POSTER SESSION 7

Computer Aided Engineering

Ingénierie assistée par ordinateur

Computerunterstütztes Ingenieurwesen

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Dynamic Analysis of Layered Beams by Exact Finite Element Analysis

Analyse dynamique de poutres à lamelles par la méthode des éléments finis

Dynamische Analyse von geschichteten Stäben anhand der exakten Finite-Elemente-Methode

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1.THEORY

The general dynamic-stiffness matrix and the calculation procedure of exact shape-functions and natural frequencies of elastic layered beams for transverse undamped free vibrations have been presented in the previous papers [1], [2] and [3]. The method is exact for beams with two faces and one core or three faces and two cores if the cross-section of the beam is symmetrical. If there are more faces then this method can be used approximatively [3]. Neglecting damping, rotational inertia and shear deformations of the faces, the differential equation for the lateral vibration is [3]

$$EI_oEI_sv^{(6)} - (EIk + NEI_s)v^{(4)} + (Nk + cEI_s)v^{(2)} - ckv$$
$$+\mu EI_s\ddot{v}^{(2)} - \mu k\ddot{v} = EI_sp^{(2)} - kp, \tag{1}$$

where EI_o is the sum of bending stiffnesses of the faces, EI_s is the Steiner-term, $EI = EI_o + EI_s$, k is the shear stiffness of the beam, N is the axial force, c is the modulus of Winkler-type foundation, μ is the mass/unit lenght of the beam, $p = p(x,t)$ is the lateral loading intensity of the beam, $v = v(x,t)$ is the lateral displacement of the beam, $v' = \partial v/\partial x$, $\dot{v} = \partial v/\partial t$, where x is the beam axis and t is time.

The equation is solved by superposing the exact shape-functions $\phi_i(x)$ (see Fig. 1.) of undamped free vibrations (the functions $\phi_i(x)$ satisfied the governing differential equation and the corresponding boundary conditions)

$$v(x,t) = \sum_{i=1}^{\infty} \phi_i(x)Y_i(t). \tag{2}$$

The homogeneous boundary conditions can be presented in the form [3]

$$/_0^L(Q\phi - M\varphi - M_o\gamma) = 0, \tag{3}$$

where Q is the total shear force of the beam, M is the total bending moment, M_o is the sum of the bending moments of faces, γ is the rotation angle due to slip of the core and $\varphi = \phi' - \gamma$. The equations for these variables can be found in the reference [3]. Substituting the trial (2) into the equation (1), multiplying the equation with a test function ψ_j and integrating over the lenght L of the beam leads us to the equations

$$\sum_{i=1}^{\infty} M_{ij}[\ddot{Y}_i(t) + \omega_i^2 Y(t)] = P_j(t), \tag{4}$$

$$M_{ij} = \int_0^L (\mu EI_s \phi_i^{(2)} - \mu k \phi_i)\psi_j dx, \tag{5}$$

$$P_j(t) = \int_0^L (-kp + EI_s p^{(2)})\psi_j dx. \tag{6}$$

The functions $\ddot{Y}_i(t) + \omega_i^2 Y_i(t)$ can be solved from the equation (4) as soon as the test functions ψ_j are chosen and then finally the functions $Y_i(t)$ can be calculated from Duhamel's integral. The functions ϕ_j are used as the test functions and then all the integrations in the equations above can be performed analytically.

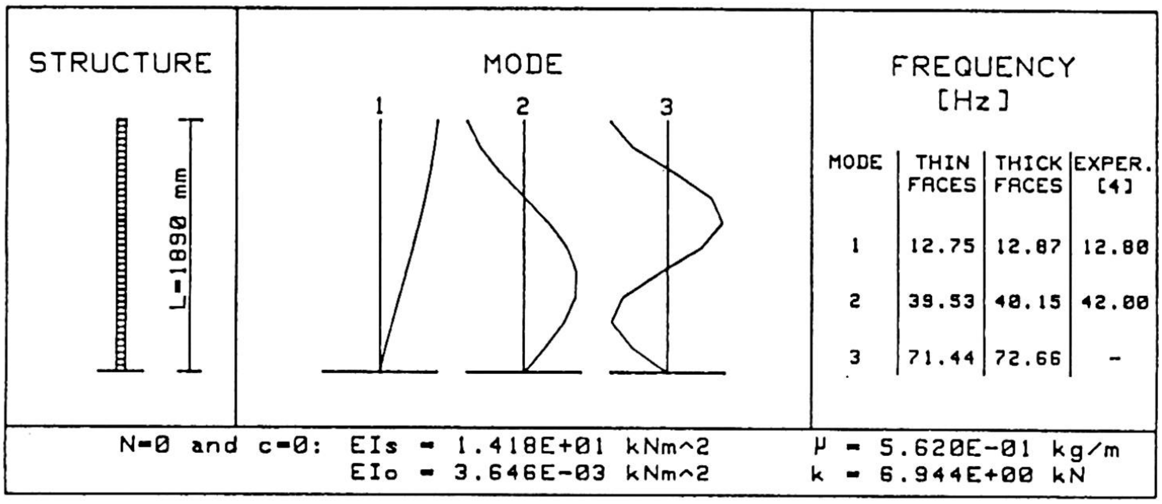

Fig. 1. Exact shape functions for a shear wall model

2.EXAMPLES

The numerical examples concerning the vibrations of wooden leight weight units and shear walls are presented in the Poster. Some test results have been calculated by using the approximative finite elements (ABAQUS). Multi-point constrains are used in these calculations, so that the results can be compared with the results calculated by using the theory of layered beams. The more accurate modelling of the structures considered is also performed so that the error due to the basic assumptions of the theory of layered beams can be seen.

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Monitoring and Prediction of Strain in a Long-Span Segmental Bridge

Contrôle et prédiction des déformations dans un pont à voussoirs de grande portée

Kontrolle und Voraussage von Spannungen in einer Segmentbrücke grosser Spannweite

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1. INTRODUCTION

In recent years, segmental bridge construction has gained popularity in the UK as a method of construction for medium to long-span bridges. It has contributed to both the aesthetics of bridge design and economy of construction [1].

In segmental construction the bridge deck is divided longitudinally into short segments which are either precast or cast-in-situ and then prestressed together in stages to form the bridge superstructure. Because of the nature of this method of construction, creep and shrinkage of the concrete and relaxation of the prestressing steel have a significant influence on the vertical deflection. Loss of prestress and the development of stress in the bridge deck both during construction and after completion of the bridge are also affected. An objective study of the actual behaviour of this form of construction, accompanied by a rigorous analytical approach is essential for successful design of segmental bridges. To this end a comprehensive instrumentation programme has been carried out to monitor the time-dependent behaviour of the River Torridge Bridge in North Devon.

2. RIVER TORRIDGE BRIDGE

The River Torridge Bridge is a glued segmental bridge crossing the Torridge estuary at Bideford in North Devon. It carries two lanes of traffic over the tidal estuary at a height of 29 m above mean high water level. It consists of eight continuous spans up to to 90.00 m in length with a total length of 645 m.

The bridge deck is a single-cell non-prismatic box girder with a depth varying from 6.1 m at the supports to 3.1 m at midspan. It is formed from 251 precast concrete segments each approximately 2.5 m length. Segments were match-cast using the short-line method and erected by means of a launching girder using the balanced cantilever technique. Segment casting commenced in June 1985 and bridge construction was complete in May 1987. A detailed description of River Torridge Bridge, its design and construction, is presented in an earlier report [2].

3. INSTRUMENTATION

Four segments were instrumented within Cantilever 4 East of Span 5, which is one of the largest 90 m spans. Segments were designated P4/1E, P4/6E, P4/10E and P4/16E corresponding to their relative positions east of pier 4. Concrete strains were measured using vibrating wire gauges of 140 mm gauge length embedded within the concrete during construction. Both single gauge elements and delta-rosette arrangements were placed on the median line of the segment cross-sections in the plane of each wall . Single gauge elements provided longitudinal strains; the rosette gauges yielded information on both longitudinal and in-plane shear strains around the cross-section. Segments P4/10E and P4/16E contained single gauges only. A few additional gauges were also placed across the thickness of wall elements of segments P4/10E and P4/16E for assessment of shrinkage strains.

Temperature measurements were made using type K thermocouples, manufactured by fusing 17 swg copper and constanton wires together to form a junction. Segment P4/10E was instrumented fully for the measurement of temperature profiles across the concrete walls. Plastic rods were fixed across the thickness of each wall element with a number of single thermocouples attached at predetermined spacings. Distribution of temperature along the bridge was monitored by placing a single thermocouple at each gauge position within each of the instrumented segments.

4. MEASURED RESULTS

Field measurements of concrete strain and temperature from bridge segments were recorded initially at three hourly intervals, for the first day after stripping of formwork, and then daily for the first week. The interval between readings thereafter was gradually increased up to approximately one month. At each major event during construction such as segment placing, temporary prestress, final prestress and launching girder movements, strain and temperature readings were also recorded.

Despite the large number of gauge elements employed, all readings were taken manually without the use of data logging equipment. An estimated 8000 readings were recorded during the 15 months period from casting up to completion of the bridge. Some typical results from field and laboratory measurements were presented in earlier papers [3, 4].

5. MATERIAL PROPERTIES

Physical properties of concrete used in segments P4/1E and P4/10E were measured using a large number of prism and cube specimens. Short term physical properties measured included compressive strength, modulus of elasticity, Poisson's ratio and coefficient of thermal expansion, at four different ages of approximately 28, 90, 180 and 240 days after casting. Creep and shrinkage tests were conducted on specimens which were either fully sealed against moisture penetration, partially sealed (to take account of the surface area/volume ratio of the segment represented) or left unsealed. Shrinkage measurements were carried out on specimens stored indoors under a controlled environment and outdoors protected from direct rain or sunlight. Creep tests were conducted indoors at a constant temperature of $23^{o}C$ and 85% relative humidity, representing average climatic conditions in the British Isles. Basic and total creep tests were initiated at ages of approximately 28, 90, 180, and 240 days from casting, under a constant compressive stress of $14.4N/mm^2$ representing approximately 25% of the 28 day characteristic cube strength.

6. COMPUTER-AIDED ANALYSIS

A computer program developed to perform non-linear time-dependent behaviour analysis for long-span segmental concrete bridges has been developed to calculate strains and deflections. It is based on a step-by-step procedure which accounts for most of the important parameters that influence the construction of segmental bridges. A procedure employing the method of superposition of specific creep curves has been used for estimating the creep of concrete subjected to a varying state of stress. Interaction between creep and shrinkage of concrete and induced prestress losses is considered. The analysis uses material properties and actual construction schedules to simulate time-dependent bridge behaviour. Material properties are used from results of laboratory and field tests or simulated from various design code recommendations. Also effects of actual loads applied on the River Torridge Bridge during each stage of construction are investigated.

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Computerprogramm BESTA

Programme de calcul BESTA

Computer Program BESTA

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1. BESCHRIEB / DESCRIPTION

Das Computerprogramm BESTA wurde vom Verfasser für die BEmessung von Stabtragwerken aus STAhlbeton und Spannbeton mit einfach-symmetrischen Querschnitten unter kombinierten Beanspruchungen aus Biegung, Normalkraft, Querkraft und Torsion entwickelt. Es berechnet die erforderliche Schub- und Längsbewehrung und führt die notwendigen Spannungs- und Bruchsicherheitsnachweise durch.

Le programme de calcul par ordinateur BESTA développé par l'auteur permet de dimensionner des poutres de section symétrique en béton armé et précontraint sous efforts combinés (flexion, efforts axiaux et tranchants, torsion). Il détermine l'armature transversale (étriers) et l'armature longitudinale. Les résultats englobent les contraintes à l'état de service ainsi que la résistance à la rupture.

The computer programm BESTA was developed by the author for the design of reinforced and prestressed concrete beams of symmetrical cross-section under the combined action of bending, compression or tension, shear and torsion. The shear reinforcement and the longitudinal reinforcement as well as the stresses under working loads and the ultimate strength are determined.

2. ANWENDUNG

Das Programm BESTA ist nicht nuf auf die heute gültigen und in Vorbereitung stehenden SIA-Normen, sondern auch auf ausländische Vorschriften anwendbar. Für SIA-, DIN- und OENormen sind sämtliche normspezifischen Parameter programmintern gespeichert. Weitere Vorzüge dieses Computerprogrammes sind:

- Bemessung und Nachweise für Stabtragwerke mit konstanter oder variabler Trägerhöhe unter kombinierten Beanspruchungen aus Biegung, Normalkraft, Querkraft und Torsion.

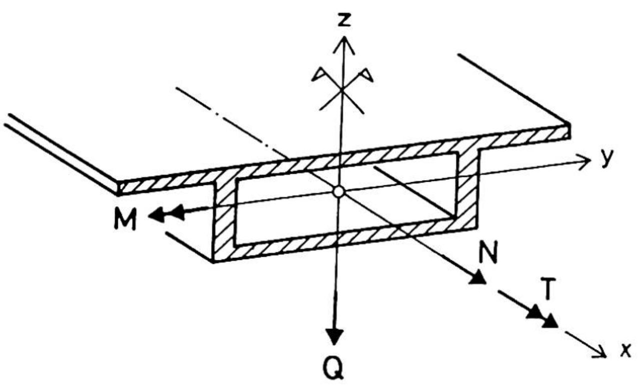

Fig. 1 Kombinierte Beanspruchung

- Bemessung und Nachweise für Einzelquerschnitte und für ganze Stabreihen mit bis zu 150 Querschnitten pro Rechengang.
- Maschinelle Uebernahme der Schnittkräfte von Resultatausgabe-Files der Programme FLASH und STATIK oder direkte Eingabe.
- Automatische Bildung sämtlicher für die Bemessung erforderlicher Schnittkraft-Kombinationen (insgesamt 24 Stück pro Querschnitt, d.h. je 12 pro Querschnitt für den Gebrauchs- und für den rechnerischen Bruchzustand).
- Bemessung und Nachweise für die massgebenden Schnittkraft-Kombinationen.
- Wählbare Resultatausgabe und direkte Verwendung als Beilage zu den statischen Berechnungen (Format A4).
- Resultatausgabe in deutscher, englischer oder französischer Sprache.
- Progammiersprache FORTRAN 77.

3. EINSATZ

3.1 Hardware

Einsatz auf Computeranlagen und Personalcomputern mit folgenden minimalen Voraussetzungen:

- Betriebssystem: Diverse
- Speichergrösse: 100 Kbytes
- Diskmenge: 1 Mbyte

3.2 Software

Einsatz als Einzelprogramm oder als Nachlaufprogramm zu verschiedenen Programmen:

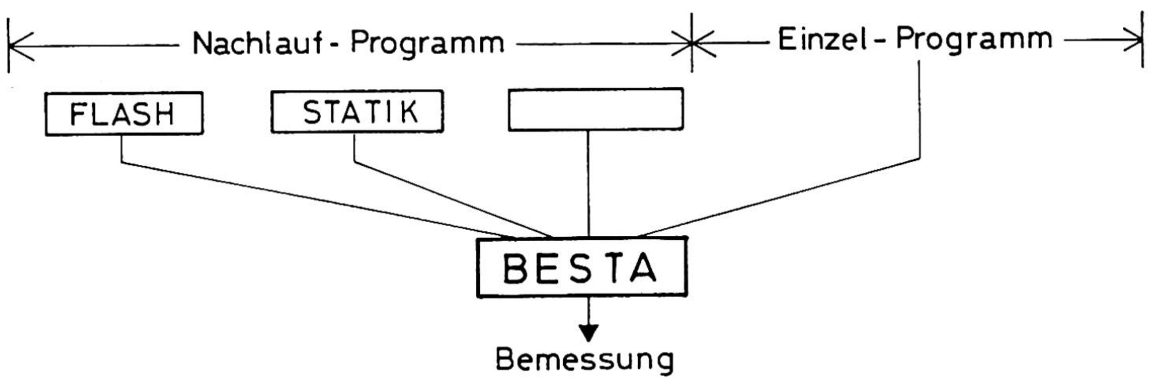

Fig. 2 Einsatz von BESTA

Beim Einsatz von BESTA als Nachlaufprogramm werden die Schnittkräfte nicht eingegeben, sondern von einem anderen Computerprogramm übernommen. Heute bestehen Uebernahmeroutinen für die Programme STATIK und FLASH, auf Wunsch können Uebernahmeroutinen auch für andere Programme geschrieben werden.

Computer-Aided Visualization of Bridges

Visualisation des ponts assistée par ordinateur

Rechnerunterstützte graphische Darstellung von Brücken

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1. INTRODUCTION

When a new bridge is to be constructed, it will always have some effects on the environment and the landscape. Whether these effects are positive or negative, depends mainly on the designer, who usually proposes a few alternative plans for criticism. Besides the the appearence of the bridge itself should be aesthetically well designed, it should also be in harmony with the environment. In order to make possible to judge whether a proposed bridge is suitable for a particular building site, three-dimensional visualization of the plans should be used, at least, in all more important projects. The applicability of CAD programs for visualizing bridges was the aim of this study. The main interest was attached to interactive 3D modelling programs. Using modelling programs it is easy to produce perspective drawings from the object from any desired viewing point to any direction.

2. VISUALIZATION WITH THE AID OF 3D MODELLING PROGRAM

The plans of Luukkaansalmi bridge were used as an application for visualizing. This bridge will be situated near the city of Lappeenranta in southeastern Finland and it will replace the ferry between an island and the mainland. Several alternative bridge designs including surroundings were modelled using CATIA program [1] installed in the IBM 4341 computer of Helsinki University of Technology.
The geometry of the bridge, road and the environment was modelled using solid and surface elements. The most laborious part in constructing the model was the topography of the ground (Fig.1). The background of the landscape was modelled using more simple plane elements (Fig.2).

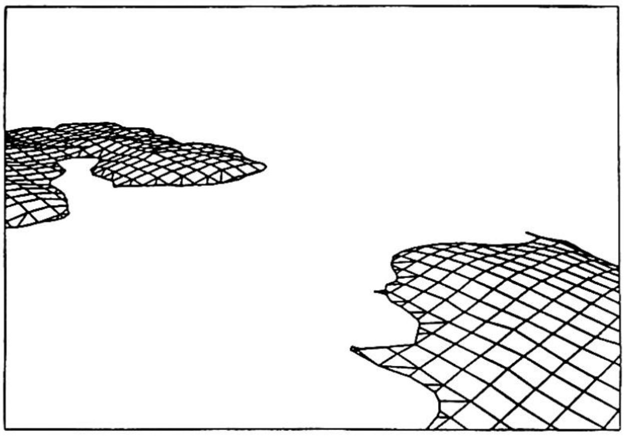

Fig. 1 Topography of the two capes.

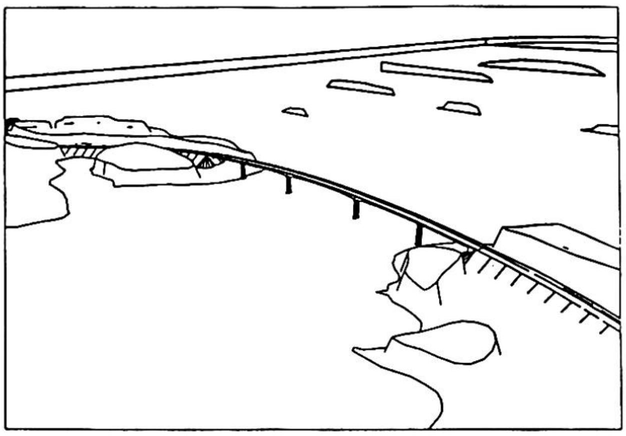

Fig.2 Over-all wiew of the designed bridge.

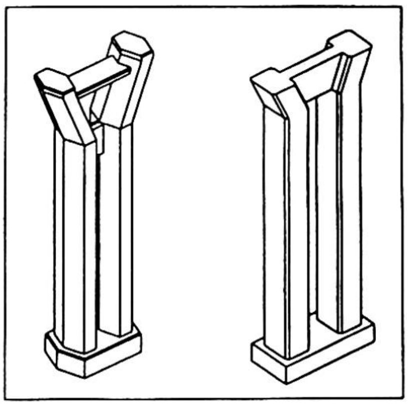

Fig. 3 Two types of piers.

In designing details, like the shape of the piers, the use of 3D program is very effective because of the fast response time (Fig.3). It was found that very realistic images can be achieved with computer graphics, however, the hidden line removal can take several minutes in large models. The main advantage of the 3D modelling compared to photo- or videomontages is that perspective drawings can easily be produced from any desired viewing point. Images can be completed by adding some details like trees, cars and ships etc. (Fig.4). Different colours, shading and shining effects of the surfaces can also be used to improve the visual realism.

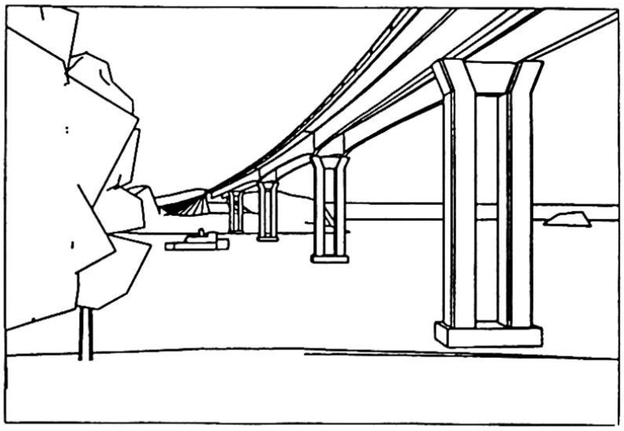

Fig. 4 Perspective view of a bridge.

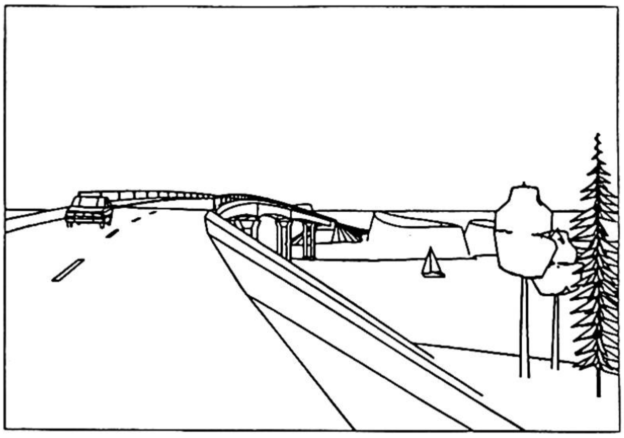

Fig. 5 Perspective when approaching the bridge.

Perspective views of the bridge, produced from the model can be used as a part in other visualizing methods like photo- and videomontage and animation to show the user's perspective when driving over (or under) the bridge (Fig.5). Examples of different visualization techniques were saved on a videotape.
From the experiences of this study, it can be concluded that 3D modelling program is applicable for visualizing the bridge and its environment. Perspective views, obtained from the model, form a good basis for judging the aesthetics of a bridge design (Fig.6).

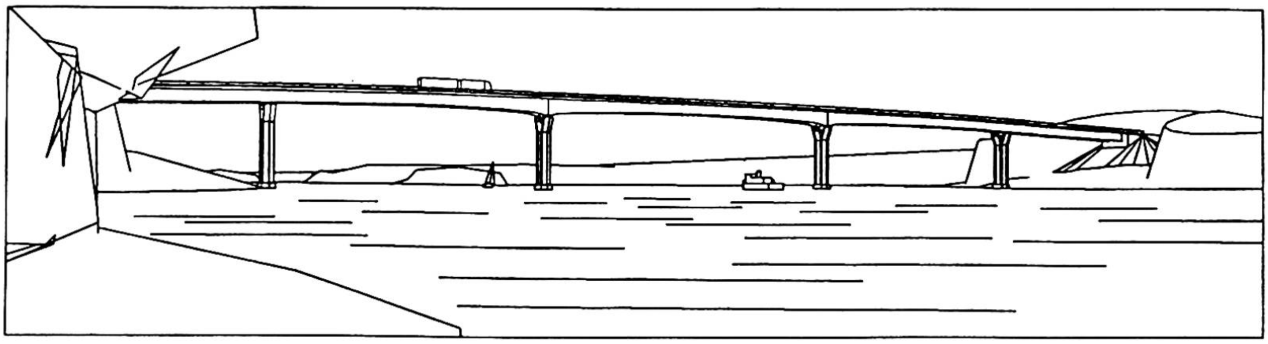

Fig. 6 Side-view of the bridge to be constructed.

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Computerunterstütztes Entwerfen von freien Schalenformen

Computer Aided Design of Free-formed Shells

Conception assistée par ordinateur de voûtes librement formées

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1. FORMEN FÜR STAHLBETONSCHALEN

Dünnwandige Schalen sind Strukturen, die vom Tragverhalten und vom Materialeinsatz her anderen Tragwerksformen überlegen sind. Der Grund für das günstige Tragverhalten der Schale ist, daß aufgebrachte Lasten vorwiegend über Membrankräfte und nur zu einem geringen Teil über Biegemomente abgetragen werden. Durch die Wahl einer geeigneten Schalenform können die unter Belastung auftretenden Biegemomente minimiert werden.

Die Druckfestigkeit des Werkstoffes Stahlbeton beträgt ungefähr das zehn- bis fünfzehnfache der Zugfestigkeit. Dieser Tatsache ist durch die Bestimmung einer Schalenform mit überwiegendem Druckspannungszustand Rechnung zu tragen.

Experimentelle Methoden zum Auffinden freier Formen für Stahlbetonschalen sind im Katalog zur Ausstellung "Heinz Isler Schalen" (Herausgeber: Ekkehard Ramm und Eberhard Schunck, Karl Krämer Verlag, 1986) beschrieben. Eine in einem Rechteckrahmen eingespannte Gummimembran verformt sich unter Innendruck zu einer Buckelschale. Eine unter Eigengewicht durchhängende Membran ergibt eine Form, die im umgedrehten Zustand unter Eigengewichtsbelastung nur Druckspannungen aufweist. Diese Formfindungsmethoden können auch mit Hilfe des Computers nachvollzogen werden.

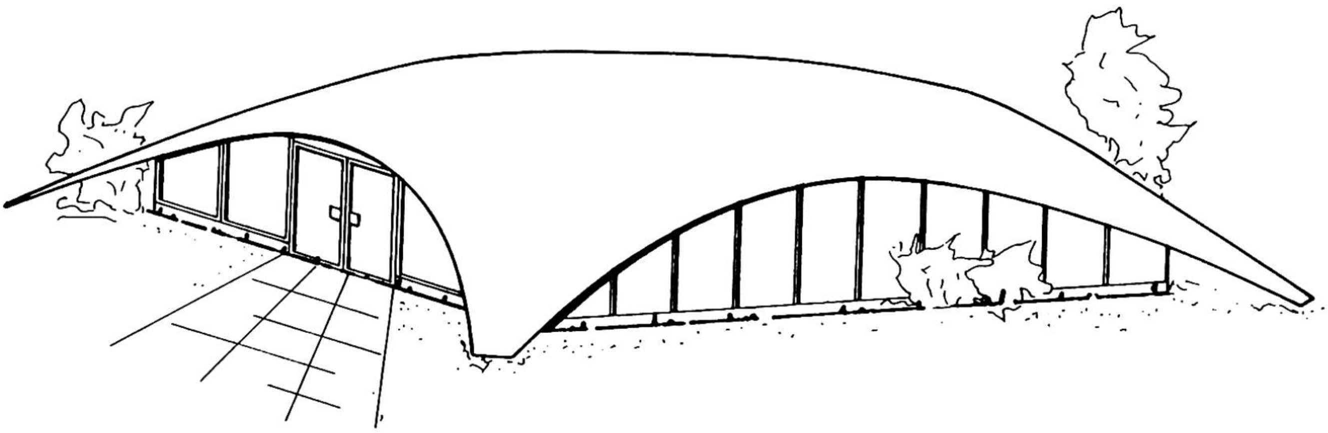

Fig.1 Freie Schalenform für Ausstellungshalle (27.5 m x 27.5 m)

2. FORMFINDUNG MIT FINITEN ELEMENTEN

Finite Elemente Berechnungen an ebenen Membranen über beliebigen Grundrissen liefern unter Belastungen durch Eigengewicht, Innendruck, Einzellasten oder Kombinationen daraus eine unendliche Anzahl möglicher Schalenformen. Die Form der im Bild 1 dargestellten Ausstellungshalle wurde zum Beispiel durch eine nichtlineare Finite Elemente Berechnung gefunden. Bild 2 zeigt den Formfindungsprozeß, der von einer ebenen Membran in einer inkrementellen Berechnung mit einer Druckbelastung normal zur Ausgangsfläche zur gewünschten Form führt. Das statische Verhalten der Schale kann unmittelbar nach dem Formprozeß kontrolliert werden, da die Schalengeometrie bereits in digitaler Form vorliegt. Die graphische Kontrolle der Spannungen und Verformungen der Schale ermöglicht ein interaktives Vorgehen, bei dem die Schalenform ständig verbessert wird. Isometrische Darstellungen der Form aus verschiedenen Blickrichtungen können Architekten und Bauherrn als Entscheidungshilfen beim Auswählen der Ausführungsvariante dienen.

An der Gesamthochschule Kassel wird zur Zeit ein Modell der Schale im Maßstab 1:5 gebaut. Die Schale wird ohne Schalung nach einem neuen Bauverfahren hergestellt. Bilder vom Bau der Modellschale und Vergleiche zwischen experimentellem und berechnetem Tragverhalten sind im Poster enthalten.

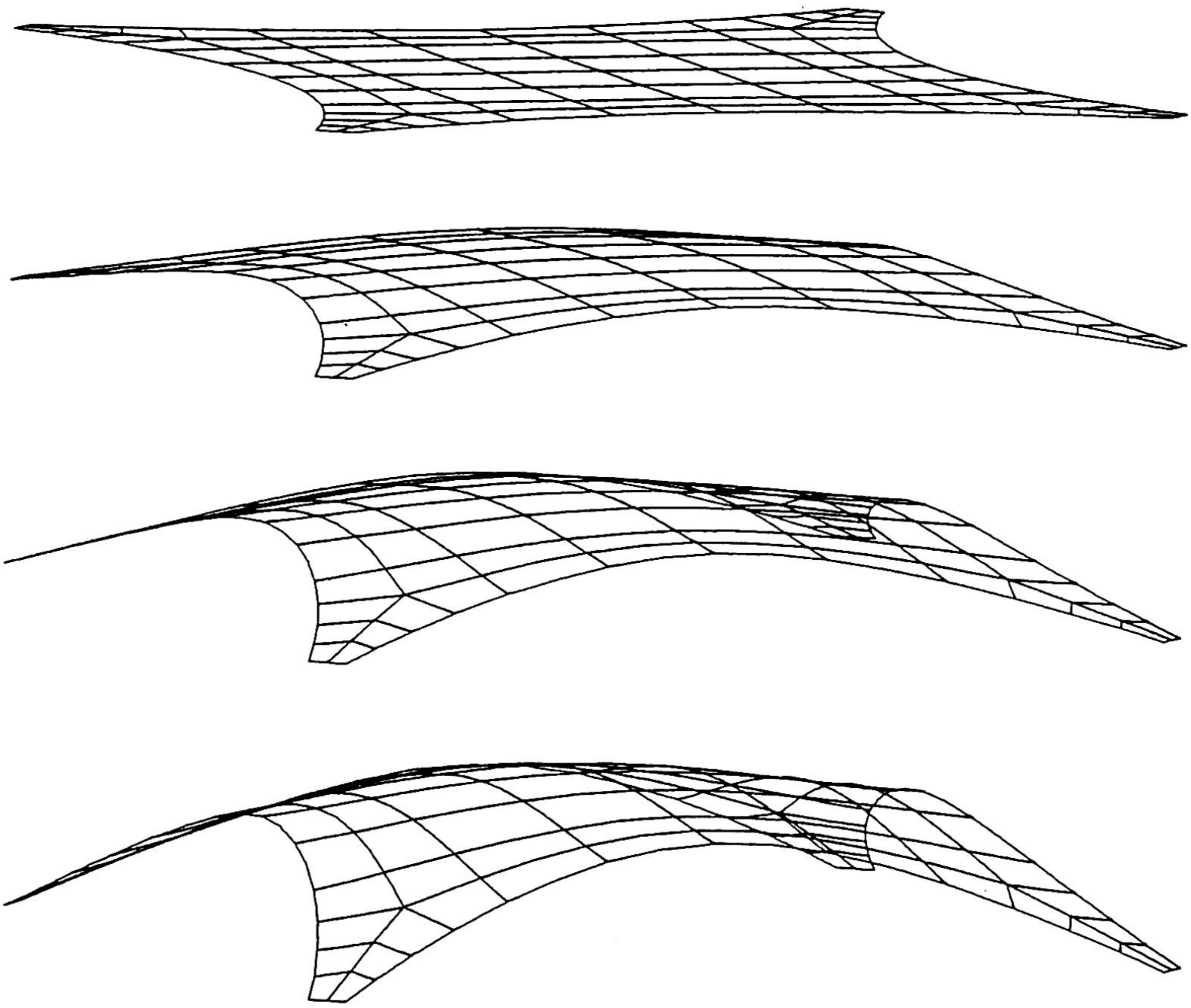

Fig. 2 Formfindung der Ausstellungshalle mit Finiten Elementen

Optimization Procedure for Horizontal Tank Prestress

Optimisation de la précontrainte horizontale des réservoirs

Optimierungsverfahren für die horizontale Vorspannung von zylindrischen Behältern

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INTRODUCTION

From the beginning of the use of cylindrical liquid retaining structures literature has made reference to discussions about the detailing of the wall-to-base connection: shall it be a sliding, a hinged or a monolithic one [1]. The major drawback of the monolithic alternative would be the high fixed-end forces under either hydrostatic load (non-prestressed tank) or under prestress (empty prestressed tank). Reduction of the troublesome fixed-end forces could be reached by adoption of ring force balancing prestress (RBP) instead of load balancing prestress (LBP). For the determination of an RBP several procedures have been proposed [2,4,5,6]. Most of these procedures focus on minimizing the fixed-end forces. Not less important, however, are the ring forces in the bottom meters of the wall, which might be tensile ones in the operational stage (fig. 1). These forces may cause vertical through-cracks, resulting in leakage and loss of durability [5,disc.]. Several authors have also pointed to an often disregarded increase of the fixed-end forces due to vertical cracking of the wall [3]. Above summarized problems compel to searching for a prestress function $F_p(x)$, which ensures a. compressive membrane forces and b. fixed-end forces as low as possible under relevant load combinations.

RING FORCE BALANCING PRESTRESS

The procedure for the determination of a RBP $F_p(x)$ starts with a prestress function with a shape identical to the course of the tensile ring forces under a fictitious hydrostatic head h_1^*. In formular form [2] (see also fig. 2):

$$F_p(x) = N_{\theta\theta,1} = (h_1^*-x).\gamma_1.R-\frac{E_c.h_w}{R}.\frac{e^{-\lambda x}}{2K\lambda^3}\{(\lambda.M_{xx}^*(o)+N_{xz}^*(o))\cos\lambda x - \beta.\lambda.M_{xx}^*\sin\lambda x\} \qquad (1)$$

in which $M_{xx}^*(o)$ and $N_{xz}^*(o)$ are fixed-end forces under the fictitious hydrostatic pressure, E_c is Young's modulus of concrete, K and λ cylinder coefficients. The factor β describes the shape of the prestress function $F_p(x)$ in the bottom part of the wall (fig. 2a). The fixed-end shear forces are linear functions of β (fig. 2b). By varying β the shape of the prestress function can be so determined that the absolute values of the positive and negative shear force under different load combinations are equal and thus minimal (intersection point β_o in fig. 2b). For the value of β_o the shear resistance of the wall at the base is checked. If no sufficient shear resistance can be realized with an initially adopted wall thickness h_w, the optimization procedure is started anew with a 50 mm thicker wall. Once the shear resistance criterion is met, it is checked whether the membrane ring forces are compressive ones in the operational stage. In case tensile ring forces are found, the value of β is adjusted so that tangential compression is ensured over the full height of the wall. The procedure finishes when tangential compression is ensured and the shear resistance criterion is fulfilled under all relevant load combinations. For the thus obtained prestress configuration the costs of the tank are determined. For a given diameter/height ratio of a tank the effects of variations in input parameters (e.g. subsoil conditions, pile characteristics, bottom slab thickness, bottom prestress, vertical wall prestress) on the structural behaviour and the costs can easily be analysed with the procedure just outlined.

DISCUSSION

The analytical optimization procedure, programmed and implemented on a personal computer, is a powerful design tool, which facilitates the designer's choice for the most cost effective alternative. Most important is that the established alternatives meet the stringent requirements for membrane stresses and fixed-end forces, thus ensuring serviceability and high durability.
Limitations: linear elastic behaviour, constant wall thickness and $H > 3\lambda^{-1}$.
It is remarked that thermal loads (temperature gradients) should preferably be carried in a partially prestressed mode [3].

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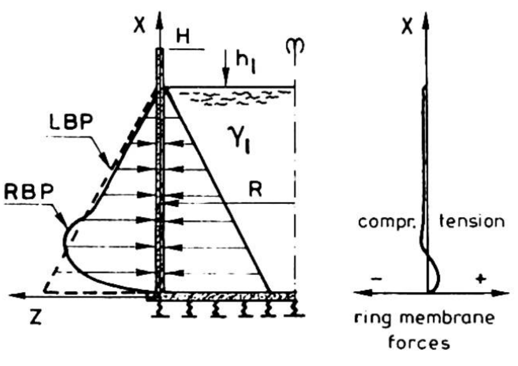

Fig 1 Load Balancing vs. Ringforce Balancing Prestress (L.B.P. vs. R.B.P.)

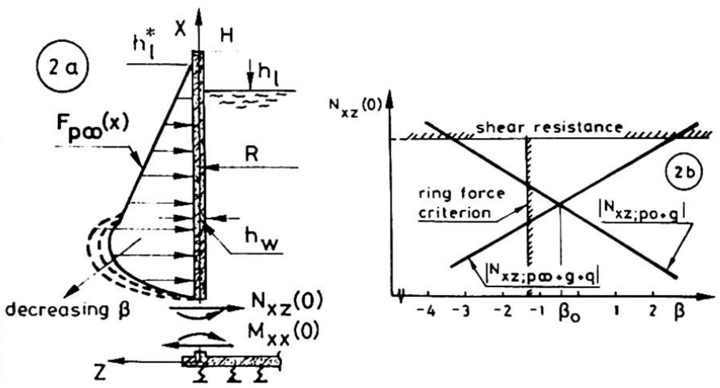

Fig 2 Schematic presentation of design procedure for determination of R.B.P. (in 2b; refrained from load factors: g = dead weight; p_o and p_∞: prestress at t=o and t=∞; q=hydr. pressure)

Education in Structural Engineering Using Small Computers

Enseignement du génie civil à l'aide de micro-ordinateurs

Lehre im konstruktiven Ingenieurbau mit Hilfe von Kleincomputern

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Is it really suitable to teach structural and computer aided engineering using small, personal computers?

At the School of Civil Engineering, Chalmers University of Technology, we have built a teaching environment based on 16-bit personal computers as a platform for computer training of our students. We have invested in two "computer laboratories", each containing 20 personal computers. From each computer there is printing, plotting and digitizing possibilities.

Fig. 1 View from one of the "computer laboratories"

In a relatively short time several applications have been developed and the laboratories are used quite extensively in several subjects. Typical applications are drafting, design exercises and simulations. The software AutoCAD is used both as a drafting tool and as a platform for the development of structural applications. Some of these developments have been carried out as diploma works. Some examples are:

- Preliminary design of a prestressed concrete beam. AutoCAD is utilized for the interaction with the user. At the end of a session the outlines of the beam is written into the AutoCAD database. Further, an input file to a special FEM-program is automatically generated.

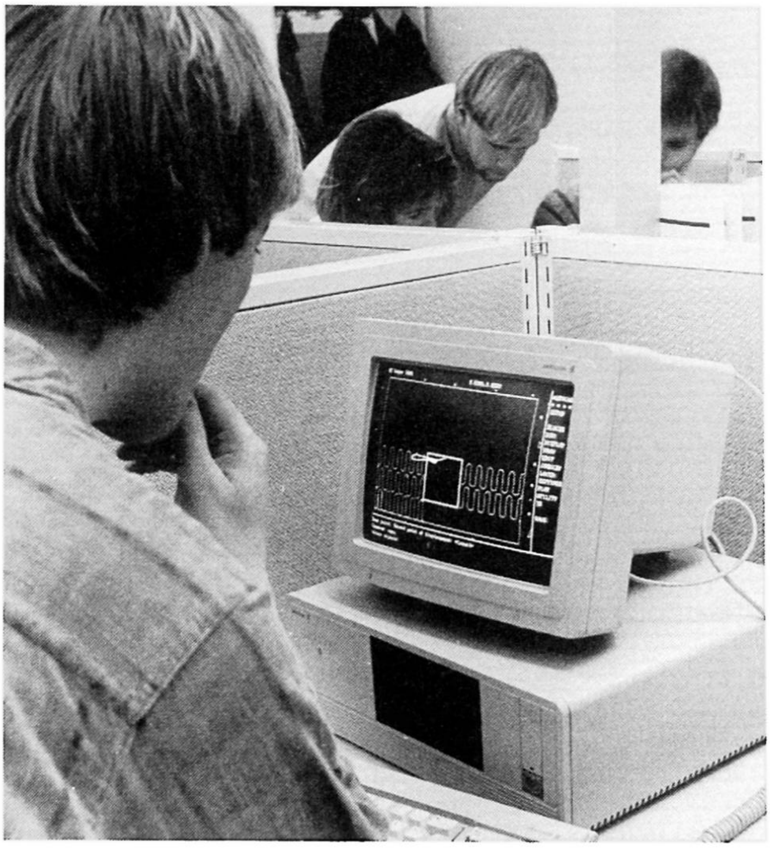

Fig. 2 Working with AutoCAD

- Calculation by yield-line theory of the load carrying capacity of reinforced concrete slabs. In this case the user draws the slab and defines the boundary conditions and the yield line geometry within AutoCAD. As result the load carrying capacity is presented both as a graph and as a numeric value. It is very easy to change any part of the input data and to perform a recalculation.

Special graphics libraries have been developed to manipulate the screen from FORTRAN programs. This feature has been used to write interactive, graphics-oriented structural applications. Some examples are:

- Calculation of the load-carrying capacity in the post-buckling range of thin-walled trapezoidal steel sheets in bending.

- A training program used in a basic course in Structural Mechanics. The program enables the study of moment and shear force distributions in a three-span continuous beam. It is possible to interactively move the supports and to change the boundary conditions as well as the load acting on the beam.

In one course in Structural Mechanics the students get familiar with the principles of a FEM-program. In this exercise, a library with several subroutines has been prepared by the teacher. The students write a subroutine of their own and then assemble the different routines together at linking time. They will end up with a simple, hopefully working, FEM-program.

Today there are also several commercial and general FEM-packages available for personal computers. It is our intention to, within half a year, install such a package and to have it available to our teachers and students.

In a future step our personal computers will be connected in a local area network and some 32-bit engineering workstations will be added. These workstations will be used to analyse larger problems.

We believe that the use of personal computers and highly interactive software is a pedagogically strong tool, when teaching students the fundamentals of structural engineering. However, for design exercises involving the analysis of "real" structures the power of bigger computers will often be needed.

Modelling of Reinforced Concrete Beam with Shear Reinforcement

Etude par modèle d'une poutre en béton armé

Modell eines Stahlbetonbalkens mit Schubbewehrung

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1. INTRODUCTION

There have been many models on shear resistance mechanism of reinforced concrete beam with shear reinforcement. The model commonly accepted is a sort of truss model which is a combination of "truss with 45° strut" and so-called "shear force carried by concrete." However, the previous experimental observation disclosed that inclination of compressive force in concrete between shear cracks was less than 45° and not constant[1], and that stress in shear reinforcement dose not follow the truss model above under repeated loading[2]. Many analytical approaches have been done on shear resistance mechanism, but no satisfactory results have been obtained except on ultimate strength. The reason for this is believed that prediction of shear crack propagation, modelling of forces transferred at shear crack, nor modelling of bond characteristics is not accurate enough in the analyses.

In this study two reinforced concrete beam with vertical stirrup were tested under repeated loading. The main parameter was bond characteristics of stirrup. Detailed measurements were done on shear crack displacements, strain in concrete between cracks, and strains in stirrup and tensile reinforcement. To estimate stresses transferred at crack, the recent study[3] was applied.

2. TEST RESULTS

Test results are summerized as follows. (1)Tensile stresses induced by shear cracking in reinforcement crossed by shear crack increase more or less linearly as external shear force increases. (2)Shear and compressive normal stresses transferred at shear crack increase approximately in a linear manner with external shear force until reinforcement crossed by the same shear crack starts yielding (see Fig.1). (3)Vertical components of forces transferred at shear crack and forces in stirrup increase also linearly as external shear force increases, but the remaining vertical component decreases. (4)Direction of compressive force in concrete between shear cracks more or less coincides with that of force transferred at the shear cracks. (5)Bond characteristics of shear reinforcement indicate negligible effects on shear resistance mechanism.

3. NEW TRUSS MODEL

It can be said that the test results are expressed by a new truss model as follows. (1)Inclination of compressive diagonal strut is less than 45° and can be assumed to be constant. (2)Shear force carried by other than truss (or concrete) decreases as external shear force increases. This truss model is given by Eq.(1) and illustrated in Fig.2.

$$V = V_c + V_s = \nu V_{co} + A_w \sigma_w (z/s) \cot\theta \qquad (1)$$

where V_{co} : shear force carried by other than truss, ν : reduction factor, A_w : area of shear reinforcement within s, σ_w : stress of shear reinforcement, s : spacing of shear reinforcement, z : distance between tension and compression chord (=d/1.15), d : effective depth, θ : inclination of diagonal strut. As shown in Fig.2, contribution of truss, V_s is greater than that with θ of 45°, but decrease in V_c compensates the increase in V_s. Since the inclination of diagonal strut is not equal to that of shear crack, force transferred by aggregate interlocking should match with compressive force in diagonal strut.

Model description so far concerns shear resistance behavior at monotonic loading. At unloading and reloading the followings were found. (1)Crack displacements (opening and shearing displacements) vary linearly with external shear force, but rate of change is smaller. (2)Inclination of compressive force in concrete between shear cracks is the smaller with the lower external shear force. These experimental facts indicate that diagonal strut in the truss model should be with variable angle at unloading and reloading. Because of plastic characteristics of shear transfer constitutive laws at shear crack, the truss mechanism indicates plastic behavior. The change in inclination of diagonal strut implies that compressive normal stress at shear crack becomes relatively the larger with the lower external shear force, compared with shear stress.

4. CONCLUDING REMARKS

The new truss model can explain satisfactorily the experimental facts. Features of the truss model, such as inclination of strut indicate strong dependency on stress transfer constitutive laws at shear crack.

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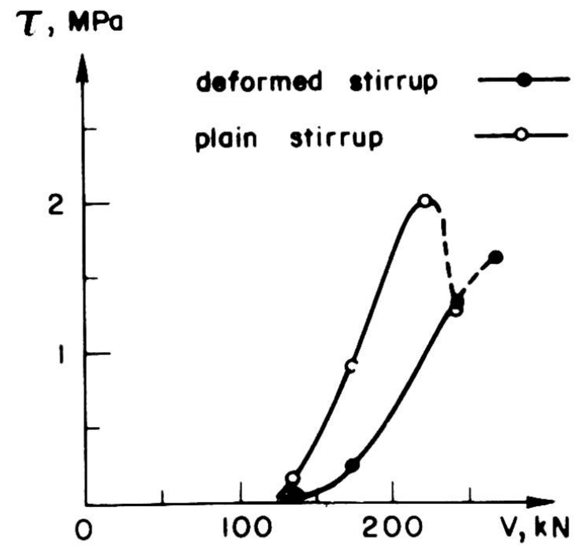

Fig.1 Shear stress at shear crack

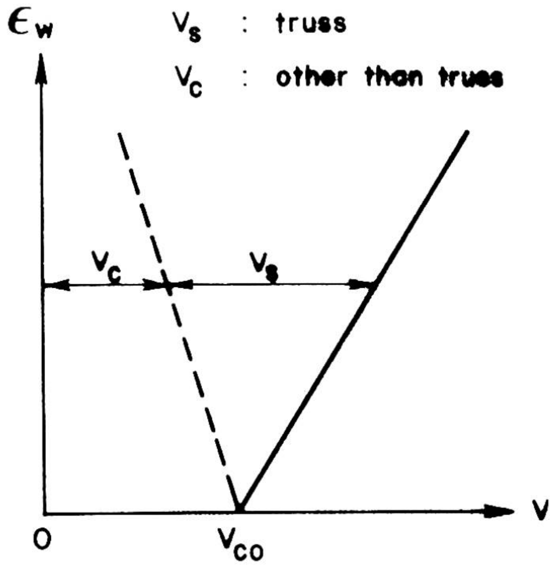

Fig.2 New truss model

Calcul de la stabilité de la nef de la cathédrale Saint-Michel à Bruxelles

Stabilitätsberechnung des Schiffes der Kathedrale Sankt-Michael in Brüssel

Calculation of the Stability of the Nave of the Cathedral Saint-Michel of Brussels

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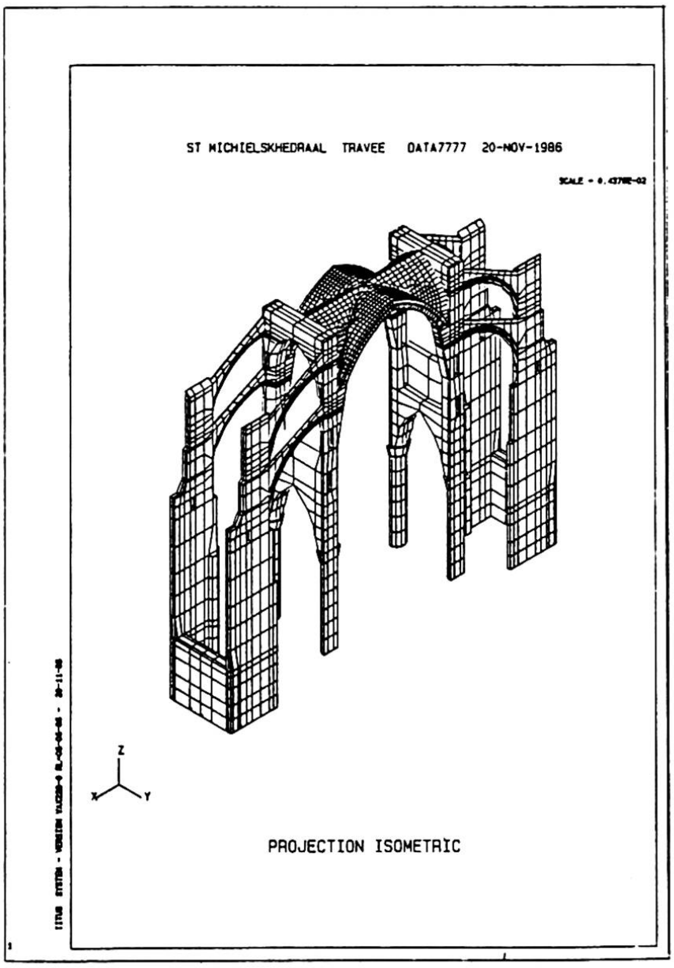
La Cathédrale Saint-Michel est l'un des plus anciens monuments de l'agglomération bruxelloise. Son histoire remonte aux 11ème siècle, alors que la contruction de la cathédrale actuelle de style gothique a débuté en 1220 pour s'achever en 1475.

Au début des années 80, les phénomènes d'altérations observés étaient tels qu'un planning urgent des travaux de restauration s'avérait prioritaire au risque de compromettre définitivement la survie de la cathédrale.



En 1982, le Ministère des Travaux Publics approuve un plan de restauration globale qui permet, dès 1983, d'entamer les premiers travaux.

Une approche globale et innovante se justifiait particulièrement dans ce cas vu l'ampleur du problème, la complexité des interventions et les responsabilités inhérentes à la restauration d'ouvrages publics de cette importance.

Les arcs-boutants et les contreforts jouent un rôle fondamental dans la stabilité horizontale de la nef centrale de la cathédrale. Les arcs-boutants génèrent sous leur poids propre des poussées, qui sont nécessaires pour résister aux efforts horizontaux engendrés par les réactions des voûtes de la nef supérieure.

L'enfilade des arcs-boutant supérieurs sert à reporter sur les contreforts les charges dues au vent qui agissent sur la toiture de la nef centrale. En outre, les réactions des arcs-boutants sur les contreforts sont équilibrées par , entre autres, le poids-propre de ceux-ci.

Vu les importantes dégradations qui se sont produites aux contreforts et arcs-boutants, cette structure complexe a fait l'objet d'une étude très précise, qui fut faite en utilisant des moyens permettant de dépasser l'application classique des méthodes connues. Un modèle tridimensionnel a été construit sur ordinateur, permettant d'exécuter une analyse structurelle très détaillée.

Cette méthode de travail s'insère dans une approche nouvelle de la restauration de la cathédrale Saint-Michel, et repose sur des fondements scientifiques et techniques sans compromettre le caractère propre à ce monument. Tant pour le projet du modèle (pré-processing) que pour le calcul et l'analyse des résultats (post-processing) l'on a fait usage des possibilités offertes par la programmation avancée. L'analyse par ordinateur a été réalisée en utilisant la methode des éléments finis.

On a utilisé à maintes reprises les possibilités du CAD liées à l'emploi de l'ensemble des modules du FEM afin de construire le modèle de calcul. La modélisation fut d'autre part, affinée par l'utilisation de modules à programmer en FORTRAN IV, compatibles avec l'ensemble des modules du FEM. Le modèle global fut érigé sur la base d'un élément à 3 dimensions (8 noeuds). Le modèle de base projeté d'une demi-travée de la nef contient environ 5000 noeuds et 2500 éléments de base. Les résultats pouvaient, comme le montrent les photos ci-jointes, être visualisés d'une manière très élégante grâce aux diverses possibilités offertes par les écrans graphiques très performants, en couleur, et, par la table traçante.
Simultanément, la vérification de la stabilité de la nef, en particulier sous les charges du vent et la recherche des zones fortement sollicitées, l'on pouvait également simuler les différentes phases d'exécution des travaux de restauration des arcs-boutants et contreforts. De cette manière, il fut possible d'évaluer quantitativement, la répercussion des différentes interventions prises sur chantier. Dans un même modèle, il fut possible de prendre en considération les différentes caractéristiques des divers matériaux et, notamment, la recherche de l'augmentation des contraintes résultantes de la composition hétérogène des piliers et des contreforts.

L'application d'une programmation avancée a permis d'optimaliser la qualité des travaux de restauration. Enfin, nous voulons mentionner que pareils calculs sont uniques en leur genre en Belgique.

Simulation of Cyclic Plasticity Behaviour of Structural Members

Simulation du comportement de plasticité cyclique d'éléments de structures

Rechnerische Simulation des zyklisch-plastischen Verhaltens von Tragelementen

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1.INTRODUCTION

Computer simulation of the cyclic plasticity behaviours of structures or structural members is very important to evaluate structural safety and make them safe against severe cyclic loading due to earthquakes, wind storms and so on. Here a cyclic plasticity theory for elasto-plastic hysteretic behaviours of structural steel is proposed and it is applied to the analyses of structural members subjected to cyclic loads.

2.CYCLIC PLASTICITY MODEL

The proposed model is derived by the refinements of a multi surface plasticity theory introduced by Petersson and Popov[1]. Petersson-Popov model used cumulative equivalent plastic strain as a state variable, two fundamental surface size functions and a weighting function as material property functions.

There are three important differences between Petersson-Popov model and the proposed model. Firstly, effective value of cumulative equivalent plastic strain is defined as a state variable to represent "Return Phenomena". Secondly, additional material property functions are introduced. These functions express strain hardening characteristics of materials after loading histories corresponding to certain values of cumulative equivalent plastic strain. Owing to this modification, the theory can express strain hardening characteristics of materials with both notable strain hardening and non-hardening strain region. Thirdly, all of the material property functions can easily and unambiguously be obtained by a combination of a simple tension test and several simple tension-compression tests.

3.APPLICATION OF THE MODEL

This method is applied to simulation of tension-compression stress-strain relationships of mild steel and high strength steel. Using material property functions measured, the authors carried out elasto-plastic finite element analyses for round-bar specimens subjected to repetitive tension-compression loading under controlled strain. Figure 1 shows stress-strain relationships predicted by the proposed model and those gained by the corresponding experiments. By comparing these results, it was confirmed that the stress-strain relationships calculated by the proposed theory was accurate.

A next application of the cyclic plasticity model is the evaluation of bending moment-curvature relationships(M-Φ relationships) of steel beams. In the calculation of the M-Φ relationships of beams and beam-columns by the proposed

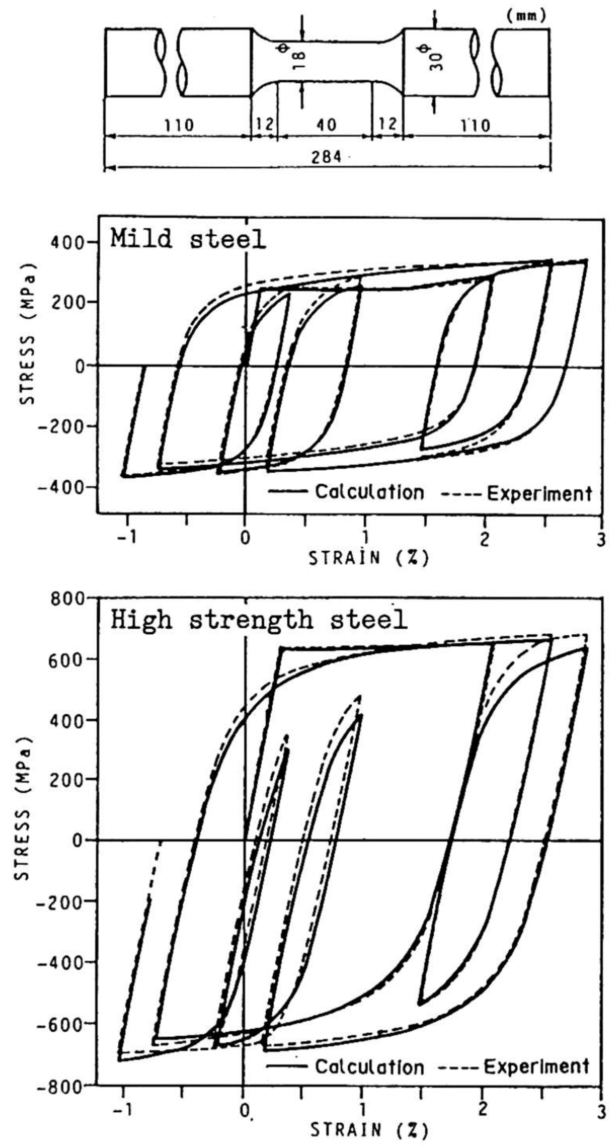

Figure 1 Stress-strain relationships of round bar specimens.

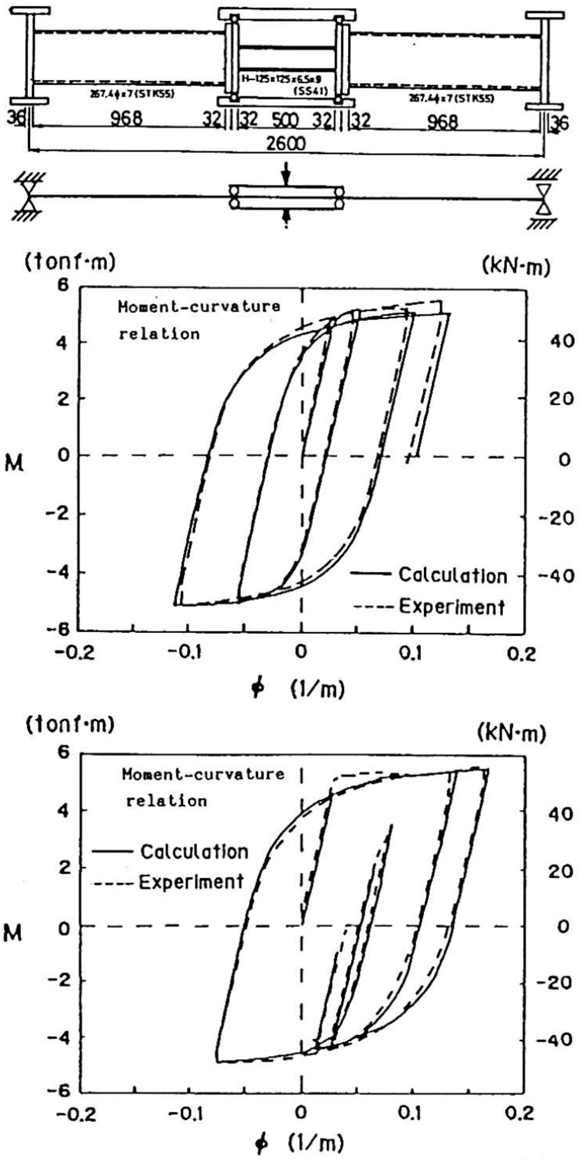

Figure 2 Moment-curvature relationships of simple beam specimens.

stress-strain model, the tangent stiffness method introduced by Chen and Atsuta[2] was used with some modifications.

JIS(Japanese Industrial Standards)-5 type specimens were shaped from a H-shaped beam specimen to evaluate material property functions. Material property functions were determined by a combination of tension tests and tension-compression tests. Residual stress characteristics were analyzed by a drilling method and simple type distribution was assumed according to these results.

The calculated M-Φ relationships are compared with those obtained by experiments which the authors had carried out. Figure 2 shows hysteretic M-Φ relationships predicted and those gained by the corresponding experiments. The accurate M-Φ relationships can be evaluated by the authors' theory.

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