Posters

Objekttyp: **Group**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

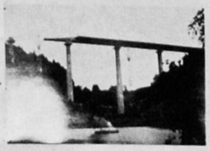
Band (Jahr): **11 (1980)**

PDF erstellt am: **15.09.2024**

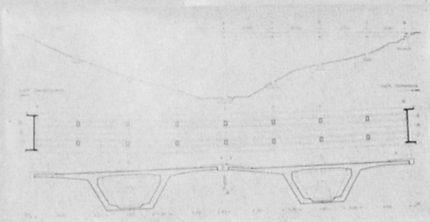
Axhausen
Fink
Katz
Rank
Stieda
Verschuer
Werner

Programmkette S E T : Entwurfsberechnungen im konstruktiven Ingenieurbau

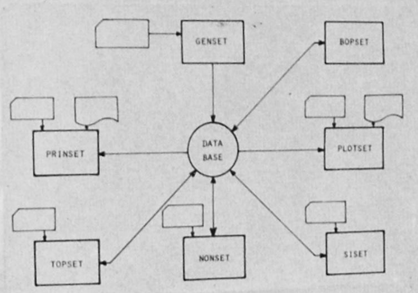
Prof. Dr.-Ing. Heinrich Werner - Technische Universität München



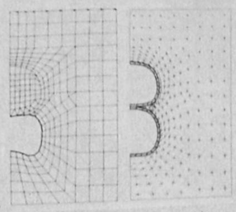
Brückenbau



Aufbau der Programmkette



Tunnelbau



GENSET: Generierung von Strukturen

- Knoten, Auflager, Elemente
- Querschnitte, Materialgesetze
- Lasten, Primärspannungszustände
- Generierungen, Fehlerprüfungen

NONSET: Lineare und nichtlineare statische Berechnungen, FE-Methode

- Nichtlineares Materialverhalten
- Große Verformungen
- Verfolgung der Bauzustände

TOPSET: Statik, Bemessung und Stabilität räumlicher Stahlbetonrahmentragwerke

- Statik nach der Theorie 2. Ordnung
- Bemessung, effektive Steifigkeiten
- Verfolgung des Baufortschrittes

SISET: Sicker- und Grundwasserströmungen

- Ebene, axialsymm., räumliche Probleme
- Lineare und nichtlineare Fließgesetze
- Freie Oberfläche mit Niederschlag
- Verknüpfung zu GENSET

Programmkette SET: Entwurfsberechnungen im konstruktiven Ingenieurbau

Heinrich Werner
Professor, Dr.-Ing.
Technische Universität
München, Deutschland

1. EINSATZMÖGLICHKEITEN DER PROGRAMMKETTE SET

Die Programme GENSET, PRINSET, PLOTSET und BOPSET sind problemunabhängig.

NONSET untersucht alle statischen Systeme, die folgende Elemente enthalten:

- Stäbe mit abschnittsweise veränderlichen Steifigkeiten (Rahmen, Fachwerke)
- Anisotrope ebene oder rotationsymmetrische Elemente (Scheiben)
- Federn und elastische Kopplungen
- Isotrope, isoparametrische Platten- und Faltwerkselemente (Platten, Schalen)
- Isotrope, isoparametrische räumliche Elemente mit 4 - 21 Knoten.

Folgende Nichtlinearitäten sind vorhanden:

- Große Verformungen für Fachwerkstäbe und ebene Elemente (z.B. Stabilitätsuntersuchungen an Gerüsten und räumlichen Fachwerken)
- Elastisch-plastisches oder nichtlinear-elastisches Materialverhalten bei Fachwerkstäben und ebenen Elementen (z.B. Plastifizierungen von Fachwerkteilen oder in geschichteten Böden)
- Schlupf, Vorspannung oder Abreißen bei Lagern und Kopplungen (Verbindungsmittel).

Das Programm TOPSET ist für Traglastuntersuchungen an Rahmentragwerken aus Stahl, Stahlbeton oder Spannbeton geeignet. Es berücksichtigt plastifizierte Bereiche mit vorgegebenen Arbeitslinien der Materialien.

2. VERFÜGBARKEIT DER PROGRAMMKETTE SET

Die Programmkette SET wurde bisher auf folgenden Maschinentypen installiert:

- CDC CYBER 175, Betriebssystem NOS 1.3
- IBM 370/125, Betriebssystem DOS/VS
- IBM 370/145, Betriebssystem OS/VS
- Siemens 7600 Betriebssystem Bs 2000
- Siemens 7.738 Betriebssystem Bs 2000

Das Programm SISET ist in Kürze einsatzbereit.

Der modulare Programmaufbau erfordert einen Kernspeicherbedarf von 128 KB. Der gesamte Datenverkehr zwischen den Teilprogrammen oder innerhalb eines Programmes erfolgt über die SET-Datenbasis. Ein in FORTRAN geschriebenes Datenverwaltungsprogramm DYNCO speichert die Daten dynamisch in freien Kern- und Massenspeicherbereichen.

Damit werden folgende Vorteile erzielt:

- Die Größe der zu berechnenden Probleme ist praktisch nicht begrenzt
- Optimale Ausnutzung der Kern- und Peripheriespeicherkapazitäten
- Einsetzbarkeit auch auf Minirechnern.

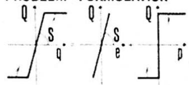
UNIVERSITA' DEGLI STUDI DI BARI
FACOLTA' DI INGEGNERIA
Istituto di Scienza delle Costruzioni

Viale Japigia, n. 182
70126 BARI (ITALY)

Computerized elasto-plastic analysis of steel reticulated plates.

Mauro MEZZINA - Giuseppe PRETE - Antonio TOSTO

PROBLEM FORMULATION



$s = \max \omega$

$\varphi = A\lambda + B\omega + k$

$\varphi^T \lambda = 0 \; ; \varphi, \lambda, \omega \geq 0$

PLCP: Parametric Linear Complementarity Problem solvable by Linear Programming algorithm (Restricted Basis entry rule).

STEP-WISE SOFTENING

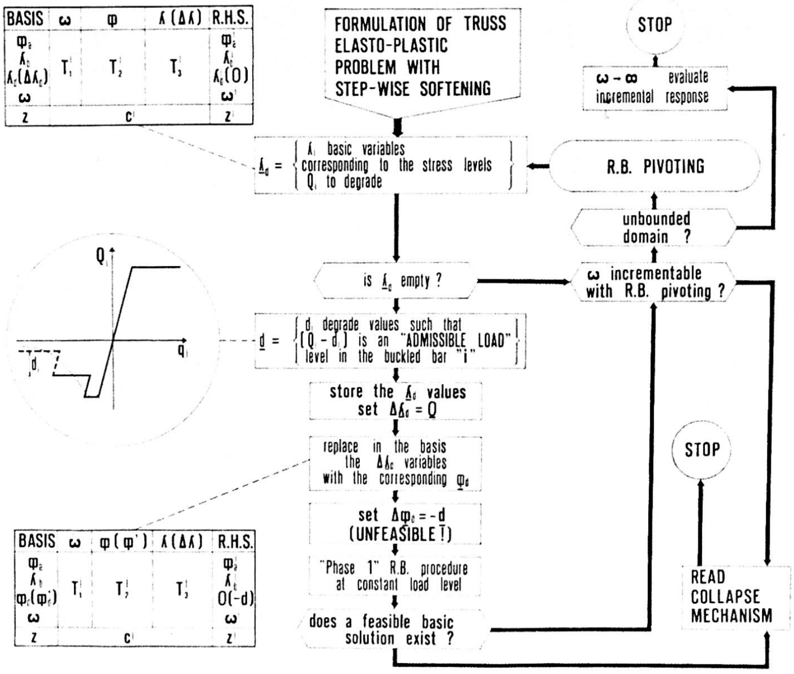
Incremental formulation from POST-CRITICAL UNLOADING (d = buckled bars):

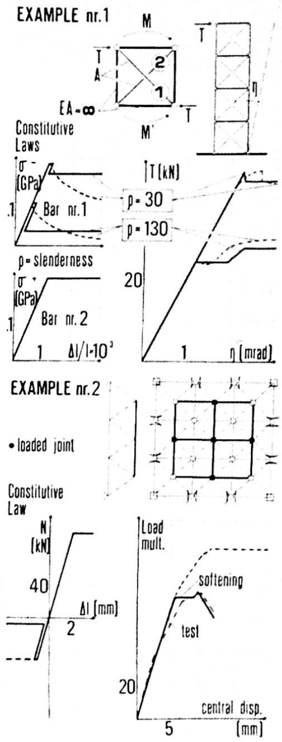
$\varphi_c = \varphi_c' + d = 0 \; ; \varphi_c', d \geq 0$

$s = \max \Delta\omega$

$\begin{Bmatrix} \varphi_a \\ \varphi_c' \end{Bmatrix} = A\,\Delta\lambda + B\,\Delta\omega + \begin{Bmatrix} \dot\varphi_a \\ -d \end{Bmatrix}$

$\begin{Bmatrix} \varphi_a \\ \varphi_c' \end{Bmatrix} \geq 0 \; ; \; \begin{matrix} \Delta\lambda \geq 0 \\ \Delta\omega \text{ unr.} \end{matrix} \; ; \; \begin{Bmatrix} \varphi_a \\ \varphi_c' \end{Bmatrix}^T \Delta\lambda = 0$





COMPUTERIZED ELASTO-PLASTIC ANALYSIS OF STEEL RETICULATED PLATES

Mauro MEZZINA - Giuseppe PRETE - Antonio TOSTO
Istituto di Scienza delle Costruzioni, Facoltà di Ingegneria
Università di Bari
Bari, Italy

In the structural collapse analysis special emphasis must be given to the knowledge of all the parameters that characterize the reached limit state. In fact this is the only instrument for evaluating the agreement in truth of such a limit state, that the specific mathematical model allows to simulate. In the special case of trusslike structures such a problem is of primary importance, due to the presence of "physically unstable" elements in the collapse mechanism. This event can invalidate the quality of the results obtainable with the "classical" limit analysis methods. In this context we propose an approach to the problem that, still remaining in the Mathematical Programming method field, arrange the solution of the evolutive elasto-plastic analysis of this kind of structures as a particular Linear Programming Problem.

The formulation starts, as indicated in |1|, from the equilibrium,compatibility and stress-strain relationships for the whole structure in elastic-perfectly plastic behaviour: after some algebra it is possible to see that the collapse load factor is obtainable by the solution of the L.P. Problem shown in the Poster, where:

$\underline{q}$ = bar elongations	$\underline{Q}$ = bar forces
s = collapse load factor	ω = load multiplier
$\underline{\phi}$ = yield functions	$\underline{\lambda}$ = plastic multipliers
$\underline{k}$ = yield limits	$\underline{N}$ = diag {1 -1} = activable yield modes
$\underline{A} = -\underline{N}^T\,\underline{Z}\,\underline{N}$	$\underline{B} = -\underline{N}^T\,\underline{Q}^e$
$\underline{Q}^e$= elastic stress response to loads $\underline{F}$	$\underline{Z}$ = influence matrix of self-stresses due to imposed strains

Such a formulation must be completed taking into account the post-critical behaviour, simulated in the $Q_i - q_i$ law with a series of constant-load steps,decreasing more and more up to the asymptotic level. The logical operations that must be performed from a "degrading" point are shown in detail in the depicted "flowchart".

The first results of the numeric-experimental comparison, presently in effect, are very comfortable. From these applications we report here the most significant ones, according to us. In particular the Example n. 2 concerns the numerical simulation of a collapse test, performed on a prototype of reticulated grid.As it is possible to see, the calculated values are very close to the experimental ones both from a static and deformative point of view.

REFERENCES.

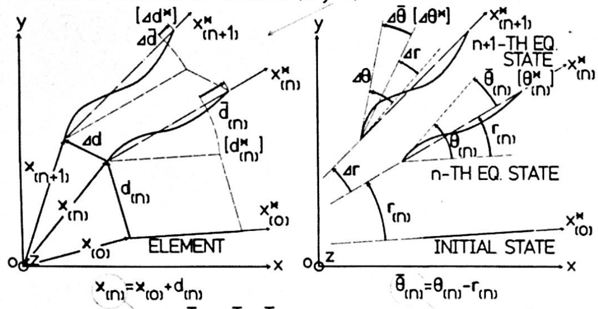
1. M. MEZZINA and A. TOSTO: Analisi elastoplastica di strutture reticolari in acciaio. Proc. of VII Congresso C.T.A., Torino, Italy, Oct. 1979.

A COMPUTER ORIENTED FORMULATION FOR GEOMETRICALLY NONLINEAR PROBLEMS
EINE COMPUTER-ORIENTIERLE FORMULIERUNG FÜR GEOMETRISCHE NICHTLINEARE PROBLEME
UNE FORMATION POUR DES PROBLÈMES NONLINÉAIRE GÉOMÉTRIQUE

Yutaka YOSHIDA TOKYO INST. TECH.
Takeo NISHIWAKI
Nobutoshi MASUDA MUSASHI INST. TECH.

DERIVATION OF ELEMENT EQUATION

1. COORDINATES x AND DISPLACEMENTS $u^T=(d^T\ \theta^T)$ IN GLOBAL COORDINATE SYSTEM $(x\ y\ z)$



$x_{(n)}=x_{(0)}+d_{(n)}$ $\quad \bar{\theta}_{(n)}=\theta_{(n)}-r_{(n)}$

2. DISPLACEMENTS $u^{*T}=(d^{*T}\ \theta^{*T})$ IN LOCAL SYSTEM $(x^*\ y^*\ z^*)$

$d^*_{(n)}=\Lambda_{(n)}\bar{d}_{(n)}=\Lambda_{(n)}x_{(n)}-\Lambda_{(0)}x_{(0)}$ $\quad \theta^*_{(n)}=\Lambda_{(n)}\bar{\theta}_{(n)}$

$\Lambda_{(n)}$: COORDINATE TRANSFORMATION MATRIX

3. INCREMENTAL NODAL DISPLACEMENTS IN LOCAL COORDINATE SYST. (FROM N-TH TO N+1-TH EQUILIBRIUM STATE)

$\Delta u^*=u^*_{(n+1)}-u^*_{(n)}=\Delta T\binom{\bar{x}}{\bar{\theta}}_{(n)}+T_{(n+1)}\binom{\Delta \bar{d}}{\Delta\bar{\theta}}$ $\quad ;\ T_{(n)}=[^{\Lambda}{}_{\Lambda}]_{(n)}$

$=\Delta T\binom{x}{\theta-r}_{(n)}+T_{(n+1)}\binom{0}{-\Delta r}+T_{(n+1)}\Delta u$

4. FORCE-DISPLACEMENT RELATION IN LOCAL COORDINATE SYSTEM

ASSUMPTION OF SMALL STRAIN

$\Delta f^*=K^*\Delta u^*$ $\quad K^*$: CONVENTIONAL LINEAR STIFNESS MATRIX

5. RELATION BETWEEN LOCAL AND GLOBAL NODAL FORCES

$\Delta f=f_{(n+1)}-f_{(n)}=\Delta T^T f^*_{(n)}+T^T_{(n+1)}\Delta f^*$ $\quad \because f_{(n)}=T^T_{(n)}f^*_{(n)}$

6. FORCE-DISPLACEMENT RELATION IN GLOBAL SYSTEM [EQ. A]

$$\Delta f=\Delta T^T f^*_{(n)}+T^T_{(n+1)}K^*[\Delta T\binom{x}{\theta-r}_{(n)}+T_{(n+1)}\binom{0}{-\Delta r}]+T^T_{(n+1)}K^*T_{(n+1)}\Delta u$$

KNOWN

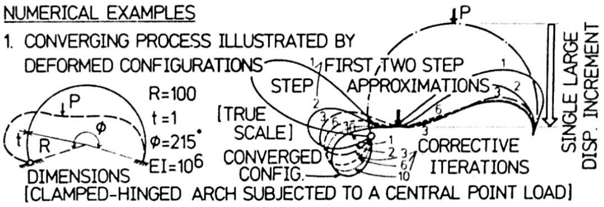
FUNCTIONS OF UNKNOWN INCREMENTAL DISPLACEMENTS Δu

DEVELOP. OF A TWO STEP APPROX. & CORRECT. ITERATION SOL. PROC.

In the first two steps, EQ.A is linearized with respect to Δu, before the assemblage of element equations. Then in iterations, only Δu_{k+1} are treated as unknowns, others are estimated by using k-th approximation of Δu.

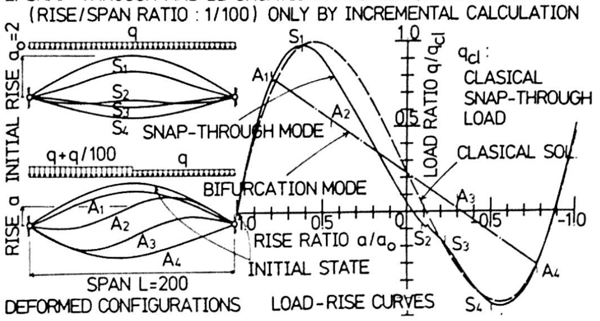
NUMERICAL EXAMPLES

1. CONVERGING PROCESS ILLUSTRATED BY DEFORMED CONFIGURATIONS



[CLAMPED-HINGED ARCH SUBJECTED TO A CENTRAL POINT LOAD]

2. SNAP-THROUGH AND BIFURCATION ANALYSIS OF A FLAT ARCH (RISE/SPAN RATIO : 1/100) ONLY BY INCREMENTAL CALCULATION



DEFORMED CONFIGURATIONS LOAD-RISE CURVES

A COMPUTER ORIENTED FORMULATION FOR GEOMETRICALLY NONLINEAR PROBLEMS

Yutaka YOSHIDA, Associate Professor, Tokyo Institute of Technology
Takeo NISHIWAKI, Professor
Nobutoshi MASUDA, Lecturer, Musashi Institute of Technology
Tokyo/Japan

FORMULATION OF THE ELEMENT EQUATION

An incremental formulation is developed here, under the assumption of large displacement but small strain. The key point of the formulation is the description of nodal locations by the coordinates themselves instead of by mere displacements. A local coordinate system (x* y* z*) is introduced which is at any time on the element itself, and the element force-displacement relation is assumed to be linear in this local system. The geometrical nonlinearity is considered only through the nonlinear transforming relations of displacement components between the space-fixed global coordinate system (x y z) and the element-fixed local coordinate system. The transformations are evaluated rigorously without neglecting small terms, so that incremental relations satisfy equilibrium conditions after deformation as accurately as poissible.

SOLUTION PROCEDURE

To construct and solve structural equilibrium equations, an effective corrective iteration solution procedure is also originally developed. The process, outlined in the following, makes use of physical properties of each term in the derived element equation. At first, the changes of transformation matrix and rigid body rotation in the element equation, ΔT and Δr respectively, are linearized with respect to incremental displacement Δu at the n-th equilibrium state A. And the transformation matrix after the increment, $T(n+1)$, is estimated at the same state A. Hence the element incremental equation is written in the following quasitangential form.

$$\Delta f=(K_f+K_u+K)_A\Delta u$$

Then summing up the element incremental equations thus approximated for overall structure and solving them, the first approximation of the n+1-th equilibrium state C is obtained. Secondly, ΔT and Δr are linearized at the midpoint B between A and C, meanwhile $T(n+1)$ is approximated at the above obtained first approximating state C. Thus the element incremental equation is written as

$$\Delta f=(K_f+K_u+K)_{B,C}\Delta u$$

Thus the second approximating solution, denoted by D, can be obtained. Then afterwards in iterations, ΔT and Δr as well as $T(n+1)$ are all evaluated by using the just preceding approximating solution, and only the incremental displacement Δu is treated as unknown variable. Namely, for the k+1-th approximation

$$\Delta f-h_k=K_k\Delta u$$

is used, where

$$K_k=T_k^TK^*T_k\ ,\qquad h_k=T_k^Tf^*_{(n)}+T_k^TK^*(\Delta T\binom{x}{\theta-r}_{(n)}+T_k\binom{0}{-\Delta r})$$

and

$$\Delta T=T_k-T_{(n)}\ ,\qquad \Delta r=r_k-r_{(n)}$$

The iteration is continued until satisfactory convergence is obtained.

FEATURES OF THE PROPOSED METHOD

- With mere incremental calculations and without special techniques such as eigenvalue analysis, not only snap-through and limit point phenomina but also bifurcation can be pursued.
- Even for an extraordinary large increment, numerical stability exists and sufficient accuracy is obtained.
- As the consequence of the above characteristics, calculating time can be reduced considerably.

Buckling Analysis of Reticulated Cylindrical Shell Roofs

TOSHIYUKI OGAWA and TOSHIRO SUZUKI, Dr. of Eng., Tokyo Institute of Technology, Japan

SYNOPSIS

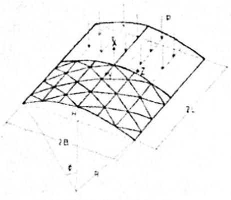
It is noted that instability failures such as snap-through are likely to occur in single layer reticulated shells as well as in true shells. Computer analysis approach in this field is usually done by the matrix method but there arise many difficulties due to large-scale degrees of freedom. In this study, we develop a simplified analytical method and apply it to the buckling analysis of single layer reticulated shells.

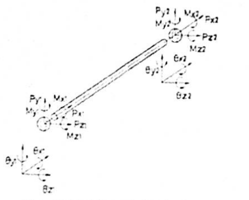
EVALUATION OF MEMBER STIFFNESS

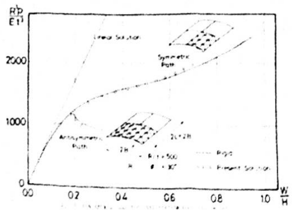
As shown in Fig.2, a reticulated shell is considered to be composed of a truss element with rotational springs at its edges. A truss element has only axial stiffness. Bending stiffness and joint rigidity is represented by these springs. Total member stiffness is obtained by the equilibrium method.

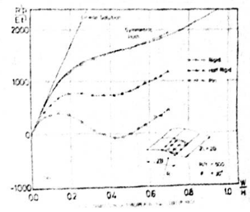
NONLINEAR BUCKLING BEHAVIORS

Fig.3 shows fundamental equilibrium and post-bifurcation path of a rigidly jointed shell. Loading condition is shown in Fig.1. Post-bifurcation path is persued by giving small asymmetrical perturbation. Present solutions give good agreement with the solid lines obtained by the conventional FEM method. Fig.4 shows the transition of fundamental path as joint rigidity decreasing. It is shown that snap-through occurs as decreasing of joint rigidity even when no snap-through exists on the fundamental path of perfectly jointed one. Although this is one example, this fact indicates the importance of joint rigidity in single layer reticulated shells. Required CPU time is much deduced by the simplified evaluation of bending stiffness.









Buckling Analysis of Reticulated Cylindrical Shell Roofs

Toshiro SUZUKI[1] and Toshiyuki OGAWA[2]
[1]Assoc. Prof., Dr.-Eng., Dep.of Architecture, Tokyo Inst. of Tech.
[2]Research Assoc., Dr.-Eng., Dep.of Architecture, Tokyo Inst. of Tech.
Tokyo, Japan

SUMMARY OF THE DISCUSSIONS AND QUESTIONS

We received several questions at the "Poster Session". These questions and discussions are classified into the following

1) Iterative procedure in the incremental step, 2) Assembly of the element stiffness, 3) Accuracy of this method, 4) CPU time.

In this analysis, the unbalanced forces appear at each incremental loading step. As mentioned before, these forces are caused by the second-order terms in the axial strain expression. These forces are extinguished by the Newton-Raphson method. In this iterative procedure, the stiffness matrix is always recomposed. Accuracy of this analysis is depend on the validity of ignoring the terms related to initial inner moments. A uniformly loaded reticulated shell is considered to be near the membrane state. In other words, the inner moments are secondary compared with the axial force, and like a 3-hinged truss structure, there may be an equilibrium state by axial force. By these reason, we consider this analytical method effective when applied to the buckling analysis of uniformly loaded reticulated shells. The computing time is much deduced by the reason that solutions converge well and the required time of forming a stiffness matrix is deduced. In comparison with the required CPU time of the conventional FEM, this simplified method need 1/5 ∿ 1/10 of that time. In addition to it, these calculations are done by Hitac M-180 which seems to have an equal ability with IBM's large computer. The assembly of the element stiffness is shown in the next section with the formulation process of a stiffness matrix [K].

FORMULATION OF A STIFFNESS MATRIX

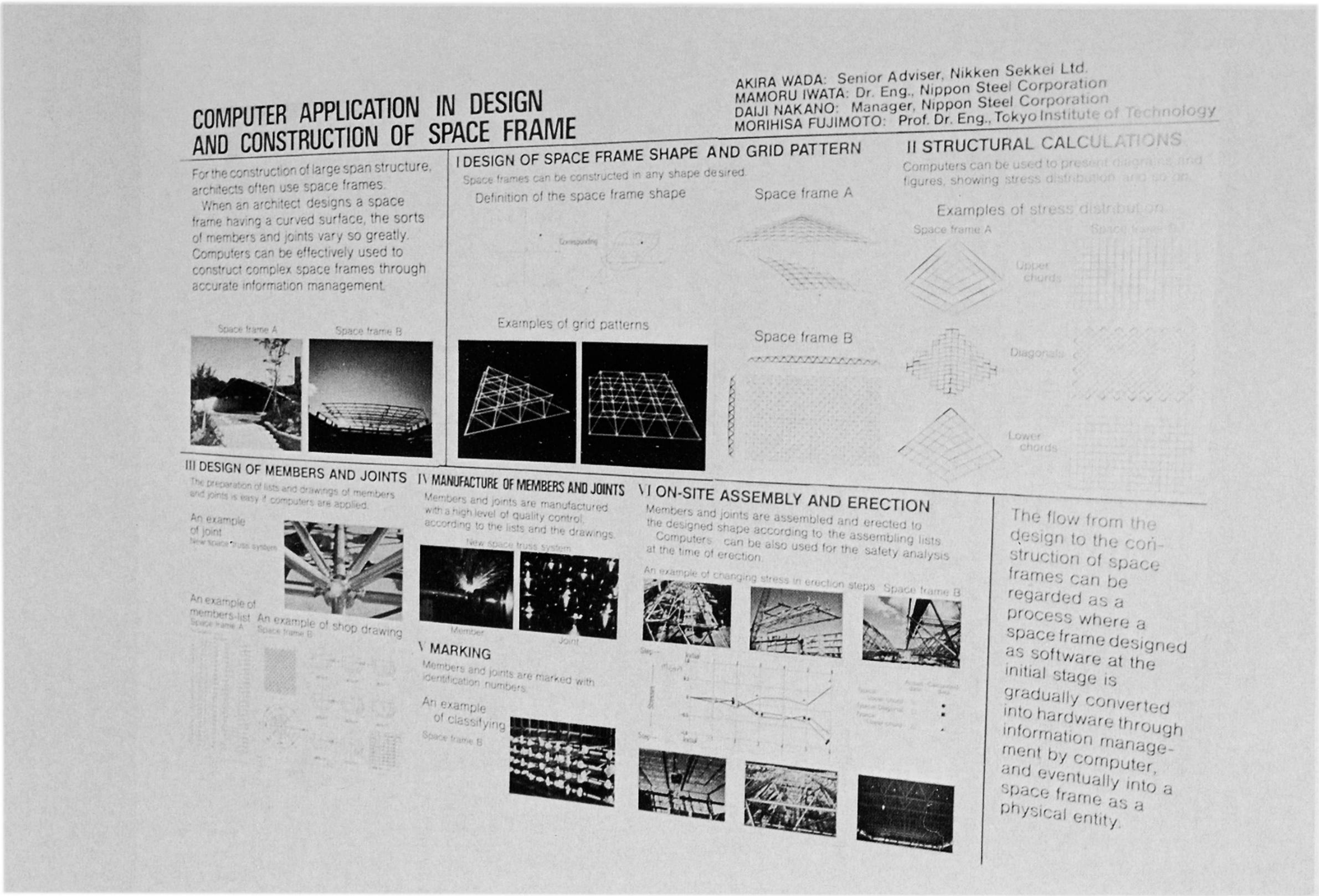
A stiffness matrix of a truss element is lead by the virtual work principle. The finite element formulation of the equilibrium is shown in quandratic form of displacement vector $\{\Delta u_e\}$.

$$\iiint [\Delta\sigma_{xx}\delta\Delta e^*_{xx}+\sigma_{xx}\cdot\frac{1}{2}\delta[(\frac{d\Delta v}{dx}^2)+(\frac{d\Delta w}{dx}^2)]-\Delta p_i\delta\Delta u_i]dv - \iint \Delta f_i\delta\Delta u_i ds - \delta\Delta Wr$$
$$= \delta\{\Delta u_e\}^t.[Ke].\{\Delta u_e\} - \delta\{\Delta u_e\}^t.\Delta F_{ex} - \delta\{\Delta u_e\}^t.R \qquad (1)$$

where, σ_{xx}=fiber stress, p_i=body force increment, and f_i=surface force increment. According to the arbitriness of the virtual displacement $\{\Delta u_e\}$, a stiffness matrix $[K_e]$ of a truss element and equilibrium equation is obtained. Total flexibility matrix [F] is obtained using flexibility matrix of spring elements $[F_1]$,$[F_2]$ and equlibrium matrix.

$$[F] = [H]^t.[F_1].[H] + [F_m] + [F_2] \qquad (2) \qquad [F_m] = [Ke]^{-1}_{jj} \qquad (3)$$

Total member stiffness matrix is obtained in inverced form of equation (2).



COMPUTER APPLICATION IN DESIGN AND CONSTRUCTION OF SPACE FRAME
AKIRA WADA: Senior Adviser, Nikken Sekkei Ltd.
MAMORU IWATA: Dr. Eng., Nippon Steel Corporation
DAIJI NAKANO: Manager, Nippon Steel Corporation
MORIHISA FUJIMOTO: Prof. Dr. Eng., Tokyo Institute of Technology
For the construction of large span structure, architects often use space frames. When an architect designs a space frame having a curved surface, the sorts of members and joints vary so greatly. Computers can be effectively used to construct complex space frames through accurate information management.
Space frame A
Space frame B
I DESIGN OF SPACE FRAME SHAPE AND GRID PATTERN
Space frames can be constructed in any shape desired.
Definition of the space frame shape
Space frame A
Examples of grid patterns
Space frame B
II STRUCTURAL CALCULATIONS
Computers can be used to present diagrams and figures, showing stress distribution and so on.
Examples of stress distribution
Space frame A
Upper chords
Diagonals
Lower chords
III DESIGN OF MEMBERS AND JOINTS
The preparation of lists and drawings of members and joints is easy if computers are applied.
An example of joint
An example of members-list
An example of shop drawing
IV MANUFACTURE OF MEMBERS AND JOINTS
Members and joints are manufactured with a high level of quality control according to the lists and the drawings.
New space truss system
Member
Joint
V MARKING
Members and joints are marked with identification numbers.
An example of classifying
Space frame B
VI ON-SITE ASSEMBLY AND ERECTION
Members and joints are assembled and erected to the designed shape according to the assembling lists.
Computers can be also used for the safety analysis at the time of erection.
An example of changing stress in erection steps Space frame B
The flow from the design to the construction of space frames can be regarded as a process where a space frame designed as software at the initial stage is gradually converted into hardware through information management by computer, and eventually into a space frame as a physical entity.

COMPUTER APPLICATION IN DESIGN AND CONSTRUCTION OF SPACE FRAME

AKIRA WADA
Senior Adviser
Nikken Sekkei Ltd.

MAMORU IWATA
Dr.Eng.
Nippon Steel Corp.

DAIJI NAKANO
Manager
Nippon Steel Corp.

MORIHISA FUJIMOTO
Prof.Dr.Eng.
Tokyo Inst. of Tech.

TOKYO JAPAN

BACKGROUND

In Japan, the construction of huge space structures, such as sports halls and exhibition halls, has recently become more widespread.
Space frames are an important method for constructing steel structures having extensive column-free space. In space frames technology, computers have been used for structural analysis, however, computer can also be used for design and construction, from the standpoint of information management.

FURTHER COMMENTS

I. Design of Space Frame Shape and Grid Pattern.

When the plane on the nondimensional coordinates is divided by a grid, such as "Example of grid patterns", the space frame on the three dimensional coordinates is also divided by a certain rule based on the isoparametric shape function of the finite element method. The member lengths, the joint coordinates and the joint angles are automatically determined.

II. Structural Calculations.

"Examples of stress distribution" shows the magnitudes of stress by the thickness of the line of the members, the qualitative tendency of stress distribution is visually portrayed. The line in black indicates compression stress, while the line in white indicates tension stress.

III. Design of Members and Joints.

"An example of member list" shows the sequential number marked on a member, the number of joints at both ends, and the length of a member. "An example of shop drawing" shows the drilling positions of a joint.

IV. Manufacture of Members and Joints.

Manufactures make the members and joints, accurately observing each specification , with no idea as to where they are used in the space frame.

V. Marking.

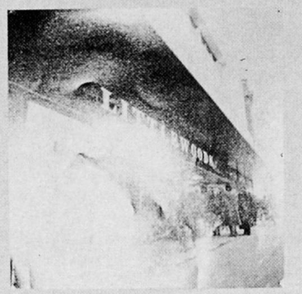
VI. On-Site Assemly and Erection.

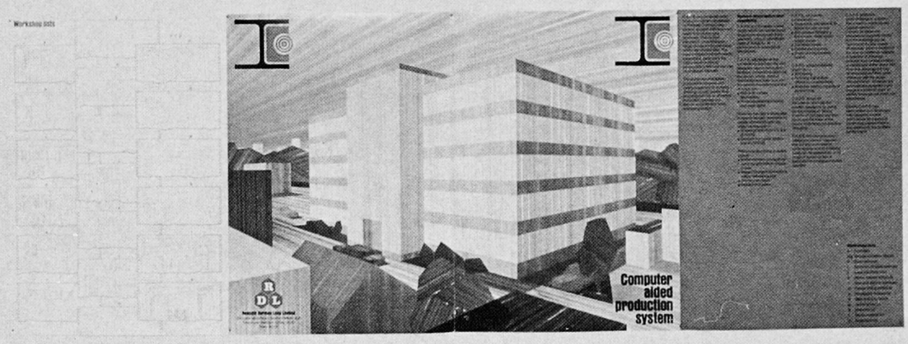
Computers can be also used for the safety analysis at the time of erection.
The figure shows the changing stress at each of erection and the calculation results.

FUTURE POSSIBILITIES

The concept of computer controlled information management is not limited to such examples and will become increasingly important to general building structures.

Littlewoods Store, Stoke on Trent, England—involved CAPS (Computer Aided Production System) detailing of 973 tonnes of structural steelwork.





CAPS - BASIC FEATURES

Some 18 man-years of research and development by RDL have gone into making the computer-aided detailing of beam and column structures a functioning reality. The benefits of this work are now available to the steelwork fabrication industry, consultants and other major users of structural steelwork.

CAPS is set up to handle beam and column structures of any geometry using a rationalised basis of design for all connections in accordance with British or National Standards.

All connections are designed automatically except where they exceed the design criteria and these are reported to the user.

The system works to a set of preferences, for instance bolt pitch and sizes, but it will adjust where necessary.

Users can easily specify their own preferences by simple commands and can select, for example:
- Bolts from 4.6, 8.8 or HSFG grades.
- Rolled sections, made-up section plate girders and also AUTOFAB.

Bolted and welded connection types available include:
- Bolted moment connections using top and bottom angles or tees.
- Welded moment connections with or without haunch.
- Double web cleats.
- Welded end plates.
- Splices, base plates.

CAPS will handle automatically:
- Eccentricity and skew, both horizontal and vertical.
- Extra fittings, etc., extra holes, shear studs, ledger angles, unusual stanchion bases, stiffeners.

CAPS will automatically check for:
- Bolt fouling.
- Notching.
- Adequate connection strength.

Users may nominate the erection sequence by giving the boundaries of 'blocks' in terms of grid lines and floor levels and the order of the blocks determining the erection sequence.

The output produced by CAPS is extremely comprehensive and includes the following schedules automatically:
- Cost estimates - materials and workmanship.
- Scheme drawings - marked and cross-referenced.
- Detail drawings - with piece marks, standard symbols and an index.
- Material lists.
- Cutting and drilling lists.
- Shop bolt lists.
- Site bolt lists.
- Erection sequence lists.
- Numerically controlled tool tapes.

All CAPS operations are carried out in a fraction of the time they would take by traditional methods.

For further information please contact:
BUROTEC, 53 Goldington Road, Bedford, England.

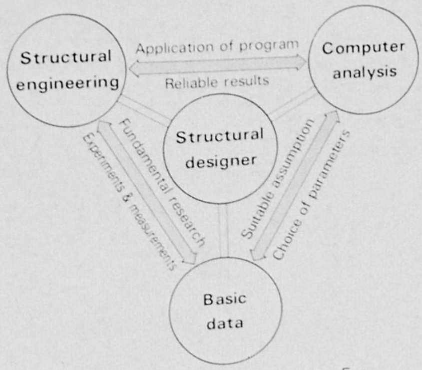
Harmonious Development of Computer Analysis and Structural Design

Dynamic Analysis of a Tall building under Strong Earthquakes

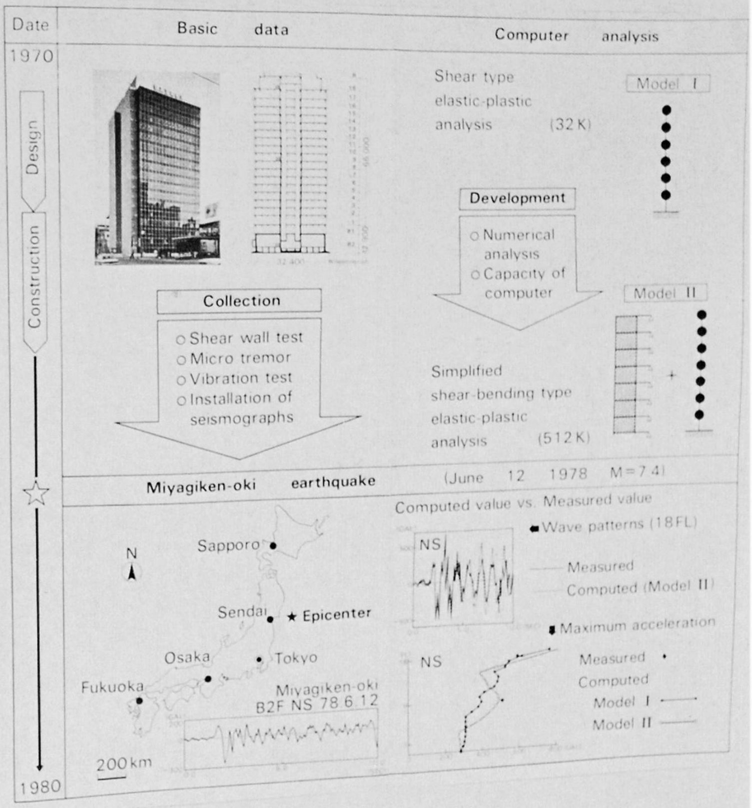
NIKKEN SEKKEI
planners architects engineers
JAPAN

T. AOYAGI N. UCHIDA H. KIRIHARA
K. MATSUNAGA M. KAWAMURA

A computer plays the following two major roles in the field of structural design. One is the fast and accurate processing of data - a task which involves enormous volume of work but requires only simple thinking. The other is the computer's aid in theoretical analysis of structure, which has tremendously expanded the domain of numerical analysis. Future development of structural engineering depends, to a great extent, on the pursuit of theoretical analysis [illegible]ough understanding of the computer's roles, and at the same time, on the storage of basic data, both requirements having to be advanced in a well-balanced state. From this viewpoint, the authors introduce here their ten year experience obtained from the design thro' simulation analysis for a tall building (in Sendai city) which encountered strong earthquakes.



Structural design in Computer Era



" HARMONIOUS DEVELOPMENT OF COMPUTER ANALYSIS AND STRUCTURAL DESIGN "

Tsukasa AOYAGI
Chief, Struc. Dept.
Koichi MATSUNAGA
Struc. Engr.
Naoki UCHIDA
Senior Struc. Engr.
Masayoshi KAWAMURA
Eng. D., Chief Advisory Struc. Engr.
Hideaki KIRIHARA
Struc. Engr.

Nikken Sekkei Ltd, Osaka, Japan

Basically, the authors' poster is separable into two segments: a theoretical vision reflecting a structural engineer's roles in the computer era; and a typical example of such correlative event and study as envisaged in that vision. Thus, the following two became the main subjects of discussion with the audience.

☐ Discussion about "Structural Design in Computer Era"

Most of audience favorably agreed to this vision, with the related discussion covering the following.

1) Development of Program: At the authors' firm, the typical way of developing computer programs is that related engineers, say, those at the computer department and the structural department associate themselves with each other by establishing an adhoc task force on a specific project. When this was explained to the audience, most of them admitted that this approach was the most preferable way because the structural engineers' participation in programming would certainly make the program easier to operate and more effective, but some audience commented that this approach was difficult to follow in their country because of professional separation of analysis from design in practice.

2) Operation of Program: The authors explained that they used their own computers installed for in-house use, viz. two machines of IBM 370-M138 and associated terminals. Besides these machines, small computers of about 100 K are provided at the structural departments for simple calculation and analysis. Though all these programs being developed for in-house use, two major programs for integrated structural analyses as authorized by the Minister of Construction are made available to outside structural engineers under licensing agreements. Such positive attitude of the authors toward computer use was favorably accepted by the audience.

☐ Discussion about "Typical Example (Dynamic Analysis of a Tall Building under Strong Earthquakes)"

The practitioners for whom seismic force is an indispensable consideration made wide-ranging inquiries, from aseismic design methods in Japan to assumption of coefficients for the analyses, eventually being followed by rewarding discussion. They regarded this example as very rare and invaluable study which certainly helped to provide a basis for the harmonious development of computer analysis and structural design.

Thus, the authors acknowledge that the poster session afforded the authors with an otherwise unobtainable chance for better communication among participants to have them thoroughly understand the authors' philosophy. With their attitude so successfully supported, the authors wish to develop further study.