_{\text{nonlin}}$ caused by the normal compression stress, is the main cause for increase of $G_{II}$.

ACKNOWLEDGEMENTS

The described experimental research is part of a comprehensive program in the Netherlands, initiated by the Royal Dutch Brickwork association (KNB) and carried out under the supervision of the Dutch Centre for Civil Engineering, Research, Codes and Specifications (CUR). Since 1991 the cooperative organisation of the calcium silicate brick manufacturers (CVK) is participating in the structural part of the program.

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SENSITIVITY CASE STUDY OF PARAMETERS
IN EUROPEAN MASONRY CODES

Harrie J. Vekemans

ABSTRACT

This paper contains the results of a continued study on the comparison of design rules in European masonry codes. The main goal is highlighting interesting developments in the design rules in the codes and the main differences in relation to the design rules. Several codes were studied: NEN 6790, BS 5628 (mainly part 1), draft Eurocode no 6 (EC 6), NBN B 24-301, DIN 1053 (mainly part 2) and SIA 177 (mainly SIA V 177/2). The original comparison of the design rules showed big differences between values for similar parameters, and also differences between parameters and rules themselves. All the codes followed a limit state philosophy. In five codes the ultimate limit state is dominant, only in the Swiss code the serviceability limit state should be checked also. A case study was calculated based on the assumption that a chosen seven storey apartment building will be constructed in the country of the matching code. Several parameters in the calculation example were changed and the results were studied. The cases that were studied were changes in the characteristic live loads, partial safety factors for the material properties (material factor), loadcases including load factors and finally the reduction factor for the effects of slenderness and eccentricity. The results showed that none of the countries had a code which was similar to another one, they all differed (more or less). Of main interest were the big differences between material factors and the reduction factors for the effects of slenderness and eccentricity.

INTRODUCTION

The European Community is developing. One of the goals is the introduction of Eurocodes for many different fields, also for masonry. Looking at the countries of the future European Community this doesn't seem to be a big problem, because most of them are situated closely together. The reality is nevertheless completely different, as some publications on this subject highlight [1, 2, 3 and 4]. The same subjects in different masonry codes differ considerably and it also happens that only one or two codes require

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a check of an important problem. This is mainly due to the empirical knowledge, hidden in many rules and values in many codes. In reality a comparison of different rules and requirements is a puzzle with many solutions. To get more insight in the differences and similarities between codes in Europe, and for gaining more knowledge about interesting international developments this research is initiated.

Masonry is particular in all the European countries, this counts for dimensions, composition, construction, details, and so on. It is for instance very difficult to find an identical, widely used brick in Europe. A masonry code from a specific country is adjusted to the types of masonry that are used and structures that are constructed in that country. This is one of the causes of differences in the codes. Tests to verify theoretical models are carried out using local masonry, and most of the time the results are adjusted to that type of masonry. This makes it difficult to compare codes with each other, most of the time you are comparing apples with pears. But through assuming that masonry is the same in all the countries, it is possible to compare at least the calculation rules in the codes and the related parameters. By following this assumption this research was carried out, with the main goal of comparing the calculation rules and their parameters.

A choice was made between several masonry codes in Europe. The countries and codes that were chosen were (in random order):

1. Netherlands (NL) NEN 6790 [5];
2. Great-Britain (GB) BS 5628 (mainly part 1) [6];
3. European Community (EC) draft Eurocode no 6 (EC 6) [7];
4. Belgium (B) NBN B 24-301 [8];
5. Germany (D) DIN 1053 (mainly part 2) [9];
6. Switzerland (CH) SIA 177 (mainly SIA V 177/2) [10,11];

The final results of the research was published in CUR-report 92-8 [12], and parts of these results were also published elsewhere [13]. Some important remarks have to be made here. For instance that the Swiss code SIA 177/2 differs a lot from all the others, not only values but also the theory, the formulas and the tables [14]. This remark is important, because the Swiss code will not be discussed on this matter in this paper, because of these differences with other codes. For both the calculation of the stability walls and (ec)centric loaded walls they use progressive approaches. The Swiss code is in general the most progressive code of the researched codes. From the other codes it is important to keep in mind that also the German code DIN 1053 'Teil 2' contains substantial differences, which makes a comparison complicated.

This paper provides more insight in the sensitivity of parameters in six European masonry codes. The sensitivity is related to a chosen case study, the calculation of a seven storey apartment building. This means that the sensitivity of certain parameters is for instance not related to dimensions of walls or the span of floors. These values are fixed by the chosen example. The seven storey apartment building (fig. 1 and 2) was calculated by using five different approaches, namely:

- case 1 by situating the apartment building in the country of the code. This means that the values for the parameters in the calculation rules, such as loads and load cases, were retrieved from the matching codes of the country. These results were
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in this paper used as reference results.

Fig. 1: Modelled stability walls A and B of the building in the calculation example.

Fig. 2: Modelled (ec)centric loaded walls C, D and E in the calculation example.

Besides calculating these 'real' results, also changes in the parameters of the design rules were studied. The aim was to get more insight in the influence of changes on the final calculation results of the chosen example. The parameters in the calculation were changed assuming four additional cases:

- **case 2** The characteristic live loads were set equal for all the countries. As a reference for the live loads the Dutch values were used, which were the same as the values used for the Eurocode (because Eurocode 1 was not available).
- **case 3** The material factor was set equal for all the countries. As a reference value the material factor was chosen as 2.5, being an average of all the possibilities varying from 1.8 to 3.5.
- **case 4** The load cases were set equal for all the countries. Again the Dutch values were chosen, which were equal to the load cases of the Eurocode.
- **case 5** The last step that was performed was an addition of the three previous cases, which resulted in a situation where only the calculation rules itself differed per country.

This last case showed the influence of the design rules on the design of a loadbearing masonry structure, because the input for the rules was equal for every country.

**CALCULATION RULES**

All the codes are using a limit state philosophy for checking the strength of a masonry structure. Fulfilling the requirements of the ultimate limit state means in five of the six codes at the same time fulfilling the requirements of the serviceability limit state. That was the reason why in this research mainly the ultimate limit state was considered. For the clearness of the comparison the structural elements that had to be checked were divided into stability walls and (ec)centric loaded walls.

The practical calculation of walls takes place by creating a load-function \( S \) and a strength-function \( R \), and checking whether \( S \leq R \). In both functions partial safety factors are taken into account, which results in \( S_d \leq R_d \). As an example some more detailed information on the calculation of stability walls will be discussed.

**Stability walls**

Essentially all the walls in a building work together to resist horizontal forces. A structure
is modelled for the sake of the calculation. In the case of checking the stability this results in calculating only the stiffest elements.

In four codes the strength-functions for stability walls are equal to the ones for (ec)centric loaded walls. Only the German and Swiss code are using separate approaches. The strength-functions are:

NEN 6790 \( \frac{\gamma_M \cdot c \cdot f_{rep} \cdot b \cdot d}{\gamma_m} \)

BS 5628 \( \frac{\beta \cdot t \cdot f_k}{\gamma_m} \)

DIN 1053 \( d \cdot \beta_R \cdot \phi_d \cdot t \cdot f_k \)

NBN B 24-301 \( \frac{\phi_d \cdot d \cdot f_k}{\gamma_m} \)

SIA V 177/2 \( \frac{f_{my} \cdot l_2 \cdot d \cdot (cos \alpha)^2}{\gamma_M} \)

EC 6 \( \frac{\phi \cdot t \cdot f_k}{\gamma_M} \)

Similar parameters in these functions are presented in table 1. Two corresponding parameters were present in all the functions, namely the characteristic compressive strength of masonry and the wall thickness. The material factor was also present in all the codes, with the exception of the German code, as already mentioned. The German and the Swiss code are not using a reduction factor for the effects of slenderness and eccentricity in their functions. The German code is only checking the stresses at the edge of the wall and the Swiss code is using a fundamentally different approach. The widely used approach in the other codes requires a reduction factor, because a linear elasticity theory is applied.

The load-functions also consist of several parameters, such as characteristic loads, partial safety factors and load cases. These parameters also influence to a great extend the final results of the calculation of a structure, and were because of that reason also changed in the different cases in this sensitivity case study.

| Table 1. Parameters in the strength-functions of horizontally loaded walls. |
|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| dimensions | masonry | factors | reduction | material | model | others |
| width | thickness | others | compr. | strength | | | | |
| NEN 6790 | \( b \) | \( d \) | \( f_{rep} \) | \( c \) | \( \gamma_m \) | \( \gamma_M \) | |
| BS 5628 | \( t \) | \( f_k \) | \( \beta \) | \( \gamma_m \) | | | |
| DIN 1053 | \( d \) | \( \beta_R \) | | | | | |
| NBN B 24-301 | \( d \) | \( f_k \) | \( \phi_d \) | \( \gamma_m \) | | | \( (cos \alpha)^2 \) |
| SIA V 177/2 | \( d \) | \( l_2 \) | \( f_{my} \) | \( \gamma_k \) | | | |
| EC 6 | \( t \) | \( f_k \) | \( \phi \) | \( \gamma_M \) | | | |

PARAMETERS IN THE DESIGN RULES
Before showing the results of the calculations it is important to know something about the input values that were used in the calculation for examining the sensitivity. The variable input values were the characteristic live loads, the material factor and the load cases. A remaining important parameter, influencing considerably the results of the calculations, is the reduction factor for slenderness and eccentricity.
Characteristic live loads
In the calculation the values for the dead load of the building were the same for all the countries, based on the assumption that differences are physical impossible. This assumption should also hold for the live loads on a building, but nevertheless the live loads that had to be applied on the building differed. To gain more insight in these differences, the live loads of every specific country were used in the reference case, later these values were changed into the values of the Dutch code NEN 6702 [15].

<table>
<thead>
<tr>
<th></th>
<th>NEN</th>
<th>BS</th>
<th>EC</th>
<th>NBN</th>
<th>DIN</th>
<th>SIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>floors [kN/m²]</td>
<td>1.75</td>
<td>1.50</td>
<td>1.75</td>
<td>2.00</td>
<td>1.50</td>
<td>2.00</td>
</tr>
<tr>
<td>roof terrace [kN/m²]</td>
<td>2.50</td>
<td>1.50</td>
<td>2.50</td>
<td>4.00</td>
<td>3.50</td>
<td>4.00</td>
</tr>
<tr>
<td>roof [kN/m²]</td>
<td>1.00</td>
<td>0.75</td>
<td>1.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>wind pressure [kN/m²]</td>
<td>0.72</td>
<td>0.97</td>
<td>0.72</td>
<td>0.69</td>
<td>0.71</td>
<td>0.70</td>
</tr>
</tbody>
</table>

The calculation for every country was performed by using the live loads belonging to an apartment building. The wind loads were calculated as an average value of the possibilities. These choices resulted in the values in table 2. The differences between the characteristic live loads influenced the results of the design rules. In the load-functions this influence is linear proportional, in relation to the load factors used in the load cases.

Material factor
A parameter that is also linear proportionally influencing the results of the design rules is the material factor (also known as partial coefficient for the material properties). Only the German code DIN 1053 does not contain such a material factor, Germany uses one general safety factor. Great-Britain, Belgium and the Eurocode are using a material factor that is related to categories of construction and manufacturing control. Both Switzerland and the Netherlands use only one value, independent of the type of control. Table 3 provides an overview of the possible values for the material factor.

<table>
<thead>
<tr>
<th>category</th>
<th>NEN</th>
<th>BS</th>
<th>EC</th>
<th>NBN</th>
<th>DIN</th>
<th>SIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA</td>
<td>2.5</td>
<td>2.0</td>
<td>2.5</td>
<td></td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>AB</td>
<td>3.1</td>
<td>2.3</td>
<td>3.0</td>
<td>or</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>BA</td>
<td>2.8</td>
<td>2.5</td>
<td>3.0</td>
<td>2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BB</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

It is important to know that the different categories in table 3 do not coincide with each other (AA in Belgium does not mean AA in Great-Britain for instance, and the Eurocode contains three categories of construction control). The German values are not really material factors, but general safety factors. There were some exceptions:
- Great Britain: where wall tests in accordance with the code have been carried out, the \( \gamma_m \) factors may be taken as 0.9 times the values given in table 3;
- Belgium: in the case of the serviceability limit state the values may be reduced
to respectively 1.05, 1.15, 1.15 and 1.3;
- European Community: in the case of accidental actions the stability may be verified by using the $\gamma_m$ factors 1.2, 1.5 and 1.8, for the three categories of construction control.

### Load cases
A structural engineer makes a choice which load cases are decisive for a given structure, although in theory he should check all the possibilities. In the calculation example also a choice was made between three load cases for the calculation of the stability of the building and two load cases for the calculation of the (ec)centric loaded walls. Table 4 is showing an overview of the different load factors that were used in the load cases.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Load</th>
<th>NEN</th>
<th>BS</th>
<th>EC</th>
<th>NBN</th>
<th>DIN</th>
<th>SIA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dead</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.0</td>
<td>1.0</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>live</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>wind</td>
<td>1.5</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>dead</td>
<td>1.2</td>
<td>1.4</td>
<td>1.2</td>
<td>1.35</td>
<td>1.0</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>live</td>
<td>0.6</td>
<td>-</td>
<td>0.6</td>
<td>0.5</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>wind</td>
<td>1.5</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>dead</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td>1.35</td>
<td>-</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>live</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td>1.5</td>
<td>-</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>wind</td>
<td>-</td>
<td>1.2</td>
<td>-</td>
<td>0.7</td>
<td>-</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>dead</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.0</td>
<td>1.0</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>live</td>
<td>1.5</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.0</td>
<td>1.5</td>
</tr>
</tbody>
</table>

The choice of the load cases was of course questionable. The intention was to put similar load cases of different countries next to each other in the calculations. This means that load case I of the calculation is consisting of similar load cases. The reason that the values for Germany were all set to 1.0, was because of the general safety factor Germany uses. Another difference can be noted in the use of a low load factor of 0.5 or 0.6 in certain countries for the live load. This was due to the fact that at least the nominal value of the live load had to be taken into account for that load case. The nominal live load means in this context a reduced part of the total live load. The peak load factors differ 5 to 20%, differences which do not cause big differences in the calculation results.

### Reduction factor for slenderness and eccentricity
The calculation of stability walls in Germany and Switzerland does not require the use of a reduction factor for the effects of slenderness and eccentricity. In the Swiss code no direct reduction factor for these effects is present, but the effect is taken into account in tables and formulas. In the German code only for (ec)centric loaded walls a reduction factor is present.

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Fig. 3: Capacity reduction factor in the case of a slenderness ratio of 0.

Fig. 4: Capacity reduction factor in the case of a slenderness ratio of 10.

Fig. 5: Capacity reduction factor in the case of a slenderness ratio of 20.

For a comparison of the different reduction factors two different parameters had to be considered, which independently influence the value of the factor. The two parameters were \( \text{elt} \) (eccentricity of the force divided by the thickness of the wall) and the slenderness ratio \( h/l \) (height of the wall divided by its thickness). For different slenderness ratios it was possible to create graphs of the reduction factor being a function of \( \text{elt} \).

Figures 3, 4 and 5 show graphs for slenderness ratios 0, 10 and 20. The resemblance is the decrease of the reduction factor with an increase of \( \text{elt} \). The shapes of the curves are for all slenderness ratios completely different. A creep coefficient was in this research not considered, mainly because only the Eurocode includes one.

The parameter \( \text{elt} \) was not equal in all the countries, because the determination of the total eccentricity was not the same in all the codes. It is especially the determination of the additional eccentricity that causes differences in the final reduction factor for slenderness and eccentricity. Comparing the differences is difficult (perhaps even impossible), because the determination of these additional eccentricities is based on different assumptions in the codes. Nevertheless an enumeration of the determination of this additional eccentricity will be presented here.

**NEN 6790.** In the case of walls in a shored framework the additional eccentricity must be determined by:

\[
e_c = 4.5 \times d \times \left( \frac{\rho \times l}{100 \times d} \right)^2
\]

with: \( \rho \)

- is 1 in the case of two sided supported walls
- is \( l_1/l \) (>1) in the case of three sided supported walls
- is \( l_2/2l \) (>1) in the case of four sided supported walls
- is the storeyheight
- \( l_1 \) is the distance from the centre of the supporting wall to the free edge
- \( l_2 \) is the distance between the centres of the supporting walls
- \( d \) is the thickness of the wall
In the case of a wall or column fixed only on one side (freestanding), with a constant normal force over the full height of the wall, the additional eccentricity must be determined by:

\[ e_c = 18 \times d \times \left( \frac{h}{100 \times d} \right)^2. \]

In the case of stability shafts the additional eccentricity must be determined by:

\[ e_c = \frac{Q_{sd}}{N_d'} \times 4.5 \times d \times \left( \frac{h}{100 \times d} \right)^2. \]

with: \( Q_{sd} \) is the design value of the vertical load of that part of the building that derives its stability from this stability shaft
\( N_d' \) is the design value of the normal force in this stability shaft
\( h \) is the height of the column or wall above the fixation
\( d \) is the biggest dimension of the horizontal cross section in the direction of bending

Additionally in the last two cases all the calculated eccentricities have to be multiplied with an extra factor \( (\xi) \), being mainly the result of the rotation capacity of the foundation of the wall or column. This results in the following formula:

\[ e_t = \xi \times (e_o + e_x). \]

**BS 5628.** The additional eccentricity in the British Standard should be calculated with the formula:

\[ e_o = t \times \left[ \frac{1}{2400} \times \left( \frac{h_{ef}}{t_{ef}} \right)^2 - 0.015\right]. \]

The total design eccentricity in the mid-height region of a slender wall is:

\[ e_t = 0.6 \times e_o + e_x. \]

The final design eccentricity \( (e_m) \) is the larger of \( e_t \) or \( e_x \).

with: \( t \) is the thickness of the wall (or depth of column)
\( t_{ef} \) is the effective thickness of the wall or column
\( h_{ef} \) is the effective height of the wall or column
\( e_x \) is the calculated eccentricity at the top of the wall

**EC 6.** In the Eurocode the accidental eccentricity depends on the quality (category) of construction control. This results in three possibilities, namely:

\[ e_x = \frac{h_{ef}}{300}, \frac{h_{ef}}{450} \text{ or } \frac{h_{ef}}{600}. \]

Although the creep eccentricity is not accounted for in this research, the formula in the Eurocode for calculating it will be explained here. In the case of clay units this eccentricity is 0, but in all the other cases the eccentricity should be calculated by:

\[ e_k = 0.002 \times \phi_w \times \frac{h_{ef}}{t_{ef}} \times t \times e_m. \]

with: \( \phi_w \) is the creep coefficient depending on the type of unit
\( h_{ef} \) is the effective height
\( t_{ef} \) is the effective thickness of the wall
\( t \) is the thickness of the wall
\( e_m \) is the final design eccentricity
NBN B 24-301. In the Belgium code the additional eccentricity is $1/300 \times h_w$. The final design eccentricity should be calculated using the maximum of the following two combinations:

- $0.6 e_1 + 0.4 e_2 + e_w$
- $0.4 e_1 + e_w$

with:

- $e_1$ is the eccentricity at the top of the wall
- $e_2$ is the eccentricity at the bottom of the wall
- $e_w$ is the eccentricity due to wind and thermal effects of the facade

DIN 1053. The additional eccentricity in the German code is: $f = \frac{\lambda}{1800} \times \frac{1 + m}{h_k} \times h_k$

with:

- $\lambda$ is the slenderness ratio of the wall
- $h_k$ is the effective height of the wall
- $m$ is the relative eccentricity at mid height of the wall

CALCULATION EXAMPLE

The seven storey apartment building that was used in this research (Fig. 1 and 2), consisted of many loadbearing walls. Only the walls A to E were calculated. The five studied cases were:

- case 1 a reference calculation, in accordance with the matching codes of the country;
- case 2 equal characteristic live loads for all countries;
- case 3 equal material factors for all countries;
- case 4 equal load cases for all countries;
- case 5 an addition of case 2, 3 and 4.

The figures 6, 7, 8 and 9 show graphically the results of these calculations and also the influence of the different steps that were taken in the cases. The results of the Eurocode show no changes, the results are the same for all the cases. This is due to the fact that the chosen cases were all in accordance with the assumptions for the calculation with the Eurocode. For the Dutch code only case 3 resulted in a higher required compressive strength, which is obvious when the material factor is increased from 1.8 to 2.5.

Case 2 (changing the characteristic live load) resulted in lower required compressive
strengths for the stability walls of Great-Britain, Belgium and Switzerland, and a higher required strength for Germany. The (ec)centric loaded walls showed an increase for Great-Britain and again Germany, and a decrease for Belgium and Switzerland. The required compressive strength did not get much closer to each other in this case.

**Case 3** (changing the material factor) resulted in higher required compressive strengths for the Dutch calculation, and for the stability walls of the German calculation. In the case of the Netherlands the increase is the result of the increase of the material factor from 1.8 to 2.5. In Germany the calculation of stability walls takes place by using a general safety factor of 2.0, this in contradiction with the use of a factor of 2.5 for (ec)centric loaded walls.

**Case 4** (changing the load cases) resulted in higher required compressive strengths for the stability walls of Great-Britain and Germany, and in lower strengths for Belgium and Switzerland. The results of the calculation of the (ec)centric loaded walls only resulted in higher compressive strengths for the German code, for all the other codes this step meant a decrease of the required compressive strength.

**Case 5** (the addition of case 2, 3 and 4) finally resulted in higher required compressive strengths for The Netherlands, Germany and Switzerland. The values of Great-Britain and Belgium, the countries with the highest values in case 1, decreased due to this calculation step.

**SENSITIVITY**

![Fig.8: Calculation results of wall A.](image)

![Fig.9: Calculation results of wall C.](image)

The structural design rules in European masonry codes are using input values which can be divided into loads, material properties and dimensions. In this research the dimensions and the dead load of the building were kept constant. The result of the calculations was the required compressive strength. The material factor, the characteristic live loads and the load cases were changed in the calculations.

The most interesting case in this sensitivity case study should be case 5, because this case was based on the same input for all the countries and therefore it should clearly show the effects of the design rules on the calculation results. Perhaps one would expect that this case would result in similar results, which is likely when the design rules in the different countries are equal. But as the results show, the design rules are not equal. All the design rules are different and it is even remarkable to note that the differences are most of the time bigger than in case 1.
In case 5 Great-Britain and Belgium required a much lower compressive strength for the masonry of the stability walls than in case 1. In both cases this was the result of the decrease of the characteristic live loads and more favourable load cases. The fact that the results were also lower than other countries in case 5 means that these countries have favourable design rules for the design of stability walls. The Netherlands, Germany and Switzerland were confronted with an increase of their required strength, caused by the increase of the material factor. In the case of Germany load factors were introduced on top of their general safety factor.

For the (ec)centric loaded walls some similar interesting changes took place as for the stability walls. In case 5 Great-Britain and Belgium required lower compressive strengths for the stability walls, this was also the case for the (ec)centric loaded walls. In Great-Britain this was now only due to more favourable load cases. The increase of the required compressive strength in the case of Germany and the Netherlands was due to the same causes as for the stability walls, only that in the case of Germany the material factor did not change.

CONCLUSIONS

This sensitivity case study of parameters in European masonry codes does not result in conclusions which point out that certain design rules are wrong and others are right. The past has proved that the design rules in all the European masonry codes are, from the matching countries point of view, right. Structures were calculated and build, without any problems up until now. For getting to acceptable design rules for the future Euro-code, it is advisable not to continue with focussing on trying to get the different design rules closer together. Efforts into that direction will not result in satisfying results for the different participating members. It is more advisable to select the best matching design rules and use them in the future European masonry code. From that point of view it is worth having a closer look on the developments that took place for the new Swiss code SIA 177/2. From a fundamental point of view the design rules are different, but nevertheless the results of the calculation match with the average of all the results. Most important reason for making a choice like this is the fact that a discussion about an important parameter in the design rules, the reduction factor for the effects of slenderness and eccentricity, is not necessary.

Besides the differences between the design rules there are also differences between the input values for the design rules. Load cases and characteristic loads differ in Europe. In several codes this is taken into account in a camouflaged way, mainly via rules regarding loads and load cases. Nevertheless codes for the design of structures with a certain material, should be independent from loads and load cases. This is an extra reason to create rules in masonry codes independent from loads and load cases.

The material factor is, and probably always will be, a stumbling-block in the development of a masonry code. Differences from 1.8 up to and including 3.5 are of course absurd, especially when the influence of this material factor is proportionally with the results of the design rules. When certain countries have the experience that with one material factor all the required safety can be achieved (from a statistic point of view), than it is not necessary to make still a selection out of the statistically possible cases. Research into the direction of finding the 'right' safety deserves more attention.
ACKNOWLEDGEMENTS

This work was carried out at the University of Technology of Eindhoven (TUE) in The Netherlands as part of a more extensive project dealing with the structural design of masonry. The work is sponsored by the Science Foundation (STW) and steered by working group 1 of steering committee C76 of the Centre for Civil Engineering, Research and Codes (CUR).

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Harrie J. Vekemans
SHEAR TESTS ON MASONRY PANELS OF 1 X 1 M²

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ABSTRACT

This paper describes shear tests on masonry panels of 1 x 1 m². The tests are carried out to verify the results of FEM and DEM calculation. The main problem with these tests were the boundary conditions. In the preceding numerical analysis the horizontal upper edge of the specimen is supposed to move parallel to the bottom edge. To be able to control this condition during testing three deformation controlled vertical 150 kN jacks are used. The horizontal deformation is applied by one 450 kN jack. The deformation of the test rig itself is minimalised by two struts with hand controlled tensile jacks. The deformations of the specimen are measured by 19 LVDT's mounted to the specimen on a 10 point grid. The test series will be completed before the end of this year. Then the results will be compared in detail with the results of the FEM and DEM calculations. To be able to do this the material parameters are established with small scale specimens made and tested simultaneously with the shear specimens of 1 m². Also the results of preceding small scale compression, tension and shear tests will be used. Until now already large resemblance is found between numerical and experimental cracking and failure modes.

INTRODUCTION

In 1988 as an initiative of the Royal Dutch Brick association a project called Brick Research Innovation and Knowledge transfer (BRiK) is started. In this project research is carried out after the behaviour of structural loaded masonry. This takes place in a numerical (A33), an experimental (B50) and an analytical (C77) way. These researches are coordinated by CUR. Research is carried out to verify numerical research carried out by CUR A33. A wall loaded with shear forces is calculated. The tests described in this paper are carried out to verificate the numerical models. This took place under the supervision of B50. At TU-Eindhoven 16 tests in total will be carried out. At this moment 12 tests are finished. In the experiments the same boundary conditions as used for the numerical research will be used. In the numerical research parameters will be varied to find an optimal relation between the experiments and the numerical model. This proces is called inverse modelling. The results of the tests will be adepted to be used in structural designing bij CUR commission C77.
NUMERICAL RESEARCH.

Numerical analysis has been carried out on the same type of specimens as concerned here with the FEM program DIANA [1] and the DEM program UDEC [2]. The model consists of a wallet of 1 x 1 m$^2$ (length x width) of 100 mm thickness. The horizontal top edge is moved parallel to the bottom edge of the specimen. Figure 1 shows how the loading and boundary conditions are modelled. Two sides of the wallet are completely free, no introduction of forces will take place there. Figure 2 shows stresses and the crack pattern calculated by UDEC for a specimen with a hole.

fig. 1: Boundary conditions

fig. 2: Calculation with UDEC, crack pattern and stresses

The following conditions are created for the bottom and upper side of the specimen:

a. Vertical displacements of the upper edge are prevented. This is realized by a stiff beam which is prevent to move in vertical direction by three vertical jacs.

b. Only the upper edge of the specimen can move horizontally. The beam at the bottom side is connected to the test rig.

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c. The horizontal load acts at the upper edge of the specimen as a uniform distributed shear load. In fig. 1 the force \( H \) acts at the upper beam. This beam distributes the load uniform over the edge of the specimen.

**CONSEQUENCES OF THE BOUNDARY CONDITIONS**

While the vertical distance between the two beams over the full length keeps the same, the specimen can not be rotated by the horizontal load. The vertical reaction forces needed are transmitted to the specimen by the edge beams. Figure 3 shows the development of the stress distribution in the upper and bottom edge of the specimen when the horizontal load increases. At low horizontal load tensile and compression stresses will occur at top and bottom edges of the specimen. See situation A at fig. 3. When the bond strength of the masonry is exceeded cracking will occur at two opposite corners, fig. 3 situation B. To be sure that these cracks develop in the masonry and not in the connection between the specimen and the edge beams, this connection has to be strong enough. After the cracking of the tensile area it is supposed that shear forces transmitted by the cracked area will be so small that they can be neglected. After that time the shear and compression forces are only transmitted by the remaining area. When the horizontal displacements and thus also the horizontal force increase the compression and shear stresses will be more and more concentrated near the corners of the specimen. See situation C in fig. 3. The horizontal load and vertical reaction force will concentrate more and more in the same corners. The direction of the resultant of these forces will be almost the same as the diagonal of the specimen.

**TEST SET-UP**

The boundary conditions discussed in the previous paragraph are used as a basis for the design of the test set-up. In fig. 4 the outline of the test setup is given. The dimensions of the specimen are 1000 x 1000 x 100 mm³ (length, height, width) The specimen is placed on a rigid base, which cannot deform or move. At the top side a beam
is connected to the specimen. This beam is longer the specimen, so jacks can be placed at both sides of the specimen, see fig. 4. The vertical jacks are numbered 2, 3 and 4. These jacks take care that the upper beam stays in a horizontal position. These jacks also create the forces needed to keep the specimen in equilibrium when the horizontal load is applied by jack number 1.

The jacks and the specimen are placed in a ridged testing frame made of HE300B members. The three vertical jacks are displacement controlled separately. Two test-specimen are made to test the controlling of the jacks and the behaviour of the specimen in the test setup.

At the start of the test it is necessary to have a vertical load. If there is no vertical load the connection could collapse to early by shear. Before the horizontal jack is used, a load of 30 kN in total will be applied by the vertical jacks. From that moment on the vertical jacks will be used to keep the upper beam in a horizontal position.

Supposing the compression strut in the specimen at failure is 250 x 100 mm² and the compression strength of the masonry is 20 N/mm² the maximal expected force in the horizontal jack number 1 equals 360 kN. The jacks number 2a, 2b and 3 have a capacity of 150 kN each. The forces in the jacks depend from the dimensions of the specimen and the distances of the jacks, see fig. 5.

Jacks 3 and 4 are placed at 500 mm and jack 2 is placed 300 mm outside the specimen. The horizontal force of 360 kN acts at 1.0 m distance from the bottom of the specimen. The center of gravity of the total vertical loads is at 100 mm from the left edge of the specimen. The forces in the jacks when they are fully loaded are equal to:

\[ F_1 = 360 \text{ kN}, F_2 = F_3 = F_4 = 150 \text{ kN}. \]

**fig. 5**: Forces on the panel.

**SPECIMENS**

The specimens are made with two kinds of bricks. A soft mud brick fabricated by "The Vijf Eiken" and a wire cut brick fabricated by "Joosten" called VE and JG respectively. The strength of the masonry made with the JG bricks is about two times that of the masonry made with VE bricks, see [3, 4, 5 and 6].

The materials used are as much as possible the same as the ones used for other masonry research carried out recently in our laboratory. To become the right parameters for the FEM and DEM calculations and to control the quality of the masonry, small scale specimens are made and tested simultaneously with the larger elements. Thus compression and bondstrength is established. Other parameters are taken from research described in [3, 4 and 5].

Figure 6 shows a detail of the principle of the connection between the beams and the specimen. At top and bottom side of the specimen one layer of bricks is poured in a high
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strength grouting mortar. Between the steel top and bottom beam a joint with a width of a few centimeters is created. In this joint provisions are taken to improve the force transmission from beam to specimen. Therefore steel bars of 10 x 10 mm² are placed in heavy bars. These bars are bolted to the beams.

The vertical dimension of the specimen is the distance between the mortar joint above the bottom layer of bricks and the mortar joint beneath the top layer of bricks. This dimension is 975 mm. The length of the specimens is 4.5 bricks and 4 header joints which is 980 mm.

Not only closed but also specimens with a hole are tested.

The dimensions of the hole were one brick wide and 5 layers high. The hole was situated 6 layers from the bottom side and two brick lengths from the left side of the specimen, see fig. 2.

All specimens are loaded with a total vertical load of 100 kN, centered in the middle of the specimen, which created a stress of 1 N/mm². With the deformations measured at this stress level the Young-module is established. The thus found Youngs-module can be compared with the one found in earlier research [3 and 4] and with the one found on the small specimens simultaneously made with the same mortar and bricks as the large specimens. After the centrally vertical pre-loading of the specimen the load is decreased to 30 kN. Then the real test started with horizontal deformation of the specimen.

MEASUREMENTS

The following measurements are made:
- The forces in the jacks by loadcells
- The horizontal displacement of the upper edge
- The deformation of the specimen by LVDT’s placed in a grid
- The displacements of the corners of the specimen by LVDT’s

The results of the measurements with the grid, along with the other results, will be used to verify the numerical results.

At different load levels the crack pattern is photographically recorded. Therefore one side of the specimen is painted white. The crack pattern of specimen J3d is shown in fig. 8.

First the horizontal cracks appear. The forces are concentrating in the corners of the specimen, as expected and also found in the numerical analysis. At the end of the test a compression strut is formed. The specimen collapses when the mortar at the two opposite corners crushes and the bricks in the compression strut crack, due to the tensile stresses perpendicular to the direction of the force in strut.

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The measurement grid originally existed of 30 points with 69 measured deformations. The deformations were measured with a removable electronic device. After the first results where analyzed it showed that the errors were too big. The removable device was useless for this purpose. It is decided to change the grid and to measure further on with LVDT's. The new grid is shown in fig. 7.

Now the grid has 10 points and 19 measurements are made. Figure 9 shows the deformed grid at different loadlevels for specimen J3d. The measured deformations are recalculated to joint displacements by means of a plane truss computer program. The deformation at 87.1 kN is measured just before collapsing of the specimen. The deformation at 80.0 kN just after that.

RESULTS OF THE ORIENTATING TESTS

During the first four tests the test rig appeared to deform too much. The horizontal jack gives a force to the specimen and an opposite force to the test rig. The lengthening of the jack is the sum of the deformation of the specimen and the deformation of the test rig. The displacement of the rig is 3 to 4 mm measured from the bottom of the specimen to the place where the horizontal jack is mounted. The displacement of the horizontal jack is controlled by a sensor in the jack, so not only the deformation of the specimen but also that of the frame is measured, thus influencing the controlling signal. When the specimen cracks, the horizontal load will drop. Then a proportional part of the deformation of the test rig is transmitted to the specimen. The specimen gets a push.

To prevent this "pushing of the frame" two diagonals with tension jacks are placed in the test rig, reducing the deformation of the rig to 0.5 mm at failure load. The tension jacks in the diagonals are stressed before the test is started to a load of 150 kN each, so the rig is "prestressed" and deformations of the joints are taken away.

It was necessary to control the vertical jacks with external LVDT's. This caused problems with the controlling of the jacks at a moment that there was no connection between the jacks and the specimen. Owing to this during one test a jack became unstable which gave
a wrongly loaded specimen which collapsed to early.
The recipe of the mortar for the first specimens was different from the recipe used for preceding other tests. After this was noticed the recipe is changed.
At this moment the tests on the first 12 specimens are almost completed. After analyzing the results, which is taking place at this moment, the four remaining specimens will be tested in a way which is depending on the results gained until now.

ACKNOWLEDGEMENTS

The described experimental research is part of a program in the Netherlands called Brick Research Innovation and Knowledge transfer (BRIK), initiated by the Royal Dutch Brick association (KNB), and supervised by the Netherlands Centre for Civil Engineering Research Codes and Specifications (CUR).
The support and work of the people of the van Musschenbroek is gratefully acknowledged.

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Research on building structures and building physics.
Mechanical properties under compression of masonry and its components.

Ad Th. Vermeltfoort

ABSTRACT

In the first part of this paper the results of compression tests on specimens of one brick wide and 5 bricks height are discussed. These tests were carried out to find accurate figures for the parameters of bricks, mortar and the interface between these two, to be used in finite or discrete element method computer programs to calculate masonry strength and deformations.

In the second part of this paper the results of more recent compression tests on masonry made of different brick and mortar qualities and different specimen sizes will be discussed. Here special attention will be paid to the influence of mortar and bricks on the strength of masonry. An in situ test method on structural masonry will be proposed. For the complete test program discussed here specimens are made in the laboratory. Later specimens made on a number of building sites in real weather conditions will be tested to verify the possibilities of the proposed building site tests.

The main goals of the tests are:
- Establishing the mechanical compression properties of masonry, bricks and mortar to be used in (computer-) calculations.
- Establishing an aspect ratio between wide and narrow specimens.
- Establishing the influence of the width of the specimen on the mechanical properties, to be able to develop a standard testing method for structural masonry on building-sites.

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INTRODUCTION

The relation between brick, mortar and masonry compressive strength has been investigated many times already, and the strength of masonry can be predicted by formulae or tables as a function of brick and mortar strength. In these kind of relations many influences are neglected. The compression strength of masonry according to the Dutch code of practice is conservative, but if higher strengths are expected, verifying tests may be done. The specimen which has to be used for these verifying tests is rather large, using a smaller one would have advantages.

This paper deals with the results of tests, on relatively small specimens which were carried out to establish the mechanical properties of masonry under compression. The results will be used for the development of a simple test procedure for controlling the properties of structural masonry on building sites.

The results also will be used in FEM or DEM computerprograms to calculate the properties of larger elements. At this moment (october 1992) tests are carried out on shear loaded walls of 1 m² to verify the numerical results of calculations with the results of the tests concerned here on this type of specimen.

SPECIMEN FABRICATION.

At the Van Musschenbroek laboratory of Eindhoven University of Technology the tests mentioned in this paper were carried out. All specimens were prepared in the laboratory at a temperature of ±15 °C and a relative humidity of ± 65 %. All masonry has been constructed under close supervision to minimize workmanship effects.

The bricks used are marked as follows:
PO : Poriso
VE : Vijf Eiken
JG : Joosten Yellow
JB : Joosten Blue
KZ : Sand lime
PO and VE are soft mud bricks, JG and JB are wire cut bricks.

The masonry used for series 1 tests is constructed from solid clay bricks (VE and JG)
fig. 1 Dimensions testspecimens.
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calcium silicate bricks (KZ) and three different types of mortar in the following cement lime sand ratios: 1:2:9 & 1:1:6 & 1:½:4½. The dimensions of all bricks were ± 52x100x210 mm³. The specimens for series 1 are kept as small as possible with bricks which are processed as little as possible.

The smallest height depth ratio of a specimen in which an area with uniaxial stresses will occur is about 1.5 to 2, so a height of about 300 mm is requested. The specimens with mortar consist of 5 bricks with 4 joints. In series 1 the contact areas of the upper and lower brick with the loading platens are ground level. All ground surfaces had a difference in height of 0.02 mm average (0.05 mm max), measured over 150 mm along the diagonal of the brick. The maximum difference in thickness of the ground bricks is 0.10 mm

In series 2 a mortar-joint on top and bottom is used. This saved the grinding work, so the fabrication of these specimens was easier. The height of the specimen was 5 bricks and 6 joints, equal to ± 330 mm. These specimens have a bedjoint on top and bottom, so the bricks can be used without processing, see fig. 1. The dimensions of the wide specimen were according to EC6, p120.

Three mortar recipes and all five brick qualities mentioned above were used for the masonry of series 2. The same bricks as in series 1 are used, but one kind of soft mud bricks and one kind of wire cut bricks are added to the program to have more differences in brick strengths. Specimens with the same brick-mortar combination were simultaneously prepared and tested. So the only intentional difference between the tests is the width.

In the proportions for the mortars for series 2 only the binder ratio is varied. The cement-lime ratio was always c:l = 1:½. The following cement-lime-sand volume ratios have been used in series 2: 1:½:1½ ; 1:½:4¼ en 1:½:9. Not all possible brick/mortar combinations have been tested.

To prevent drying out of the specimens they are covered with plastic during the first three days. After that they are stored in the climate room at a temperature of 20±0.5°C and a relative humidity of 60±3 % until testing.

During hardening the top bricks of the specimen can get loose. This is prevented with three layers of bricks, piled on top of the specimen, which give a uniform pressure of ± 300 N/m².

The specimens of series 1 are tested after 90 days minimum. Those of series 2 are tested after 28 days. Before applying the bricks they are saturated in water for 7 minutes and kept in plastic for 12 hours, so the water could spread in the bricks. With this method the Haller-ratios are (in gr/dm²/min): PO 31.6; VE 11.4; JG 9.1; JB 3.3 and KZ 5.0.
MEASUREMENTS

The following deformations were measured (see fig. 2):

a: The deformation of one brick with two joints. The measure length was 78 mm for series 1 and 112 mm for series 2.
b: The deformation of the brick on different places around the specimen. The measure length was about 40mm. In series 2 a special developed measure clip with strain gauges was used.
c: The deformation in lateral direction of the brick.
d: The deformation in lateral direction of the brick.
l: The deformation of the complete specimen at the four corners of the loading platens.

Not all measurements are carried out at all specimens. Small changes are made in the measure method during the course of the tests.
The displacement velocity of the loading platens was 0.30 mm/min. Testing one specimen took about 30 minutes.

Figure 3 shows the load-deformation relation of the complete specimen measured between the loading platens (measurement type l).
BRICK STRENGTH.

To establish the properties of the bricks themselves a pile of seven bricks that were ground level on both sides, is used. The height of these specimens is about 320 mm, which is about the same as the height of the specimens with mortar.

Measuring the deformations of such a specimen showed that, due to deformations in the contact area the strain of a single brick was smaller than the strain of the complete pile. Comparing the deformations of the bricks with those from the complete specimen no correspondence could be found.

The average brick deformation as measured on a pile of ground VE-bricks is shown in fig 4. It is remarkable that for one of the specimens the average deformation only started when the applied load was over 180 kN and that the recorded deformation-line is parallel to the lines from the other tests.

To be able to measure these deformations accurately, ground bricks were glued together with a thin layer of a two component material called Bolidt. The deformations of these specimen measured over two seams and two bricks and the deformation of a single brick in the same specimen were proportional to the measure length. The influence of the joint filling on the strength is small. The thickness of this layer is smaller than 0.5 mm. That is why the strength and stiffness of the pile with a joint filling can be seen as representative for the properties of the brick. The measured strengths and E-moduli of the bricks are presented in table 1.

Table 1. Strength and E-moduli of bricks, measured on piles of ground bricks glued together with Bolidt. Values in N/mm².

<table>
<thead>
<tr>
<th>Type</th>
<th>Strength</th>
<th>E-module</th>
</tr>
</thead>
<tbody>
<tr>
<td>PO</td>
<td>10.0</td>
<td>6530</td>
</tr>
<tr>
<td>VE</td>
<td>14.2</td>
<td>6050</td>
</tr>
<tr>
<td>JG</td>
<td>35.8</td>
<td>16700</td>
</tr>
<tr>
<td>JB</td>
<td>64.8</td>
<td>15400</td>
</tr>
<tr>
<td>KZ</td>
<td>39.0</td>
<td>13400</td>
</tr>
</tbody>
</table>

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MORTAR STRENGTH

To control the mortar quality prisms of 4 x 4 x 16 cm were made. These prisms were tested after 28 days. The results are shown in table 2. The hardening conditions in the steel mould, and later in the climate room, were very different from the conditions of the mortar in the joints between two bricks. Thus specimens made to establish the quality of a mortar cannot be used to establish the mechanical properties of the mortar between bricks. These properties have to be determined via tests on masonry specimens.

DEFORMATION OF BRICKS IN SPECIMENS WITH MORTAR.

It was expected that, during testing, a certain measure inaccuracy due to the inclination of the connecting puns would develop. Therefore special test are carried out, which showed that this error was to big to give reliable results for the masonry specimens made with Vijf Eiken bricks. As a global view it can be mentioned that the deformations cover a large area and that a number of sensors only started recording after a loading of 150 kN or more. The scatter in the measured brick deformations in series 1 is large. Because of that a measure-clip with strain gauges is developed for series 2. Now the deformations of the brick could be measured more accurately and close to the surface of the specimen. Figure 4 shows the mounting of these clips and a graph of the load deformation relation for Sand lime bricks measured with these clips.

![Graph of load-deformation relation for KZ-bricks in a specimen with mortar.](image-url)

fig. 4 Mounting of a clip.
Load-deformation relation for KZ-bricks in a specimen with mortar.
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**MASSONRY STRENGTH: WIDE VERSUS NARROW**

The strength of masonry is determined by the strength of its components and by the way they are processed. In most codes of practice and formulae the strength of the brick and the mortar is used. Then the brick strength is established with a single brick, the mortar strength is established with compression tests on prisms and the influence of the processing, the thickness of the joints and other execution-factors are not taken into account. An adequate procedure is needed for fabricating specimens on the building site and for testing, because the parameters used in calculations have to be controlled and guarded.

Table 2. Compression strength [N/mm²] of mortars, bricks and wide and narrow specimens. Each value is the average of three test results.

<table>
<thead>
<tr>
<th>mortar c:l:s</th>
<th>strength</th>
<th>brick type</th>
<th>strength wide</th>
<th>narrow</th>
<th>wide/narrow</th>
<th>calc</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:½:9</td>
<td>3.3</td>
<td>PO</td>
<td>10</td>
<td>4.6</td>
<td>3.7</td>
<td>1.26</td>
</tr>
<tr>
<td>1:½:9</td>
<td>2.9</td>
<td>VE</td>
<td>32</td>
<td>14.2</td>
<td>7.1</td>
<td>6.5</td>
</tr>
<tr>
<td>1:½:9</td>
<td>2.9</td>
<td>JG</td>
<td>66</td>
<td>35.8</td>
<td>8.6</td>
<td>9.0</td>
</tr>
<tr>
<td>1:½:4½</td>
<td>12.5</td>
<td>PO</td>
<td>10</td>
<td>10.0</td>
<td>5.1</td>
<td>3.0</td>
</tr>
<tr>
<td>1:½:4½</td>
<td>9.8</td>
<td>VE</td>
<td>32</td>
<td>14.2</td>
<td>10.4</td>
<td>8.7</td>
</tr>
<tr>
<td>1:½:4½</td>
<td>9.8</td>
<td>JG</td>
<td>66</td>
<td>35.8</td>
<td>15.9</td>
<td>16.3</td>
</tr>
<tr>
<td>1:½:4½</td>
<td>10.3</td>
<td>JB</td>
<td>120</td>
<td>64.8</td>
<td>23.2</td>
<td>20.9</td>
</tr>
<tr>
<td>1:½:1½</td>
<td>47.9</td>
<td>VE</td>
<td>32</td>
<td>14.2</td>
<td>12.8</td>
<td>10.5</td>
</tr>
<tr>
<td>1:½:1½</td>
<td>47.9</td>
<td>JG</td>
<td>66</td>
<td>35.8</td>
<td>36.9</td>
<td>31.2</td>
</tr>
<tr>
<td>1:½:1½</td>
<td>41.0</td>
<td>JB</td>
<td>120</td>
<td>64.8</td>
<td>45.5</td>
<td>39.8</td>
</tr>
<tr>
<td>1:½:4½</td>
<td>10.4</td>
<td>KZ</td>
<td>--</td>
<td>30</td>
<td>20.0</td>
<td>20.0</td>
</tr>
<tr>
<td>1:1:6</td>
<td>6.3</td>
<td>KZ</td>
<td>--</td>
<td>30</td>
<td>20.0</td>
<td>20.0</td>
</tr>
</tbody>
</table>

VE = Soft mud brick  JG = Yellow wire cut brick
PO = Poriso      JB = Blue wire cut brick
* Brick strength from bricks 50 mm thick, according to NEN 3838
** Brick strength of a pile of 7 ground bricks with joint filling.

Mortar strength according to NEN 3835, prism strength after 28 days.

calc = calculated with: \( f_k = 0.4f_b^{0.75}f_m^{0.25} \) according to [1]

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In table 2 the compression strength is calculated by dividing the maximum force by the gross sectional area. Extreme values were found for the poriso brick. This is a very porous brick which is cracked heavily inside, due to its fabrication process, which declares these extreme values.

The average compression strength of the series 1 tests compared with those from series 2 is given in table 3.

Table 3. Compression strength of the masonry specimens of series 2 compared with those from series 1.

<table>
<thead>
<tr>
<th>Mortar</th>
<th>Brick</th>
<th>B50 [4]</th>
<th>Old</th>
<th>200x100</th>
<th>Old</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:4:4:4</td>
<td>VE</td>
<td>10.2</td>
<td>95</td>
<td>8.7</td>
<td>32</td>
</tr>
<tr>
<td>1:4:4:4</td>
<td>JG</td>
<td>19.2</td>
<td>102</td>
<td>16.3</td>
<td>32</td>
</tr>
<tr>
<td>1:4:4:4</td>
<td>KZ</td>
<td>19.2</td>
<td>--</td>
<td>16.3</td>
<td>32</td>
</tr>
<tr>
<td>1:1:6</td>
<td>KZ</td>
<td>20.0</td>
<td>---</td>
<td>20.0</td>
<td>32</td>
</tr>
<tr>
<td>1:2:9</td>
<td>VE</td>
<td>10.8</td>
<td>188</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:2:9</td>
<td>JG</td>
<td>20.6</td>
<td>195</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:2:9</td>
<td>KZ</td>
<td>19.4</td>
<td>200</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

VE = Soft mud brick  JG = Yellow wire cut brick
PO = Poriso          JB = Blue wire cut brick
KZ = Sand lima brick
old = averaged age in day at testing.

The age of the series 1 specimens was ±100 and ±200 days, the age of the series 2 specimens was ±32 days.

The compression strength of the mortars is established at the same time as the compression strength of the masonry made with it.

Comparing the results of the tests of series 1 and 2 it has to be taken into account that there are differences in mortar recipes and age of the specimens.

Figure 2 gives an impression of the masonry strength in relation to the brick and mortar strength. Stronger mortars give a stronger masonry, according to the expectations.

Estimation of the masonry strength with the formula: $f_k = 0.4 * f_b^{0.75} * f_m^{0.25}$, according to [1], in which mortar and brick strength are taken into account, gives useless results, see the last column in table 1.

In table 2, the relation in strength between wide and narrow specimens is presented. These ratios are classified after type of bricks. Neglecting the values for Poriso the average aspect ratio is 1.11, but closer investigation, for example by calculation with FEM-computer programs, or experimental research on an "ideal" material, could give...
more insight in the found values.
The Belgian Code NBN 24-301 [2] gives a formula to calculate the influence of the dimensions of the specimen on the strength of the specimen. The ratio in strength for the wide and the narrow specimens calculated with this formula is 1.1. This is almost equal to the experimental value.

Table 4. Ratio between strength of wide and narrow specimens.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>PO</td>
<td>1.71</td>
<td>1.26</td>
<td></td>
</tr>
<tr>
<td>VE</td>
<td>1.22</td>
<td>1.20</td>
<td>1.10</td>
</tr>
<tr>
<td>JG</td>
<td>1.18</td>
<td>0.98</td>
<td>0.96</td>
</tr>
<tr>
<td>JB</td>
<td>1.14</td>
<td>1.11</td>
<td>1.13</td>
</tr>
<tr>
<td>Gem.</td>
<td>1.18</td>
<td>1.25</td>
<td>1.11</td>
</tr>
</tbody>
</table>

Average of all values: 1.185  1.110
Standard deviation: 0.199  0.098
Standard deviation in %: 16.79%  8.80%
*Average without values for Porio

Crack growth and failure

The start of the beginning of cracking can hardly be seen in the load deformation graph. At about 80 to 85% of the maximum load the first cracks show up. They often can be heard first. They become visible later. Especially with the harder wirecut bricks often cracking suddenly occurs accompanied by a loud noise. The cracks in the wire cut specimens were almost straight and parallel with the axis of the load.
The tests were deformation controlled. When the load is decreasing, with still increasing deformation, small vertical slabs get lose laterally from the specimen. The thickness of these slabs is about 15 to 20 mm, often more then one brick high. The specimen falls apart when the deformation goes further. Only a pyramidal piece is left over.
The crack process for the specimen made of softmud bricks takes place much slower.
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Flakes coming loose from the specimen were smaller. Also grains of brick come free. Cracks were almost straight and parallel to the force axis, in the upper part of the specimen they sometimes run under 45° to the corner of the specimen. This indicates the confining by the loadplatens. Also with soft mud bricks a pyramidal part remains. The structure of these bricks is clearly different from the wire cut bricks, because of the different production methods.

After the test had been stopped in a few cases the specimen completely fell apart. The collapse and crack pattern for wide and narrow specimen is the same. For the wide specimens the form of the centre-area is equal to the narrow specimens but its area is relatively wider. The stiffness of the header joints influences the deformation in lateral direction of the specimen. The strength of specimens with a soft mortar in the header joints will be smaller. Closer investigation of this effect is needed.

BUILDING SITE TESTS

Until now, from the tests carried out over the differences in strength between wide and narrow specimens, the conclusion can be drawn that the compression strength of masonry can be established with a specimen of 5 bricks. Closer investigation has to be carried out to establish which correction factors for different structure types are needed. These factors also depend on building site conditions.

A procedure for fabricating the specimens is developed and tested at this moment. A joint on top and bottom of the specimen is easily made and results in a sufficient levelled surface. Close attention is paid for the possibilities of reproduction of the tests and sensibility for mistakes during construction.

MASONRY, BRICK and MORTAR STIFFNESS

The deformation in the middle of the specimen over two joints and two bricks is measured to get an impression of the deformation of the masonry. With the available equipment it is not possible to measure the deformation of mortar between bricks adequately. The deformation of the mortar only can be established by subtracting the deformation of the brick from the measured deformation of the bricks and joints. From the tests of series 1 it is learned that when the deformations of the brick itself are measured on the same specimen as on which the deformations of joints and bricks are measured the results can not be used in the calculation of the deformation of the mortar. The deformation of the brick at the outside of the specimen is much smaller then can be expected on the basis of an equal stress distribution and on the deformation of the
complete specimen with mortar joints. That is why the Young moduli of the bricks established on specimens with a joint filling are used.

The tests of series 1 also showed that the stiffness of the mortar between bricks is completely different from the stiffness measured on mortar prisms. This is also found in the tests of series 2. Table 5 shows the E-moduli of mortar in masonry specimens.

Table 5. E-moduli in N/mm² of mortar in masonry specimens, calculated with the deformations measured over 112 mm and E-moduli of the bricks established with Bolidt tests.

<table>
<thead>
<tr>
<th>Brick—&gt;</th>
<th>PO</th>
<th>VE</th>
<th>JG</th>
<th>JB</th>
<th>KZ</th>
<th>KZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar</td>
<td>wide</td>
<td>narrow</td>
<td>wide</td>
<td>narrow</td>
<td>wide</td>
<td>narrow</td>
</tr>
</tbody>
</table>

Results series 2 wide and narrow specimens

<table>
<thead>
<tr>
<th></th>
<th>1:1/2:4 1/2</th>
<th>1:1/2:4 1/2</th>
<th>1:1:6</th>
<th>1:1/2:9</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1:1/2:4 1/2</td>
<td>1:1/2:4 1/2</td>
<td>1:1:6</td>
<td>1:1/2:9</td>
</tr>
<tr>
<td></td>
<td>2123</td>
<td>1952</td>
<td>4512</td>
<td>3688</td>
</tr>
<tr>
<td></td>
<td>12010</td>
<td>4512</td>
<td>5315</td>
<td>3534</td>
</tr>
<tr>
<td></td>
<td>9328</td>
<td>4991</td>
<td>4704</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3688</td>
<td>4704</td>
</tr>
</tbody>
</table>
| Results series 1 narrow specimens

<table>
<thead>
<tr>
<th></th>
<th>1:1/2:4 1/2</th>
<th>1:2:9</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1410</td>
<td>2860</td>
</tr>
<tr>
<td></td>
<td>13300</td>
<td>14900</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3350</td>
</tr>
</tbody>
</table>

The E-modulus for the mortar used with the Poriso masonry is established with E-brick = 6500 N/mm². Due to the structure of this type of brick this value only can be seen as an indication.

In series 2 extremes for the mortar recipes and brick strengths are used. This can be found in the results for the E-moduli from the mortars. The mortar qualities are influenced by the bricks. When a certain mortar is used with different bricks different Youngs-moduli for this mortar are found. This is found both in the tests series 1 and 2.

The E-moduli for the mortars used in series 1 is in accordance with those found in the tests from series 2. The E-modulus is bigger when the E-modulus of the brick is bigger. The indication for the mortar recipes is the same for both types. But there are (off course) differences. The mortar for the tests series 1 is delivered by a factory in bags in dry condition. The mortar for series 2 is made in the van Musschenbroek laboratory.

11 Vermeltfoort
CONCLUSIONS

- To be able to design, mechanical properties of the materials have to be known. During erection of the structure the structural properties of load bearing masonry have to be guarded. A simple method of control is needed.
- The strength of specimens which were two bricks wide is about 11% larger then the strength of specimen consisting of 5 bricks with 6 joints. For calcium silicate bricks the strength of both wide and narrow elements is the same.
- A masonry specimen of 5 bricks high with a mortar joint on top and bottom gives good numerical information of the mechanical compression properties of masonry. The results can be used in computer simulations.
- The properties of the mortar are influenced by the bricks. For bricks with a small compression strength the E-module for the mortar is smaller than that from the same mortar when used with stronger bricks.
- Further research over the properties of masonry in building practice has to take place to verify the results. A test procedure is developed. The research over the differences between wide and narrow specimen will be finished.

ACKNOWLEDGEMENTS.

The described experimental research is part of a program in the Netherlands called Brick Research Innovation and Knowledge transfer (BRIK), initiated by the Royal Dutch Brick association (KNB), and supervised by the Netherlands Centre for Civil Engineering Research Codes and Specifications (CUR).
The support and work of the people of the van Musschenbroek is gratefully acknowledged, in special C. Naninck who carried out all the tests described in this paper,

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[3] Vermeltfoort, A.Th., and Van der Pluijm, R., 1991. Strength and deformation properties of masonry to be used in computer calculations. 9th IBMAC Berlin, pp 244-252
Concrete & maintenance
Research on building structures and building physics.


Research on building structures and building physics.

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Technical University of Wrocław, POLAND

Dr Piet STROEVEN  
Delft University of Technology, THE NETHERLANDS

CRACKING BEHAVIOUR OF CEMENT-BASED COMPOSITES  
AS A RESULT OF THE BAR EXPANSION

INTRODUCTION

The problem of linear steel elements in concrete increasing with time in radial direction is met with tendons at early prestressing, fire conditions or in case of corrosion of the steel bars reinforcing concrete. Such expanding effect provokes an unfavourable tensile stresses in tangential direction around the steel bar and inevitably leads to concrete cracking. As a result of this process the durability resistance could be seriously reduced once cracks open more than about 0.2 mm [1]. Thus, the delaying of crack formation and of their further development appear as fundamental problems.

As a reference the phenomenon of an expanding steel bar in the bottom corner of a reinforced concrete beam can be considered. For a theoretical analysis the bar is assumed being concentrically embedded in a concrete cylinder of infinite length, so that a 2-dimensional solutions can be formulated. Such situation can by simplified for the present purpose to the so called "ring element", presented in Figure 1.

![Fig.1 Idealization of the stress situation around the expanding steel bar.](image)

The mechanical behaviour of concrete stressed internally by expanding steel bar was analytically discussed by de Wind and Stroeven [21]. Some quantitative information and preliminary test results on the early prestressing case were presented by Dantuma and den Uijl [3]. Interesting test results pertaining to particular aspects of corrosion case can be found in [4] (assessing the
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The present paper summarizes first part of the test results pertaining to evaluation of the cracking behaviour of concrete composites due to an expanding steel bar, obtained hitherto within the Polish-Dutch joint research project on: "Bond and crack development in cementations composites", which was granted by the NWO (Netherlands Organization for Scientific Research).

EXPERIMENTAL DETAILS

The tests were executed in the Stevin Laboratory of the Civil Engineering Faculty, Delft University of Technology. Two concrete mixes listed in Table 1 have been designed assuming equal volume of both compositions.

<table>
<thead>
<tr>
<th>MIX CODE</th>
<th>AGGREGATE (kg/m³)</th>
<th>GRADING</th>
<th>FIBRES (kg/m³)</th>
<th>CEMENT (kg/m³)</th>
<th>WATER (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>193</td>
<td>281</td>
<td>263</td>
<td>263</td>
<td>299</td>
</tr>
<tr>
<td>FC</td>
<td>189</td>
<td>274</td>
<td>258</td>
<td>258</td>
<td>292</td>
</tr>
</tbody>
</table>

A Portland Cement type A, satisfying the Netherlands Standard NEN 3550 was employed. Further use was made of a good quality river aggregate with a maximum grain size of 8 mm, tap water and 1.5% by volume of plain steel fibres with a length and diameter of 12.5 and 0.4 mm, respectively. The mixing, vibration and casting procedure was similar for all series and performed according to relevant Netherlands Standards.

The tests have been carried out on specially designed prismatic specimens of 100x100x300 mm. The main element of the testing set up was constituted by a cone-shaped steel bar with a diameter of 20mm which is pushed with a constant rate of displacement through a similarly shaped hole in a concrete specimen. The rate of applied cone displacement, amounting to 7.5 μm per second, was controlled by two LVDT's. The friction between all contact zones was eliminated by polishing the specimens and greasing all surfaces with vaseline. Strain gauges with a length of 20 mm were glued onto the specimen's top surface at testing. Crack formation was recorded by means of series of three parallel clip gauges at the side surfaces of the specimens. The acoustic emission response has been monitored additionally with an acoustic emission analyzer, type EA-3, designed and produced by the Institute of Fundamental Technological Research of Polish Academy of Sciences in Warsaw. More detailed information of test procedure can be found in [4].

Independently, compressive and tensile splitting strengths were determined using standard cubes.
The results obtained reflect the appreciable differences in cracking behaviour between plain and steel fibre concretes. It was manifested by post-peak differences in the load-displacement curves and in the acoustic emission response. Figure 3 shows examples of two selected 28 days old specimens.

It can be noted that the acoustic emission activity properly reflects the damage evolution process in the tested concrete composites. It is seen that the maximum of acoustic emission rate occurs around the BOP (Bend Over Point). Nevertheless, the character of this phenomenon is essentially different for plain and fibre reinforced concrete.

In plain concrete an impetuous acoustic discharge is associated with a considerable load drop following BOP. In case of steel fibre concrete, the post-peak load stabilizes after an insignificant decline. The acoustic emission activity in the post-peak range is characterized by the absence of a distinct peak. It can also be observed that the total number of AE counts registered over the post-peak cracking range is considerably less for the fibre concrete than for the plain one.
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Fig. 3. Development of load and AE activity under controlled conditions of bar expansion. Examples of 28 days old specimens, plain concrete at the left and steel fibre concrete at the right, are presented.

For further analysis the idealization of the observed damage evolution has been given on the Figure 4.

Fig. 4. Idealization of load - bar expansion curve.
The accepted code notation can be explained as follow:

- **BOP** - bend over point; notion which is usually attributed to the end of ascending branch of load-displacement relationship.
- **F_{u}** - ultimate load; load corresponding with bend over point.
- **F_{cr}** - critical load; load corresponding with the critical width of the leading crack, assumed to as 0.2 mm.
- **\( \delta_{u} \) \( \delta_{cr} \)** - ultimate, resp. critical bar expansion.
- **\( \Delta \delta \)** - bar expansion over post-peak cracking range.
- **\( \delta_{o} \)** - fictitious point of zero bar expansion.
- **E** - cracking energy dissipated over post-peak cracking range.

To properly characterize material behaviour the ratio of dissipated energy over the post-peak cracking range and total AE counts (\( \xi_{AE} \)) has been proposed.

- \( \xi_{AE} = \Sigma_{AE}/E \) - total AE counts over post-peak cracking range per unit of cracking energy

This parameter expresses the relationship between the acoustic emission response and the external energy provoking this phenomenon. It can be denoted as Degree of Cracking Instability (DCI).

The obtained results indicate that fibers substantially decrease the rate of AE. It is resulting from the fiber's ability to control the growth of micro- and macrocracks. The acoustic emission response becomes more uniform, without the impetuous discharges. The rate of acoustic activity is reduced in spite of a larger number of cracks then in the plain concrete case.

---

**Fig. 5** Idealization of the damage evolution in concrete
The results obtained have also shown that the evaluation of damage in tested specimens can be generally classified into three main regions, schematically given in Figure 5, where:

- DP - discontinuity point; notion describing the end of elastic stage,
- COP - crack opening point; notion referring to the stage in which the process of crack opening of the leading crack is initiated,
- $F_D$ - discontinuity load,
- $F_{co}$ - crack opening load,
- $\delta_D$, $\delta_{co}$ - bar expansion at discontinuity, ultimate, resp. crack opening stages.

At the first stage of loading (elastic range) the stress-strain situation is in accordance with elastic analysis. Further increase of the stress concentration around the expanding steel bar inevitably leads to microcracking, so that the material is gradually losing its integrity. Finally, in the last stage one, so called "leading" macrocrack appears, causing the material to loose its section continuity.

It is worthy of notice that the examinations also reveal to distinguish in the cracking range (Fig. 5) two different stages. In the first one (stable crack formation) scattered microcracks are created which gradually develop and coalesce. This process provokes a gradual reduction of the load-bearing structure and around ROP crack propagation is becoming unstable (unstable crack propagation). This phenomenon manifests itself by sudden acoustic emission discharges which are associated with a considerable load drop and a visible increase in the slope of the crack opening curve.

CONCLUSIONS

- It has been found that the evolution of damage in concrete composites stressed internally by expanding steel bar can by generally classified into three main regions.
- The tests have shown that monitoring of acoustic emission allows to clearly follow the cracking process of concrete.
- In the post-peak range even small amounts of steel fibres were found to considerably improve toughness by controlling crack development.
- The Degree of Cracking Instability is proposed as a relevant parameter for evaluation of structural integrity of concrete.

ACKNOWLEDGEMENTS

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CRITERIA OF CO-OPERATION OF STIFF ANTICORROSIVE COATINGS WITH CONCRETE

Jerzy Karpś

1. INTRODUCTION

Polymer coatings are most widely used to protect against corrosion. In media of strong corrosive power heavy multilayer coatings, which successfully replace the traditional ceramic and stone linings, are employed.

Considering the different character of the collaboration of heavy coatings with a construction material in comparison with the work of light coatings, one should take a different approach to the design of heavy coatings, particularly in the cases when mechanical loading collaborates with chemical loading at high temperatures (power station smokestacks, tanks containing chemical solutions).

The requirements concerning heavy anticorrosive coatings are related to the kind of a polymer and the possible reinforcement in the form of glass cloth and sometimes to the reduction of run-off on vertical surfaces. Moreover, the standards limit the use of heavy stiff coatings to the instances when the spacing of mobile cracks is smaller than 0.1 em.

It seems, however the requirements are too strict.

This paper presents criteria that polymer coatings working in corrosive media at high temperatures, normally below 100 °C (373 K), should meet. Moreover, it has been found, both theoretically and experimentally, that heavy stiff coatings can work on foundations with mobile cracks of slightly over 0.1 mm spacing.

2. CRITERIA FOR STIFF ANTICORROSIVE COATINGS LAID ON CONCRETE FOUNDATION

The durability of anticorrosive coatings depends on several factors and above all on the collaboration of the coating with the foundation.
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The crack resistance of the coatings, in conditions when cracks exist in the concrete foundation, determines this collaboration. The detailed relationships are shown in Fig. 1. The individual properties - criteria conditining the collaboration of coatings with concrete can be written as follows:

a/ The criterion of adhesion to concrete:
\[
\frac{F_I}{B_I} \frac{E_I}{\alpha_I} - 1 > 0 \quad (1)
\]

Where: \( F_I / B_I \) - the shear strength of the coating - concrete connection,
\( E_I / \alpha_I \) - the shear strength of the concrete

b/ The criterion of interlayer adhesion:
\[
\frac{F_{I+1}}{B_{I+1}} \frac{E_{I+1}}{\alpha_{I+1}} - 1 > 0 \quad (2)
\]

Where: \( F_{I+1} / B_{I+1} \) - the shear strength of the connection between successive layers

c/ The criterion of thermal resistance:
\[
\frac{E_F}{\alpha_F} - \Delta T > 0 \quad (3)
\]

Where: \( \Delta T \) - the difference in temperatures existing in successive media,
\( E_F / \alpha_F \) - the tensile strength of the coating,
\( E_F / \alpha_F \) - the coefficient of elasticity of the coating,
\( E_F / \alpha_F \) - the coefficient of thermal expansion of the coating

d/ The criterion of thermal shock resistance
\[
\frac{\lambda^I}{\lambda^B} \frac{\alpha^I}{E^I - \alpha^B - E^B} - 1 > 0 \quad (4)
\]

Where: \( \lambda^I, \lambda^B \) - the thermal conductivities of the coating and concrete, respectively,
\( E^I - \alpha^B - E^B \) - the coefficient of elasticity of the concrete,
\( \alpha^I, \alpha^B \) - the coefficient of thermal expansion of the concrete
Fig. 1. Factors determining the durability of stiff anticorrosive coatings laid on concrete.
c/ The criterion of thermal compatibility:

\[
\Delta T = \frac{R^B}{T^B (\alpha^F \cdot E^F - \alpha^B \cdot E^B)} - 1 > 0
\]  

(5)

Where: \( R^B \) - the tensile strength of the concrete

f/ The criterion of crack resistance:

\[
\frac{a_{\text{max}}}{a_T} = 1 > 0
\]  

(6)

Where: \( a_{\text{max}} \) - the maximum spacing of a crack which does not damage the coatings,

\( a_T \) - the actual spacing of a crack.

3. COLLABORATION OF STIFF ANTI-CORROSIVE COATINGS WITH CRACKED CONCRETE FOUNDATION

Starting with criterion of crack resistance, the following relations for stiff anti-corrosive coatings collaborating with the foundation, can be derived:

a/ without the separation of the coating at the moment of rupture

\[
\frac{L}{a_T} \left[ \frac{R^F}{E^F} \left( 1 + \frac{R^F}{E^F} \right) - 1 > 0 \right]
\]  

(7)

and \( a_{\text{max}} = \frac{L}{E^F} \left( 1 + \varepsilon^P_{\text{max}} \right) \)  

(8)

b/ with a possibility of the separation of coating at the moment of rupture

\[
\frac{L}{a_T} \left[ \frac{R^F}{E^F} \left( 1 + \frac{R^F}{E^F} \right) + 2 \frac{c \cdot L}{E^F} \right] - 1 > 0
\]  

(9)

and \( a_{\text{max}} = \frac{L}{E^F} \left( 1 + \varepsilon^P_{\text{max}} \right) + 2 \frac{c \cdot L}{E^F} \)  

(10)

Where: \( l_v \) - the work length (length of bonding), along which additional stresses exist in the coating \( (\gamma^F) \) after the appearance and widening of a crack; assumed to be in the range 0.035 - 0.05 m,
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\[ \varepsilon_{\text{max}} \text{ - the maximum relative strain in the coating at the moment of rupture,} \]

\[ e \text{ - the length of the separation of the coating; the assumed value: 0.0012 m.} \]

4. ANALYTICAL AND EXPERIMENTAL VALUES OF \( \varepsilon_{\text{max}} \)

Calculations were done for stiff epoxy and polyester coatings and laminates using formulas 7-10. The results are in table 1.

<table>
<thead>
<tr>
<th>Kind of coating</th>
<th>( \varepsilon_{\text{max}} ) ( \text{calc} ) by 7-10</th>
<th>( \varepsilon_{\text{max}} ) ( \text{from experience} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. Coatings ruptured without separation from foundation: ( l_v = 0.05 \text{ m} )</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- epoxy coating</td>
<td>1.28</td>
<td>1.57</td>
</tr>
<tr>
<td>- polyester coating</td>
<td>1.22</td>
<td>1.49</td>
</tr>
<tr>
<td>- epoxy laminate</td>
<td>1.44</td>
<td>1.66</td>
</tr>
<tr>
<td>- polyester laminate</td>
<td>1.71</td>
<td>1.91</td>
</tr>
<tr>
<td><strong>B. Coatings ruptured with separation from foundation; ( l_v = 0.035 \text{ m, } c = 0.0012 )</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- epoxy coating</td>
<td>1.38</td>
<td>1.61</td>
</tr>
<tr>
<td>- polyester coating</td>
<td>1.36</td>
<td>1.58</td>
</tr>
<tr>
<td>- epoxy laminate</td>
<td>1.54</td>
<td>1.68</td>
</tr>
<tr>
<td>- polyester laminate</td>
<td>1.77</td>
<td>1.93</td>
</tr>
</tbody>
</table>

5. CONCLUSIONS

The results of the analytical and experimental testing of stiff anticorrosive coatings indicate larger spacing of cracks at the moment of rupture than the one known from the literature. The spacing of cracks reaches about 2 mm in these coatings and it is 15-20 times larger than the acceptable value in the standards. This means that the work of the coatings when the spacing of cracks is over 0.1 mm is possible but it is necessary to consider the effects of the fatigue of the coatings at the moments of change in the spacing of cracks. These effects increase the corrosivity of the medium and decrease the durability of coatings.
The quantitative determination of the influence of these phenomena requires long-term research. The results and formulae given in this work are valid when the solids of the tensile stresses in the coatings do not coincide, i.e. the mobile cracks are spaced at distances greater than $2 \cdot l_v = 0.10 \, \text{m}$.

LITERATURE


Application of Boundary Elements Method to analysis of large deflected plates.

Assuming that external, in-plane loads are equal zero, the basic equations for nonlinear theory of plates can be written out as

\[ D\Delta \Delta w + L(w,\phi) = q \]  
\[ \frac{1}{Eh} \Delta \Phi = \frac{1}{2} L(w,w) \]

where:  
- \( D \) - plate bending stiffness;  
- \( h \) - thickness of the plate;  
- \( E \) - Young modulus;  
- \( \phi \) - Airé function;  
- \( L(w,\phi) = \frac{\partial^2 w \cdot \partial^2 \phi}{\partial x^2 \partial y^2} - 2 \cdot \frac{\partial^2 w \cdot \partial^2 \phi}{\partial x \partial y \partial x \partial y} + \frac{\partial^2 w \cdot \partial^2 \phi}{\partial y^2 \partial x^2} \)  
- \( L(w,w) = 2 \cdot \frac{\partial^2 w \cdot \partial^2 w}{\partial x^2 \partial y^2} - 2 \cdot \left( \frac{\partial^2 w}{\partial x \partial y} \right)^2 \)

At these equations, influence of lateral, normal and tangential stresses is neglected.  
Equations 1.1 and 1.2 can be solved in a numerical way, using computer programme calculating plates with the help of boundary
element methods. As the first approximation, the values calculated through the classical small-deflection theory can be used. Having obtained in that way the surface of deflection "w", we can calculate the operator \( L(w,w) \)- the influence functions of \( L(w,w) \) can be obtained with the help of Green’s functions for plates. Now treating the right side of the Equation 1.2 as an external load of some kind, this equation can be solved with the help of boundary element methods on the analogy of the equation of classical theory

\[
\Delta \Delta \omega = q
\]

Similarly, we can accept the basic influence function of the Aire function as

\[
\bar{\varphi} = \frac{Eh}{16\pi} \left[ (x-\xi)^2 + (y-\eta)^2 \right] \ln \frac{(x-\xi)^2(y-\eta)^2}{R^2}
\]

(1.3)

The values of the function \( \varphi \) caused by the "load" \( L(w,w) \) can be expressed now as

\[
\varphi = \int_{\mathcal{F}} L(w,w) \cdot \bar{\varphi} \cdot d\mathcal{F}
\]

(1.4)

Having in mind appropriate boundary conditions, Eq.1.2 can be solved for plate of any shape. The support conditions are:

- edges free to move in plan

\[
N_{xx} = - \frac{\partial^2 \varphi}{\partial y^2} = 0
\]

(1.5)
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-edges restrained

\[ h \cdot E \cdot u = \int \frac{\partial^2 \phi}{\partial y^2} \cdot dx - \nu \cdot \frac{\partial \phi}{\partial x} + F_1(y) = 0 \]  \hspace{1cm} (1.6)

\[ h \cdot E \cdot v = \int \frac{\partial^2 \phi}{\partial x^2} \cdot dy - \nu \cdot \frac{\partial \phi}{\partial y} + F_2(x) = 0 \]

where: \( u, v \) - in-plane displacements.

Additional condition for functions \( F_1(y) \) and \( F_2(x) \)

\[ \gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \]  \hspace{1cm} (1.7)

Hence, in terms \( \phi \) we get

\[ \int \frac{\partial^3 \phi}{\partial y^3} \cdot dx + \int \frac{\partial^3 \phi}{\partial x^3} \cdot dy + \frac{d F_1(y)}{dy} + \frac{d F_2(x)}{dx} = -2 \frac{\partial^2 \phi}{\partial x \partial y} \]  \hspace{1cm} (1.8)

Having function \( \phi \), we can calculate the operator \( L(w, \phi) \).

Transferring operator \( L(w, \phi) \) on the right side of Eq.1.1 and treating it as an external load of some kind, we can calculate each plate subjected to the substitute load

\[ \bar{q} = q - L(w, \phi) \]  \hspace{1cm} (1.9)

Equations 1.1 and 1.2 are solved by turns up to obtain the required accuracy of deflection "w". The computer programme calculating the plates in that way was arranged by the author of that report. For comparison with other published results, shown in Table 1, the calculations with the help of this programme were made for the square steel \((v = 0.3, E = 30 \times 10^6 \text{ psi})\) plate 8 ft by 8 ft, 1/4 in. thick, \( P = 0.1 \text{ psi} \).
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Tab. 1. Values of the center deflection $w$.

<table>
<thead>
<tr>
<th>Boundary conditions</th>
<th>Published results</th>
<th>$w$ [in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>simply supported, edges restrained</td>
<td>[1]</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>[2]</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>[3]</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>this study</td>
<td>0.29</td>
</tr>
<tr>
<td>simply supported, edges free to move</td>
<td>[1]</td>
<td>0.48</td>
</tr>
<tr>
<td>in plane</td>
<td>[3]</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>[4]</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>this study</td>
<td>0.43</td>
</tr>
</tbody>
</table>

The received results are comparable with the others. The programme is still tested. Simultaneously is arranged the fragment of the programme, allowing to calculate in that way the RC plates with cracks, where the crack is treated as a defect described by the distributive calculus.

References.

GENERALIZATION OF THE P-DELTA METHOD

Summary

The general P-Δ approach (GΔPN) for the finite element method (FEM) has been presented in this paper. It has been shown that P-Δ effects and suitable solutions algorithms comes from some modifications of the standard secant stiffness method for the second order theory (MNII) in general.

The GΔPN method is accurate approximation of the MNII with opposite to a standard (SΔP) and a modified (MΔP) P-Δ methods. Moreover SΔP and MΔP methods are use to be applied to the rectangular frame. The GΔPN approach doesn't need this limitations.

With respect to structures, especially rectangular frames ,the GΔPN method, its first linearization (GΔPL), standard and modified P-Δ methods, full nonlinear MNII and its linearization NMIIL has been compared.

One representative example of the frame has been chosen. A behavior of mentioned methods in noncritical and critical area of loading with respect to their convergence has been considered.

A numerical example shows that all considered P-delta methods with comparison to the exact second order analysis gives satisfied results in noncritical area but the best are received by the GΔP methods. Moreover from the P-Δ approaches, only the general P-Δ method is convergent in a critical area of loading.

1. The fundamental P-Δ approach.

In the static analysis of a rectangular high structures is necessary to consider the influence of so called second order factors on internal forces distribution. These are caused by displacements and deflections of a construction, and finally leads to increasing the value of internal forces with comparison to linear static analysis.

For example in two dimensional structural analysis, these increments are relate to shear forces and overturning moments but in three dimensional structures it concern to torsional moments also.
The overturning and torsional moments, acting at a given story of the structure have two components:

(i) primary moments received from the linear static analysis in which suitable differential or finite element equations are deviated of formula for undeformed state of structure,

(ii) second-order moments calculated for deformed state of the structure and are caused by the vertical forces acting over their respective incremental lever arms.

This additional second order components are called as the P-delta (P-\Delta) effects [8].

For matrix displacement method, the P-\Delta factors are introduced into linear elastic stiffness matrix for one member, before it transformation from the local to the global coordinate system. However for small deflection, i.e., when the influence of the local member curvature on the second order effects are negligible small, the P-delta effect can be introduced into matrix equilibrium equations of the structure after the assembly the global elastic stiffness matrix \( K_S \) [10].

Let the matrix equilibrium equation has the form [6]:

\[
K_S \cdot V = F
\]  

where \( V = \{ V_1, \ldots, V_N \} \) is nodal displacement vector of construction, \( V_i \) is displacement vector for i-node (i.e. \( =\{v_{i_x}, v_{i_y}, o_1\} \) for plane structures), \( F = \{ F_1, \ldots, F_N \} \) is lateral nodal loading vector and \( F_i \) is lateral loading acting on i-node (i.e. \( =\{P_{i_x}, P_{i_y}, M_i\} \) for plane structures), \( N \) is a the total number of nodes.

Equations (1) is linear and present the standard linear method (LM). The P-\Delta method modifies the right hand side of this equations by one factor. Let consider j-column and i-story of a rectangular structure, and assume that each story has \( l_j \) -columns. The P-\Delta effects involved a second order moment \( M_{1i} \) which have to be calculated at each frame story. This moment has the form [1, 4]:

\[
M_{1i} = \sum_{j=1}^{l_j} N_{i,j} \cdot (v_{1x} - v_{(1-1)x}), \quad i = 1, \ldots, N, \quad (2)
\]
where $N_{1,j}$ is an axial force in j-column of i-story, $v_i$ and $v_{i-1}$ are horizontal displacements of the i and i-1 stories and $m$ is a number of stories.

Let define a lateral force couples $F_{s1}$ at each story as:

$$F_{s1} = \alpha_i \cdot \frac{M_i}{h_i}$$  \hspace{1cm} (3)

where $h_i$ is a story height. Then lateral P-Δ vector has the form:

$$\vec{F}_s = \{ F'_{s1}, \ldots, F'_{sM} \} , \quad F'_{s1} = F_{s(i-1)} - F_{s1}$$  \hspace{1cm} (4)

where $i$ is a story number, $M$ is a number of stories.

Making the transformation of the $F_s$ vector, using the T matrix, into the vector space for lateral loading $F$, the influence of the P-Δ effects in (1) can be express then in form:

$$K_s \cdot \vec{V} = \vec{F} + \vec{F}_d$$  \hspace{1cm} (5)

where $\vec{F}_d = T \cdot \vec{F}_s$.

2. Iterative solution for P-Δ method.

In the standard P-delta (SΔP) method, when the influence of the axial force for deformation is neglected coefficient $\alpha_i$ is used to take as equal 1 or 1.21 [2]. In modified (MΔP), it is calculated from geometrical stiffness matrix [3], has an algebraic form and is dependent only on the suitable rotations of nodes of stories for linear model LM [9]. It can be write as:

$$\alpha_i = f ( V_i )$$  \hspace{1cm} (6)

Hence and (5) the fundamental P-Δ matrix equations can be written:

$$K_s \cdot \vec{V} = \vec{F} + \vec{F}_d (\vec{V})$$  \hspace{1cm} (7)

and its solution has an iterative form as follow:

(i) first step  \hspace{1cm} $\vec{V}_0 = (K_s)^{-1} \cdot (\vec{F})$  \hspace{1cm} (8)
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(ii) \( l \)-step

\[
V_1 = (K_s)^{-1} \cdot (E + E_d(V_{(1-1)})) \quad (9)
\]

The iteration process stops when:

\[
\| V_1 - V_{(1-1)} \| < \epsilon \quad (10)
\]

where \( \epsilon \) is a tolerance and \( \| . \| \) is euclidean norm.

A convergence of SAP method was considered in a papers [5],[10]. It can be found that for some structures, the process over five iterations, is not convergent and structure could lose its global stability.

3. Generalization of the P-\( \Delta \) iterative procedure.

For small displacements under linear - elastic conditions, prismatic members with neglecting the share shape deformation, the stress-strain relations for construction can be written in a form [3]:

\[
K_T(V) \cdot V = F^n - F^e(V) \quad (11)
\]

where \( K(V) \) is tangent stiffness matrix nonlinear dependent on a displacement vector \( V \), \( F^n \) is the loading vector acting on the nodes, \( F^e(V) \) is loading vector acting on the elements.

This equations can be solve by incremental iterative technique, which steps are as follow:

(i) first step

\[
V_0 = (K)^{-1} \cdot F^* \quad (12)
\]

next are calculated

\[
F_0 := K_T(V_0) \cdot V_0, \; F^* := F^n - F^e \quad (13)
\]

\[
\Delta F^*_1 := F_0 - F^* \quad (14)
\]

\[
\Delta V_1 = (K_T(V_0))^{-1} \cdot \Delta F^*_1 \quad (15)
\]

first step continued

\[
V_1 = V_0 + \Delta V_1 \quad (16)
\]
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(ii) $i$ - step

\[ F_{(i-1)} := K_1(V_{(i-1)}) \cdot V_{(i-1)} \]  \hspace{1cm} (17)

\[ \Delta F_{(i-1)} := F^* - F_{(i-1)} \]  \hspace{1cm} (18)

\[ \Delta V_1 = (K_T(V_{(i-1)}))^{-1} \cdot \Delta F_{(i-1)} \]  \hspace{1cm} (19)

\[ V_1 = V_{(i-1)} + \Delta V_1 \]  \hspace{1cm} (20)

All process is finished when :

\[ \| \Delta F_{(i-1)} \| < \varepsilon \]  \hspace{1cm} (21)

for small arbitrary chosen $0 < \varepsilon << 1$.

This iterative increment procedure direct leads to the secant process (NMII), in which $i$-iteration is described as :

\[ V_1 = K_T(V_{(i-1)})^{-1} \cdot F^* \]  \hspace{1cm} (22)

what has been presented on figure 1.
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Now let assume that on each iteration in (22), the inverse matrix will be calculated from elastic matrix \( K_S \). Then above procedures, after same modifications, can be expressed in a form:

\[
K_S \cdot V_1 = F^* + (K_S - K_T(V_{(1-1)})) \cdot V_{(1-1)} \tag{23}
\]

This equation is called the generalization of the P-\( \Delta \) method. It can be note that it has similar form then equation (7).

After introducing incremental loading defined as:

\[
F_i = F_{(i-1)} + \Delta F_i \tag{24}
\]

where

\[
\Delta F_i = F^* - F_{(i-1)} \tag{25}
\]

equation (24) can be written in form:

\[
K_S \cdot V_1 = F^* + \sum_{j=0}^{1} \Delta F_{(j-1)} \tag{26}
\]

which give the sense of the P-\( \Delta \) effects in general nonlinear expression (G\( \Delta \)PN).

When the global stiffness matrix \( K_T \) is linearized by elastic \( K_S \) and geometric matrix \( K_G \) i.e.:

\[
K_T = K_S + K_G \tag{27}
\]

then substituting this expression into (25), it can be obtained a linearized general P-\( \Delta \) method (GA\( \Delta \)PL), which for \( i \) iteration has the form:

\[
K_S \cdot V_1 = F^* + K_G(V_{(1-1)}) \cdot V_{(1-1)} \tag{28}
\]

It can be noted that both GA\( \Delta \)PN and GA\( \Delta \)PL methods are convergent because this fact resulted from the convergency of the MNII and MNIIL methods.
4. Numerical example

On a figure 2 it has been shown a frame for which all mentioned above P-Δ methods with the LM, MNII, MNIIL methods have been considered.

In this example the $V_{1x}$ displacement, for x direction in joint number 1 and the $M_{12}$ internal moment in joint number 12 have been calculated. It has been introduced a relative error defined as:

$$\varepsilon_{H,x} = \frac{\|w-w'\|}{w} \cdot 100\% \quad (29)$$

where $w$ is a solution received by the MNII method (treat as accurate) and $w'$ is a calculated value by the rest methods.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure2.png}
\caption{Topology, geometry, material properties and loading of a three bay, five storey frame.}
\end{figure}
This relative error gives the information about effectivity of the one method, in an accuracy sense of the one value, due to accurate solution.

Also the time of the computer operations has been measured. For this purpose the relative coefficient has been defined as:

\[
\varepsilon_T = \frac{t_H}{t_L, M}
\]

(30)

where \( t_H \), \( t_L, M \) are times of calculations for chosen and ML method respectively. This definition gives the most proper look for an effectivity of one method because the direct calculation time is in great manner perturbed by the chosen compiler and linker and the type of a computer.

All calculation has been made under ASTAT II PC computer program (static linear and nonlinear analysis for structures) written in overlay technique with the use MS FORTRAN v. 4.0, MS PASCAL v 4.0 and MS C v. 6.0 and MASM v5.0, on the base PC 486/25MHz/EVGA unit.

In a table 1 are presented results for loading case I, which is non critical area and loading case III which is a critical area with the critical loading \( R_{1}=0 \) [kips] and \( P=830\) [kips] for NMII method.

<table>
<thead>
<tr>
<th>Calculated displacement, mean relative errors</th>
<th>LM</th>
<th>( \Delta P_{&lt;10} )</th>
<th>( \Delta P_{&lt;40} )</th>
<th>( \Delta P_{&lt;40} )</th>
<th>NMII</th>
<th>( \Delta P_{&lt;40} )</th>
<th>NMII</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_{u} ) <img src="%5Bv%5D" alt="( [u] )" /></td>
<td>0.7634</td>
<td>0.8864</td>
<td>0.8869</td>
<td>0.8878</td>
<td>0.8909</td>
<td>0.8877</td>
<td>0.8910</td>
</tr>
<tr>
<td>( \varepsilon_{T} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>14.32</td>
<td>0.51</td>
<td>0.46</td>
<td>0.36</td>
<td>0.01</td>
<td>0.28</td>
<td>0.0</td>
</tr>
<tr>
<td>( \varepsilon_{T} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>11.02</td>
<td>9.38</td>
<td>3.48</td>
<td>0.25</td>
<td>0.01</td>
<td>0.28</td>
<td>0.0</td>
</tr>
<tr>
<td>( M_{12} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>1.0</td>
<td>2.41</td>
<td>2.86</td>
<td>3.01</td>
<td>3.44</td>
<td>3.01</td>
<td>3.44</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Calculated displacement, mean relative errors</th>
<th>LM</th>
<th>( \Delta P_{&lt;10} )</th>
<th>( \Delta P_{&lt;40} )</th>
<th>( \Delta P_{&lt;40} )</th>
<th>NMII</th>
<th>( \Delta P_{&lt;40} )</th>
<th>NMII</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_{u} ) <img src="%5Bv%5D" alt="( [u] )" /></td>
<td>0.000026</td>
<td>0.00030</td>
<td>0.00030</td>
<td>0.00033</td>
<td>0.06307</td>
<td>0.00033</td>
<td>0.23069</td>
</tr>
<tr>
<td>( \varepsilon_{T} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>99.89</td>
<td>80.68</td>
<td>80.87</td>
<td>83.21</td>
<td>1.18</td>
<td>83.24</td>
<td>1.0</td>
</tr>
<tr>
<td>( M_{12} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>-0.0237</td>
<td>-0.02419</td>
<td>-0.02577</td>
<td>-0.02788</td>
<td>-2.26497</td>
<td>-0.02789</td>
<td>-8.00549</td>
</tr>
<tr>
<td>( \varepsilon_{T} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>99.7</td>
<td>99.87</td>
<td>99.87</td>
<td>99.86</td>
<td>72.66</td>
<td>99.86</td>
<td>1.0</td>
</tr>
<tr>
<td>( \varepsilon_{T} ) <img src="%5B%5Ctheta%5D" alt="( [\pi] )" /></td>
<td>1.0</td>
<td>2.72</td>
<td>3.23</td>
<td>3.4</td>
<td>3.9</td>
<td>4.11</td>
<td>4.77</td>
</tr>
</tbody>
</table>
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For all methods it has been assumed that \( c \), which ends the iterations, is equal to the value 1E-5.

Received results, for four iterations, shows that in non-critical area standard and modified P-\( \Delta \) methods gives relative error in the range of 3.5 - 9.5 %, but general P-\( \Delta \) methods are much more accurate with relative errors in the range of 0.25-0.28 %.

From the point of view of the relative time coefficient, the differences between GAPL and MAP are small (relative range ~ 2.4%), but between GAPN and MAP are considerable (relative range ~ 20.3%). In critical area of loading, within the four iterations, only the MNII method has given the results near the analytical solution. The MNIIL with relation to the MNII has given 1.18 % error but all P-\( \Delta \) have given this error huge, in a range of 80.7 - 83.2 %.

Increasing iterations until 50 the MNII has oscillated around value received in fourth iteration (table 1), MNIIL, GAPN and GAPL has been convergent to solutions of MNII method, but SAP and MAP methods have not been convergent.

5. Conclusions

The general P-\( \Delta \) approaches, described in this paper, are more effective then thy standard and modified P-\( \Delta \) methods.

It is not the consequence of the direct comparison of these methods on the base of a rectangular frames, as it was shown on a presented example.

Above all it is the consequence of the matrix iteration equations (23) - (27) which:

(i) doesn't require any special topology and geometry of the structure, so any structure can be consider,

(ii) is convergent if the MNII and MNIIL methods are convergent

(iii) has general form what caused that all second order factors are considered, not only those mentioned in SAP and MAP methods.
References


SAND CONCRETE AS A MATERIAL FOR PARTIALLY PRESTRESSED MEMBERS

by Dr Andrzej Kmita

Introduction

One of the projects being now elaborated in the Institute of Building Engineering at the Wroclaw Technical University is the test of the applicability of sand concrete in prestressed structures. A part of information and results is given in this work. The reliable anchorage of the strand in the sand concrete produces some troubles in the contrary to the ordinary concrete which is considered as the most important result of the executed tests.

Test description

Test have been executed on the pre-tensioned partially prestressed concrete elements prepared in two sets containing three elements each. The first set - I contains members made of the concrete with granite crushed stone aggregate, the second set contains members made of the sand concrete with the median of the grain \( d_m = 0.60 \) mm.

Dimensions of the members for both sets and the distribution of the reinforcement (fig. 1) were the same, whereas the strength parameters different for both sets have been performed in the tab. 1. The degree of prestressing was the same \( \eta = 0.6 \).

![Fig. 1. Structure of tested elements.](image-url)
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Tab. 1. Strength parameters of tested elements

<table>
<thead>
<tr>
<th></th>
<th>$R_d$[MPa]</th>
<th>$R_{bx}$[MPa]</th>
<th>$E_b$[MPa]</th>
<th>$\varepsilon_0$[%]</th>
<th>$V_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>57.6</td>
<td>3.1</td>
<td>30120</td>
<td>0.490</td>
<td>0.19</td>
</tr>
<tr>
<td>II</td>
<td>60.5</td>
<td>2.6</td>
<td>31500</td>
<td>0.311</td>
<td>0.20</td>
</tr>
</tbody>
</table>

The load for a given distribution of forces (fig. 1) acted according to the histogram shown in fig. 2. It is visible that the beams were loaded in cycles in the range $P = [0.2 - 0.6]P_n$ (where $P_n$ - load-bearing capacity) in order to determine the effect of the repeatable load on the members behaviour.

Fig. 2. Load histogram.

In the fig. 3 the relationship deflection $y$ - load $P$ for the first and hundredth cycle of loading for both series have been presented. The load level $0.6P_n$ answers the maximal opening of the crack $a_{f_{\text{max}}} = 0.2$ mm, whereas for $P = 0.3P_n$ the member works as a member having cracks. As it was shown in tests after ten cycles of loading the response of members was stabilized and the later response got an elastic character.

Considering such parameters as rigidity or crack resistance in the range $P \in [0.0 - 0.6]P_n$ it was found that the beam response for both series is similar. That means that the application of sand concrete has certain chances.

Fig. 3 Relationship $y = f(P:P_n)$ of the element

$I_1(100)$ - first set and first [hundredth] load cycle
$II_2(100)$ - second set and first [hundredth] load cycle
A separate problem which requires an individual analysis is the behaviour of mentioned members in the supporting zone. This zone gives some difficulties in the interpretation of the distribution of the internal forces, morphology of the cracks and the failure mechanism.

For the destructive test the first skew cracks appeared for \( P = 0.75 \, P_n \). For \( P = 0.85 \, P_n \) the slide of the prestressing strand recorded for the lower tensioned fibres causes the displacement of the cracked plane to the anchorage zone (fig. 4). The growing load causes afterwards the slide in the second row which produces the members failure. Therefore the determination of both the adhesive forces and the length of the anchorage in the sand concrete becomes basic for the appreciation of the applicability of the sand concrete in prestressed structures.

**Fig. 4. Distribution of cracks in the supporting zone of the tested element of set II.**

In order to determine the length of the strands \( 6 \, 2.5 + 1 \, 2.8 \) the individual elements have been prepared (fig. 5). In the upper part of the beams the hinges have been fixed and the both parts of the tested elements have been separated by a metal sheet with thickness 1 mm. The beams have been loaded by two concentrated forces up to failure or up to slide. At the end of strands the inductive gauges have been fixed for the permanent recording of the likely splice slide. The differentiation of the anchorage length has been reached by a plastic pipe placed in the central zone of the beam. The full length of the anchorage has been defined as the length which was recorded when the prestressing strand had been snapped.

The test shows that the length of the anchorage in the concrete with crushed granite aggregate is nearly 35% less from that in the sand concrete.
During the tests it was found that apart of the different concrete strength the sand graining has a significant effect on the anchorage length. For a small graining the strand slide is fluent. For sand concrete having bigger grains the stabilization of the anchorage was observed after initial slip and strand snap. The determination of this relationship requires separate extensive investigation which will be continued in the future.

Fig. 5. Test of the anchorage length - a view of the element on the testing device.

Conclusions

The applicability of the sand concrete in pre-tensioned prestressed structures has been confirmed by tests. Such structures may be designed and constructed in the practice under separate standards like that established for ordinary concrete. A big deal of attention has to be paid to the supporting zone where the safe length of the anchorage of the prestressing steel has to be determined.

It is planned to test both the different sand concretes in order to determine the corresponding anchorage lengths and sand concrete subjected to long-term loads.
CONSTRUCTION PROJECT MANAGEMENT OF TALL BUILDINGS

1. INTRODUCTION

Recent progress of high-rise buildings [2,8] shows the shortage of management theories that could meet the requirements of both the new construction technologies of tall buildings and the economy factors. Some directions that the author is going to follow in the future research to build a model of construction project management of tall buildings are presented in this paper. Construction of a tall building - to a certain extend - can be treated as a large engineering project, according to the terminology of Production Management Theory. However, large projects are usually understood as located on a large area (highway, airport, etc.) and here we should rather consider "tall" projects. So, the large project management theories should be adjusted according to skyscrapers' specificity. Very often those projects are managed by joint venture construction management teams what increases complexity of them considerably. The statements presented in this paper, which is an introduction to the future research, are based on the study of recent professional literature and visits to several construction sites in State of Connecticut (USA), last summer.

2. PLANNING

Planning of the whole construction process is critical for the success of a project. Some real examples to prove that statement were presented by Vanderpool in his paper [11]. Looking into planning, the financial planning seems to the very important in the present recession on the building market. A systematic approach of capital planning and budgeting process can be adopted from standard applications [4] as outline in fig. 1.

1. Define planning period
2. Establish decision making criteria
3. Define capital improvement needs
4. Estimate costs and benefits
5. Evaluate priorities, timing, and strategies
6. Identify financing sources
7. Assess risks
8. Make decision

Fig.1. The capital planning and budgeting process.
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Special attention should be paid to the risk analysis. The risk of tall building construction should be carefully identified, qualified and finally allocated. Because of the relatively long construction period the effect of inflation on the project should be forecasted. The British index escalation system NEDO [1] may be used for that purpose. Because of many subcontractors and complexity of the project, performing sensitivity analysis could be included into the financial modeling of the project. Sensitivity to income and to interest rate can be helpful to identify a break-even point. To compare alternative projects (in the phase of planning) the concept of Internal Rate of Return seems to be very proper and let to forecast cash flows. Important element of financial planning is cost estimating for one square meter (sq ft) of the building. Reasonably accurate method to estimate that cost can be derived on the basis of historical building costs. Location of the site, the type of building and the number of subcontractors should be considered in the method. The formula for steel-framed office building has been already derived by Karshenas [5]:

\[
C = A^{0.1045} \times H^{1.1258} \times e^{-5.6735}
\]

where: \(C\) is the predicted cost of a building with the typical floor area of \(A\) sq ft, and height of \(H\) ft.

Similar method (historical review) can be used to derive equations for other types of tall buildings. All other components of the planning phase can be directly adopted from suggestions presented by Kerzner [6].

3. ORGANIZATIONAL STRUCTURES

The structure of an organization is reflected in how information is transmitted, how decisions are made and where responsibilities lie. The whole project phase can be divided into some parts of different extends of complexity, uncertainty and interdependency. From the strategic point of view the organizational structure of the project should be the same during the whole project phase. In the progress of the construction process some departments of the structure would change their activity. Among organizational proposals reviewed in paper [10] the Functional Organization with Area Coordination presented in fig. 2 is suitable for the considered projects. Managing the construction phase in a shared responsibility so the "Construction Manager" component (fig. 2) may be departmented according to number of subcontractors or subprojects. Such an approach is very similar to that used in Britain as described in [7]. Building the organizational structure and departmenting its components one should take into account that integrating cost and scheduling data is easier to achieve in the engineering phase than in the construction phase.
4. DECISION MAKING

Construction of tall building is a very dynamic project. As mentioned before, changes in project phase stimulate adjustments in the organization. There are many environmental factors influencing decision making style in the project organization, such as:

- organizational structure itself (type, position, culture),
- organizational groups (committees, unions),
- personal traits (experience, background),
- information available (certainty, uncertainty, risk).

Usually managers decide using few models of Decision-Making Theory at the same time. However, the Rational-Contingency Model presented by Tatum [9] fits very well to tall buildings construction projects. To meet the requirements of the model it would be critical to use proper Decision Support System. Information is an important strategic resource on the tall building site. So, using software packages could meet many needs of managers within the process of decision making. Despite numerous criticism Critical Path Method (CPM) meets most of managers', contractors' and subcontractors' needs in the decision making process. The CPM scheduling concept is being used on many complex sites of USA. Success of CPM scheduling depends of course on realistic estimation of productivity of crews and time buffers. An excellent analysis of CPM for project analysis is presented in [3].

5. CONCLUSIONS

Very specific characteristics of tall building construction projects requires specific management methodology. Some components of the methodology can be taken from management approaches for large projects and others should be developed taking into account complexity of tall buildings. Engineering, design and construction
should be integrated in one system. The three dimensional CAD model should be used to build a project database for facility development process. The more detailed approach of the model of project management of tall buildings require further research.

LITERATURE:


RESEARCHES ON THE PHYSICAL AND MECHANICAL PROPERTIES OF SPUN-CAST CONCRETE.

by Prof. Mieszysław Kamiński
D. sc Janusz Kubiak
D. sc Aleksy Łodo
M. sc Adewole Adesiyun

1. Introduction

Spun-cast concrete is different from the normal concrete in this respect that during spinning, a segregation of components of concrete mix takes place. The components having a greater mass are displaced to the outer layer of the wall while those of the lowest mass remain in the inner layer. The above mentioned segregation during spinning takes place because the working pressure is not uniformly distributed across the whole thickness of the concrete mix layer. The maximum value is on the outer surface of the concrete layer and gradually drops down when moving toward the inner surface where it assumes the minimum value. This difference in the working pressure is also responsible for the fact that water is not removed to the same extent from the whole thickness of the wall. The value of (W/C) for inner layers is greater than that for the outer layers.

This structure of spun concrete gives in effect not only the physical and mechanical differences between normal and spun-cast concrete but also between the individual layers of the latter.

2. Literature review

The published papers on the properties of spun-cast concrete is limited. Two of these properties, namely the strength and frost resistance are discussed below.

2.1 Strength

The strength of the spun-cast concrete at the outer layers is higher than in the inner zone (1). This results from the fact that, as it was mentioned earlier, the components of higher mass tend to settle at the outer side.

It is commonly known that the spun-cast concrete has the higher strength than the normal concrete but up to now no agreement was reached on how to determine this strength. In some manufacturing plants, the cube strength for normal concrete is used and corresponding strength of spun concrete is determined by the use of correcting factor $k = 1.25$. In other studies however, the values
obtained for factor k fall into the range 1.20 to 2.00 (2).

According to Archiedow (1), the simplest method of checking the strength of spun-cast concrete is by testing the sample cubes prepared either in special forms using the spinning machine or thickening the concrete by vibrating it from the initial (W/C). In the latter case, it will be necessary to apply the intermediate factor k.

2.2 Freeze thaw tests

As stated by Dilger and Ghali (3), resistance of concrete to freeze-thaw cycles is one of the most important criteria for spun concrete poles. In the experiment they carried out (3), five samples with different mix parameters were first soaked in water for two weeks and then subjected to temperature cycles between 4°C to -17.8°C and back to 4°C in not less than 2 and not more than 4 hours. The results are shown in Fig.1. The test on specimen 5 (no air entrainment) was discontinued after 105 temperature cycles because the dynamic modulus dropped to 19% of its original value.

Fig.1. Change of relative dynamic modulus vs. no. of freeze-thaw cycles (3)

The other specimens were considered frost resistant because there was little or no drop in their dynamic modulus. From these results it was concluded by the authors that air entrainment is an absolute necessity to warrant frost resistance of spun concrete poles.
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3. Individual research

The main goal of this research which is been undertaken (by the authors of this article) in the laboratory of the Institute of Building Engineering, Wroclaw Technical University is to investigate the physical and mechanical properties of spun-cast concrete. Another important aspect of this work is to find a correlation between these properties and those of normal concrete.

3.1 Materials and mix parameters

The mix parameters to be varied in this research are as follows:

1. Type and content of cement
   Portland Cement (P 45) which contains 8% C_3A
   This cement is supplied by the cement plant "Malogoszcz".

2. Type of aggregate
   These include gravel, crushed stone, sand and basalt.

3. Water-cement ratio
   The effect of low water-cement ratio (leads to high concrete strength) and high water-cement ratio are to be investigated.

4. Spinning speed and duration
   These parameters are very important since too high a speed or long duration may lead to segregation while too low a speed, to insufficient compaction (3).

Apart from these parameters, another parameter that is of great interest is the type of admixtures. Admixtures help in increasing workability, concrete durability, strength and in reducing the water-cement ratio (3). In this research, superplasticizer Betoplast is to be used. It is used in increasing the workability of the concrete with low water-cement ratios. This also leads to increase in the concrete strength.

Shown in Table 1 are details of two concrete mixes for the spun-cast specimens.

Table 1

<table>
<thead>
<tr>
<th>Cement</th>
<th>Water</th>
<th>Sand</th>
<th>Coarse aggregate</th>
<th>Water Cement Ratio</th>
<th>Admixture Betoplast</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg/m³</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td>kg/m³</td>
<td></td>
<td>kg/m³</td>
</tr>
<tr>
<td>647</td>
<td>225</td>
<td>507</td>
<td>1018</td>
<td>0,35</td>
<td>-</td>
</tr>
<tr>
<td>634</td>
<td>220</td>
<td>497</td>
<td>998</td>
<td>0,35</td>
<td>9,0</td>
</tr>
</tbody>
</table>
3.2 Concrete forms and specimens

The concrete specimens are produced in cylindrical forms (190 x 230 mm). The form with a spun-cast concrete specimen are shown in Fig. 2, while Fig. 3 shows how the form is fixed by means of screws to the spinning machine. The diameter of the concrete specimen corresponds to the minimum diameter of the spun-cast concrete poles manufactured in the Institute's plant.

Fig. 2 The form with the specimen.

Fig. 3 The form fixed to the spinning machine.

3.3 Experiments

The spun-cast concrete properties to be investigated include strength, modulus of elasticity, Poisson's ratio, frost resistance, porosity, creep, shrinkage etc. The properties of the different layers on the wall thickness are to be tested by subjecting them (among others) to chemical and mechanical attacks.

The differential shrinkage of the layers is also of great importance in this research. In an earlier experiment carried out at the Institute of Building Engineering in Wroclaw (results not published), cracking of the inner layer of mortar was attributed to the differential shrinkage that occurs along the wall thickness.
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Figs. 4 and 5 show the measurement of shrinkage on a concrete plate (cut from a spun-cast specimen). Providing two dial indicators on the same level helps in monitoring the differential shrinkage of the layers. Dilger and Ghali (3) performed similar experiments. The aim of their work was to design concrete mixes which would not segregate during spinning, thus eliminating differential shrinkage.

Figs. 4 and 5. Measurements of Shrinkage Strains

Conclusion

Little has been done so far not only on the physical and mechanical properties of spun-cast concrete but also on the correlation between spun and normal concretes. It is hoped that this research will shed more light on the concrete technology of spun concrete.

References

2) Batasev V.M. "Prochnost, trespinostokost i deformacii zelezobetonych elementov s mnogoriadnym armirovaniem" Kijejev 1978.
Research on building structures and building physics.
Research on building structures and building physics.

Steel research
Research on building structures and building physics.
EXPERIMENTAL STABILITY OF SPACE STRUCTURES
BASED ON DYNAIMICAL CRITERION

1. Introduction.
In many theoretical works the three-dimensional structures [fig.1] are considered as a system of nodes and bars with pinned ends.

![Fig.1.](image)

This simplification is proper when the internal axial forces in individual bars have to be computed. The same assumption gives a significant inaccuracy in the process of dimensioning of those bars which are compressed i.e. part of diagonals and an upper layer of a space structures. In the true structures all bars have elastically build in ends with a different degree of fixity (system Nodus, Octaplatte, Mero and others) which leads as to the hypothesis that the reduced length \( l_r \) of such a bars are not constantly equal to their theoretical length \( l_t \),

\[
1_r \leq l_t
\]

or

\[
1_r = a \cdot l_t \quad \text{and} \quad a \leq 1
\]  

[1]

where \( a \) is the coefficient of the reduced length.

Thus the overall load capacity of the true system can be significantly higher from that which is modelled by bars having pinned ends exclusively. Disclosing this reserve we can design many complex latticed structures in a more economical way.

2. Program of experimental tests.
Experimental test have been divided into three steps:

2.1. Destructive tests executed on small models of the three dimensional structures.

2.2. Destructive tests executed on the full-size pyramids as integrated parts of a space structure.

2.3. Non-destructive test executed on a true-three dimensional 21.0 x 27.0 m by the method checked in the step 2.2.

In the step 2.1, we were looking for the load-bearing capacity of
models which was afterwards compared with the results of the theoretical analysis made for an idealised system modelled by bars with pinned ends. The disclosed experimentally reserve of the load bearing capacity reached in some cases 35 - 40 %. These optimistic results lets us to continue the experiment.

In the step 2.2. the applicability of the dynamical method of testing of the stability was checked. This part of investigations will be presented below.

3. Experimental determination of critical load based on dynamic criterion of stability - theoretical background.

The dynamic criterion of the stability of the bar is based on the relationship between the frequency of the lateral free vibration and the compressive axial load of the bar. This relationship is presented in the literature [1,2] in the form of an approximate formula:

$$\omega_s^2 \simeq \omega_o^2 \left(1 - \frac{S}{S_{cr}}\right)$$

which indicates that the circular frequency of the free vibrations $\omega$ decreases when the axial force $S$ increases and when the load reaches his critical value $S_{cr}$ the frequency becomes zero. The circular frequency $\omega_o$ of the lateral vibration for a prismatic pinned end bar occurs without the axial load $S$. Putting $\pi^2 E/[a1]^2$ in an equation (2) we are getting:

$$a \approx \pi \sqrt{-\frac{E}{S}} \left(\frac{\omega_s}{\omega_o} - 1\right)$$

For a case when the measurement of $\omega_s$ can be executed for two different stages of loading $s_1$ and $s_2$ it is profitable to use this equation in a modified form:

$$a = \frac{\omega_1^2 - \omega_2^2}{s_2 \omega_1^2 - s_1 \omega_2^2} \frac{S}{E}, \text{ for } S_1 < S_2$$

where $S_E = \pi^2 E J/1^2$ (Euler's load).

4. Experimental investigations.

As it was mentioned before a set of single full - scale pyramids have been subjected to a destructive test for finding:
- the shape of the empirical relationship between the basic mode of the lateral free vibration $f_s = \omega_s/\pi$ and the axial compressive force $S$ acting in the individual diagonals in order to obtain the critical load basing on the dynamical criterion of stability ($f_s = 0$)
- the load bearing capacity $S_{lb}$ of diagonals obtained directly by a destructive test and to compare it with the critical load $S_{cr}$ determined accordingly to the dynamic criterion of stability (non-destructive method).

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The geometry and the cross section of the pyramid members subjected to the test are shown in fig. 2:

The diagonals have been made of pipes with steel R-35 (yield stress design value 235 MPa) and connected in the top by cold-pressed hat node [5]. The vertical external load - P_i forced by a hydraulic jack has been applied to the top point. The axial forces in diagonals - S_i have been recorded for each step of loading within the range P_i = 0 → P_i = P_{1b} (destructive load) by especially constructed mechanic gauges [3]. The remote measurement of the lateral free vibration - f_s1 have been executed by means of piezoelectric gauges fixed in the middle point of each of tested diagonals and recorded on the light-sensitive tape. A part of results have been given in the table 1.

The experimental relationship \( f = f(S) \) has been described by the regression curve [6] and shown in fig. 3.

\[
f_{s1} = b + c \ln (S - S_{1}) \quad \text{for } S > S_{1}
\]  

(5)
Substituting $f_{S_i} = 0$ when $S_i = S_{cr}$ (dynamical criterion of stability) we can estimate the critical load:

$$S_{cr} = S - e^{-b/c}$$

(6)

where $S$ is a constant force fullfilling the condition $S > S_{lb}$. $b$ and $c$ coefficient of regression.
Having the value of $S_{cr}$ (col.4, tab.2) calculated according to equation (6) the mean value of reduced length $\bar{a} = \pi l [E]/S_{cr}$ (col.6, tab.2) have been determined and compared with that calculated directly as $a^* = \pi l [E]/S_{lb}$ (col.5, tab.2) for $S_{lb}$ obtained as a load-bearing capacity of the individual diagonals in destructive tests (col.3, tab.2). The coefficient $a^*$ contains the physical and geometrical imperfections of the member giving an integrated and more objective value.

Table 2.

<table>
<thead>
<tr>
<th>Pyramid type and model number</th>
<th>Number and diagonal cross-section</th>
<th>Load-bearing capacity</th>
<th>Critical load</th>
<th>Integrated coeff. of reduced length</th>
<th>Coeff. of reduced length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_{lb}$ [kN]</td>
<td>$S_{cr}$ [kN]</td>
<td>$a^*$</td>
<td>$a$</td>
<td></td>
</tr>
<tr>
<td>P.7 - M.2</td>
<td>1234 060/3</td>
<td>70.14</td>
<td>71.73</td>
<td>0.8028</td>
<td>0.7911</td>
</tr>
<tr>
<td>P.7 - M.3</td>
<td>1234 060/3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P.1 - M.6</td>
<td>12 083/3.5</td>
<td>158.84</td>
<td>162.34</td>
<td>1.0588</td>
<td>1.0275</td>
</tr>
<tr>
<td>P.1 - M.7</td>
<td>12 083/3.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5. Remarks and conclusions.

The experimentally studied relationship between the frequency of the lateral free vibration of the member and axial compressive force (fig.3) gives the basis of a simple estimation of the critical load.

The comparison of load-bearing capacities of the diagonals of the full-scale pyramids estimated after both destructive and non-destructive tests shows a satisfactory agreement (table 2).

The non-destructive method described above may have a practical application when the reduced length of members with elastically supported ends has to be determined. Thus the load-bearing capacity of the spatial lattice structures can be estimated more objective which in many cases leads to the more economical designing.

References

Research on building structures and building physics.
WEB CRIPPLING OF COLD-FORMED STEEL MEMBERS
by Monique C.M. Bakker

ABSTRACT
Cold-formed, thin-walled flexural steel members are frequently used as structural elements in building. When such a member is subjected to a concentrated load or reaction, its web may cripple due to the high local intensity of the load. This type of failure is denoted as web crippling. This paper focuses on the description of an analytical web crippling model and the comparison of the model and tests results.

INTRODUCTION
Objectives
Web crippling of cold-formed steel members has been studied since the 1940s. Because theoretical analysis was found to be rather complicated, most studies concentrated on the development of design formulas based on curve fitting of test results (see for example Wing, 1981). These empirical formulas have a limited and often not well described range of applicability. Of course, each formula correlates well with the test results it is based on, but the correlation is worse for tests from other sources (Bakker and Peköz, 1985). The research reported in this paper was prompted by the lack of real understanding of the web crippling behavior. The primary object of the research was to develop an analytical model. This model should then be used to develop more reliable and theoretically based design formulae.

Scope
Web crippling may occur under various loading conditions. This research was restricted to web crippling at interior supports of continuous members (Fig. 1). At these supports the concentrated load is accompanied by a bending moment, which reduces the resistance of the member against the concentrated load. Only cold-formed hat and deck sections with unstiffened web and flange elements have been considered (Fig. 2).
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Fig. 1 Web crippling of a thin-walled cold-formed steel member at an interior support (Reinsch, 1983)

Fig. 2 Section types included in the research

In the following, first the approach of the existing design formulae is explained. Then a short description will be given of the experimental research, which was carried out to enable the development of the model. Subsequently the model is presented, and the comparison of the model with tests results is discussed. Finally some conclusions and recommendations for further research are summarized.

In a short paper like this it is impossible to explain and justify all the details of the model. For more information the reader is referred to Bakker (1992).

EXISTING DESIGN FORMULAE

In the approach of the current (Eurocode and AISI) design formulae it is assumed that small bending moments have negligible influence on the web crippling behavior: the web crippling formulas predicting the ultimate web crippling resistance $F_u$ are based on curve-fitting of the results of web crippling tests, which are performed as three-point bending tests on members with very short spans. The influence of larger bending moments is taken into account by means of an interaction formula (Fig. 3). This interaction formula is based on curve-fitting of the results of three-point bending tests on members with longer spans.
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Fig. 3 Graphical representation of an interaction formula

EXPERIMENTAL RESEARCH

Test program
For the development of an analytical model it was found necessary to perform new web crippling tests. The web crippling tests described in the literature could not be used, since in most of these tests only the ultimate loads were reported. A test set-up was designed to perform deformation controlled web crippling tests. The tests were carried out as short span three-point bending tests (Fig. 4). Besides the load also the web crippling deformation, the mid-span deflection and the rotation at the end supports were measured. The varied parameters in the tests are the span length, the plate thickness and yield strength, the length of the load bearing plate, the web angle, the width of the web and the width of the top flange.

Fig. 4 Measured deformations in the performed web crippling tests
Failure modes
From the experimental research it was concluded that cold-formed hat and deck sections, subjected to the combined action of a concentrated load and bending moment, may fail by two different types of web crippling failure modes: the yield arc mechanism occurring in members with small corner radii, and the rolling mechanism occurring in members with large corner radii. In three tests an asymmetrical failure mode was observed, which might be caused by the attainment of the ultimate bending moment resistance.

The yield arc and rolling mechanisms are characterized by different web deformation modes (Fig. 5). In the yield arc mechanism the web buckles and the web deformation is enabled by a curved yield line. In the rolling mechanism the corner radius rolls down through the web, a process which can be modeled with moving yield lines. In both mechanisms the flange deformations result in the formation of a plastic hinge mechanism (Fig. 6). This hinge mechanism affects the global load-deformation behavior of the member.

![Fig. 5 Idealized web deformation modes in the yield arc and the rolling mechanism](image)

![Fig. 6 Idealization of the hinge mechanism](image)
In the yield arc mechanism the initiation of the hinge mechanism (that is, the initiation of non-linear deflections) more or less corresponds to the attainment of the ultimate load (Fig. 7), in the rolling mechanism it merely marks a change in the load-web crippling deformation behavior, the ultimate load being reached at larger web crippling deformations and some mechanism rotation (Fig. 8). For members failing by the rolling mechanism it was therefore found a useful idealization to define a mechanism initiation load as the load corresponding to the initiation of the hinge mechanism.

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**Fig. 7** Typical load-deformation behavior of a member failing by the yield arc mechanism

**Fig. 8** Typical load-deformation behavior of a member failing by the rolling mechanism
A MODEL FOR THE ROLLING MECHANISM

The research has resulted in a model for predicting the mechanism initiation load of members failing by the rolling mechanism (for the yield arc mechanism no model has been developed yet). According to the model, the mechanism initiation load is determined as the point of intersection of an elastic curve and a rigid-plastic mechanism initiation curve (Fig. 9).

Fig. 9 Modeling of the load-web crippling deformation behavior of a member failing by the rolling mechanism

The analytical determination of the elastic curve has not been treated yet, the curve is taken simply as a straight line with a slope equal to the initial web crippling stiffness measured in the tests.

The rigid-plastic mechanism initiation curve is derived by using generalized yield line theory. Based on the observed failure modes in the tests, a simplified yield line pattern with straight yield lines is proposed (Fig. 10).

Fig. 10 Idealized yield line pattern in a member failing by the rolling mechanism
All yield lines are treated as stationary yield lines, except the yield lines at the curved transition between the webs and top flange, which enable the rolling down of the web. It is assumed that at the initiation of the mechanism, the rolling radius (that is, the instantaneous radius of the curved transition between the web and top flange) is equal to the initial corner radius between the web and loaded flange, and that the other yield line pattern parameters (that is, the distances $L_{yt}$ and $L_{yb}$, Fig. 10) can be determined by minimizing the mechanism initiation load.

To predict the load-deformation behavior after the initiation of the mechanism, so far only constant-rolling-radius mechanism curves have been derived, based on the assumption that the rolling radius and the yield line pattern do not change during deformation. In the tests it was observed however that the rolling radius decreases with increasing web crippling deformation. A typical feature of the rolling mechanism is the increase in the load-carrying capacity after the initiation of the mechanism (Fig. 8).

From the path of the constant-rolling-radius mechanism curves (Fig. 9) it can be derived that this increase is probably caused by a decreasing rolling radius. Note that Fig. 9 does not explain why the rolling radius decreases.

The calculation of the mechanism initiation and constant-rolling-radius mechanism curves is based on a simplified work method. The work method means that the mechanism (initiation) load is calculated by equating the external incremental work by the applied loads to the internal incremental energy dissipation in the yield lines. It is a simplified work method, because the coexistent stresses in the yield lines are determined from global equilibrium considerations, and not from the normality rule, as in the exact work method. The influence of these stresses (that is the stresses caused by the global bending moment) on the energy dissipation in the yield lines is taken into account by means of stress factors.

**COMPARISON OF THE MODEL AND TEST RESULTS**

**Prediction of the mechanism initiation load**

Figure 11 shows an example of the comparison of the predicted and measured mechanism initiation load in one specific test. In Fig. 12 an overview of the ratios $F_{imec, test}/F_{imec}$ is given, for all the tested members failing by the rolling mechanism. It can be seen that the model gives reasonably accurate predictions of the mechanism initiation load for members with an interior top flange corner radius of 10 mm. For members with a radius of 5 mm, the model tends to overestimate the mechanism initiation load. This may partly be caused by the fact that the mechanisms occurring in these members are at the transition from the rolling to the yield arc mechanism.
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Fig. 11 Predicted and measured mechanism initiation load in test 16

Fig. 12 Accuracy of the prediction of the mechanism initiation load as a function of the corner radius

It is suspected that especially the influence of flange curling (that is the downward deflection of the top flange underneath the load bearing plate) and the influence of stresses caused by the bending moment on the energy dissipation in the moving yield lines are not yet modeled sufficiently accurate.

It is striking that both model and test results indicate that a decreasing web angle results in an increasing mechanism initiation load. According to the current web crippling prediction formulae the ultimate web crippling resistance $F_u$ decreases with a decreasing web angle.

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Prediction of further mechanism behavior

The calculation of mechanism initiation load implies the calculation of the distances \( L_{yt} \) and \( L_{yb} \) (Fig. 10). The values of \( L_{yt} \) and \( L_{yb} \) predicted by the model correspond to reasonable yield line patterns, but do not correspond exactly to those observed in the tests. This does not matter for the calculation of the mechanism initiation load, since this load is not very sensitive to the assumed yield line pattern. It is more important for the calculation of mechanism curves (and the ultimate load), since the sensitivity of the mechanism load to the distances \( L_{yt} \) and \( L_{yb} \) increases with increasing web crippling deformation.

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

The research has resulted in an analytical model for predicting the mechanism initiation load of members failing by the rolling mechanism. According to this model the influence of the bending moment cannot be treated separately from that of the concentrated load. Potentially, the model can be further developed for the prediction of the decreasing-rolling-radius-mechanism curve. From this curve then the ultimate load can be determined. However, an accurate prediction of the ultimate load will be much harder than that of the mechanism initiation load.

Both model and test results indicate that the difference between the ultimate load and the mechanism initiation load decreases with increasing span lengths. Therefore the hypothesis is set forth that for practical span lengths this difference is so small that it would not be unduly conservative to develop design formulas based on the prediction of the mechanism initiation load instead of the ultimate load. To check this hypothesis, web crippling tests on longer span tests are needed.

For the development of design rules not only a model for the rolling mechanism is needed, but also a model for the yield arc mechanism. However, the above formulated hypothesis draws attention to the fact that in studying failure modes one must make sure that one studies relevant failure modes. Before starting to work on the development of a model for the yield arc mechanism, it would be wise to check whether this mechanism indeed occurs in practical loading conditions. Long span members with small corner radii might fail by the attainment of the ultimate moment resistance rather than by the yield arc mechanism.

Since the model for the rolling mechanism was based on an idealization of the observed failure mode in hat and deck sections with unstiffened webs, it cannot automatically be generalized to other types of sections.
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ACKNOWLEDGEMENTS

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THE BEHAVIOUR OF A HEADED-STUD-CONNECTION IN A COMPOSITE SLAB.

ir. Pieter G.F.J. van der Sanden.

ABSTRACT: Nowadays in the Netherlands composite beams (Fig. 1) are still little used. These beams consist of a composite slab with a profiled steel sheeting connected with a headed-stud-connection in the rib. The design rules for the strength of this connection in a composite beam are related to the design rules concerning solid slabs by means of a simple reduction-factor. However the failure-strength and the failure-mode predicted by the design rules do not correspond with those observed in push-out tests. Sometimes the design rules are resulting in an unsafe prediction of the failure-strength and in other cases the design rules are far too conservative. At Eindhoven University of Technology a research project is started in which a model is made to predict the failure-strength, the failure-mode and the load-deformation-behaviour of a headed-stud-connection in a composite slab.

INTRODUCTION.

Dutch building-contractors are using among prefab concrete slabs and in situ casted concrete slabs also composite slabs. However in the Netherlands the proportional amount of realised composite slabs is less than the amounts that are realised in the United Kingdom, the United States and Japan. Before explaining why these composite slabs are less used in the Netherlands, the system in which composite slabs are used in combination with steel beams will be described. In this paper a composite beam (Fig. 1) consists of a composite slab (top layer) and a steel beam, which are connected with each other by means of headed-stud-connections. In this paper only the situation is regarded in which the ribs of the composite slab are perpendicular to the steel beam.

Fig. 1: Composite beam.

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The composite slab is cast by using a permanent profiled steel sheeting, which acts compositely with the concrete layer on top of it. The steel sheeting is used as a permanent shuttering and as a reinforcement, because the sheet is connected with the concrete layer by means of chemical bond and physical keying. Mainly there are 2 forms of the profiled sheeting, namely trapezoidal and re-entrant (Fig. 1). The profiled steel sheeting can be made with indentations and/or embossments. In the Netherlands profiled steel sheetings with rib-heights varying from 50 to 80 mm are used. The thickness of the 2-sided galvanised sheet varies from 0,70 to 1,25 mm.

![Fig. 2: Connection between concrete and sheeting.](image)

The connection between the steel sheeting and the concrete can be divided in 3 groups:
- frictional interlock for profiles in a re-entrant form (Fig. 2, upper part),
- mechanical interlock by means of indentations and/or embossments (Fig. 2 middle part), and
- end anchorages (Fig. 2, bottom part).

The end anchorage can be made in different ways, from which the headed-stud-connection (Fig. 2, bottom part, right side) is used a lot in the Netherlands.

As the profiled steel sheeting is laid over the structural steelwork, there is no direct connection between the concrete and the main steel section. Therefore the shear connectors can be welded through the steel sheeting in situ. In this way a composite beam with a composite slab as top-layer is realised. This combination is suitable for realising large spans for both the beam as the slab. As the composite beam is loaded in bending there is a longitudinal shear force (S in Fig. 3) in the plane between the composite slab and the top of the upper-flange of the steel beam. This shear force has to be transferred from the composite slab to the steel beam by means of the headed-stud-connection.

![Fig. 3: Forces composite beam.](image)
One of the reasons why composite slabs are relatively less used in the Netherlands finds its origin in the fact that in general Dutch building-participants (architects, engineers and contractors) have only little experience with the advantages of this floorsystem and the efficient use of the materials concrete and steel. Dutch building-participants prefer to use large spans (3.6 to 7.2 m), which can be realised with rib-heights varying from 100 to 200 mm. However when the composite slab and the steel beam are combined (and designed) as a composite beam, in both the Dutch code (RSBV 1990 [1990]) as the European code (Eurocode 4 [1990]) the maximum rib-height is limited to 80 mm, which limits the use of large spans. The missing knowledge of the behaviour of the headed-stud-connection in a composite beam is an impediment for the use and the development of new profiles (higher sheets and narrower ribs). It is important to know how large the maximum shear force (S in Fig. 3) is that can be transferred, and how the failure-strength, failure-mode and load-deformation-behaviour are depending on, for instance the rib-geometry, material-properties, geometry composite slab and geometry steel beam.

Recently at Eindhoven University of Technology a student-research [Wijte, 1989] has been finished, in which the behaviour of a headed-stud-connection in a composite beam was investigated. It should be possible to describe this behaviour with a model, and to make design rules which are also suitable for rib-heights larger than 80 mm. In the next chapters the following topics are described: design rules which are nowadays available, research recently finished at Eindhoven University of Technology and research at Eindhoven University of Technology which is done nowadays.

**CODES AVAILABLE NOWADAYS.**

The characteristic connection resistance of a headed-stud-connection in a composite rib (= rib in a composite slab) is related to the characteristic connection resistance of the same stud-connection in a solid slab. According to (RSBV 1990 [1990]) the design shear resistance of a headed-stud-connection in a solid slab ($S_{um}$) has to be determined with (1) and (3). The notations used are given in Fig. 4.

\[
S_{um} = 0.32 \alpha d^2 \frac{\sqrt{f'_{ct} E'}}{\gamma_{mv}} \quad (1)
\]

\[
\alpha = 0.2 \left( \frac{h}{d} + 1 \right) \leq 1.0; \quad \frac{h}{d} \geq 3 \quad (2)
\]

\[
S_{um} \leq 0.7 \pi d^2 \frac{f_{ad}}{\gamma_{mv}} \quad (3)
\]

- $\alpha$: reduction factor, -
- $f'_{ct}$: characteristic compressive strength of concrete, N/mm²
- $E'$: Young's modulus concrete, N/mm²
- $\gamma_{mv}$: partial safety factor = 1.25
- $f_{ad}$: ultimate tensile strength of the material of the stud, N/mm²

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Equations (1) and (3) are based on 2 failure-modes. Failing by local concrete crunching around the foot of the stud-connector (Fig. 5, above) is represented by (1). Equation (3) is based on shearing of the connector just above the weld collar (Fig. 5, under).

The design shear resistance of a headed-stud-connection in a composite rib has to be determined by multiplying (1) with the reduction-factor (4). The smallest value of (3) and the product of (1) and (4) is the design shear resistance, which has to be used in design calculations. Equation (4) is only valid if the ribs and the steel beam are perpendicular to each other. The number of studs (n) in (4) may not exceed 3.

$$0.85 \frac{b_p}{h_n} \left( \frac{h}{h_a} - 1 \right) \leq 1.0$$

(4)

Some remarks have to be made with respect to determining the design shear resistance as described in the last paragraph.

- A stud connection in a composite rib has 4 more possible failure-modes, next to the 2 failure-modes mentioned before:
  - punching from the stud through the composite rib (Fig. 6a),
  - shearing of the composite rib (Fig. 6b),
  - shearing of the composite rib in combination with pulling out of a concrete cone (Fig. 6c), and
  - cracking from the concrete slab (Fig. 6d).

With the simple reduction-factor (4) it is not possible to determine the design shear resistance for every possible failure-mode.

- By composite beams the equations (1) and (3) are according to the Dutch code (RSBV 1990 [1990]) and the European code (Eurocode 4 [1990]) only valid in a limited area of use (for example: $h_s \leq 80$ mm), and are based on research (on mostly push-tests with solid slabs) done in the seventies.

- The failure-strength and failure-mode can be determined with an experiment (for example: push-test Fig. 7). If the failure-strength and failure-mode from these experiments are compared with the shear resistance and expected failure-mode according to the codes, 2 aspects become clear (Wijte [1989]).
In several push tests failure-modes occurred which could not be described with (1) or (3).

When the experimental determined failure-strengths are compared with the failure-strengths, which are calculated according to the codes, it seemed that in 50% of the considered cases in stead of 5% (characteristic) the experimental determined failure-strength is smaller than the calculated one (Fig. 8).

Fig. 7: Shear-test.

Fig. 8: Comparison measured and calculated shear-force.
In recent research (Wijte [1989], Lloyd [1991]) it has been shown that headed-stud-connections in a composite rib often have been failing according a wedged-shear-cone-failure-mode (Fig. 6c, Fig. 9). As composite beams are realised with the desired greater rib-heights (to realise large spans) this failure-mode becomes even more important. Therefore at Eindhoven University of Technology a first attempt was made to describe the behaviour of a headed-stud-connection loaded in shear with a model (Wijte [1989]). In the next chapter the model has been described. In 1991 a PhD-research has been started in which both the failure-strength, the failure-mode and the load-deformation-behaviour of the connection is subject of research. At the beginning, as contrasted with most research in this field, there is no special interest in one particular failure-mode. Later on the interest will be concentrated on the wedged-shear-cone-failure-mode.

![Image](image_url)

**Fig. 9: Wedged-shear-cones in failed test-specimen.**

**MODEL EINDHOVEN UNIVERSITY OF TECHNOLOGY (Wijte [1989]).**

According to (Wijte [1989]) the wedged-shear-cone-failure-mode starts with cracking in the concrete plate (Fig. 6d). With the model it is possible to calculate the cracking-load (force at a load, as the first crack occur) and the corresponding displacement of the connection. The geometry of the composite rib (trapeziodal) has been modelled with a 2-dimensional truss-model (Fig. 10).
In the sections A and B (Fig. 10) the moments are calculated. The bending-deformations of the rib and the plate as well as the shear-deformations of the plate were taken in to account. With an arbitrary chosen equivalent width of the plate the maximum tensile stress in the section A and B was calculated. As this stress is equal to the tensile strength of the concrete it is possible to calculate a cracking-load for both sections. The smallest one is representative. From 8 push-tests the cracking-load was known, and compared with the calculated cracking-load. With this model the cracking-load could not be determined exact enough. An experimental and numerical research was done to get more knowledge of the behaviour of the connection, and of how this behaviour is affected by different parameters.

Three of the 11 experiments have been carried out to verify the numerical model (2-dimensional, plane strain, finite-element-package DIANA). As main result of this verification was concluded that there is a minimum width, above which increasing the width has no effect on the cracking-load. From the other 8 push-tests it seemed that the profiled steel sheeting affects the behaviour of the complete connection a lot. Wijte also stated that it is important to standardise the specimen-width as well as the manner how these specimens are supported. With this standardisation it becomes possible to use and compare test-results from other researchers.

From the results of the numerical model and the experimental results the physical model has been adapted. The main adaptions include: equivalent width rib, tensile strength concrete, equivalent section concrete rib and the influence of the shear stress on the maximum principal stress. However the model still seemed not accurate enough. Although there has been done a lot of research in this field, there is still little knowledge about the behaviour of a headed-stud-connection in a composite beam. There is not any appropriate theoretical model to predict the failure-strength, the failure-mode as well as the load-deformation-behaviour. In the next chapter the results at the beginning of a PhD-research at Eindhoven University of Technology are presented.

RESEARCH AT EINDHOVEN UNIVERSITY OF TECHNOLOGY.

Research-program.
The research-program is divided in a numerical and an experimental part. At the beginning a 3-dimensional finite-element-model has been made (Fig. 11). The results with this model will be verified with experiments. In a next phase the model will be extended with non-linear material-behaviour. With this model headed-stud-connections can be designed. However it is meant to be an engineering-tool to get more knowledge about the behaviour. With this knowledge it is probably possible to make a simple to use model.
Research on building structures and building physics.

A part of the experimental verification is supposed to be carried out with specimens made of a material, which material-parameters only have a little standard-deviation. In this way it is easier to verify the quality of the model.

Results beginphase research.
With the truss-model of Wijte it is possible to give a prediction of the load, which develops at the moment that the first crack at the corner between the plate and the rib appears. This crack is according to (Wijte [1989]) the beginning of the wedged-shear-cone-failure-mode. However the model is a 2-dimensional model and the length of the rib has been substituted in the model with a equivalent width. The choice of this width is arbitrary.

With the numerical model (3-dimensional) the equivalent width can be determined. Also the position of the maximum tensile stresses can be determined. At the beginning the geometry of the rib is simple (rectangular) and the model is carried out without the profiled steel sheeting (Fig. 11). This model consists of the half length of a composite rib.

Provisional qualitative conclusions can be drawn of the shown calculation-results (Fig. 12).
- The shape of the deformed model gives no reason to suppose that the chosen boundary conditions of the model don't correspond with the boundary conditions in a composite rib in a push-test-specimen (Fig. 12, left).
- The studhead is hardly necessary to transfer the tensile-force in the stud to the surrounding concrete, because the stresses in the studshaft near the studhead are very small (Fig. 12, right).
Research on building structures and building physics.

- From the distribution of the support-reactions (Fig. 13) in the concrete plate (face a in Fig. 11) it becomes evident that the shear-force (S in Fig. 3) is distributed over a width equal to 4 times the centre-to-centre-distance (L in Fig. 4) of the studs. It is plausible that this width will increase as non-linear material-behaviour appears.

- From the stress-distributions shown in Fig. 14 from the xy-plane (z=0, 60≤y≤120, Fig. 11) it becomes evident that in this plane the maximum tensile stress appears in the corner between the plate and the rib. Also it becomes evident that the shear stresses $\sigma_{yz}$ and $\sigma_{xz}$ cannot be neglected.

![Fig. 12: Deformed model by a little rib-width (left) and principal stresses in the front-face (right).](image)

The research will be continued with describing the composite-beam-behaviour according the sandwich-theory and with some adaptations of the numerical model. These adaptations are described below.

- The geometry of the composite rib will be changed to a trapezoidal shape.
- The model will be extended with the profiled steel sheeting.
- The elementsizes near the studfoot will be decreased. The elementsize has to be a couple times smaller than the studdiameter. Also a manner has to be sough to minimize used computermemory as well as calculationtime.
- The model will be extended with plasticity and with cracking of the concrete.

During this modelling experimental verification will take place. It depends of the experimental results whether it is necessary to adapt the model.

![Fig. 13: Support-reactions.](image)

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Fig. 14: Primary stresses in the xy-plane (z=0, 60≤y≤120).
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Research on building structures and building physics.
"Comparison of different nail bending apparatus."

ir. H.P.C.A. Vianen
ir. F. Schot

ABSTRACT.
A research to develop a registered testmethod to define the allowable bending moment of a nail was started in spring of this year. A request for a registered testmethod is caused by the final project of ir. H.P.C.A. Vianen’s study. The consequence of developing new codes in The Netherlands and Europe is part of this research. Different testmethods on nails have been carried out: a tensile test, a four-point-bending test and a lever test. The first method gives a stress-strain relation for calculate the maximum bending moment, the other methods results in force-displacement relations and gives values for the maximum bending moment. A comparison between these testmethods has been made.

INTRODUCTION.
The aim of the research is to develop a testmethod which makes it possible to determine the allowable bending moment of a nail. Therefore a comparison between different testmethods has been made. During a literaturestudy it became clear that a standardized testmethod is not available. In a draft version of the European code EN409 a four-point-bending testmethod is required, while in the new Dutch code NEN6760 it’s required to use the yield stress to calculate the yield moment. The yield stress should be determined experimentally, with a specimen which has an increased cross-section. This is a strong requirement, because the product is not tested in its fullsize. Tests have been carried out to come to a comparison between the different testmethods.
TESTMETHODS.
As mentioned different testmethods are carried out; a tensile test, a fourpoint-bending test and a lever test. The lever test was described in an informative appendix of the draft version of the EN409 code. On each testmethod three series of nails with different diameters, 2.7, 3.5 and 4.0 mm, have been tested. Each testserie consists of five specimens.

Tensile test.
The common wire nails are tested as shown in fig. 1, having an increased cross-section and screwed ends. The deformation and load are registrated and calculated to the stress-strain relation of the testspecimen. The values for the yield and ultimate stresses are shown in table 1.

Table 1 \( f_{0.2\%} \) and \( f_{u,t} \) of the nails.

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>2.7</th>
<th>3.5 (1)</th>
<th>3.5 (2)</th>
<th>4.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{0.2%} )</td>
<td>888</td>
<td>687</td>
<td>812</td>
<td>763</td>
</tr>
<tr>
<td>( f_{u,t} )</td>
<td>917</td>
<td>756</td>
<td>846</td>
<td>797</td>
</tr>
</tbody>
</table>

Graph 1 M-\( \kappa \) relations for the nails of the tensile tests.
In the calculations of the stresses is the increasing crosssection by contraction ignored. This makes that the stresses are not the real stresses.

With an iteration method the stress-strain relation is calculated to a moment-kappa relation, see graph 1. All the results in the graphic show an ultimate bending moment for a kappa of 0.015 or 0.020 mm⁻¹. Especially the nail with diameter 3.5 mm has an increasing bending moment after the topvalue. The fact that the crosssection increases in the higher stressregions during the tensile test has an important influence on the calculation for the bending moment which can be concluded by the increasing moment in the graph. More about this problem later.

**Fourpoint-bending test.**

In the draft version of the European Code EN409 relations are required between the diameter and the distances \(l_1\), \(l_2\), and \(l_3\). Based on these requirements a test setup has been developed, see fig. 3. Distances \(l_1\) and \(l_3\) see fig. 2, are both 10 mm, distance \(l_2\) is 5 mm. The nail can now be tested in fullsize. The loading points are created by bullits to create a small contactsurface to be sure about the enddistances \(l_1\) and \(l_3\). With the load and the enddistance the bending moment on the nail can be calculated. The deformation is also registrated, which makes it possible to generate load-deformation or moment-kappa relations. The resulting diagrams makes it hard to decide what the yield or ultimate bending moment of the nail is. There is not a

![Fig. 2 Principle of the test.](image)

![Fig. 3 Fourpoint-bending test.](image)

![Fig. 4 Deformation in bending test.](image)
For the used configuration in the test setup (10, 5, 10 mm) the relation between kappa and delta will be: \( \kappa = 35/175 \). The M-\( \kappa \) relations for the three different nails are presented in graph 2.

![Graph 2 M-\( \kappa \) relation for the nails of the four-point-bending test.](image)

For all the drawn relations the moment is still decreasing. The tests are stopped when reaching a deflection of the specimen of 4 mm. These large deformations of the specimen influence the load distribution and the mechanics model. The nails are no longer a stiff beam but also a cord. This makes that the mechanics model, with the deformations as showed in fig. 4, is no longer available. Further research about this behaviour will be done.
Lever test.
According to the principles of the fourpoint-bending test another testsetup is described in the European Code EN409 in the informative appendix, see Fig. 5.
By rotation of the left part (the arm) of the setup around \( A \), a bending moment is carried out to the nail. The nail is fixed in a bushing (2) on the left part. At the other side the nail is put into a cylinder (4) which is fixed to a lever (5). The right side contains a load cell (7) to measure the reaction force. This testsetup is developed by Larsen in Denmark and Werner/Ehlbeck in Karlsruhe, Germany\(^1\). Profound analyses make clear that this testsetup will never be able to create a pure bending moment in the nail, which is easy to understand with the reactionforce in the cord (6). The bending moment on the nail will be the product of the reactionforce and the distance to the nail: \( M = F \times h \) or \( M = F \times (l_4 + l_2) \), so in the middle part of the nail there will not be a constant bending moment. Both shear stresses and normal stresses are in the crosssection of the nail.
Based on this concept a testsetup for the lever test has been developed where in the distances \( h \) and \( b \) become nil. The nail is now fixed at the centre of the lever, see Fig. 6. With these changes in the testsetup the deformation of the nail is realized without a horizontal translation as a consequence of the distances \( h \) and \( b \). This makes that the angle between the lever (5) and the cord (6) stays 90 degrees. Than the reactionforce is no more influenced by a horizontal force.
During the test the reactionforce is measured in relation to the time as a kind of testperformance control. As in the fourpoint-bending test a recognizable point of yielding or cracking is not found. Therefore the ultimate moment will be calculated with the highest value of the reactionforce by a maximum rotation of 45 degrees of the arm at the left side of the testsetup.
It might be clear that with this testmethod only a value for the ultimate bending moment of the nail will be reached. Further research with this testmethod is planned and should result in a \( M-K \) relation to compare with the other testmethods.

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Erik Vianen & Frank Schot
COMPARISON OF TESTMETHODS.

M-κ relations.
The M-κ relations from the test results of the fourpoint-bending test in graphic 2 are about similar to the M-κ relations in graph 1. The difference between the calculated and the experimental values is the increasing values of the experiments Graph 3 shows this very well.

Graph 3 Comparison of calculated (lines) and experimental (markers) M-κ relations.

The differences in the M-κ relations between the calculations and experiments are influenced by phenomena, a. the decreasing crosssection of the nail in the tensile test, and b. the decreasing enddistances in the fourpoint-bending test. Other testmethods will be carried out to solve these problems.
Ultimate bending moment.
All three test methods can be used to determine the ultimate bending moment of the nails. As shown in graph 3, the ultimate bending moment is for both methods reached with a different kappa. With these differences it is hard to come to an identical statement for the ultimate bending moment of the nail without further research. The

![Graph 4 Comparison of the test results.](image)

Graph 4 Comparison of the test results.

Tests with the lever test setup determine only the ultimate bending moment. Graph 4 shows the ultimate bending moment found with the experiments. For the four-point-bending test two values have been given, one is the bending moment at a deflection of 0.05 of the free span and the other is the maximum value in the tests.

For all nails the bending moment calculated with the results of the tensile tests has the lowest value. For all nails the results of the lever test and the four-point-bending test are not directly related to each other. The lever test gives the highest values for the 3.5 mm nail, the four-point-bending test higher gives values with the 4.0 mm nail.

Because of the small size of the research, further research is planned to solve these problems experimentally and analytically.
Research on building structures and building physics.
LANGSSTEIFIGKEIT DER STAHLSTÜTZEN MIT KASTENFÖRMIGEM QUERSCHNITT

Dr. hab. Ing. Ernest Kubica

Einleitung

Gegenstand der vorliegenden Arbeit sind aussermittig gedruckte Stahlstützen mit symmetrischem dünnwandigem Querschnitt, geschweißt aus Blech (Bild 1).

Bild 1. Stützenschema und Parameter, die ihre Tragfähigkeit bestimmen

zur kritischen Charakteristik der Kastenprofile werden.

Über die Längssteifigkeit, teilweise auch über die Biegesteifigkeit der Stützen, entscheidet die Stauchung der Wände, die als Plattenstreifen, belastet durch Einflüsse der Schweissimperfektionen, betrachtet werden. Diese Steifigkeiten verringern sich in dünnwandigen Elementen mit der Belastungszunahme, vorwiegend im Bereich der elastisch-plastischen Arbeit der Stahlelemente. Die sollte bei der Ermittlung der Konstruktionsverformungen sowie bei der Berechnung der inneren Kräfte in statisch unbestimmten Systemen berücksichtigt werden.


Stützenstauchung

Die Stützenstauchung $S_C$ entstammt dem gedrückten Material, den wellenförmigen Stützenwänden in Stablängsrichtung sowie der Krümmung der Längsachse d.h. der Stützendurchbiegung. Unabhängig von der Form des Vorbeulens der Wände, wenn die Plattenbelastung einen bestimmten Wert überschreitet (ähnlich der kritischen Plattenbelas-

Die Stauung des Stützenabschnitts um die Länge a (Bild 1b), beträgt im Falle der axialen Belastung

\[ S = \frac{\delta_{av}}{E} + \frac{\pi^2}{8a^2} \left( f^2 + f_0^2 \right), \]  \hspace{1cm} (1)

worin:
- \( \delta_{av} = N/A \) - Vergleichsspannung,
- A - Querschnittsfläche der Stütze,
- N - Stützendruckkraft,
- \( f_0 \) - Vorbeulamplitude der Wand,
- f - Beulamplitude bei Wandspannungen unter Berücksichtigung von \( f_0 \), \( \delta_{rc} \), \( E_t \), \( E \), \( \delta_{cr} \),
- \( \delta_{rc} \) - Druckschweißspannungen,
- \( \delta_{cr} \) - Druckschweißspannungen,
- \( E_t \) - Elastizitätsmodul und Tangentialmodul,
- k - Beulungsbeiwert einer Einzelplatte unter Berücksichtigung des Schweisseinflusses auf die Randbedingungen der Plattenabstützung laut [5].

Bei der aussermittigen Belastung in einer Fläche

\[ S = \frac{\delta_{av}}{E} + \frac{\pi^2}{16a^2} \left[ \left( f^2 + f_0^2 \right) - 2f_0^2 \right], \]  \hspace{1cm} (2)

worin \( \delta_{o} \) und \( \delta_{p} \) - Spannungen in den Wänden entsprechend auf der konkaven und konvexen Stützenseite.

Die Stützenstauung als Ganzes beträgt

\[ S_c = S + \pi^2 \nu^2/4L^2. \]  \hspace{1cm} (3)
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Der Pfeil der Stützenbeulung $v$ muss unter Berücksichtigung der Imperfektionen für den gesamten Belastungsbereich ermittelt werden.

Sollte man anstatt dieser Beziehungen die Eigenschaften des Ersatzquerschnitts im überkritischen Bereich nutzen, so kann man einfacher Formeln zur Stützenstauchung, d.h. zur Bestimmung ihrer Längssteifigkeiten erhalten. Dazu lässt sich ein neuer Vorschlag der Forschungsgruppe aus Singapur [6] verwenden. Er beruht auf Bestimmung der mittragenden Breiten als Funktionen der steigenden Belastung $\beta$, der Charakteristik der Platte $\beta_y$ und des Imperfektionseinfusses $R_r$. Die mittragende Breite $b_e$ einer axial belasteten Einzelplatte (vgl. Bild 1c) beträgt in gekürzter Form

$$b_e = \varphi_1(C_1, \beta)$$

worin $C_1 = \varphi_2(\beta_y, R_r)$,

$$R_r = \varphi_3(E, E_t, f_y, \sigma_{rc}, \beta_y)$$

$f_y$ - Fließgrenze.

Dabei wurde angenommen, dass sie Verteilung der Schweissspannungen rechtwinklig verläuft; die Zugbereiche mit den Breiten $3_t$ haben Eigenspannungen $f_t$, und die Beulamplitude $f_{01} = 0,001b_i$.

Die von Verfasser [5] durchgeführte Analyse des Einflusses der Imperfektionen auf die Tragfähigkeiten und Steifigkeiten der Stützen mit kastenförmigem Querschnitt erlaubt eine Modifizierung der genannten Beziehung und schafft Möglichkeiten, beliebige Imperfektionen zu berücksichtigen. Die Formeln für die mittragende Breite lassen sich in Form von

$$b_e = \left(\frac{R_r}{\beta^2} \right) \left( C_1 + C_2 \beta + C_3 \beta^2 + C_4 \beta^3 \right),$$

worin:

$C_1 = 0,277(1-1,901C_2 - C_3 - 0,526 C_4)$,

$C_2 = -3 C_4 \beta_y^2 - 2 C_3 \beta_y$,

$C_3 = 0,526/(0,526 - \beta_y) - 1,5 C_4 (0,526 + \beta_y)$,

$C_4 = 2 (0,526 \beta_y - 0,277) / (0,526 - \beta_y)^3$,

$\beta = (\delta_{av} b / \delta_{cr} b_e)^{0,5}$, $\beta_y = (f_y / \delta_{cr})^{0,5}$

Somit ist $\delta_{av} = \delta_{cr} / \beta^2$ $b_e / b$ und nach Substituieren von (5) erhalten wir
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\[ \delta_{av} = \delta_{cr} R_r \left( C_1 + C_2 \beta + C_3 \beta^2 + C_4 \beta^3 \right). \]  (6)

In diesem Zustand beträgt die Wandstauchung

\[ S = \delta_{av} b / E_b = \delta_{cr} \beta^2 / E, \text{ und } \beta = \left( \frac{SE/d_{cr}}{S} \right)^{0.5}, \]  (7)

ihre Grenzlast hingegen, bestimmt laut Kriterium von Horne

d \( \delta / dS = 0, \)
d.h.

\[ d \delta / dS = E R_r \left( C_2 / \beta + 2 C_3 + 3 C_4 \beta \right) = 0, \]  (8)

beträgt

\[ \delta_{ult} = \delta_{cr} R_r \left( C_1 + C_2 \beta_{ult} + C_3 \beta_{ult}^2 + C_4 \beta_{ult}^3 \right), \]  (9)

worin \( \beta_{ult} \) aus der Beziehung (8) ermittelt wird.

Ist die Grenzlast der Platte direkt proportional von \( R_r \) abhängig, dann ist \( R_r \) ein Multiplikator, der die Tragfähigkeit der Platte in Hinblick auf Imperfektionen verringert. Er lässt sich somit aus dem Vergleich der ideallen Platte \( \delta_{ult perf} \) mit der Tragfähigkeit der Platte, belastet mit Imperfektionen \( \delta_{ult imperf} \), ermitteln, d.h.

\[ R_r = \delta_{ult imperf} / \delta_{ult perf} \]  (10)


Die Stauchung des Stützenabschnitts \( a \) beträgt somit

\[ S = \frac{2 \delta_{av} b}{E (b_{eo} + b_{ep})}, \]  (11)

worin \( b_{eo} \) und \( b_{ep} \) sich aus der Formel (5) für \( \beta \), die den Spannungen \( \delta_0 \) und \( \delta_p \) entsprechen, ermitteln lassen.

Ermittelt werden soll noch die Stützenbeulung. Die Beulung \( v \) der aussermittig gedrückten Stütze mit kastenförmigem Querschnitt (Bild 1a) wird aus der Differentialgleichung vierten Grades mit nicht linear veränderlichen Faktoren ermittelt. Ein annähernd genaues Ergebnis lässt sich auch aufgrund der Formel (8)
ermitteln,
in der $k_k = \frac{N}{EI}^{0.5}$, $I$ - Trägheitsmoment, $\varepsilon_z = \varepsilon + \nu_0$.

Im elastisch-plastischen Bereich verringert sich die konstante Steifigkeit der Stütze $EI$ mit der zunehmenden Belastung. Die Stauchung der gedrückten flachen Platte beschreibt die Abhängigkeit $\varepsilon = \delta/E$ oder allgemein die Beziehung $\varepsilon = \delta/E_t$. Die gebaute biegsame Platte belastet mit Imperfektionseinflüssen unterliegt einer nicht linearen Abhängigkeit $\delta - S$. In diesem Zusammenhang wird vorgeschlagen, in den Formeln, die die Stützenbeulung bestimmen, eine Beziehung $\delta - S$ anstatt der gewöhnlich gebrauchten Beziehung $\delta - E$ einzuführen.

Aufgrund des Karman-Engesser-Verfahrens lässt sich der Ersatzelastizitätsmodul $E_k$ für den kastenförmigen Querschnitt im nicht elastischen Beulbereich aus der Formel

$$E_k = EE_t \left( \frac{A_d + A_p}{EA_d + E_t A_p} \right)$$

bestimmen.

$E_t = d\delta/dS$, was bereits in der Formel (8) berechnet wurde.

Daher ist es einfach $E_k$ zu berechnen, und in die Gleichung für die Stützenbeulung (12) $k_k = \left( \frac{N}{E_k I} \right)^{0.5}$ einzusetzen.

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Bild 2. Stauchung der Stützen $S_c$ und der Stützenabschnitte $S$

Schlussfolgerungen

Die Nutzung des Ersatzquerschnitts der Stütze, laut der Theorie der Breite der mittragenden Platten, ermöglicht die Längsteifigkeit der Stützen zu bestimmen, ohne eine eingehende Analyse der in den Platten dieser Stützen auftretenden Spannungen durchführen zu müssen. Dabei lässt sich genau der Einfluss der Schweissimperfektionen im Beiwert $R_f$ berücksichtigen, da dies in der Bestimmung der Tragfähigkeit einer einzelnen Platte enthalten ist. Die Schweissimperfektionen bewirken eine Verringerung der Längsteifigkeit der dünnwandigen Stützen im gesamten Belastungsbereich. Der Einfluss einzelner Imperfektionen auf die Verringerung der Längsteifigkeit verhält sich so wie im Falle einer Verringerung der Plattenlast. Den größten Einfluss auf die Stützenstauchung hat der wellenförmige Verlauf der Wände, der
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umso schneller ansteigt je größer die Beulung und Vorspannung waren.

Literatur


LOAD BEARING CAPACITY AND SAFETY OF THIN - WALLED STEEL BEAMS IN BENDING

1. Introduction.

The relatively wide use of corrugated sheets in the building industry is not always accompanied by a full awareness of their specific properties and in particular, of the behaviour of the structure in the boundary effort conditions.

This paper presents an analysis of load - displacement curves and the limiting states of the load capacity corrugated sheets in bending. The analysis was made on the basis of tests on natural scale models. A mathematical model describing the effort of thin - walled beams in bending is discussed. Both the load bearing capacity and safety of considered type of structures were analysed.

2. Testing corrugated sheets in bending.

In paper [1] results of tests on the random load bearing capacity corrugated sheets 0.75 and 1.00 mm thickness in bending are discussed. The tests were carried out on 10 natural scale models (fig.1). At the load of about 0.3 P ul local buckle appeared in the crest of the corrugated sheets used in the tested models. The destruction of models consisted in the formation of a local plastic hinge in one of the extreme crests followed by successive failures of the other crests of the plate. The phenomenon had an avalanche character and it resulted in the depletion of models load capacity. The local buckling hinges, in the form of a local plastic failure of the compressed wider flange, were visible under the place of load application.

Fig.1 shows diagrams of average displacements y and displacement variation coefficients y as a function of load P for the models built out of corrugated sheets of 0.75 and 1.00 mm thickness. The load - displacement curve diagrams for the tested corrugated sheets indicate that the stage of failure of the thin - walled sections is accompanied by an increase in the displacement variation coefficient. The load - displacement curve can be described by a nonlinear - elastic - brittle model.

3. Load bearing capacity of cold - formed steel beams in bending.

The effort of cold - formed beams in bending within the elastic range under the assumption that the local compressive stress of the crest is smaller than the critical stress $\sigma_1 < \sigma_{cr}$ is described by the equation

$$E J y^{IV} = q(x)$$

After the critical stress of the local buckling of the crest or webs has been exceeded $\sigma_1 > \sigma_{cr}$, the stress distributions in the thin - walled cross - section stops to be linear and becomes nonlinear. According to the Winter's theory, the effective width of the sections compressed flange changes. The moment of inertia of cold - formed steel area, being a function of transverse load changes, too.
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\[ J(a, b, c, h, t) \delta_1^{<} \delta_{cr}^{>} J_{ef}(a, b_{ef}, c_{ef}, h_{ef}, t) \delta_1^{<} \delta_{cr}^{>} = J(q, x) \quad (2) \]

The position of the neutral axis changes as well. In the case of bending, a change in the position of the neutral axis does not result in additional effort of the cross-section in contradistinction to the compression and bending case.

The differential equation that describes the boundary equilibrium curve after local buckling of the crest or webs has the form:

\[ E \int J(q, x) y^{IV} = q(x) \quad (3) \]

It is a differential equation of the fourth order with nonlinearly changing coefficient of flexural rigidity \( E[J(q, x)] \). Equilibrium equation \( (3) \) can be solved by means of trigonometric series \( [4] \) by applying finite sine and cosine Fourier.
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transformations to an appropriate differential operator.

4. Load - displacement curves of corrugated sheets in bending.

The load - displacement curve of a thin - walled beam in bending is nonlinear due to the reduction of rigidity. Strain - gauge measurements [1] reveal that the bearing load capacity of the bended cross - section is reached when the compressive stresses in the flange and the adjacent corners reach the yield point.

Fig. 3.

Fig. 3 shows the load - displacement curves for thin - walled beams and hot - rolled beams in bending. The hot - rolled beams made of elastic - plastic model material are described by load - displacement curves with clearly marked plastic phases of the cross - sections load capacity.

The load - displacement curve of a thin - walled beam in bending is described by a nonlinear - elastic - brittle model [1]. In the load - displacement curve of the thin - walled beam (fig. 3) one can distinguish: linear - elastic phase OA, nonlinear - elastic phase AB (after local buckling of the crests or webs) and a failure phase characterized by a drop in the load capacity. The curve in the tested models changes abruptly. It was accompanied by a reduction in the load capacity and the formation of a peculiar
articulated hinge - a permanent local plastic hinge. There is no plastic phase in the structures behaviour work. The bearing capacity of a bended thin - walled cross - section made of material having \( R_y \) yield point can be determined using the formula

\[
M_{ef} = W_{ef} R_y
\]

where \( W_{ef} \) - effective static moment of cross - section

After the bearing load capacity state was reached, a local plastic hinge was formed in the thin - walled cross - section and the bifurcate phenomenen of failure is accompanied by a drop in the load capacity ["passive" hinge]. The behaviour of the thin - walled beams in bending in the ultimate state has a significant for the estimation of the reliability of these structures.

5. Limiting states of statically indeterminate thin - walled structures.

In statically indeterminate structures with a thin - walled cross - section we deal with a qualitatively different depletion of load capacity than in hot - rolled structures. The development of a first hinge in the form of a local plastic hinge leads to the depletion of the load capacity of this cross - section [classic hinge \( M = 0 \) is created] which increases the internal forces in the remaining critical cross - sections. The elimination of the load capacity of the first critical cross-section [where the first local plastic hinge appeared] may lead to such an increase of the internal forces in the remaining cross - sections that forces will be greater than the bearing capacity of these cross - sections. The failure of such a structure may have a chain - failure character. This case of the load capacity depletion may occur e.g. for the beam of thin - walled cross - section shown in fig.4.

\[ \text{Fig. 4.} \]

The beam is fixed on the two sides and loaded uniformly. The load bearing capacity of any thin - walled cross - section denote as \( M_{ef} \). At the first stage of loading, load capacity depletion will take place in the bearing cross - sections for the load...
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\[ q_1 = 12 M_{ef} 1^{-2} \]  \[ (5) \]

The depletion of the load capacity of the beams bearing cross-section leads to the change in the static scheme. At the second stage of loading, the beam considered as freely supported. The transverse load of the beam is larger than the load capacity of the critical cross-section of placed the midspan

\[ q_2 = 8 M_{ef} 1^{-2} < 12 M_{ef} 1^{-2} \]  \[ (6) \]

In the considered case, there are no reserves of load capacity at the second stage of loading of the beam. A third local plastic hinge in the midspan will appear immediately after the formation of bends in supporting cross sections. In the considered case, the load bearing capacity of statically indeterminate thin-walled structures is limited by the load bearing capacity of the first local plastic hinge.

One can prescribe a series of model of structure reliability [3] for thin-walled beams in bending and a model in the form of a parallel bunch of chains (fig.5) can answer the corrugated sheets in bending.

6. Conclusions.

The mathematical model of thin-walled beams in bending is described by equation [2]. This is an equation of the fourth order with a nonlinearly changing coefficient in the form of the sections flexural rigidity.

The load-displacement curve of thin-walled beams in bending can be describe by a nonlinear-elastic-brittle model. A change curve of thin-walled beams in bending occurs abruptly and is accompanied by both the reduction in the load capacity and formation of a permanent local plastic hinge of \( M = 0 \) load capacity.

The bearing load capacity of statically determined thin-walled structures is limited by the load capacity of the first local plastic hinge.

7. References.


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ALCO-DOME
Design and erection of the steelstructure
by
Wim Huisman
Marcel Vullings

ABSTRACT

Recently a single-layer lattice dome of 52m clear span was erected on the grounds of the Eindhoven University of Technology. This dome has been developed by members of the light weight structures group of the department of Structural Design (BKO) in co-operation with Siemer B.V., Geertruidenberg. Attention will be paid to some special features of the dome, including the geometry, the node (Adjustable Lattice Connector) and the erection method.

Technical data

<table>
<thead>
<tr>
<th>Clear span</th>
<th>52m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>15m</td>
</tr>
<tr>
<td>Diameter sphere</td>
<td>60m</td>
</tr>
</tbody>
</table>
| Design load    | - windspeed: 144 km/h
               | - snowload: 50 kg/m²
               | - safety factor: 1,5 |
| Bars           | 300 steel tubes ø152,4 x 4mm
               | 6 HEA 140 (entrances)
               | 25 different length’s between 4,5 and 7,5m |
| Reducers       | cast steel |
| Nodes          | 30 adjustable, 61 fixed, 33 semi-spherical supports and 6 entrance supports. |
| Foundation     | concrete ringbeam 500 x 800mm |

Introduction

The existing laboratories for building research at the Eindhoven University of Technology do not offer the possibility of large full-scale experiments. A current housing project requires shelter for the next three years for a three story experimental module and some other extensive experiments are awaited. Renting or buying a temporary structure proved to be very expensive, so it was decided to use some results of the research and development by the light-weight structures group of the department of structural design and Siemer B.V.

A single-layer lattice dome with a prestressed fabric skin
inside and a retractable roof structure were proposed and the first one was chosen.

Geometry

The geometry of the dome is based on the conditions given by a new type of low-cost covering for dome structures developed by Siemer B.V. This will probably be tested on the dome in the future and subsequently reported. The structure is a segment of a sphere with a centre angle of 120° and a diameter of 60m. The segment is divided in six mainly equal parts; three of them contain an entrance. The segment borders are meridians and divided in six equal parts. Five sets of steel ring bars are not horizontal (except the first ring around the top) but segmentwise vertically curved, while the sixth ring, the concrete ring beam, is horizontal.

To create an entrance, two bars, emerging from the central support node of three segment parts, are omitted and replaced by vertical HEA 140 posts, which stiffen the by the left-out tubes weakened area around the entrance. These posts connect the dome nodes on top of the door with an inserted piece of foundation, as can be seen in picture 1. A special detail has been developed to prevent stress-concentrations near the post top. This quite unusual entrance solution was chosen not only for aesthetic reasons but also for erecting considerations as will be explained later on.

Nodes

It is a well-known fact that lattice domes can’t be made from indentical bars and identical nodes, except the icosahedron dome, which is only suitable for relatively small spans and hardly can be considered as a spherical structure. The dome at issue requires 22 different nodes, including the support nodes. This number increases rapidly for larger domes with higher 'frequencies'. So it is very attractive to try to find a universal node which fits all over the structure. The first author developed an Adjustable Lattice Connector (ALCO) which indeed fits everywhere and guarantees a centric connection of the bars in all positions. Thirty ALCO's were fabricated by the Central Technical Service of the University and spread over the dome structure to test the suitability. The main body of all the nodes is spherical. In order to minimize the node it is necessary to reduce the diameter of the tubes.
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This reduction is archived by cast steel reducers, welded to the end of the tubes. In the surface of the dome stiffness is ensured by triangulation of the geometry so a large reduction from $\phi 152.4$ to only 50mm is possible. In the given special dome configuration this results in a node diameter of only 175mm. Perpendicular to the surface the reduction was limited up to 100mm in order to obtain sufficient bending stiffness in view of the erection method.

Erection method

The usual erection method for single-layer domes, as for most other structures, starts at the foundation and works gradually to the top. Only "high frequency" domes can be built this way without temporary supports because of the relatively short members. With members up to 7,5m in this case these supports are necessary. People working at increasing heights with heavy parts and tools means a certain safety risk. In order to increase safety in the first place and secondly to avoid temporary supports in the successive mounting steps the procedure was turned upside down. Starting with the top and working to the foundation, by lifting the finished part by means of a central mast and adding members and nodes at ground level, both aims were reached. Double-layer domes, like for example Temcor domes can be built this way quite easily because of their relatively high stiffness perpendicular to the surface of the sphere in the free-edge situation. Naturally single-layer domes are very weak in this respect. That's why the reducers are shaped as mentioned before. To allow mounting of the crown of the dome at first, a two-legged mast was positioned on a ball-bearing in the centre of the dome, so the top node could move between the legs. The mast was stayed in three directions by cables anchored to the foundation. To prevent the dome from meeting the stays during lifting three temporary bucks were places on the inserted entrance foundations. The stags were attached to the top of the bucks, thus giving enough space for the dome to rise, and subsequently anchored to the ringbeam. This was a major design consideration for the choice of the dome geometry and the solution for the entrances. Fig. 1 shows the dome during construction just before the final lift of about 4,5m. At that moment the diameter of the finished part is 46m and the weight about 200 kN. 60 Bars are waiting to connect this part to the concrete ring beam. Fig. 2 shows the suspension of the dome during erection and gives an impression of the nodes.
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Fig. 1: Just before the final lift.

Fig. 2: Suspension during erection.
NUMERICAL SIMULATION OF LIMIT-STATE CONCRETE BEHAVIOUR LOADED IN COMPRESSION

ir. J.P.W. Bongers

ABSTRACT: Limit-state behaviour of concrete is influenced strongly by a multiaxial stress state. Taking this influence into account in finite element analysis is the subject of much current research. In this paper two numerical models are discussed, one describing concrete on a micro-scale and one on a macro-scale.

INTRODUCTION

The subject of this paper is a study of limit-state behaviour of concrete loaded in compression. This means that results of compression tests, both uniaxial and multiaxial are analysed.

Recently, a micromechanical model is developed at the University of Technology in Eindhoven. By means of this model numerical simulations of laboratory tests were carried out by using the computer program UDEC (Universal Distinct Element Code, Itasca Consulting Group [1989]). An important material characteristic of concrete is its heterogeneity: a two-phase structure of aggregates in a hardened cement paste matrix. At this level concrete has been modelled in the micromechanical model. Discrete cracking is modelled in interfaces between aggregates and mortar and in the mortar itself.

The results of the numerical simulations with UDEC proved that the model is able to realistically describe:
- crack patterns
- softening as a result of crack growth
- localization of cracks and stress release elsewhere
- size effect in peak stress and softening

Application of the micromechanical model for the computation of large structural elements like beams, columns, floors etc., is yet not very practical. Using the model on this level results in very large calculation times and makes a great demand on computer capabilities. The object of this study is therefore:
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--- bond interface
--- mortar interface

(a) (b)

Fig. 1. Micromechanical model, (a) basic pattern (b) randomly disturbed pattern

The development of a macro-model which describes properly the limit-state behaviour of concrete under multiaxial stress states.

A macro-model, in this context, is then defined as:

A numerical model which can be used effectively in structural design, being applicable for the computation of large structural elements by means of the Finite Element Method.

For this purpose the computer program DIANA has been chosen.

EXPERIMENTAL RESEARCH

Both uniaxial and triaxial compression tests have been carried out with a triaxial apparatus developed at the University of Technology in Eindhoven. This apparatus consists of three identical loading frames with a capacity of 2000 kN each. In this way concrete cubes can be loaded either in force control or in deformation control. Results of both uniaxial and triaxial experiments, performed by Van Mier [1984] and Vonk [1992], indicate that:

1. Compression in the lateral direction involves a large increase of peak-stress and axial deformation (Fig. 2).
2. Use of a rigid loading platen results in a restraint to lateral deformations due to friction between loading platen and specimen. Due to these frictional stresses peak stress increases and the specimen shows a more ductile behaviour. In order to reduce these frictional stresses, Vonk used teflon layers between loading platen and specimen. With the addition of some grease this resulted in a very low coefficient of friction.
3. Softening of concrete is a structural property, rather than a material property. This is obvious when we observe localization of deformations. The reaction of a specimen to loading conditions is a combined reaction of local and global behaviour, which makes it dependent on the size of the specimen. Global, concrete behaviour can initially be described adequately by the theory of homogeneous continua. At a certain moment this continuum behaviour is disturbed by local behaviour. It is important to determine when this moment occurs.
Fig. 2. Nominal stress-strain curves for different compression tests

Fig. 3. Crack propagation in concrete loaded in compression

In this respect research done by Slate & Olefski [1963] is of great importance. They developed an x-ray technique to detect cracks. This technique enabled them to investigate crack propagation in loaded concrete.
Slate & Olefski divided cracks into three types:
- mortar cracks
- aggregate cracks
- bond cracks

Their research showed that a significant number of cracks already exists prior to loading. Most of these cracks are bond cracks between mortar and coarse aggregates. They are a result of volume changes in the cement paste during hydration, bleeding and drying shrinkage. Up to 30 % of peak stress, the increase in number and length of cracks is negligible. At higher stress levels, new bond cracks are formed and the existing bond cracks start to grow round the aggregates. At 70 to 90 % of peak stress cracks appear through the mortar. The mortar cracks form bridges between adjacent bond cracks. As the stress further increases, more and more cracks coalesce and crack growth becomes unstable. At this stage it is likely that continuum behaviour is becoming disturbed by local behaviour.

Finally this process of crackgrowth results in the formation of localized macro-cracks. At this stage further deformation occurs mainly in these macro-cracks while the stress in the continuum parts of the specimen releases. Analysis of failure patterns of loaded specimen (Fig. 4 and 5) shows that macro-cracks can be divided into two types of cracks:
- shear cracks
- tensile splitting cracks

Failure of a specimen occurs due to the formation of macro-shear cracks. In uniaxial compression this crack pattern is modified by the formation of tensile splitting cracks. The formation of these splitting cracks is however not essential to failure in uniaxial compression.

Fig. 4. (a) Surface crack pattern for a uniaxial compression test (teflon platen) and (b) internal crack pattern (vertical cross section).

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Fig. 5. (a) Surface crack pattern for a triaxial compression test (teflon platen) and (b) internal crack pattern (vertical cross section).

NUMERICAL MODEL

Development of a macro-model as defined in the introduction implies that the heterogeneity of concrete on a micro-scale is implicitly taken into account. Two such possibilities have been investigated:
- Modelling of concrete as a continuum
- Modelling of concrete as a discontinuum

CONTINUUM-MODEL

DISCONTINUUM-MODEL

Material behaviour:
Non-linear elasto-plastic (Mohr-Coulomb plasticity)

Material behaviour:
Continuum: Linear-elastic
Interface: Non-linear elasto-plastic (Mohr-Coulomb plasticity)

Fig. 6. Modelling as a continuum versus modelling as a discontinuum
The differences in modelling are shown in Fig. 6. In the discontinuum model heterogeneity is modelled on a large scale by means of interfaces which represent the large shear cracks formed beyond peak stress in laboratory tests. The idea behind this model is that all crack deformation localizes in these interfaces, both during hardening and during softening. Unless one uses a very fine finite element mesh, which is not desirable in the macro-model as described above, this localization of deformations is not possible in the continuum model. In other words, a homogeneous stress state leads to deformations which are distributed homogeneous over the entire specimen.

From research it is concluded that, on a macro-scale, concrete hardening is described best by the continuum model and that concrete softening is described best by the discontinuum model. This conclusion has resulted in the development of a combined model, characterized by continuous deformations during hardening and localization of deformations, in the interfaces, during softening.

Material behaviour:

Continuum: Non-linear elasto-plastic (Mohr-Coulomb)
Interface: Non-linear elasto-plastic (Mohr-Coulomb)

Fig. 7. Combined model

In the combined model both Mohr-Coulomb interface-plasticity and Mohr-Coulomb continuum-plasticity are applied. In both cases Mohr-Coulomb plasticity is described by three parameters:
- cohesion (c)
- friction angle (\(\phi\))
- dilatancy angle (\(\psi\))

After the failure criteria are met, hardening and softening can be modelled by relating these three model parameters to an additional hardening/softening parameter. All parameters are determined from the simulation of two independent laboratory tests. Chosen is for the numerical simulation of one uniaxial and one triaxial compression test, both with a teflon loading platen.

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Fig. 8. Nominal stress-strain curve for a simulation of a uniaxial compression test

Fig. 9. Nominal stress-strain curve for a simulation of a triaxial compression test
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Fig. 10. Relation between nominal axial strain and nominal lateral strain (simulation uniaxial compression test)

Fig. 11. Relation between nominal axial strain and nominal lateral strain (simulation triaxial compression test)
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The figures on the previous pages show some results of these numerical simulations. In Fig. 8 and Fig. 9 the nominal stress-strain curves in axial direction are plotted. Figure 10 and Fig. 11 show the relation between the nominal axial strain and the nominal lateral strain. From these figures it can be concluded that both hardening and peak stress are described well by the model. The description of the softening behaviour of concrete is however rather poor. The reason for this poor description is that the material parameters of the Mohr-Coulomb plasticity model for the interfaces can not be given the correct values in the present version of DIANA. In case of applying the correct values numerical difficulties arise and the problem does not converge to the correct solution anymore. For this reason the friction and dilatancy angle have to be kept constant with respect to the softening parameter, and equal to each other. This means a simplification of the interface plasticity model, resulting in the rather poor approximation in the post-peak region of laboratory tests.

CONCLUSIONS

- Use of a numerical model including only interface- or continuum plasticity does not result in a good simulation on a macro-scale of both uniaxial and multiaxial experiments.
- Use of a combined model results in a correct description of hardening behaviour and peak stress. Limitations in the computing method (F.E.M.), however, require for the time being certain simplifications, resulting in a rather poor description of post peak behaviour.

RECOMMENDATIONS FOR FURTHER RESEARCH

- Further research on this subject should be concentrated on the combined model. This model physically comprises the right parameters. We've seen, however, that numerical difficulties arise. Further research should therefore firstly be directed towards solution of these numerical difficulties.
- At this stage of research application of the combined model is only possible when the compressive failure mode of the structure is known. This is a severe limitation. Implementation of some kind of detection method of this failure mode in the computing method is therefore desirable.
- Application of a multi-linear or a curvilinear plasticity model, instead of the linear Mohr-Coulomb plasticity model, will probably enhance the approximative capability of the method. Use of such more sophisticated models implies on the other hand the need for more and refined test results.

9 ir. J.P.W. Bongers
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