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SUPPLEMENTARY SESSION

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Session IV — Etudes de cas

IV. Sitzung — Fallstudien

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COLLOQUIUM on:
"INTERFACE BETWEEN COMPUTING AND DESIGN IN STRUCTURAL ENGINEERING"
August 30, 31 - September 1, 1978 - ISMES - BERGAMO (ITALY)

Computerized Structural Analysis of Cantilever Bridges

Calcul statique de ponts à travées en porte-à-faux, au moyen de l'ordinateur

Computergestützte Berechnung von freitragenden Brücken

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Summary

The amount of calculations for a complete structural analysis of post-tensioned cantilever bridges is very extensive due to the stepwise construction and the live loads. The main features of a special computer programme (CoBe) for these calculations are described. Some opinions pertaining to the rational and safe use of computer programmes are put forward.

Résumé

La quantité de calculs est très grande pour le calcul statique complet de ponts construits en porte-à-faux, car il faut tenir compte des états successifs de la construction ainsi que des charges utiles. Les aspects principaux d'un tel programme spécial (CoBe) sont présentés. Quelques opinions sont émises sur l'utilisation rationnelle et sûre des programmes d' ordinateur.

Zusammenfassung

Wegen dem schrittweise ausgeführten Bau und den Nutzlasten, ist die Anzahl der Berechnungen für die Gesamtberechnung einer vorgespannten, freitragenden Brücke sehr gross. Die Grundcharakteristiken eines besonderen Computerprogramms (CoBe) werden für diese Berechnung beschrieben. Gedanken im Zusammenhang mit wirtschaftlicher und sicherer Benützung von Computerprogrammen werden unterbreitet.

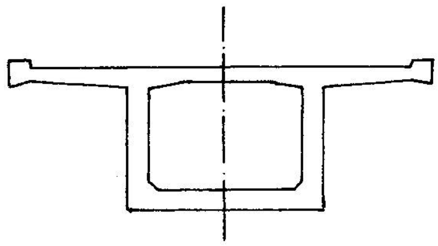
1. INTRODUCTION

Since 1960 about 50 bridges of the posttensioned cantilever type with spans from 60 m to 210 m have been built in Norway.

The cross section of the beam usually is a single box cell with cantilevers as shown in Fig. 1.

FIGURE 1

TYPICAL CROSS SECTION



The amount of calculations connected with a complete structural analysis of this bridge type is very extensive. The main complication compared to other bridge types is the stepwise construction which involves analysis of a series of structural systems gradually loaded and additionally influenced by creep.

Further there is considerable work connected with the calculation of the effects from mobile loads (live load).

The first bridges were calculated by hand, but for the above mentioned reasons designers soon started searching for aids that could relieve them of the greater part of the numerical calculations.

At that time (at the end of the sixties) the programmes available in Norway were for general 3-dimensional frames.

However, none of these had the possibility of calculating the step-by-step construction in a reasonable way. In addition the designer had to calculate the node coordinates, the cross section constants, the necessary load cases etc., beforehand. For these reasons it was decided to work out a special computer programme that should be able to execute all the necessary calculations in connection with a complete structural analysis of such bridges as 3-dimensional frames.

In the programme specification it was emphazised that the data for the programme input should be transfered from drawing, sketches etc. with a minimum of effort for the designer.

It was also made a point that the designer should have the possibility of giving data in more ways and with different levels of accuracy according to the purpose of the analysis.

The first version of the programme appeared in 1970, the last one in 1978.

The development of the programme has been fully sponsored by the Norwegian Highway Authority.

2. WHAT IS THE PROGRAMME ABLE TO CALCULATE?

First of all displacements and forces of beam and supports, but also intermediate results that may be useful for the designer.

To give an impression of the capacity and the level of automatization rendered by the programme a survey of the main input data and results is presented.

2.1 Reference line, system line

Parameters are given for a reference line, usually the centre line of the road or a line parallel to the centre line.

The horizontal alignment may have straight lines, circles and spirals.

The vertical alignment consists of straight lines and parabolas.

The system line is defined as a line through the centres of gravity in the beam. The distance between the reference line and this line is calculated on the basis of the cross sectional data, hence the coordinates and bearing of all the beam-nodes are determined.

2.2 Cross sectional constants (beam)

These may be specified by the designer or calculated in the programme on the basis of shape and dimensions of the cross sections. The cross sections are symmetrical to a vertical plane through the system line.

Otherwise they may have any shape and the dimensions may vary at random along the bridge.

2.3 Tendon forces/moments

Effective tendon forces may be specified by the designer or calculated in the programme based on tendon alignment, prestressing forces and parameters for losses etc. The prestress moments (load) are calculated on the basis of effective force and eccentricity of the tendon in the cross sections.

Length of tendons, elongations during tensioning and effective forces/moments are intermediate results presented in the output if wanted.

Effective forces/moments are introduced as loads for the structural analysis.

2.4 Stepwise construction

To describe the variable time, structural system and load relationship the designer may specify the following activities:

- activate beam segments.
- " prestressing tendons.

- activate dead load.
- introduce, move and remove travellers.
- introduce/remove supports/hinges.
- execute structural calculations.
- move the point of time and calculate the effects of shrinkage and creep during the preceding period.

Displacements and forces can be printed out at any point of time.

To save time the designer has the possibility to use the LOOP-concept enabling him to specify series of activities in a very short way.

The results (displ. and forces) from the different points of time are added and saved for later use.

The programme may of course also calculate bridges which are not stepwise constructed.

2.5 Live loads

The live loads for bridges are specified by various National Bridge Load Codes.

Although such Standard Live Loads have more or less different compositions they have one common denominator: The loads have to be positioned in such way that the maximum effect on the sectional forces is obtained.

To this end the programme will calculate influence lines, search for the dimensioning load position and compute the maximum (or minimum) effect in accordance with the rules of the specified Standard Loading.

Thus the designer may receive results from hundreds of load cases by giving only a few lines of Live Load input (type, magnitude of loads and position of lanes).

At present the programme can handle Live Loads set out in the Internordic Standard Loadings for Highway Bridges, however, if required any Standard Loading can be built into the programme.

2.6 Combination of results

The Norwegian Bridge Code defines three combinations of loads:

1: ordinary loads.

2: dead load + prestressing + extraordinary loads.

3: ordinary + extraordinary loads.

The following loads are defined as ordinary:

- dead load.
- prestressing.

- creep.
- shrinkage.
- water pressure.
- earth pressure.
- standard live load.

The following loads are defined as extraordinary:

- wind pressure.
- temperature.
- brake forces.
- ice pressure.
- collision forces.

The different loads may be of the following types:

- ordinary/extraordinary.
- permanent/transient.
- independant/alternative.

Two limit states are defined:

- serviceability limit state.
- ultimate limit state.

With different load factors for the different load combinations/ load types/limit states it is easily understood that the number of calculations connected with the combinations of loads may be quite extensive.

However, the programme carries out these combinations in a speedy and reliable way and presents the results in well arranged tables.

3. FURTHER DEVELOPMENTS

In connection with projects in development countries some additional features will be added to the programme in the near future:

- the possibility of having all texts in English.
- British Standard Highway Live Loads.
- earthquake response calculations.

A logical extension of the programme would be to include the calculation of the necessary capacities of the beam sections. This has not been done until now because of the rapid change and national variations of the design rules.

4. RATIONAL USE OF THE PROGRAMME

A necessary precondition for the rational use of this and other computer programmes is that the designer should have an understanding of the underlying theory and its implications to such an extent that he in principle could have done the calculations himself.

Rational use also means that the designer will estimate the most critical features beforehand and will compare these with the computed results. The estimates may be carried out on the basis of more or less exact formulas or based on modifications of similar existing structures.

Such procedure will have more advantages:

- errors in data and/or programme can be detected and corrected at an early stage prior to possible serious consequences.

- the basis of the estimate may be improved which in turn leads to fewer computer runs, faster design and less cost.

When the programme is used in this way the risk of serious errors is minimized, the designer is relieved of timeconsuming and costly numerical calculations and the effects of changes in the structural system can be fast and economically calculated.

It is on the other hand quite clear that computer programmes may be misused.

Poor understanding, time-pressure and the absence of adequate checking are the main reasons for miserable results, however, these conditions would also jeopardize the calculations if they were carried out in the old manner.

During the design period it may also sometimes be observed that extensive calculations are run after only small changes in the structural system. This is due to lack of understanding and experience on the hand of the designers and should be prevented by proper supervision.

The supervision of computer-calculations may in most cases be limited to checking the structural system, loads and the reasonableness of the results. If the supervisor is unacquainted with the programme or the structure is complicated he may carry out his own calculations either by hand or by mean of another computer programme. The clear documentation of the structural system, loads and results obtained by the use of a computer programme is in every case of great value to the supervisor.

5. CONCLUSION

General computer programmes do not always suit the designers need for calculating assistance. If there is sufficient know-how and capacity available, the development of a special programme may be a possible solution to the problem.

The advantages of making programmes "in house" are several:

- the programmes may be well adapted to the problems in such a way that a high degree of automatization is obtained.

- changes in the specifications may be built in when required.

- the application is simpler and safer because people with intimate knowledge of the programmes are readily available.

The disadvantages are mainly the high costs involved in programme development and maintenance.

Rational and safe use of well suited computer programmes relieves the designer of timeconsuming, tedious and costly numerical calculations and leads to faster and safer design at a lower cost.

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"BC: Bridge Construction" a Case History

"BC: Construction de ponts": une étude de cas

"BC: Bau einer Brücke" ein Studienfall

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Summary

The general methodology followed to develop a large program is presented. This methodology is aimed at standardizing the communication between the engineers and the program. It is based on a simple in-house package which facilitates the programmer's work. A high quality documentation, yet limited to the minimum, is produced.

Résumé

Cet exposé a pour but de présenter la méthodologie générale suivie lors du développement d'un grand programme. Cette méthodologie est basée sur la standardisation de la communication entre l'ingénieur et le programme. Elle est basée sur un package simple-maison qui facilite le travail du programmeur. La documentation produite est limitée au minimum mais elle est de bonne qualité.

Zusammenfassung

Die generelle Methodenlehre wird für die Entwicklung eines grossen Computerprogramms dargestellt. Diese Methode soll die Kommunikation zwischen dem Ingenieur und dem Programm vereinfachen. Sie ist auf einem eigenen "Package" begründet und erleichtert die Arbeit des Programmierers. Die Dokumentation ist aufs Minimum beschränkt, aber doch von hoher Qualität.

1. GENERAL

1.1. The Firm

Europe Etudes Gecti is a Civil Engineering consulting firm which essential specialization is prestressed concrete. It employs approximatly 300 persons in France, scattered in nine locations. It has five subsidiaries in different countries.

EEG has been involved in the design of many outstanding prestressed concrete structures (offshore platforms, Montreal olympics....) and is currently involved in the design of the Key West bridges in the US and of F9 freeway in Australia.

1.2. Data Processing

The duty of the EDP (electronic data processing) team is to make programs available to the engineers. The EDP team doesn't use the programs itself but for testing purposes. The programs results are placed under the final total responsibility of the engineers.

The service provides user's manuals which are the communication tool between the machine and the users. Training sessions are also organized.

Programs are only developped when they are not available somewhere else. This implies that the service has access to the most important computers available. This is done with the use of CADUCEE which is a high quality data processing network.

2. THE CASE

A simulation program for the verification of prestressed concrete linear bridges had to be developped. In house, programs were already available but did not meet new requirements. The problem related with this type of bridges is that each individual construction phase must be checked. A three span bridge for example may require up to one hundred computations, each done with a structure slightly different from the previous one.

The existing "chain" of programs was very limited both in the methods of construction it could modelized and in the mathematics it was based on. In addition these old programs were very difficult to use by the engineers and even more difficult to either maintain or modify.

Beside the strength of materials and mathematical requirements, which are not the topics of this conference, the objectives were :

- A verification program which could be used for design.
- An engineering oriented program
    - . user's oriented input language
    - . high quality output
- Stand alone user's manual
- An international target
    - . easily adaptable to any bridge regulation
    - . an output translatable into different languages
- A good quality in house documentation.
- A program which has to run on both IBM and CDC.

The final product is a program called "BC:BRIDGE CONSTRUCTION".

The methodology presented in this paper is the result of an evolution which started over four years ago. The large project represented by the building of BC enabled to standardize the approach.

3. THE METHODOLOGY FOLLOWED

The final objective is to obtain a complete program which satisfies the needs of the users.
A program can be considered as complete when :

- it can be used without a direct contact between the engineer and the programmer
- documentation is provided to satisfy
  . the client who is going to look at the output
  . the user
  . the programmer who will have to maintain the coding
  . the technical management who is going to request future changes

The final documentation is standard at EEG. It is illustrated on figure 1.
It contains :

- The USER'S MANUAL which is itself split into two basic levels
  . the general and client oriented level (Presentation, Hypothesis and conventions, presentation of results). This part can be referred to as the functionnal specification of the program.
  . the user's level (Data input, methodology, running the program, tests and examples, error messages).

- The EXTERNAL SPECIFICATION which is a complete desciption of the program and associated information. It contains :
  . A description of the overall program structure
  . The mathematics on which the program is based
  . The data base definition (either in core or on disk)
  . The functional description of each module and the detailed algorithm when necessary
  . The graphs representing the user's oriented input language.

- The INTERNAL DOCUMENTATION which is directly included in the FORTRAN coding.
This list contains from 30 to 50% of comments among the final 70000 cards of BC.

- The TESTS DOCUMENTATION where the main tests are logged together with their results.

3.1. The design phase

The programming of an application like BC is a permanent iteration process which converges towards finer and finer documentation. It is basically a topdown analysis which links with some existing utility modules or sets of modules.

The initial request usually comes either from the technical Management or from the Data processing itself.

The user's manual is used to express the functional requirements. It is written by the EDP team. Only a skeleton of the manual is first written, it will gradually "grow" into the final version.

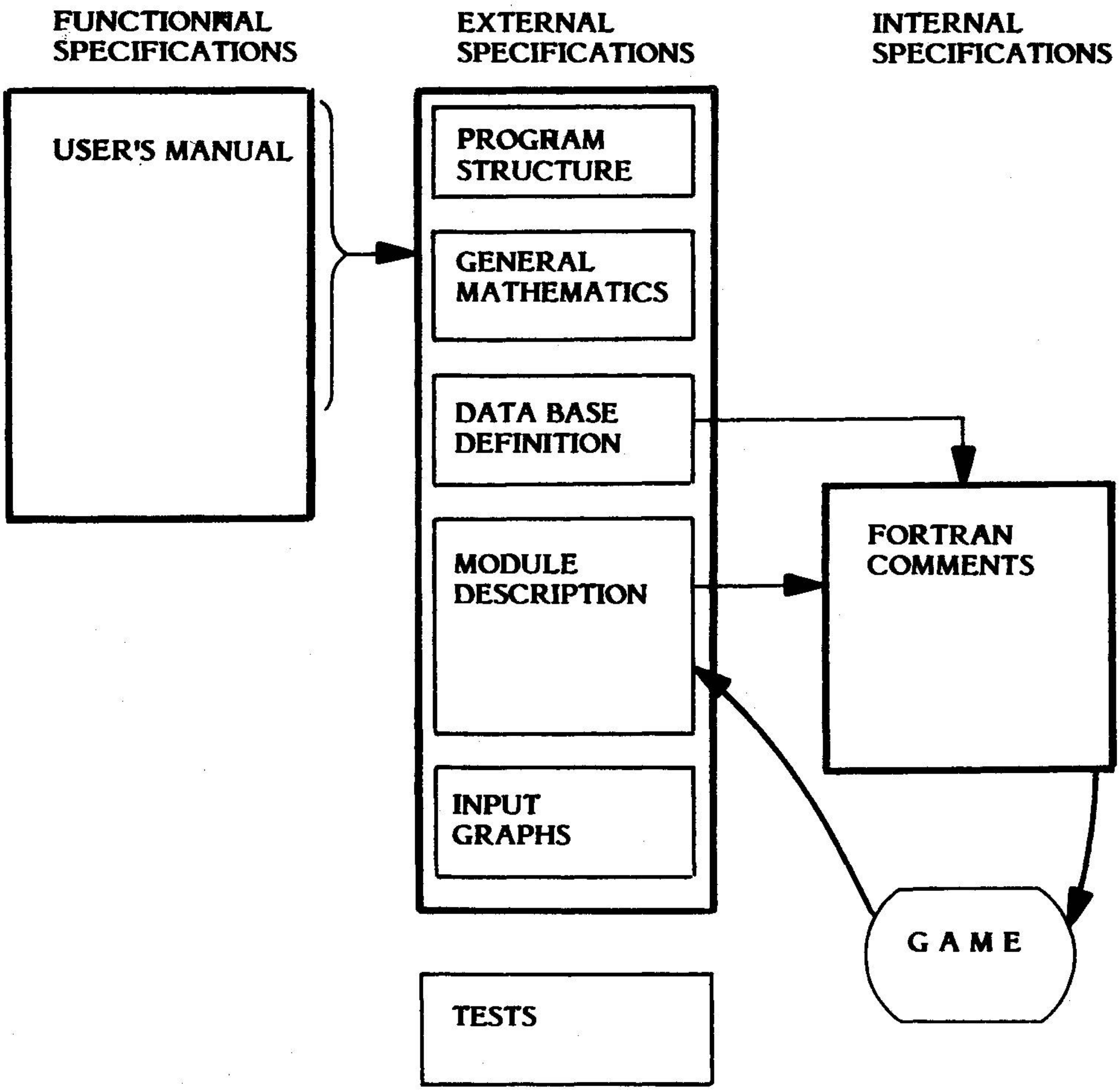

Figure 1

Structure of the Documentation

The initial manual only contents the following section :

- presentation
- hypotheses and conventions
- presentation of results
- data input (very brief)

After a few iterations between technical management and EDP team, the external specificationsstart growing, beginning with the program structure.
As soon as possible cost estimates are made and "Go ahead" or "kill" decisions are taken.

3.2. The GO AHEAD phase

Once the GO AHEAD has been granted, the external specifications are detailed

- the data base is defined
- the graphs of the user's oriented language are established
- the modules are described

Because a top down approach is followed, all the modules need not to be defined before programming begins.

3.3. The Programming phase

The programming is done in a very standard FORTRAN IV as the program must be able to run on both IBM and CDC machines.

Structured "egoless" programming is used. It is based on very strict written rules which were discussed by the team before being applied. Figure 2 shows an example of the coding produced.

The basic rules which are followed can be summarized as :

- it must be possible to follow the basic logic of a module by simply reading the comments
- the text is indented as requested by branches, computed GOTO's or DO LLOP's.
- the statement numbers normally contain five digits
    - . two for the chapter number
    - . two for the section number
    - . one for the part number. A zero part number represents a "clean" instruction and a non zero one an instruction which is implied in a surrounding flow.
- variable names are formed by using two letter codes common to the entire program.

The functional description of each module is placed as comments at the very beginning of the list following the "SUBROUTINE" or "FUNCTION" statement.

One of the difficulty found is to keep the initial functional hand written description of the module up to date while some functions are slightly modified. To overcome this problem, it was decided to throw away the initial hand written functional description and replace it by that contained in the FORTRAN source. This new functional description is obtained automatically by a specific program.

```
      SUBROUTINE KPRT1
C
C PURPOSE
C -------
C     PRINT A PHASE RESULT
C
C INPUT
C -----
C     PRINTING CODES FOR EACH OF THE LOADCASES
C     STARTING AT ADDRESS  IDPRT WITH NBPRT  VALUES
C     FOR EACH LOADCASE WHICH ARE IN ORDER
C               1 != DEAD LOAD
C               2 != SURCHARGES
C               3 != TOTAL OF ALL LOADS
C               .
C               .
C        NBLDPR != PRESTRESSING TOTAL
C        NBLDMX != MAXIMUM NUMBER OF LOADINGS IN R.F.
C
C     PRINTING CODES ARE IN ORDER
C               0 != 1 SOMETHING TO PRINT
C                    0 NOTHING TO PRINT
C               1 != STATE OF THE STRUCTURE
C               2 != DISPLACEMENT OF THE NODES
C               3 != REACTIONS AND ELASTIC SUPPORTS
C               4 != INTERNAL FORCES


C
02024           CONTINUE
C               **** THIS COORDINATE IS SLAVED
                NDI=NRK
C               .... IS I NODE ACTIVE ?
                IDNDI=IADNO(NDI)
                IF(IDNDI)02025,02025,02026
C
02025               CONTINUE
C                   .... I NODE IS NOT ACTIVE
C                   .... NUMBER THIS COORDINATE WITH K
                    NBCO=NBCO+1
                    IBUF(IDNRK+3)=NBCO
                    GOTO 02020
C
02026               CONTINUE
C                   .... NODE I IS ACTIVE AND WILL BE NUMBERED
C                   .... AT THE END OF THE LOOP
                    IWAIT=IWAIT+1
                    CALL GETVAR(IDW,2)
                    IBUF(IDW)=IDNRK+3
                    IBUF(IDW+1)=IDNDI+KOORP1+3
02020       CONTINUE
02000 CONTINUE
C
C     **** FINAL LOOP TO NUMBER THE LEFT OVER SLAVE COORDINATES
```

Figure 2

Egoless FORTRAN

3.4. GAME : An in house package

One of the problem of communication between engineers and the programs is that of the standardization of both the input and the user's manual. Using a new program requires a time for discovery.

This discovery goes through the following phases :
- global look at the manual
- detail look at the hypothesis
- understanding of the input
- trying to use the program

It is highly important to ease the task of the engineer by simplifying this discovery. This can be achieved if :
- all user's manual have the same structure
- all user's manuals have the same presentation
- all programs use the same user oriented input language

This is one of the goal of the in-house package GAME (Graph - Analyzer - Memory - Editor).

GAME comprizes four main functions (see figure 3) :
- a syntaxic language analyzer
- an output formater and an editor
- a memory manager
- a specification regenerator

These basic functions can be found in large systems like GENESYS or ICES. However GAME is a very basic simple tool, which runs on IBM and CDC, and which does not require a complex environment.

3.4.1. The syntaxic language analyzer.

This analyzer is made of two parts

- a table builder program which takes the programmer's description of the requested input language to build the necessary FORTRAN tables. These tables are included in the program being developped

- a series of routines which analyze user's input data (see figure 4) according to the above tables and returns to the programmer the decoded informations. A series of basic functions are also included in the language and are therefore common to all programs. (editing functions, setting program parameter functions).

3.4.2. The output formater and editor.

This is a tool to the programmer. It consists of two parts :

- a program which takes as input all the formats containing some alphabetical informations to create a format file.
- a series of routines which are called instead of the standard write. Figure 5 shows a typical output together with the summary of the job which is automatically generated.

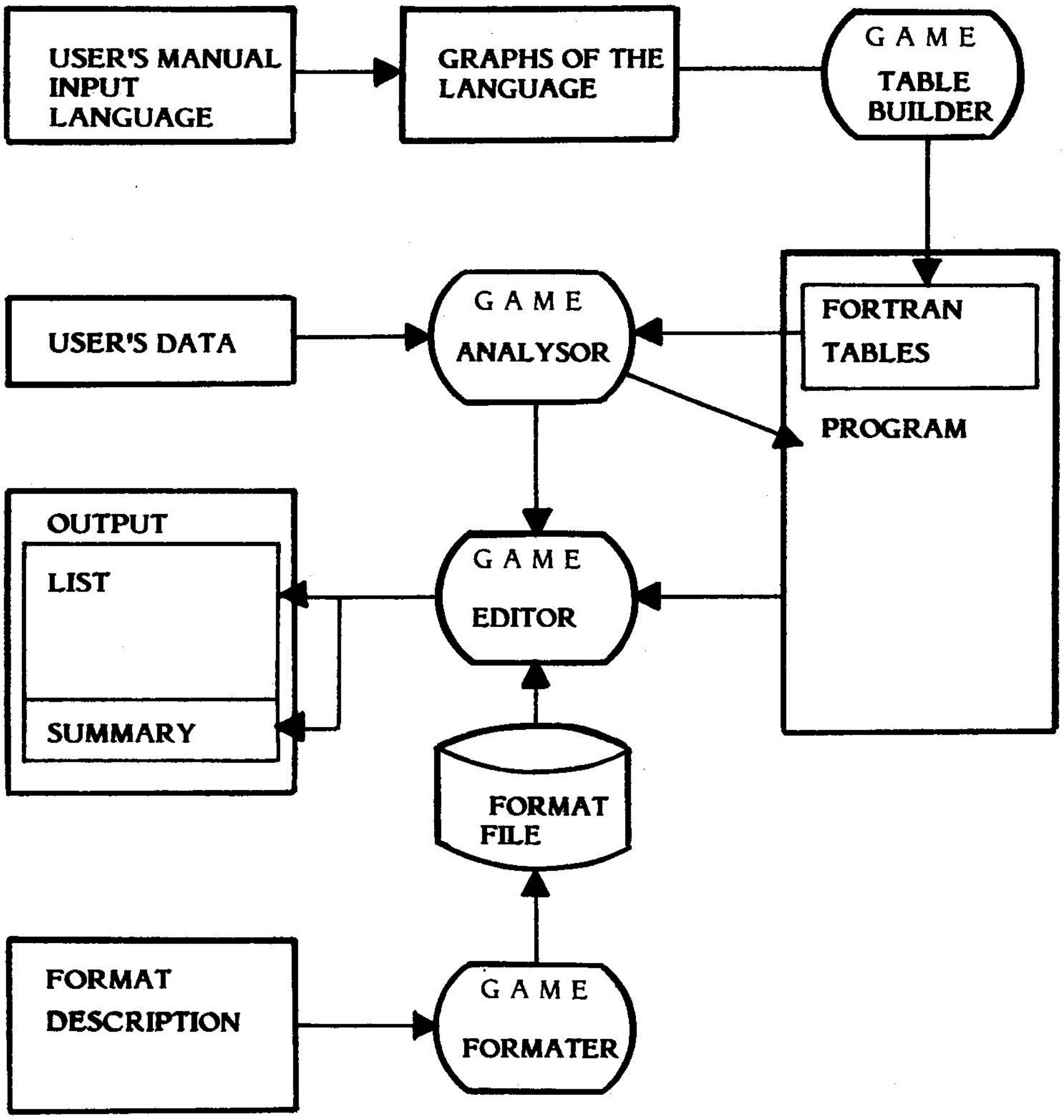

Figure 3
Functions of the package "GAME"

```
EUROPE - ETUDES                PROGRAMME  BC                    APPENDIX -B-
CONTRACT  26203.00       A 35 OA 203
06/15/78                                                        PAGE     1

PARA 84  0 / 88 2 / 27 3 / 163 2
CONTRACT 26203 01 / A 35 OA 203
TITLE EEG STRASBOURG A 35 OA 203 PONT SUR LE CANAL DU RHONE AU RHIN
TITLE DEFINITION S1
REPORT
DEFINITION
MATERIAL
CONCRETE 10/ BETON STANDARD
STEEL 1 / 8 T 15 CLASSE 3 TBR POUR CABLES D ENCORBELLEMENT
   AREA 0.001112
   GUTS 184900 JACK 157000
   FRICTION 0.17 WOBBLE 0.0016 DRAW 0.005
   RELAXATION INITIAL .55 10.3 .8 13.1
STEEL 2 / 8 T 15 CLASSE 3 TBR POUR CABLES DE CONTINUITE
   AREA 0.001112
  GUTS 184900 JACK 157000
   FRICTION 0.17 WOBBLE 0.0016 DRAW 0.005
   RELAXATION INITIAL .55 17.7 .8 18.2
   RETURN
NODES 1 2 / 0 .50 -0.681        -.681
   3 THRU 10 / 4.27 THRU 30.52 STEP 3.75 -0.681 -0.684 -0.697 -0.727
* -0.785 -0.882 -1.028 -1.240 / 11 / 37.52 -1.849
  111 / 37.52 -1.849 / 211 / 37.52 -8.43
   12 THRU 19 / 44.52 THRU 70.77 STEP 3.75 -1.236 -1.024 -0.876 -0.776
* -0.716 -0.682 -0.666 -0.663 / 20 /73.97 -0.661 /21 / 76.11 -0.661
  22 THRU 29 /79.31 THRU 105.56 STEP 3.75   -0.663 -0.666 -0.682
* -0.716 -0.776 -0.876 -1.024 -1.236 / 30 / 112.56 -1.849
  130 /112.56 -1.849 / 230 /112.56 -8.43
  31 THRU 38 / 119.56 THRU 145.81 STEP 3.75 -1.24 -1.028 -0.882
* -0.785 -0.727 -0.697 -0.684 -0.681 /39/149.58 -0.681
   40 /150.08 -0.681
  SUPPORT V 1 40 H V R 211 230 RESTRAINT H V R 11 111 30 130
   RETURN
SECTIONS
   1 IB 19.341 AREA 8.476 C 1.849 CP 2.101 Q1 5.89  B 1 RESAL PARTIEL
```

Figure 4

Example of user's input data

```
EUROPE - ETUDES                    PROGRAMME  BC                    VERSION 1.38
CONTRACT  26203.00       A 35 OA 203
06/15/78                                                            PAGE     3

         INPUT  DATA  -GENERAL-             (CONTINUED)
         ----------------------

         MATERIAL PROPERTIES   (CONTINUED)

         STEEL       1   1    8 T 15 CLASSE 3 TBR

               RELAXATION COEFFICIENTS

               RELATION RELAXATION/INIT.STRESS

                   INIT.STRESS              RELAXATION
               (PERCENT OF GUTS)    (PERCENT OF INIT.STRESS)

                      .550                    10.3
                      .800                    13.1
```

```
EUROPE - ETUDES                    PROGRAMME  BC               APPENDIX -A-
CONTRACT  26203.00       A 35 OA 203
06/15/78                                                       PAGE     1

         ---------------  S U M M A R Y  ---------------

     INPUT  DATA  -GENERAL-                                             1
     -----------------------------

            MATERIAL PROPERTIES                                         2
            COORDINATES OF THE NODES                                    5
            DATA OF SUPPORTS                                            6
            DATA OF INTERNAL RESTRAINTS                                 6
            CROSS-SECTION GEOMETRIC PROPERTIES                          7
            CROSS-SECTION TEMPERATURE PROPERTIES                        7
            DATA OF ELEMENTS                                            8
            DATA OF SCAFFOLDS                                          10
            DATA OF UNIFORM LOADS                                      10
            DATA OF PRESTRESSING                                       11
                   STANDARD EXTREMITIES                                11
                   EXTR NB.     3                                      11
                   EXTR NB.     4                                      11
                   EXTR NB.     5                                      11
                   EXTR NB.     6                                      11
```

Figure 5

Typical output and summary

3.4.3. The memory manager . (see figure 6)

This is a set of simple subroutines which manages the in core memory. It prevents using the cumbersome FORTRAN rigid DIMENSION statement.

In addition, a standard memory organization enables the user to access program parameters at any time during his natural flow of data.

For example :
Normal flow of data
"CONSTRUCTION PHASE SEGMENT ASSEMBLE ALL"

Setting parameter 200 to 1 before assembling :
"CONSTRUCTION PHASE SEGMENT PARA 200 1 ASSEMBLE ALL"

Because this specific data input is handled directly by the package, it is common to all programs. Here again, discovery is made easier to the engineer.

3.4.4. The specification re-generator.

This is a very simple module which extracts and prints comments from a FORTRAN list. It does it with two options :

- prints the functional description comments only (i,e the comments place at the beginning of a module)
- prints the above comments plus all those found in the coding

The output from this program replaces the hand written functional specifications used initially. The new specifications are therefore always up to date as the programmer easily accepts to update the comments in the FORTRAN list at the same time as he modifies the coding.

4. PROGRAMMING THE REGULATIONS

One of the difficulty when programming traffic regulations for bridges is that they are subject to change and that they are different from one country to the other. This is always a source of problems to a programming team, more so when the user is thousands of kilometers away.

The decision was taken to implement in BC a user oriented traffic regulation language. It enables the user to describe the logic he requires. He does so using elementary orders which deal with variables and operators located at a very high level (trucks, influence lines, combinations, searches....).

Standard traffic regulations (AASHTØ, CPC) are available by default but they can either be modified or completely changed upon data input.

Just to give an idea, only approximately 70 statements are sufficient to represent the AASHTØ requirements.

This gives the user a possibility to gain confidence into the program in an area which is usually only accessed by the programmer.

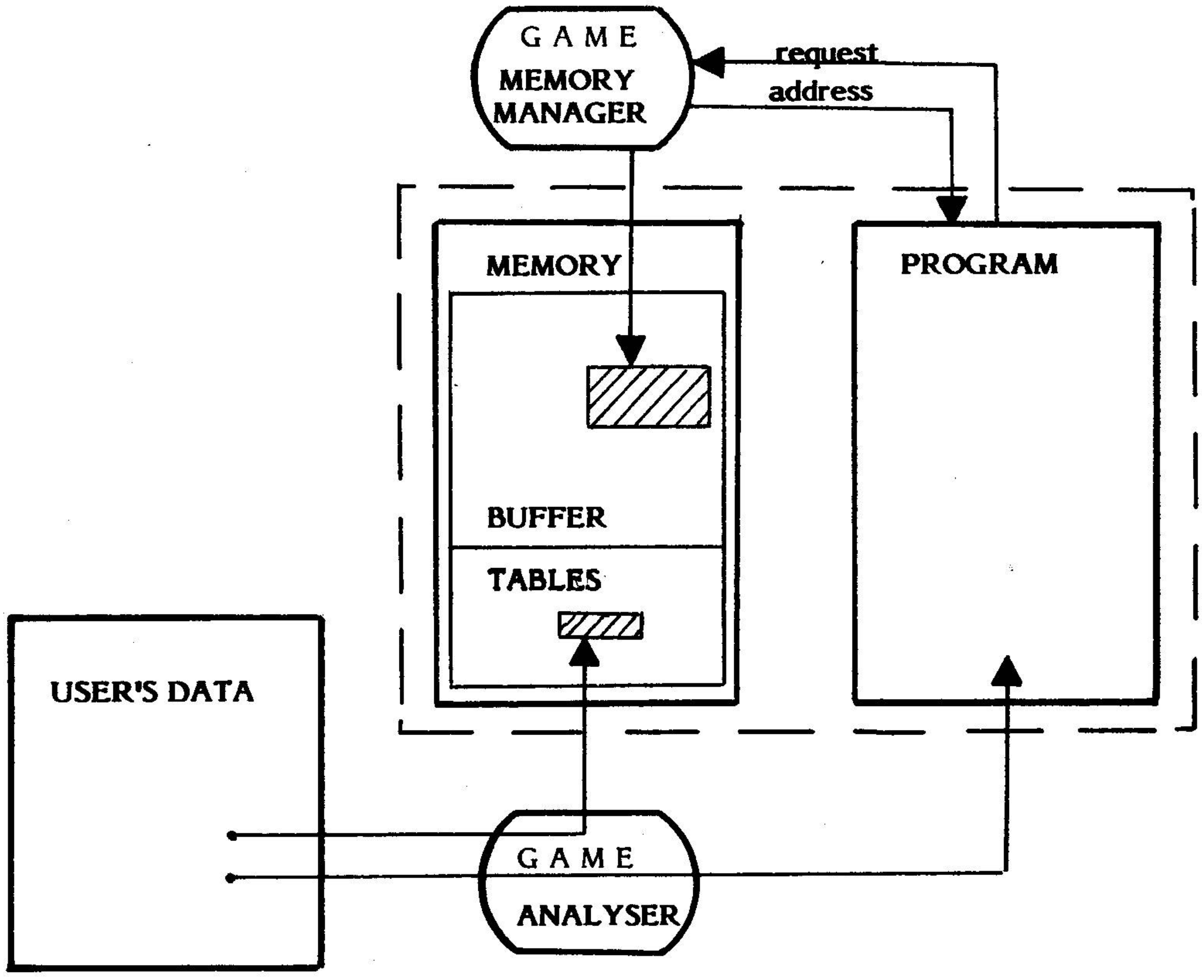

Figure 6
The memory manager

5. THE USER'S MANUALS

A full time secretary is in charge of updating, printing and dispatching the user's manual. Only one working copy is available to the EDP team. Upon correcting this copy, the programmer tags the page to be modified. Every so often, if there is no emergency,the secretary updates the original and goes ahead with the dispatching.

The entire user's manuals are kept on cassettes using a Xerox 800 machine.

The updates can be done at a chapter, a section, a part or a page level which is the smallest entity which can be updated.

The contents of the manual is standard. The following sections can be found :

- Presentation
- Hypothesis and conventions
- Presentation of results
- Data Input
- Methodology
- Running the program
- Tests and examples
- Error messages

Each page of the manual can be clearly located in its position. (See figure 7). A "control" sheet is sent together with each update thus giving the opportunity to the user to check the entire contents of his manual.

The mailing list of BC contains as of today 65 addresses.

6. CONCLUSION

This paper was supposed to present a case history. Because EEG's EDP team has continuously aimed at standardizating it's methods,the resulting paper is more a presentation of the general methodology followed.

The key points of this methodology are :

- a standardization of the interface between program and engineer
- a high quality documentation
- a documentation which evoluates in order to be kept to a minimum.
- a simple in-house package which is used for data input, output, memory management and documentation generator.

E · E CALCUL SCIENTIFIQUE	CHAP : 20 - PROGRAMME B C	REF : 2.20.4.2
	SECT : 4 - Data Input	PAGE : 42
VOL : 2		DATE : 24.06.76
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The following set of commands is available :

```
                  { AXIS a      }
                  { SIDEWALK w  }
---■   PRØFILE  n { CURB        }
                  { ROADWAY r   }
                  { SPACE s     }
                  { RAILING     }
              1 ________________/
```

where :

PRØFILE n gives and I.D. number to the profile. This number will be used when defining spans/piers. Usually only one profile will be necessary for a given bridge.

AXIS a enables the user to position the profile with respect to the vertical x - y plane.

Figure 7

Frame of the user's manual

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COLLOQUIUM on:
"INTERFACE BETWEEN COMPUTING AND DESIGN IN STRUCTURAL ENGINEERING"
August 30, 31 - September 1, 1978 - ISMES - BERGAMO (ITALY)

Computational Aspects of a General Purpose Program for the Analysis of Creep and Shrinkage in Concrete Structures

Programme général pour le calcul des effets du fluage et du retrait du béton, du point de vue de la programmation

Computeraspekte in einem allgemeinen Programm zur Berechnung der Answirkungen des Kriechens und Schwindens im Betonbau

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Summary
Two examples are presented which show the requirements of a general - purpose program for the analysis of creep and shrinkage in concrete structures. The impact of the different construction phases and of the different design standards on the formulation of the stress-strain law and on the data and program organization are discussed. Results of the two calculations are presented.

Résumé
Deux exemples montrent la capacité qu' un programme pour le calcul des effets du fluage et du retrait du béton devrait avoir. On tient compte de l' influence de la réalisation de la construction en plusieurs étapes et avec différents systèmes statiques sur le comportement des matériaux et sur l'organisation du programme. Des résultats de deux calculs sont présentés.

Zusammenfassung
Die Anforderungen an ein Programm zur Berechnung der Auswirkungen des Kriechens und Schwindens werden an je einem Beispiel aus dem Hoch - und Brückenbau erörtert. Dabei werden die Auswirkungen der Baupraxis, z. B. der abschnittsweisen Herstellung des Bauwerks in Bauzuständen, auf die Formulierung des Stoffgesetzes und die Daten - und Programmorganisation erörtert. Für zwei Beispiele werden abschliessend einige Berechnungsergebnisse erläutert.

1. INTRODUCTION

With regard to the design of structural analysis programs including the effects of time-dependent behaviour of concrete (e.g. creep and shrinkage) attention is usually restricted to the mechanical and numerical aspects, such as the form of suitable stress-strain laws or the development of stable and accurate time-integration schemes.

In considering these undoubtedly important problems one often neglects the organizational aspects of the task resulting, for example, from the fact that concrete structures are always erected in a sequence of construction phases with often different structural systems, or, that cross-sections may be subsequently supplemented by concrete or prestressing cables. Moreover, the dead weight and the prestressing are usually set up in stages corresponding to the construction phases. This loading history may continue over the course of several years.

The first purpose of this contribution consists of discussing some stress-strain laws for the time-dependent behaviour of concrete from the point of view mentioned above. In addition this paper describes some organizational aspects of programs for the analysis of the effects of creep and shrinkage in concrete structures in conjunction with two example problems. Results of both examples are discussed.

2. STRESS-STRAIN LAW FOR THE TIME-DEPENDENT BEHAVIOUR OF CONCRETE

As most design codes [1], [2], [3], [4] include statements on uniaxial stress-strain behaviour only, the present discussion is restricted to one-dimensional stress-strain laws. In addition shrinkage, being independent of stresses, will not be taken into account in the following equations.

In formulating a stress-strain law for the creep behaviour of concrete, the validity of the priciple of superposition [5], [6] or a linear stress-strain law [7] is assumed. This assumption, which appears to be valid for stress levels up to about 30 % of the ultimate strength, is very useful for the analysis. This assumption and the assumption that the cross-sections of beams will remain plane, for example, result in a linear stress distribution over the cross-section.

Based on this assumption, the linear stress-strain law can be expressed as Stieltjes' integral [8]:

$$\varepsilon(t) = \int_0^t \frac{1}{E(\tau)}\left[1+\varphi(t,\tau)\right]d\sigma(\tau) \tag{1}$$

Here ε denotes the strain, σ the stress, E the time-dependent elasticity modulus, φ the creep function, τ the time at which the stress variation $d\sigma(\tau)$ occurs and t the time of observation.

The integral representation of equation (1) is unsuitable for implementation into a program due to the large amounts of data involved [9], [10].

An extremely simple stress-strain law can be obtained if the equation for a constant stress σ_0 applied at time t_0.

$$\varepsilon(t) = \sigma_0 \frac{1+\varphi(t,t_0)}{E(t_0)} \tag{2}$$

is also used for time-dependent stress histories $\sigma(t)$.

This results in the procedure of the effective modulus E_{eff} , which is based on the following equations:

$$\varepsilon(t) = \frac{\sigma(t)}{E_{eff}} \tag{3}$$

with

$$E_{eff} = \frac{E(t_o)}{1+\varphi(t,t_o)} \tag{4}$$

With this equation it is possible to calculate the effects of creep and shrinkage as in an elastic structure with the modulus of elasticity $E = E_{eff}$. However, as can be seen from equation (4) the effective modulus E_{eff} depends on the time of loading t_o and thus each loading case of the loading history will have its own effective modulus E_{eff} . If, as is the case in the two examples to be described later, the loading history consists of a large number of loading cases with different t_o the procedure becomes complex and uneconomical. Apart from this, the procedure also becomes inaccurate if a considerable change in stress $\Delta\sigma$ occurs between the loading time t_o and the time of observation t and the concrete ages considerably [11].

A similarly simple analysis of the effects of creep and shrinkage for one loading case in one step can be done with Trost's ϱ -formula [6].

$$\varepsilon(t) = \frac{\sigma(t_o)}{E(t_o)}\left[1+\varphi(t,t_o)\right] + \frac{\sigma(t)-\sigma(t_o)}{E(t_o)}\left[1+\varrho\,\varphi(t,t_o)\right] \tag{5}$$

The relaxation factor ϱ allows to take into account the influence of the aging of concrete more accuratly than by the effectiv modulus method. For this reason the procedure is also known in the USA as the "age-adjusted effective modulus method".

The great advantage of equations (3) and (5) is that they allow to calculate the stress resultants and displacements at the time of observation t for one loading case in one step (one-step procedures). However, if the various construction phases consisting of different structural systems are to be considered, the changes in stresses and deformations within each construction phase must be computed using the respective structural system. For this reason, the total changes in the stress resultants and the deformations up to the time t must not be calculated in one step (i. e., using one particular structural system). In such cases the above-mentioned advantages of the equations (3), (5) may no longer be relevant, and thus a rate type stress-strain law is preferable.

Let us now take a closer look at the differential equation (6) of the aging three-parameter solid (Fig. 1).

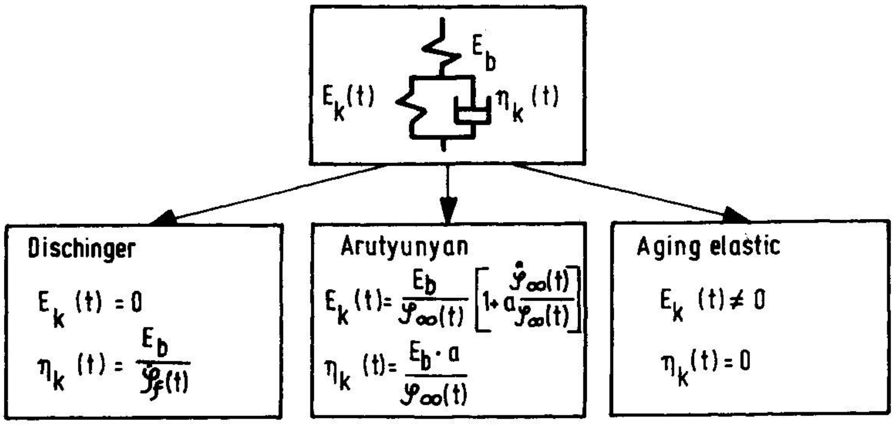

Fig. 1 Some creep laws which can be represented by the aging three-parameter solid

This model consists of a spring E_b and a Kelvin element with a dime-dependent spring $E_k(t)$ and a time-dependent dashpot $\eta_k(t)$:

$$\dot{\sigma}\left(1+\frac{E_k+\dot{\eta}_k}{E_b}\right)+\ddot{\sigma}\frac{E_k}{E_b}=\dot{\varepsilon}(E_k+\eta_k)+\ddot{\varepsilon}\eta_k \tag{6}$$

If we assume the functions $E_k(t)$, $\eta_k(t)$ to be piecewise constant, this differential equation can be transformed to an integro-differential equation suitable for numerical integration [10].

Up to now the aging three-element model has been used almost exclusively in the limited form of Arutyunyan's creep law [13], [14]. This creep law assumes the parameters $E_k(t)$ and $\eta_k(t)$ to be interconnected by the following equation:

$$a=\frac{\eta_k}{E_k+\dot{\eta}_k}=\text{const.} \tag{7}$$

a has the dimension of time and is usually called "retardation time".

The creep law of Arutyunyan yields the following equations for the creep function

$$\varphi(t,\tau)=\varphi_\infty(\tau)\cdot f(t-\tau) \tag{8}$$

with

$$f(t-\tau)=1-e^{-\frac{t-\tau}{a}} \tag{9}$$

In many cases, actual creep curves cannot be represented by eqs. (8), (9) with sufficient accuracy due to the exponential expression in eq. (9) [11].

These difficulties, however, can be avoided by omitting the assumption of a constant retardation time a; in this case the functions of $E_k(t)$ and $\eta_k(t)$ can be determined independently. This results in a large degree of adaptability to a large variety of creep laws (cf. Fig. 1).

In the special case

$$E_k(t)=0 \tag{10}$$

$$\eta_k(t)=\frac{E_b}{\dot{\varphi}_f} \tag{11}$$

we obtain Dischinger's creep law (rate of creep method) which yields the following equation for the creep function.

$$\varphi(t,\tau)=\varphi_f(t)-\varphi_f(\tau) \tag{12}$$

In this equation creep is described in terms of a flow function φ_f which is irreversible upon stress removal.

The equation for the creep function laid down in the prestressed concrete guidelines [3] and the CEB recommendations [1]

$$\varphi(t,\tau)=\varphi_f(t)-\varphi_f(\tau)+\varphi_v(t-\tau) \tag{13}$$

is obtained from e. g. (12) by adding the another term $\varphi_v(t-\tau)$ that is completely reversible when the stress has been removed, and which is referred to as "delayed elasticity".

This creep law, too, can be represented by means of the aging, three-parameter solid with an accuracy perfectly adequate for practical requirements. Delayed elasticity with a final value, as prescribed by the guidelines for prestressed concrete, of $0.4\,E_b$ can be taken into account if $E_K(t)$ is suitably determined. Also the development of creep in time can be approximated to a sufficient degree of accuracy by appropriately choosing the viscosity function $\eta_K(t)$ [15].

3. CREEP AND SHRINKAGE ANALYSIS OF THE "HYPOHOCHHAUS"

The "Hypohochhaus" is a 137 m high administration building presently under construction in Munich. In this building the vertical loads from all storeys are carried by a prestressed supporting floor half-way up the building which itself is supported, by four stairway towers (cf. Figs. 2 and 3).

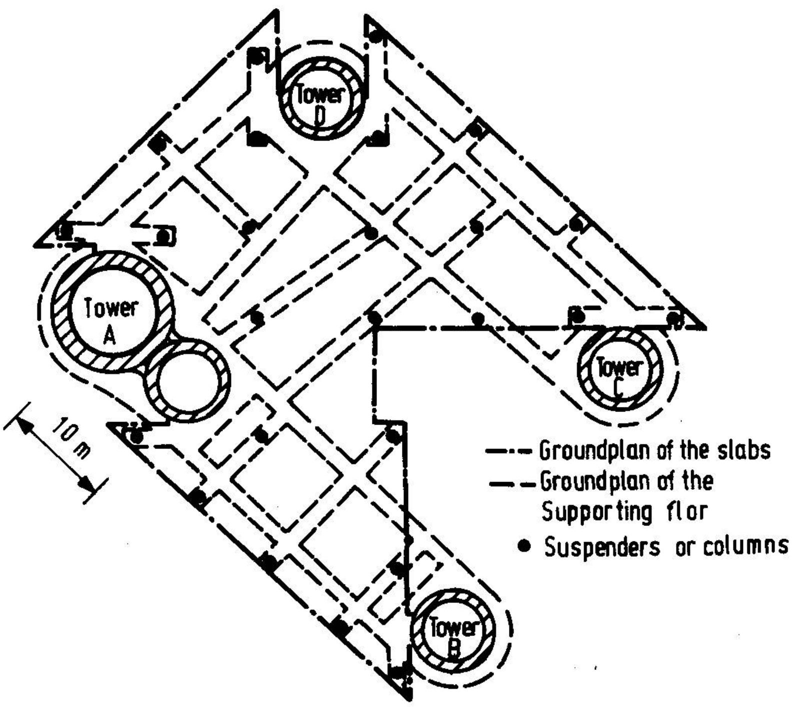

Fig. 2 Ground plan of the "Hypohochhaus"

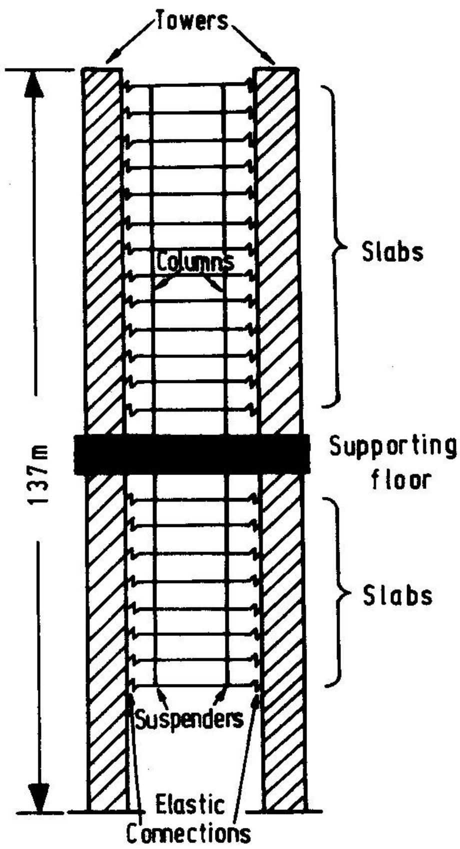

Fig. 3 Sketch of the structural behaviour of the "Hypohochhaus"

Thus, the slabs above the supporting floor rest on it, while the slabs below the supporting floor are suspended. In horizontal direction, however, the slabs are linked to the towers by means of elastic connections.

The slabs are constructed of lightweight concrete, to keep their dead-weight low; but the towers and the prestressed supporting floor are built of normal concrete. The considerable differences in the creep and shrinkage behaviour of these two materials (cf. Fig. 4) cause large changes of stresses,the history of which had to be calculated.

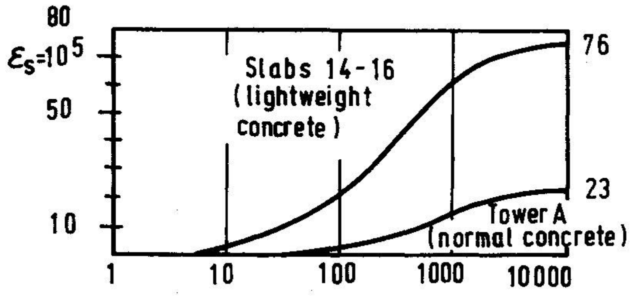

Fig. 4 Shrinkage of normal and lightweight concrete

For the creep and shrinkage analysis the structure is idealized as a space frame with 224 beams, 242 joints, 126 rigidity conditions and 36 hinged joints with elastic connections. The supporting floor is idealized as a grid structure. The 32 lightweight concrete slabs are grouped into nine blocks and each block is represented by a system of four beams, forming a cross. The moment of inertia of these beams I_2 is assumed to be very large and the cross-sectional area is determined, such that the total stiffness of a slab block connecting the four towers is represented correctly.

These simplifications are justified, since the main purpose of the analysis is not the determination of the state of stress in the slabs, but the history of internal forces in the elastic connections, the towers and the prestressed supporting floor.

The building is erected in a sequence of seven construction phases with different structural systems. The different slab blocks are constructed and linked to the staircase towers at four different instants of time.

The loading history consists of 8 permanent loading cases "dead and live loads" and 3 loading cases due to prestressing, all of which are applied at different times. The prestressing includes a total of 124 tendons in different positions. One cross-section could contain up to 48 tendons.

The creep and shrinkage analysis of this example is based on Arutyunyan's creep law. By distorting the time scale it is possible to approximate the prescribed material curves for the normal concrete and the lightweight concrete with sufficient accuracy using the exponential expression in equations (8) and (9). The significant differences in the creep and shrinkage characteristics of the different materials and the extremely varying effective thicknesses imply that the maximum stresses do not occur at $t \rightarrow \infty$ but at an intermediate point in time which is to be determined.

Therefore, the entire histories of the internal forces and deformations have to be computed in order to be able to find their maximum values.

These calculations are carried out for two cases: one, shrinkage alone, and two, shrinkage plus permanent load. Some typical results are shown in Figs. 5 and 6.

Fig. 6 indicates the remarkable fact that the force in the elastic connection of the first slab block above the supporting floor due to shrinkage alone is higher than the force due to shrinkage plus permanent load. The reason for this lies in the fact that the tensile stresses in the connections due to shrinkage of the slabs are considerably reduced by the compressive stresses induced by the permanent loads.

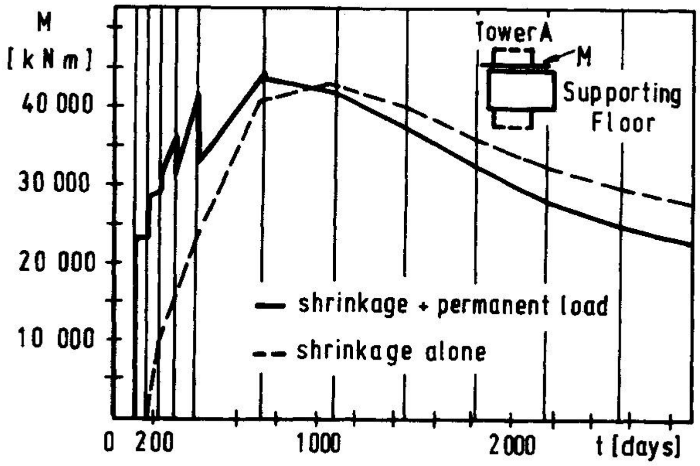

Fig. 5 Bending moment in tower A at the height of the upper surface of the supporting floor

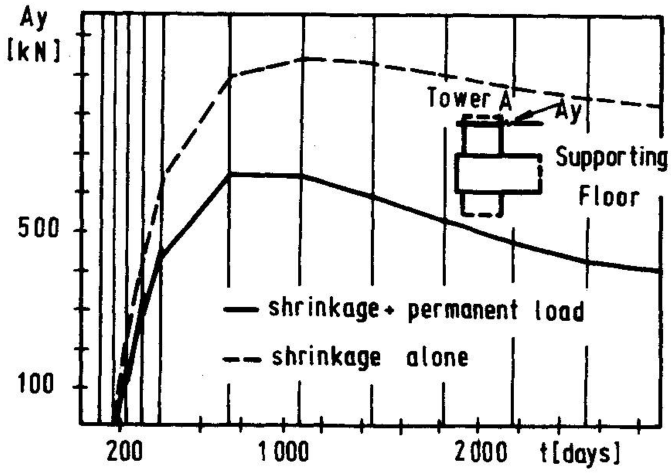

Fig. 6 Force Ay in the elastic connection of the first slab block above the supporting floor at tower A

4. CREEP AND SHRINKAGE ANALYSIS OF THE KOCHER VALLEY BRIDGE

The Kocher Valley Bridge is a prestressed-concrete motorway bridge at present under construction as part of the extension of the Weinsberg - Nuremberg motorway between Schwäbisch Hall and Künzelsau. Its maximum height above the valley floor is 185 m.

The structural system of the bridge consists of a frame structure, the four central columns of which are rigidly connected with the superstructure. The regular width of the spans of the superstructure is 138 m (cf. Fig. 7).

The six-lane highway is supported by a box girder 6,5 m high with widely overhanging plates. The superstructure is constructed in stages by cantilevering out using an advancing girder. In this procedure the box girder section without the lateral plates (i.e., the core cross-section) is first constructed by cantilevering out in both directions from the tops of the piers. As soon as one section extending from one centre of a span to the centre of the next span has been completed, one begins to concrete the lateral plates on both sides of the core structure, the so-called "follow-up" structure, thus completing the final cross-section of the bridge. These concrete plates are braced against the

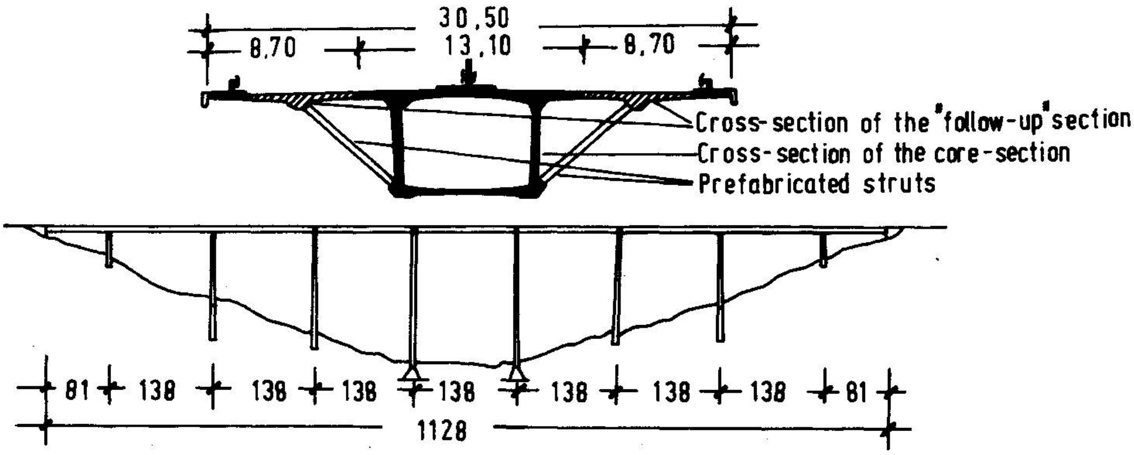

Fig. 7 Cross-section and side view of the Kocher Valley Bridge

core by means of prefabricated struts. Simultaneaously, the next section in the cantilever construction of the core proceeds as before. Due to the short period of time available, the construction of the superstructure is advanced, starting from both abutments simultaneously.

The delayed addition of the "follow-up" cross-section to the core cross-section causes stress redistributions in the cross-section ("cross-sectional creep") and deformations which, in turn, produce stress redistributions in the entire structural system ("system creep"). Both types of cree-effects can, in this case, no longer be considered independently [14]. The construction of the bridge in 14 construction phases with different structural systems also yields stress redistributions in the total structure. In order to keep these effects to a minimum, so-called joint-expansion moments and forces are applied by means of presses to the coupling joints before these are closed. These joint-expansion moments and forces provide the internal stresses which would be present if all loads had been applied right from the start to the final structural system. But as the bridge contains concrete of varying ages this measure cannot completely suppress system creep. In addition, the temporal progression of the deflections has to be calculated in order to determine the required camber of the bridge.

In calculating the effects of creep and shrinkage it is necessary to analyse half of the bridge only as the structural problem is symmetric. It is idealized as a plane frame with 13 beams. Each of these beams may have varying cross-sectional properties along its axis and each cross-section may be a composite cross-section. With this extraordinarily efficient beam we are able to account for the gradual increase in cross-sectional values due to the concreting-on of the "follow-up" cross-section, and the successive prestressing and injecting of the tendons.

The following loading cases are considered:

- 32 loading cases of dead load due to the weight of the core cross-section
- 32 loading cases of dead load due to the weight of the "follow-up" cross-section
- 39 loading cases for various positions of the cantilevering out scaffold and the advancing girder
- 9 loading cases due to joint expansion
- 9 loading cases related to pavement, railing etc.
- 41 loading cases due to prestressing of 175 tendons in the core cross-section and 90 tendons in the "follow-up" cross-section

These loading cases were applied at 42 different instants of time.

The analysis was carried out with the improved three-parameter solid. The creep curves of the prestressed concrete guidelines were used as input data by a preprocessor to compute the time dependent material characteristic values of the three-parameter solid.

The integration in time of the stresses and displacements was carried out in 90 time steps. It is interesting to note that 83 steps were required to describe accurately the different construction phases and the loading history and only 7 steps to determine the changes in the internal forces and displacements during the period beginning with the opening of the bridge to traffic (t = 1120 days) up to the assumed date of termination (t = 12.000 days).

Fig. 8 shows the changes of the normal forces in the core cross-section and the "follow-up" cross-section versus time.
It indicates that the "follow-up" cross-section gradually carries a considerable proportion of the load.

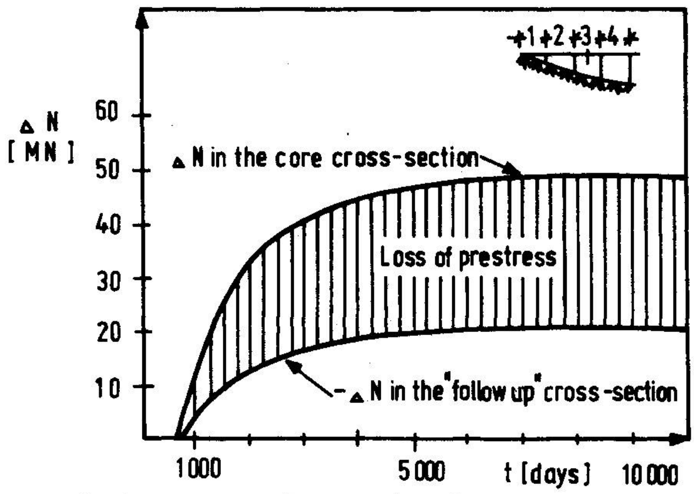

Fig. 8 Changes of the normal forces in the core cross-section and the "follow-up" cross-section in the middle of the third span of the bridge

In Fig. 9 the normal forces and their changes at $t \to \infty$ are shown. 68 % of the normal force at $t \to \infty$ in the "follow-up" structure are caused by the effects of creep and shrinkage. Another result of creep and shrinkage is a reduction of the normal force in the core of 50 MN. Of this figure, 29 MN are prestress losses and 21 MN have been taken over by the "follow-up" cross-section.

	Core	"Follow-up"
Normal forces $t \to \infty$	- 82	- 31
Changes in the normal forces, $t \to \infty$	+ 50	- 21

Fig. 9 Normal forces and their changes in the middle of the third bridge span (unit = MN)

Fig. 10 shows the changes of the bending moment at the middle of the third span. The inflections at the beginning of the curve are due to the different construction phases during this period. The total change in the bending moment ΔM = 12.1 MN at the time $t \rightarrow \infty$ is about 5 % of the bending moment due to the total dead weight of the superstructure on the final system. Thus, the expansion of the joints has greatly reduced system creep.

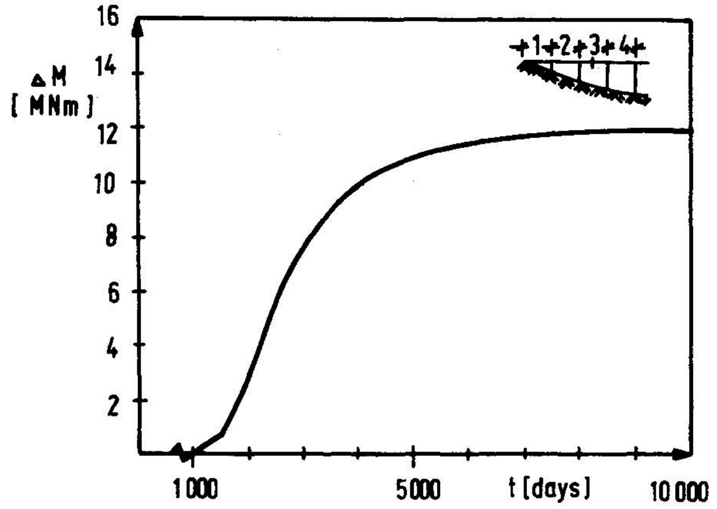

Fig. 10 Changes of the bending moment in the middle of the third span of the bridge

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COLLOQUIUM on:
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A Wide Variety of Computer Appearance

Des nombreuses possibilités d'utilisation de l'ordinateur

Die vielen Anwendungsmöglichkeiten des Computers

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Summary

This paper deals with the computer use for the design and engineering of the Dunlin A gravity type production and storage platform 156 m deep North Sea waters. After explaining the platform concept and functions, this paper deals with the different applications as could be distinguished. Computer aided dimensioning and parameter studies, computerrun simulation, computer control and administration of the design and engineering process and the pure structural FEM analysis for the platform caisson base and the dynamic frame analysis for the platform towers and deck are explained and illustrated.

Résumé

Cet article présente l' utilisation de l'ordinateur lors du projet et de la conception de la plateforme de production et de stockage, de type gravitaire Dunlin A, réalisée sur des fonds de la Mer du Nord, à 156 m. La concéption et le fonctionnement de la plateforme sont expliqués, de même que diverses applications possibles. Le dimensionnement à l'aide de l' ordinateur, les études de paramètres, la simulation à l' aide de l' ordinateur, le contrôle et l'administration du projet d' engineering, l' analyse purement structurale au moyen des éléments finis de la base en caissons de la plateforme, et l' analyse dynamique des tours et du pont de la plateforma font l'objet de cet article.

Zusammenfassung

Der Artikel behandelt die Anwendung des Computers für die Planung und den Entwurf eines Produktion - und Lager-Platforms des Gravitätstyps, Dunlin A, im Nordmeer bei 156 m Tiefe. Die Planung und Funktionen des Platforms werden erklärt, sowie die verschiedenen Anwendungsmöglichkeiten. Computergestütze Bemessungen und Parameterstudien, Computer-Simulierung, - Ueberwachung und - Verwaltung des Entwurfs und Planungsverfahrens wowie die anhand von finiten Elementen durchgeführte reine Berechnung des Platformcaissons und die dynamische durchgeführte reine Berechnung des Platformcaissons und die dynamische Berechnung der Platformtürme und - Brücke werden erklärt und illustriert.

1. INTRODUCTION.

The 1973 oil-crisis caused an acceleration in the construction of oilproductionfacilities in the Northern North Sea. Several factors including the boom in steelconstruction causing a scarcity in steelworkconstruction capacity favoured the development, design and construction of fourteen concrete gravity type platforms.

The Dunlin A, production and storage platform designed and constructed by an Anglo-Dutch consortium "ANDOC" for Shell Expro, is one of these platforms.

This paper will deal with the computerwork used for design and engineering of this simultaneously designed and constructed, US $ 300 millions worth, job.
After a brief description of the platform, it's functions and the way it is constructed, the different types of computer usage to distinguish are described in their function for the design and engineering process. They are the administration and control functions, the parameter studies, the dimensioning applications, the simulating exercises and last but not least, the bulky structural analysis which can be basically divided into the shell analysis for the base and the frame analysis for deck and columns.

2. THE DUNLIN PLATFORM.

2.1 It's functions.

The Dunlin platform serves for the production of about 200.000 barrels of crude oil per day in 156 m deep water of the Northern North Sea. This required a working platform of over 4.500 sq.m. to support the necessary equipment. This includes the drilling rig and support units for the 48 wells to be drilled from the platform, living accomodation for 150 people, heli-deck, flare boom hoisting equipment, heat exchangers to cool the produced oil before storage, a gasturbine for the platform energy consumption etc. The wet weight of all this equipment amounts to a 20.000 tonnes in total. The unit is further capable of storing a 1 million barrels of crude in the base as buffer storage for waterseparation.

2.2 How it looks like. (See figure 1)

The platformdeck consists of a 85 x 65 m grid of 6 m high boxgirders. This deck supports the facilities housed in modules on top of it and serves further as part of a space frame consisting of the four columns and this deck.
The 143 m high columns built up from 113 m corical concrete towers and a 30 m steeltop do in the first place support the deck on a safe height of 23 m above the sea. These columns further have to resist the horizontal forces from waves and wind, where they act as a space frame with the deck. In between two of the columns three tubular girder bracings are spanning to guide and support the conductors against lateral loads by waves, current and wind.
The columns do contain further the equipment to control and operate the storage function of the platform and a lot of piperuns. During certain constructionstages the volume of the columns is used as the stabilizing floatation capacity.

The 100 x 100 m wide and 32 m high caisson is built up of rectangular cells of 11 x 11 m. It has arched outer walls, a cassinishell type roof and a ribstrengthened bottom. The caisson forms the basic floatation body for horizontal and vertical transport, it will function as the storage reservoir for crude, it provides the foundationplane, and by it's own weight together with the added solid ballast it provides the stability during towing and after final installation on the seabed.
Underneath the caissonbottom 4 m long steel H-beam shaped skirts are penetrating in the seabed, serving mainly for the transfer of horizontal forces to deeper soillayers and to protect the foundation strata against scouring.

2.3 How it was constructed.

After a 6 week tenderperiod and another 6 weeks of negotiations Andoc was awarded the design and construct contract of the Dunlin A platform on the 1st May 1974. The first pours of concrete took place 4 months later in the graving dock on the "Maasvlakte", a reclaimed area close to the Rotterdam harbour entrance.
Sheetpile wall strenghtened trenches in the graving dock bottom were provided to house the skirts.

After completion of the 80 cm thick slab, the 4 m high ribs, 4 m height of the walls and just 4 m extra height of the outer walls (functioning as splashboard), the 4.80 m heigh "saucer" of 100 x 100 m square was floated up in the flooded dry dock with the aid of an aircushion in between the skirts to reduce the draft. This happened in the beginning of June 1975. Another year of construction in the Rotterdam harbour area followed. Whilst floating in 22 m deep water the caisson walls and concrete towers were slipformed and all the other concrete works were completed.
In June 1976 the structure was towed to a Norwegian fjord. Here solid ballast was placed in the caisson and after completion of all the installation facilities the platform was immersed to place the 30 m high steel columns on top of the towers. The deck was installed by floating it on a barge above the submerged platform from which only the top 8 m of the columns, were above the water. After the necessary hook up the platform was towed to it's installationsite in the North Sea, early June 1977, only 3 years after design and construction started.

3. COMPUTER AIDED DIMENSIONING.

It was new and complex. This required two types of computer usage that are usually not distinguished as a purpose in itself for designers. The first one was the parameter study as used for overcoming the problem of quantitatively unknown physical phenomena and the other was the aid of the computer for optimizing dimensions. Chapter 4 will deal with parameter studies as used.
Most of the many functions of the different components of the platform are interactive as may be clear from the description in chapter 2.2. Especially characteristics, such as draft, stability, payload, membrame strength from columns and platform have a linear relationship to main dimensions such as caissonwidth, caissonheight, columndiametre, centre to centre distance of columns and solid ballast inserted in the caisson.
It has proved to be very useful to develop a computerprogram with such relations built in. Providing some of the basic dimensions or series of basic dimensions, the other basic dimensions were calculated in view of pre-set boundary conditions for draft, freeboard after inserting solid ballast, minimum meta centre height during caisson roof immersion, minimum meta centre height during tow with full payload on deck, minimum storage capacity.

It was possible to get a good impression of the range of optimum caisson-dimensions in a few computerruns. This impression was based firstly on the selection of series of platformdimensions that fullfill the pre-set boundary conditions and secondly on the total quantities of concrete and solid ballast incorporated in the selected platforms that become available as output simultaneously.
Further fast straight forward computer evaluation of the lateral forces and overturning moments caused by waveforces and foundation resistance were used as a basis for the platform selected for further evaluation.
To start with, this was basically an electronic pocketcalculator handcheck of all the assumed characteristics by an experienced designer!

All with all a useful pragmatic computer exercise to increase the understanding of the behaviour and characteristics of the project we were designing.
Before the actual program and basic design concept of a platform was made, some detail studies for dimensioning were carried out with the aid of standard programs. For instance the amount of load that was transferred by the caisson internal walls to roof and bottom had to be known before considering the reduction of wall thickness towards the caisson central part (figure 2). Just with some computerruns of a disc program in which different caisson heights and reductions of wallthickness from the outside towards the centre were inserted, simple design graphs were developped.

4. PARAMETER STUDIES.

The dimensioning studies with standard and purpose-written programs were in fact trial and error methods to enable us to select the optimum solution in an unknown field. Parameter studies however, normally used in the same trial and error way, were basically encouraged to proof that some characteristic being unknown was not important for the design. In case the unknown characteristic proved to be really important, the design had to be adapted to the highest possible level one could imagine.
For example the whole field of damping was extremely important for the response of the structure to dynamicloads with a period close to the platforms own period. Damping forces from the foundation strata generated in rocking and sliding modes of the platform, on which hardly any information existed,proved to be of great influence on the internal loaddistribution.

The same applied for the material damping values of concrete and steel and hydrodynamic damping of water around the platform legs. As different sets of values produced extremes on different places in deck and columns, one can imagine that a very elaborate parameter study had to follow to find these extremes.

Parameter studies in the sence as defined were also quite often performed in order to establish the effect of a certain magnitude of tolerances in the dimensions. Results were often encouraging, hence facilitating survey and inspection work.

5. ADMINISTRATION AND CONTROL.

Although this chapter may apparently got lost in this context, the purpose of it is that it presents a very useful field of computerapplications for the designer and the structural engineer in case they are working on something big fast and complex as the subject described here.

Fifteen years ago, a designer used to have a mechanic calculating machine of very limited capacity and performance according to our present day view, but being more expensive than a small car. In those days the investment for one item of output were considerable.

Nowadays a small electronic pocket calculator, being much faster and more versatile than the machine mentioned above is cheaper than one working hour of our designer. We now have a situation that the costs per item of output are neglible and that the designer, in a moment of recklessness can approach the computer in such a way that he gets more output than he can ever digest during the rest of his life.

It is this phenomena that forced us to promote a most conscious computeruse during the design of the Dunlin platform. In certain cases we selected a physical laboratory tests instead of a computersimulation, for instance for the detailed analysis of the deck to column joints. Instead of an elaborate finite element run for which "only a few elements for stiffened panels" still had to be developped a perspex scale model was used. Our observation that computeroriented engineers always are to optimistic in time and consequentially in costs was the basis of that decision, although the estimates for a finite elementrun and a perspex scale model were about the same.

A wide variety of computerprograms for administration and control of the job was written and used. This was inspired both by the magnitude and the complexity of the job.
We developped two extremely useful programs "TEHREG" and "CALREG" for the administration of drawings and calculations. Distributions, requests for approvals, approvals themselves, revisions and the dates of all of these actions from all calculationpapers and drawings were fed into the computer through these programs. We could not have done without it. One should realize that over 15 subcontractors worked on the design, that had to be by the client with his consultants in the U.K. and the certifying authority D.N.V. in Norway.

In chapter 3 we mentioned the activity of calculating the stability of the platform. During the actual detailed design and construction, with it's many changes, a daily "bookkeeping" program "CASTA" for the dimensions, and distribution of weights was activated.
The programs output presented the stability heel during preset wind and draft conditions including the safety and the meta centric height.

Another exercise in this category, although not consequentially used for various reasons, was the computercontrol of the operating manuals including their updates, and internal references. The lack of sufficient staff experience in this type of work against the required speed aborted this exercise.

6. SIMULATION EXERCISES.

Beside the trial and error exercises encountered during dimensioning and parameter studies, simulation has been done by means of digital computer-programs to check if certain designed operations were correct, but especially to help the imagination of the crew who had to perform such operations and to train them consequentially.

The different towing stages, from Rotterdam harbour into the North Sea, the tow through the Norwegian fjords with the fully loaded 450.000 dwt platform and the final approach in wave and current of the installationsite in the North Sea were simulated on the computer of the Netherland Shipbuilding Research Station at Wageningen.

Not only the final arrangement and layout of the 8 tugs of 80.000 hp's all together around the platform was designed in this way but even the selection of the final operating crew was decided by means of this provision.

Another area of careful simulation to check the envisaged procedure, was the ballast and leveling system to control vertical motions of the platform during immersion. One should realize that all pairs of cells could be individually ballasted, but that overloading of seperating walls between cells by differential waterlevels in adjacent cellpairs could lead to disastrous damage. This simulation exercise, which included the full ballast system with valves, pipes and cells, proved itself extremely useful for simulating emergency procedures that had to be selected by malfunctioning equipment and monotoring instruments. By careful use of this simulating program we were able to estimate better maximum loadings that could occur during operations governing the structural integrity.

7. THE SHELL AND DISC ANALYSIS.

7.1 Introduction.

The analysis of the caisson and concrete towers was mainly a job of straight forward finite element analysis.

The main problem areas here were:

- the selection of subdivision of the platform and of the proper steps from a course mesh to a fine mesh in subsequent runs.
- the proper analysis of differential temperature loading caused by partly stored hot oil inside the caisson.
- the nonlinearity of certain loadcases due to heavy membrane loading by hydrostatic pressure together with the effect of creep and construction tolerances in the dimensions.

7.2 Mesh selection.

From the start on, subdivision for FEM-analysis was selected with total units as small as possible. The base was considered infinitely stiff for the towerframe analysis as well as for the interaction of the soil with the caissonbottom. Such interactions were only evaluated in the detail-analysis of for instance towerwall embedment in the caissonroof.

As most important loadcases on the caisson are symmetrical, the basic start of all FEM-analyses was performed on a 1/8 section of the caisson (figure 3). With other, easier to use programs, using boundary conditions from the first run more detailed stress distributions were investigated. Asymmetrical loads, due to wave action and the consequential transfer from column forces into the base had to be investigated in more detail after high shear forces were found on certain places. These runs were also used for the global effect of asymmetrical temperature loading caused by only partial storage of hot oil.
In figure 4 a diagram is given, showing the different FEM-computerruns with their purpose.
The lesson we learned was that the idea of runs of limited size going from a coarse overall mesh to a fine local mesh was good. We however, learned as well that a careful planning, right from the first run onwards, considering the maximum versatility for detail runs in later stages is extremely important. Direct modelling in such a way that part of the final model can also be used for construction stages to be analysed proved to be very useful. In such an early planning the selection of the computerprogram(s) to be used with their specific features, such as accessibility, graphic display, in house experience, level of support, costs, capacity and all type of other characteristics play an important role.

7.3 Temperature loading.

As the caissonroof was a cassini type shell structure with wallthicknesses that varied from 2.5 m to 0.80 m being obviously loaded under certain temperature conditions with compression stresses on the inside up to tensile stresses far over the tensile strength of the concrete on the outside, one can imagine that no straight forward FEM-analysis could be made. Also special programs, dealing with cracked concrete could not be used as the temperature forced cracks, results in a not fully developped crack pattern as with bending.

With different approaches it was checked that no resulting membrane tension occurred in the roof and to abandon the possible damage by uncontrolled cracks the reinforcement was galvanized.

7.4 Non-linearity.

In many places the elastic stability of the shells and discs loaded with high compression pressures had to be checked. This was especially important for loadcases during immersion of the platform. As creep plays an important role in such cases, where initial tolerances causing prime deformations have to be assumed, load history curves had to be evaluated. Special computerprograms were written to perform this work as the evaluation could have to be very fast in case certain modifications on the load history dictated by the envisaged operationprogram had to be absorbed.

In one occasion, at the bottompart of the concrete towers, the tolerances were very critical. Twice a day the presently performed towercross-section-shape was loaded into a computerprogram. This program first evaluated the circularshape closest to the real cross-section and than checked the occuring moments incorporating creep effects.
From this the actual safely factor against buckling performed was calculated and checked with the requirements.

FRAME ANALYSIS.

The internal face distribution in the superstructure (= the portalframe) has been calculated using a 3D finite element computer simulation and the STRUDL program package. The columns and deck have been modelled as a space-frame where the caisson is thought to be a stiff, rigid foundation. During the course of the project significant progress has been made in analysis techniques, especially where dynamics and fatigue are concerned. Looking back now, the frame analysis can be divided in three subsequent stages, the static part remaining basically unchanged and the dynamic part keeping up with the developping state of the art and the growing capabilities of the computer software.

Stage I: Static and quasi static analysis.

Once the structure has been modelled using adequate elements and element connections, the main effort consisted of the definition of loading cases. The dynamic character of the waveloadings has been simulated by applying a dynamic load factor on a static loading which is calculated for a number of "frozen" wavepositions.
The number of loading combinations for such a type of analysis is enormous. The following loading conditions are considered:

selfweight	1 condition
dead load	3 conditions
life load	7 conditions
wave heights/periods	10 conditions
wave directions	5 conditions
wave positions	10 conditions
analysis conditions	4 conditions

The number of loading combinations becomes:

1 x 3 x 7 x 10 x 5 x 10 x 4 = 42.000.

Using a 300 element frame model and requiring 2 fines of output per element this results in 300 x 2 x 42.000 = 25.200.000 lines of output. Physically this means 504.000 pages which, printed on normal paper, gives a stack with a height of 25 m!
Of course this has been reduced by some handcalculations and engineering feeling.

A number of 600 loading combinations have been analysed.
The computer, although it has calculated more loading cases and checked more types of structures than any of our engineers, still missed the capability of performing this significant step.

Stage II: 2D dynamic response analysis.

The use of dynamic loadfactors to simulate the dynamic character of the waveloadings was not appropriate. It appeared that the effect of dynamic amplification of the response was different for several parts of the structure, so a multi degree of freedom dynamic response calculation was required. This has been realized by using a 50 degrees of freedom lumped mass finite element idealization.
Using this model response analyses have been carried out for four wave periods.

For these waves the time histories of the loadings were calculated for each submerged joint of the structure, which in turn were used as loading input for the transient response calculation.
According to this analysis the waves near the resonance period (4.2 sec.) procedure severe stresses in the deck column connection, these stresses being of course very dependent on the dused damping.
The dominating influence of the damping and the resonance waves in a fatigue calculation, made this type of analysis insufficient for realistic lifetime predictions.
Because of the nature of the oceanographic data and the strong influence of resonance waves, a 3D frequency domain fatigue analysis had to be the basis for a trustworthy lifetime calculation.

Stage III: 3D spectral analysis. (figure 5)

The necessary software has been developped to enable us to carry out a full 3D probabilistic analysis.
The problems which we encountered during this development had to do with the transfer of a known study room theory to a fully operational and usable stage.
Theoretical problems concerning non-linearities and wavestatistics had to be solved and the resulting calculation methods implemented in the STRUDL program.

Also organisational aspects had to be looked at to avoid exceptional cpu-time usage and input preparation. This had been achieved by writing preprocessors which generate direct loading input and by using the harmonic response calculation technique for a considerable part of the frequency band. The results gave great confidence in the wed method and showed a more realistic dependency on parameters like natural period and damping.

IV. 48

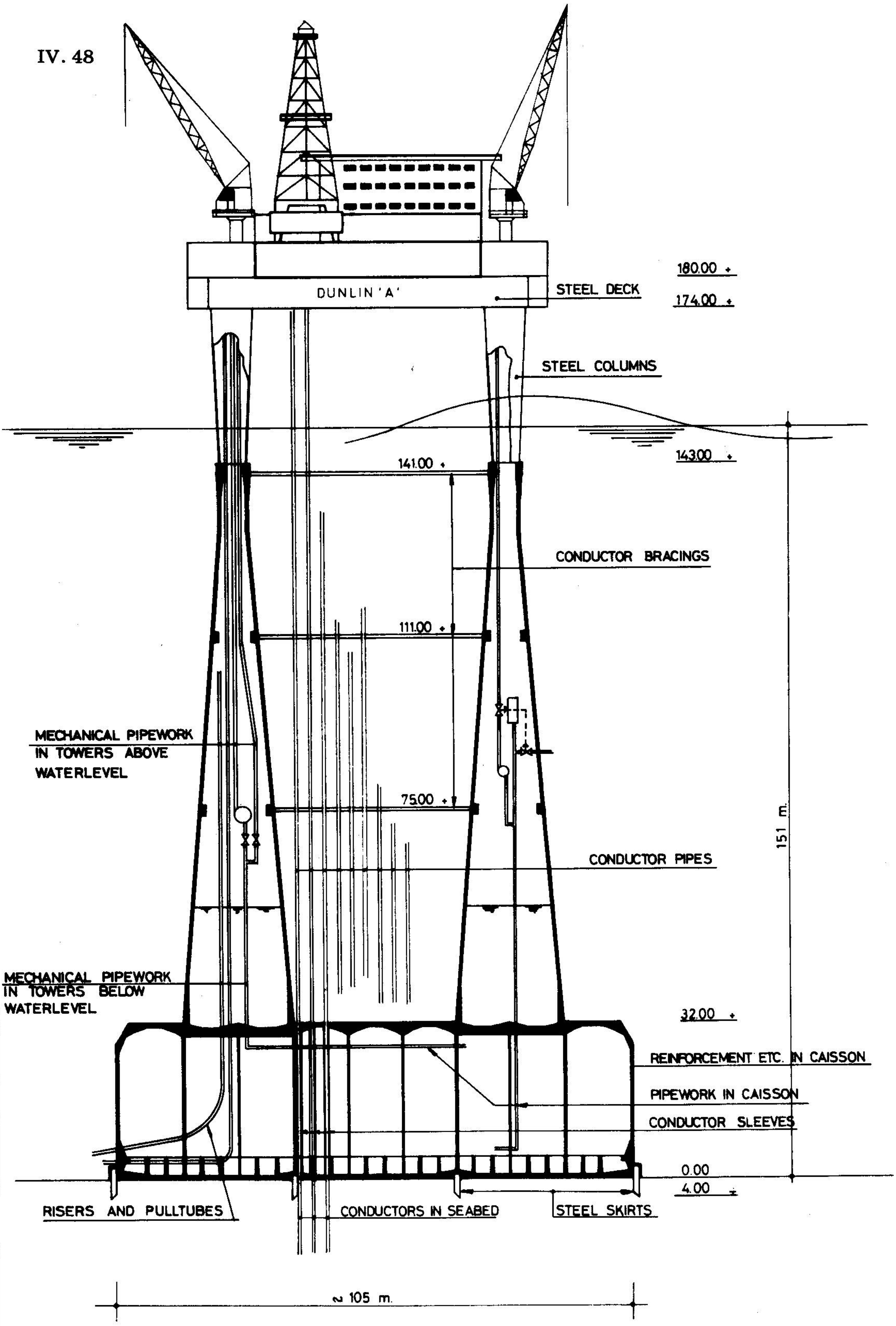
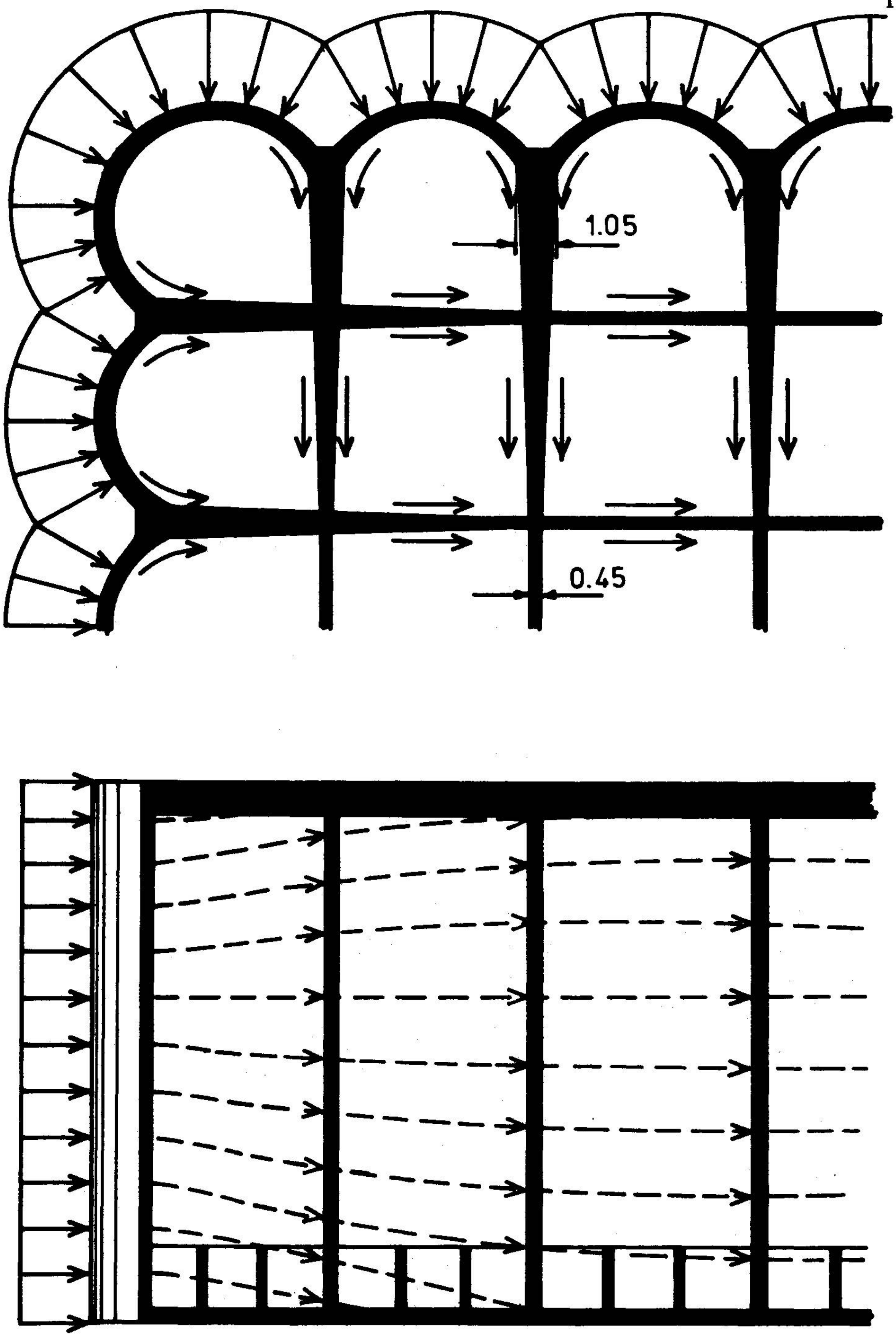


fig. 1

ANDOC



Distribution of outer arch loads over shearwalls, roof and bottom

Figure 2.

SUBDIVISION

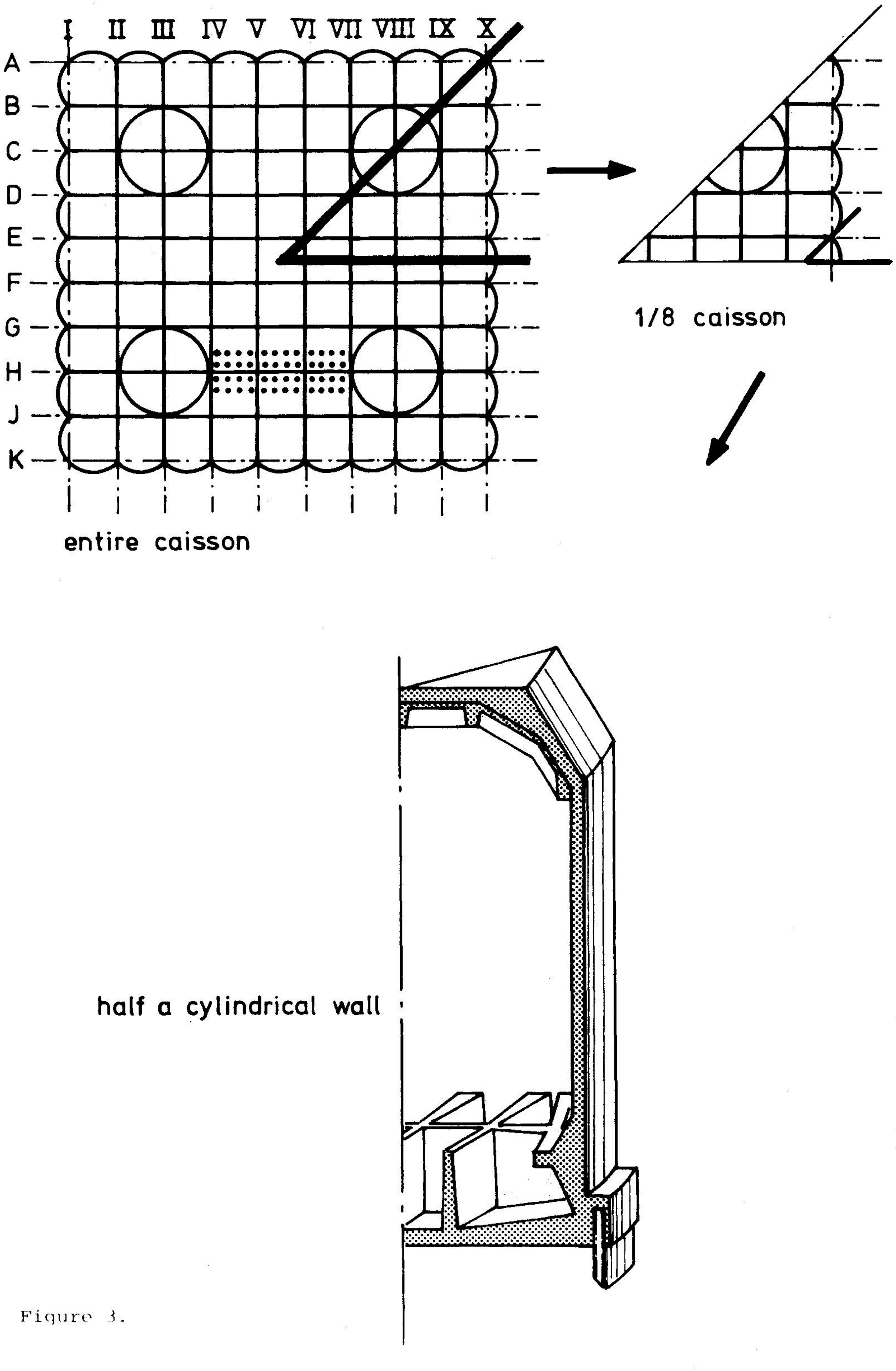


Figure 3.

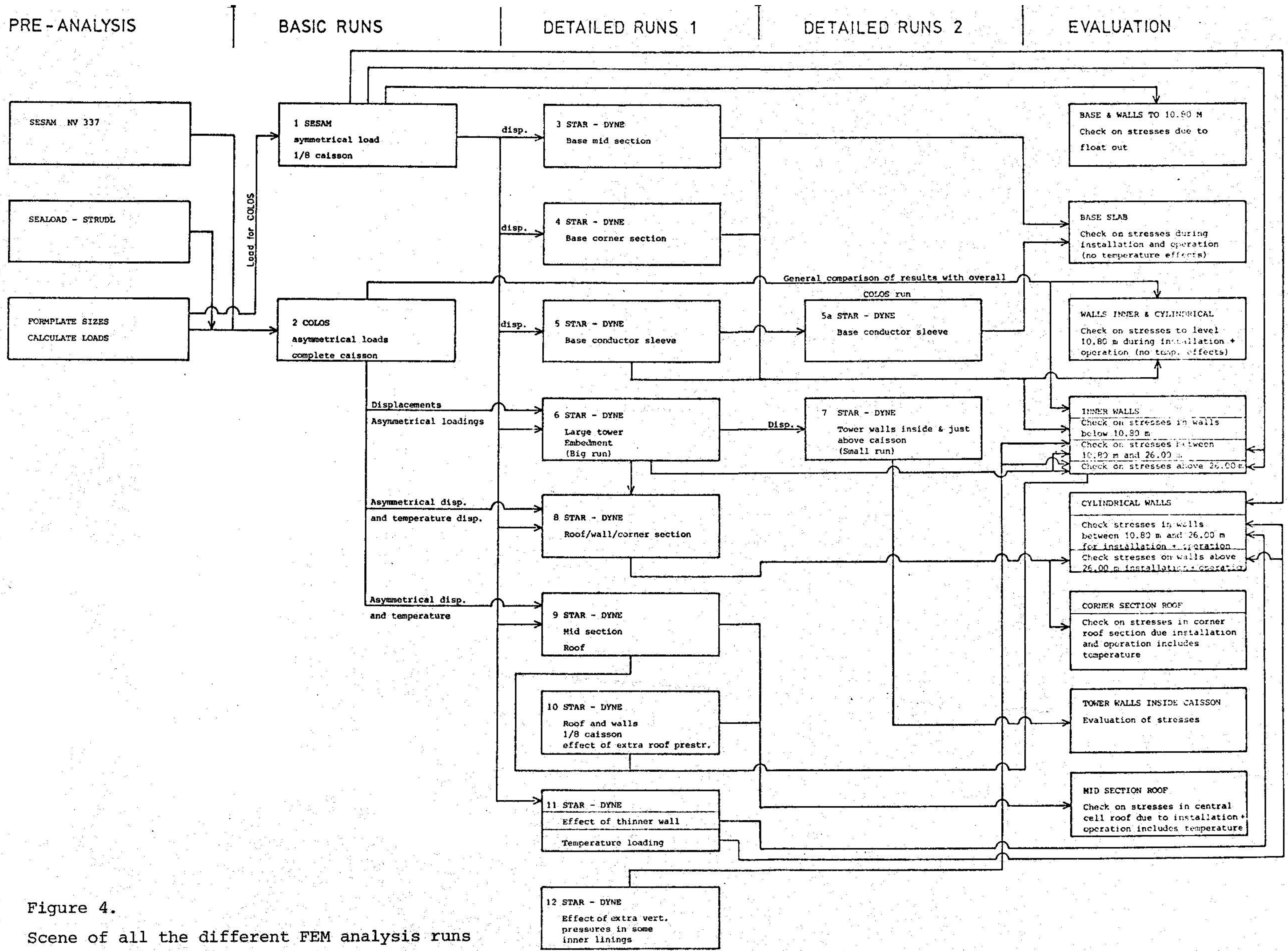


Figure 4.

Scene of all the different FEM analysis runs of the Dunlin A caisson.

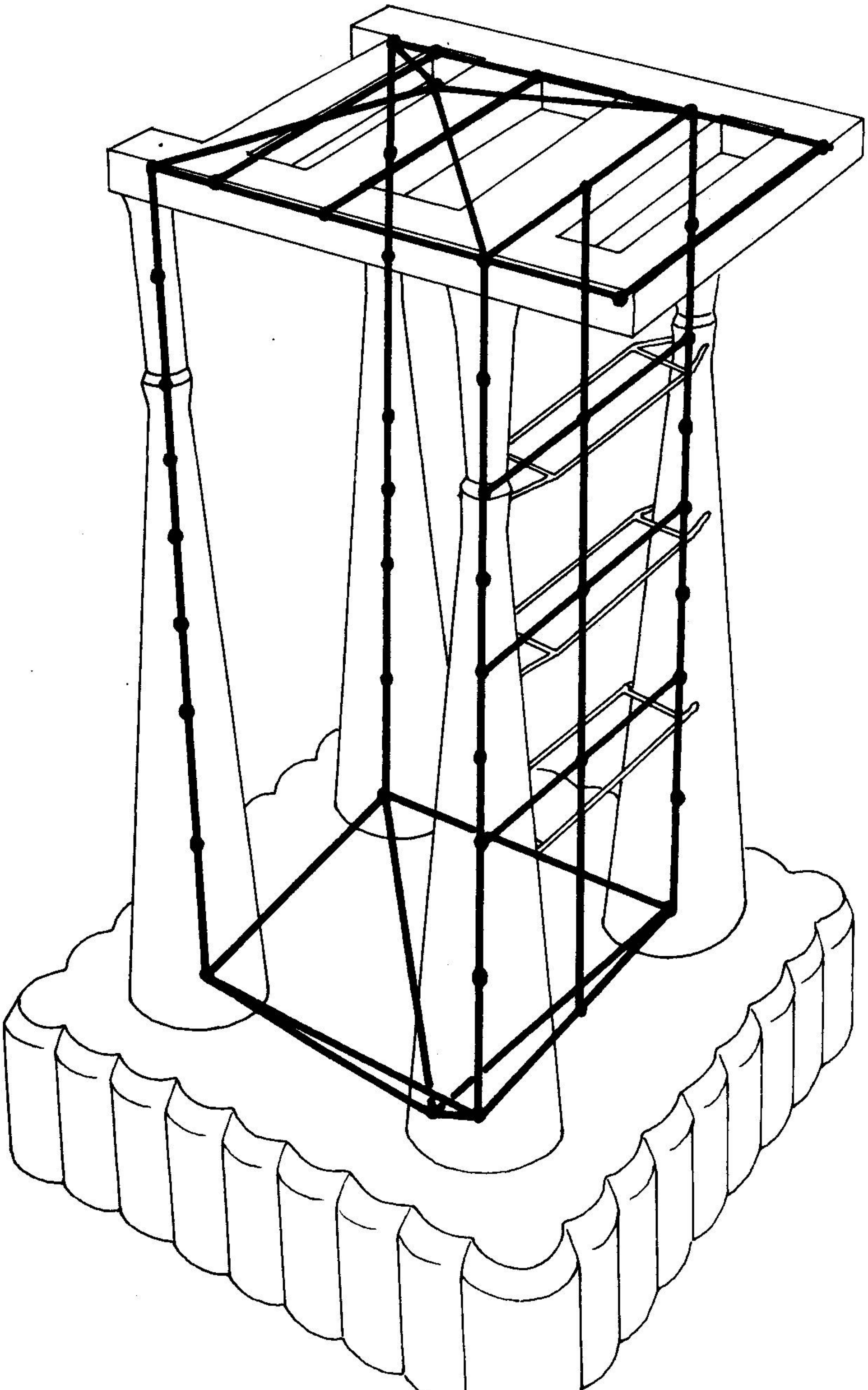


Figure 5. 3D Dynamic Model.

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Computer Aided Design of Bridges Using STRAINS and KABE Systems

Project des ponts à l'aide de l'ordinateur et des systémes STRAINS et KABE

Mit den Systemen STRAINS und KABE hilft Computer beim Brückenprojektieren

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Summary

The applications of two systems to facilitate the design of prestressed concrete bridges are presented. Examples of applications of STRAINS, a structural analysis system - to calcul ate the internal forces and KABE, a prestressed concrete design system - to determine the cable trajectories and the prestressing force are discussed. The examples of printer output and three colour BENSON drawings are given.

Résumé

On présente l'application de deux systèmes STRAINS et KABE qui facilitent l'établissement des projets de ponts en béton précontraint.
STRAINS : pour le calcul statique, KABE : pour la détermination des forces de précontrainte et de la position des câbles. Quelques exemples de sorties numériques et graphiques - trois dessins faits sur la table autotraçante BENSON sont présentés.

Zusammenfassung

Es werden zwei Systeme vorgestellt, die das Projektieren von Spannbetonbrücken erleichtern. Anwendungsbeispiele von STRAINS, einem Programm zur Berechnung der Schnittkräfte, und KABE, einem System zur Bestimmung von Kabellagen und Spannkräften werden behandelt. Beispiele der numerischen und der mehrfarbigen graphischen Ausgabe werden gezeigt.

1. INTRODUCTION

STRAINS /Structural Analysis Integrated System/ is a package of automatically operated programs for the analysis of the internal forces and displacements in the skeletal, surface or massive elastic structures. It was developed in 1970-75 with the aim to assure widest acceptance and good maintenance over a long period of time. It has two versions: STRAINS 71 and STRAINS 75. They differ by the scope of the problems which can be solved and by the hardware required to operate the system. [1], [2], [5].

KABE is a computer system for the design of prestressed bridges. It is available in two versions: KABE 73 and KABE 76. The difference between the versions is that the 76 version is equipped with the devices to facilitate data preparation. [3], [4], [6]. Both systems operate on ODRA 1300 series computers which are an equivalent of ICL 1900 series and are running under the ICL operating systems. To run STRAINS 71, KABE 73 or KABE 76 a 32k word core memory is required and 4 to 6 Magnetic Tape units, but not the random access memory. STRSINS 75 requires access to disc memory. Both systems operate in batch mode: they are simple yet effective, user-oriented and friendly.

2. CHARACTERISTICS AND FUNCTION OF THE SYSTEMS

2.1 General characteristics of both systems.

Both systems were intended for use in the Design Offices as well as in Civil Engineering Faculties: they had therefore to deal efficintly both with large and small size problems. Both of them have user-oriented languages.

2.2 STRAINS

The system is intended for the calculation of internal forces and displacements in trusses, frames /both plane or space/, grillages, plates, plates in bending, shells, solid three dimensional structures and structures composed of bars and shell elements. It can deal efficiently both with large and small size problems of analysis, leaving out only the not so often encountered problems of more than 6000 Degrees of Freedom to be computed using other systems of analysis.

STRAINS is not only user oriented but is also very friendly. The command GENERATING causes an incremental metod of data generating to be used.

In the extreme case of the regular, straight line grids, the entire data needed for the node numbering and the definition of nodal coordinates can be given in just three lines for any plane structure and in four lines-for a space structure.

Similar generator can be applied to the description of the topology.

In the case of more complicated, but regular grids the user can either devise his own data genetaring program or use data prepared independently and input it via the magnetic or paper tape, using the command EXTRA INPUT.

The problem-oriented language compiler checks the input data for errors and in the cases they occur, a message is output.

Geometrical and topological errors can be spotted at a glance in a picture output on the line printer. The errors can be corrected by writing just a few lines.

The data can be easily modified, changed, added or deleted in subsequent computer runs.

In fact, this is the nearest one could get to the interactive mode, without actually having the necessary hardware.

The form of the printouts /both formats and tables/ is automatically adjusted.

When the user wants to limit the amount of printouts, he can specify the required results.

The scale of the line printer sketch as well as the scales of the plotter drawings are automatically adjusted, so thet only one command DRAW is needed.

In the rare cases only when the drawing output is expected to be / or found to be / unsatisfactory, the user can give his own specifications concerning the scale and - in the case of space structures-the viewing angle.

The graphical output received special attention.

In the case of Finite Element Analysis, a map of stresses can be produced on the line printer.

Owing to the fact, that not all computer users have direct access to the plotters and also to save computer time, the graphical output of STRAINS is registred on a Magnetic Tape and processed off-line on a plotter /in our case it is a BENSON 122 drum plotter/.

A two-level program structure has been devised, hovewer, to make the graphical output device - independent.

Some examples of the graphical output available are shown in the examples.

At the moment STRAINS-75 is operational on ODRA 1300 series, ICL 1900 and 2900 series computers. In particular, it runs very well under George 2 and George 3 operating systems,and on ICL 2903 mini, although in this last case the computing times are fairly slow owing to the 1900 emulator.

STRAINS-75 is installed in some 11 out of the 15 Civil Engineering Faculties in Poland and in about 15 other computing centres, serving regularly some 30 Design Offices and occasionally further 40 or 50.

2.3. KABE

The other system developed at the Institute for Highway and Bridge Design, is KABE-76 - a system for the preliminary design of the prestressed concrete bridges.
The statical scheme of the bridge is a multi-span beam.

It may have varying cross section along the span.

The system can cope with any shape of cross-section as long as it can be described using less than a 100 points with straightline connections between them.

Each span is automatically divided into 10 equal parts: all 10 cross sections at these points can be different from each other. The dead loads include self weight, continuous load along the entire span /due, for istence to road surface or track ballast/ and point loads /due to pipe hangers/.

The moving loads include the standard road - rail - or tramway loading as well as "special loading", consisting of up to 30 point loads of arbitrary magnitude and spacing.

The results at each of the 10 points along every span include:

- cross section propertes
- envelope lines for Bending Moments, Shear Forces and support reactions for each type of the moving load
- maxima and minima for the combinations of loads
- parameters of the permissible cable zone
- prestressing cable trajectory
- extreme values of the prestressing forces
- stresses in top and botton fibres
- quantities of the prestressing steel and concrete for each of the four values of the prestressing force.

The system is composed of a problem-oriented language compiler and 9 programs, of which one provides the graphical output.

In the initial stages, system KABE was very much different from STRAINS and had no problem-oriented language.
The experience gained during the several years of using both systems brought KABE more into line with STRAINS as far as user-system interface is concerned. From the point of view of hardware, KABE is similar to STRAINS in that it is operational on ODRA 1300 and ICL 1900 and 2900 series and runs well under GEORGE 2 and GEORGE 3 operating system.
32 K words core capacity only is required.

The latest development of KABE is its inclusion in a large system for checking the strength of the existing bridges under extra-ordinary loading. KABE is used to compare, point-by-point, the Bending Moment and Shear Force envelopes under the standard and under the special loading. In case the values of special loading envelopes exceed those of standard loading, the warning is output and the bridge is considered unsafe for that particular special load.

3. EXAMPLES OF APPLICATION

3.1 STRAINS

3.1.1 The analysis of the internal forces in a space frame bridge.

A space frame bridge in which same nodes are pin-jointed was analysed. The structure consists of 143 joints and 296 bars. The fragments of the program are shown in Table 1. The printer sketch is shown in Fig.1

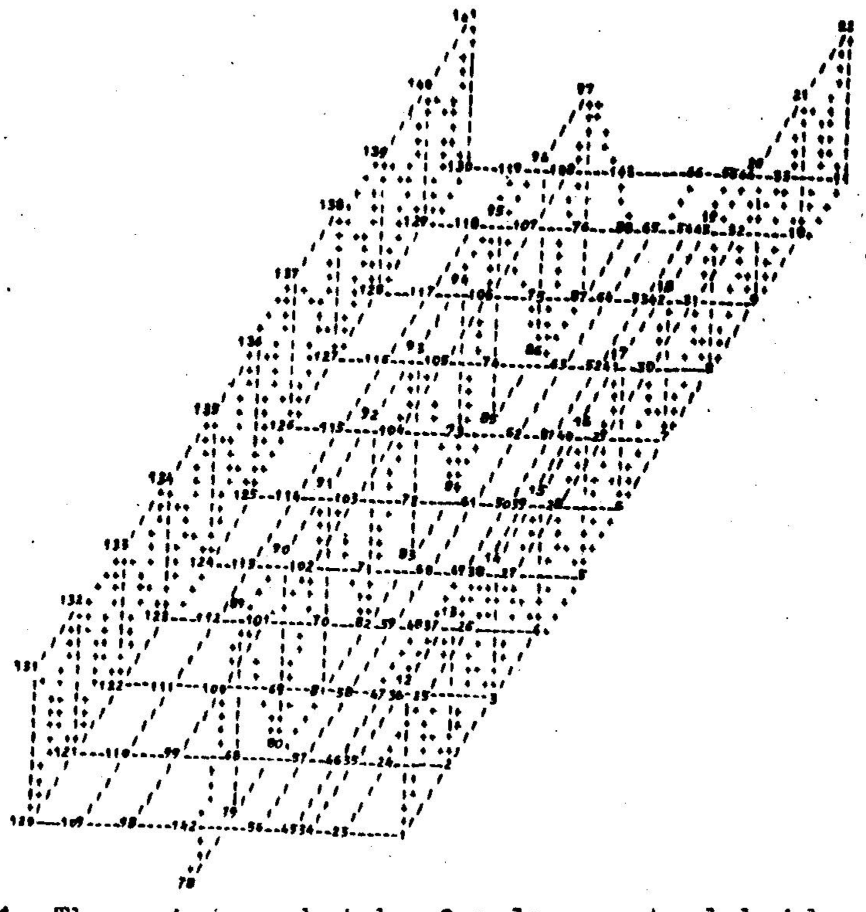

Fig. 1. The printer sketch of a large steel bridge frqme.

```
'STRUCTURE' MOST T-2
'TYPE''SPACE FRAME'
'JOINTS'143
'ELEMENTS' 296
'STIFNESS METHOD'
'GEOMETRICAL DATA'
'JOINT COORDINATES'
1,1120,0,0,
2,1120,361,0,
3,1120,722,0,

'SYSTEM OF ELEMENTS'
1,1,2,
2,2,3,

296,117,128,
'PROPERTIES'
'ELEMENTS''E''V''ALPHA'
'ALL'2100000,0.3,0,
'ELEMENTS''A''IY''IZ''IX'
4,41,42,191,210,98.8,11889,22691.56,

9,121,122,123,124,125,126,127,128,129,222.4,46803,47541,420,
'ALLREST'303,2,568930,5711,765,
'BOUNDARY CONDITIONS'
1,120,'FIXED IN' 'Y' 'Z'
78,142,'FIXED IN''X''Y''Z'

'LOAD DATA'
'JOINT LOAD'
1,11,'Z',1875,
2,10,'Z',2860,

106,119,'Z',7220,
33,44,55,66,'Z',3610,
'GEOMETRICAL OUTPUT'
'JOINT ROTATIONS'
'JOINT DISPLASEMENTS'
'INTERNAL FORCES'
'SOLVE'
'END'
```

Table 1. A fragment of STRAINS program.

3.1.2 The analysis of the internal forces in the web of a prestressed concrete beam.

A diafragm of a box section highway bridge was analysed. Out of a complete set of the results consisting of printouts, printer graphics and the graphical output, drawing obtained on BENSON 122 graph plotter are shown in Fig. 2, 3 and 4.

3.1.3 Analysis of a plate in bending type of viaduct

A plate in bending viaduct was analysed. A printer scatch of stress distribution is shown on Fig.5.

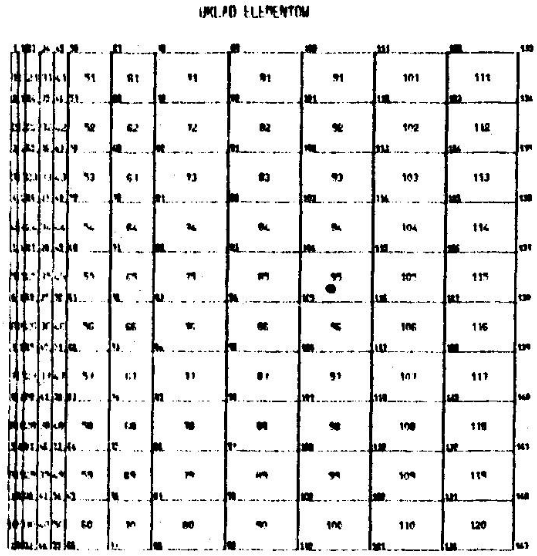

Fig.2. Division into finite elem.

3.2 KABE

3.2.1 An analysis of a prestressed concrete highway bridge.

An analysis is carried out of the internal forces under the Polish Standard Road Loading and calculation of the excentricity and the prestressing force.

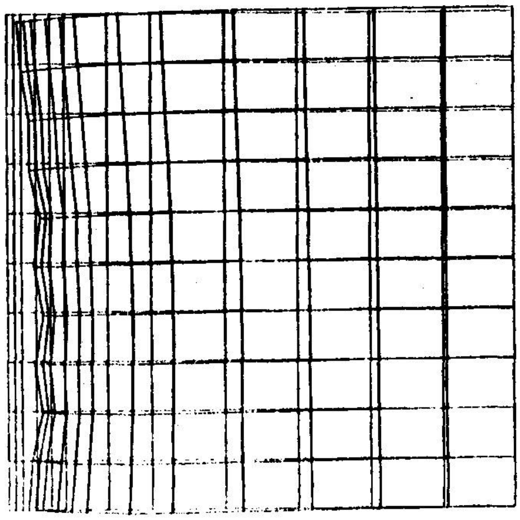

Fig.3. Deformation of the web

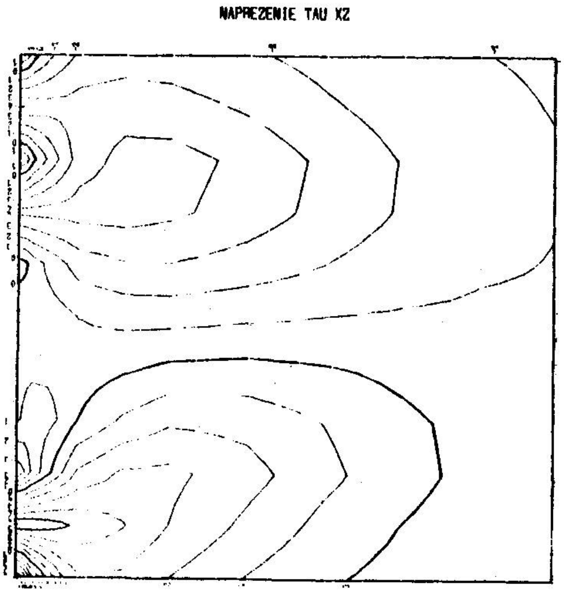

Fig.4. TAU XZ stress distribution

Fragment of program is shown on Table 2. The results are output on the printer and graphplotter.

The drawings of the beam cross section influence lines of bending moment and the cable trajectory are presented in three colours as the graphical output /Fig.6,7,8/.

3.2.2 An analysis of a prestressed concrete railroad bridge

An analysis of internal forces under Polish Standard Railroad Loading and calculation of the excentricity and the prestressing force is carried out. The results are output on the printer and graph plotter. The printer sketch of the beam cross section is show on Fig. 9.

```
  TRANSLATOR JEZYKA *KABE* IDIM PW      DATA:06/06/78
 *MOST DROGOWY,PRZYKLAD NR 1,PRZEKROJ SKRZYNKOWY,
 *LICZBA PRZESEL 3
 *DANE GEOMETRYCZNE
 *ROZPIETOSCI 56 70 56
 PRZEKROJE POPRZECZNE
 *PARAMETRY PRZEKROJU 1
 LICZBA WEZLOW 21
 *WSPOLRZEDNE   NR X Y
 1 0 0
 2 13,5 0
 *CIAGLE 2,52,2
 *CIAGNIK D
 *KROK ITERACJI 0,5
 *OBCIAZENIA NIERUCHOME
 *PRZESLOWE CIAGLE
 *WSZYSTKIE PRZESLA 3,0
 *WYNIKI
*SPREZENIE
 *OTULINA DOLNA 0,15
 *OTULINA GORNA 0,2
 *KONIEC
```

Table 2. A fragment of KABE program.

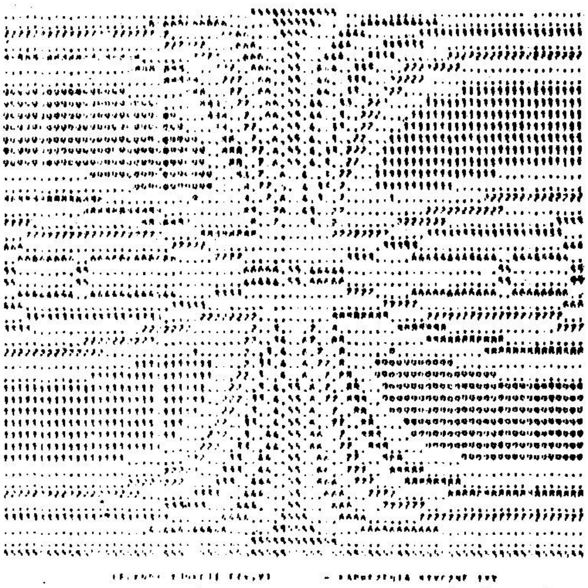

Fig.5 A printer sketch of TXY stress distribution

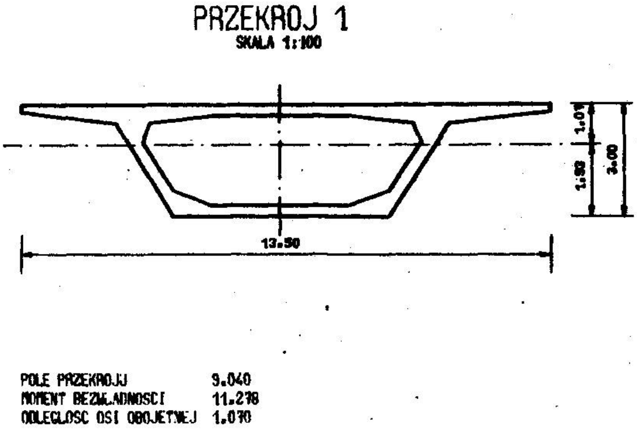

Fig.6 A drawing of the beam cross section

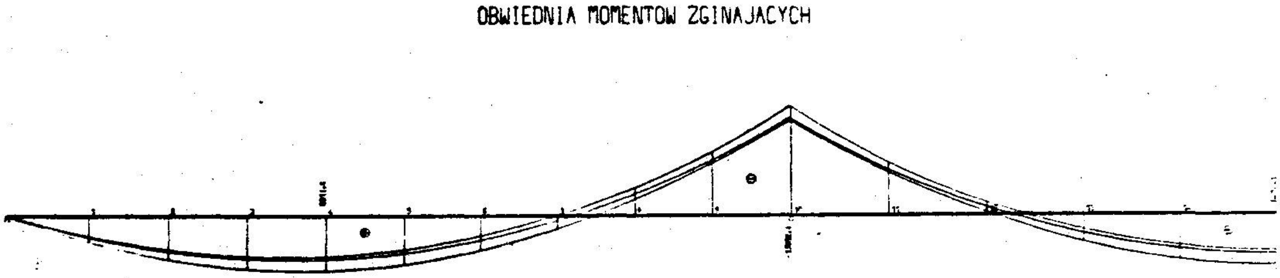

Fig.7. A bending moment diagram.

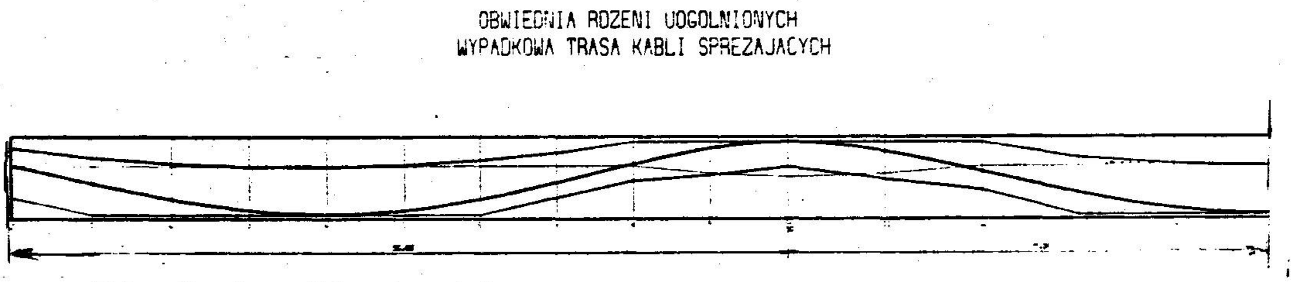

Fig.8. A cable trajektory in prestressed beam.

4. CONCLUSIONS

Both systems, STRAINS and KABE afford a very good interface between designer and computer. They are suitable to the medium size computers.Thanks to the problem-oriented languages they are friendly and easy in application.

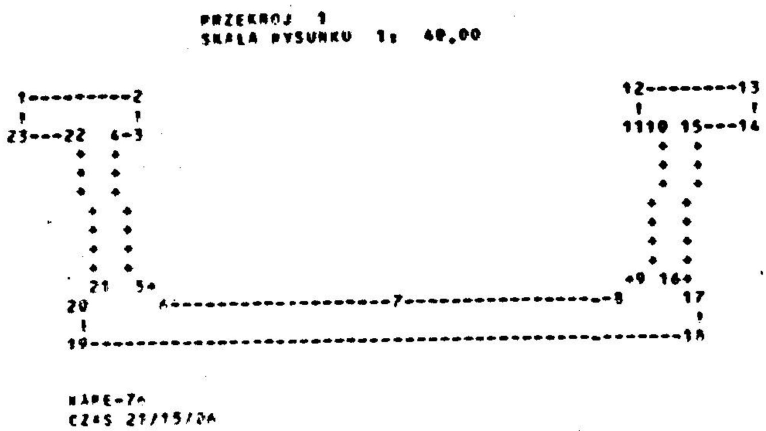

Fig.9. A printer sketch of the beam cross section.

Thanks to the two parallel language versions they may be used in other countries where, instead of the Polish language, other language versions could be used.

Thanks to the printer sketches and three-colour plotter output, the results produced by both systems are very easy to grasp and any mistakes can be easily located.

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COLLOQUIUM on:
"INTERFACE BETWEEN COMPUTING AND DESIGN IN STRUCTURAL ENGINEERING"
August 30, 31 - September 1, 1978 - ISMES - BERGAMO (ITALY)

Use of Computers for the Static Analysis of a Containment

L'utilisation des ordinateurs pour le calcul statique d'une enceinte de réacteur

Die Anwendung von Computern bei den statischen Berechnungen für einen Sicherheitsbehälter

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Summary
The paper renders a survey of the history of the static analyses of a concrete reactor containment and deals with the relation between computer calculations and conventional "by hand" calculations. The paper describes the considerations which were made for the choice of programmes and for the modelling of the structures. It ends with some conclusions gained from the project.

Résumé
Le rapport donne l' histoire du calcul statique d' une enceinte de réacteur en béton. Il traite le rapport entre des calculs sur ordinateur et des calculs conventionnels "à la main". Le rapport décrit les considérations faites pour le choix de programmes et pour le modèle de dimensionnement. Il finit par quelques conclusions tirées du projet.

Zusammenfassung
Dieser Bericht gibt eine Übersicht über die statischen Berechnungen für einen Sicherheitsbehälter und zeigt im besonderen das Verhältnis von Computerberechnungen zu konventionellen betreffs Wahl des Programms und der Berechnungsmodelle und abschliessend die bei dem Projekt gemachten Erfahrungen behandelt.

1. SCOPE

The objective of this report is to render a survey of the history of the static analysis of a concrete reactor containment. The paper deals with the relation between computer calculations and conventional "by hand" calculations in different stages of the design work and concludes with some conclusions gained form the projects.

2. BRIEF DESCRIPTION OF THE CONTAINMENTS

The nuclear power plants of TVO I and TVO II are situated at Olkiluoto on the west coast of Finland approximately 200 kilometres from Helsinki. The plants are "twin" stations and are delivered by Asea-Atom, Sweden to the Finnish company TVO. The reactor is a BWR (boiling water reactor) with a rated capacity of 660 MW_e. The reactor is enclosed by the containment, which shall protect the environment from radiation and from leakage of radioactive substances.

The main containment vessel consists mainly of a cylindrical wall of concrete, a base constructed of concrete slabs and walls, a concrete roof slab in conjunction with and stiffened by the pool structures. (The pool structures constitute accommodation for the handling and storing of reactor fuel etc.) A detachable steel lid forms the crest of the containment. The containment is founded on solid bedrock. The cylindrical wall has an inner diameter of 22.0 m, a height of 32.5 m and a thickness of 1.1 m (Figure 1).

The layout of the containment is based on the pressure suppresion (PS) principle, i.e. in case of a major pipe rupture the steam is led from the upper part, dry well, through a number of pipes to the lower part of the containment, wet well, where condensation takes place. Thanks to this principle the volume of the containment can be kept small, about one-fifth of that of a dry containment with the same design pressure. It is assumed that the pressure in the containment can rise in 10 - 20 seconds to 0.37 MPa overpressure and that the temperature in the atmosphere can go up to 175°C (Design Basis Accident, DBA).

3. DESIGN CRITERIA

The tightness of the containment is secured by the embedded steel liner, at least 5 mm thick, of carbon steel which is cast in to a depth of at least 200 mm in the concrete components.

The pressure retaining function of the containment is secured by the concrete structures. The cylindrical wall is prestressed horizontally and vertically. The longitudinal pool walls are prestressed horizontally. The design philosophy implies the following main rules as the basis for the design:

1) For all normal loading conditions, as well as for loads originating from the design basis accident situation, the design shall be carried out in accordance with applicable parts of the Finnish code. Considering allowable stresses and safety factors, the design pressure (0.37 MPa) is regarded as a normal loading condition.

2) The prestressing shall be sufficient so as not to permit any tension stress resultant forces (i.e. membrane tension) in the pressure-retaining prestressed structural parts for maximum accident design pressure loading conditions.

3) For the condition resulting from a 50 per cent exceeding of the design pressure the reinforcement and the prestressing steel shall be within 0.9 times the yield strength.

4. LOADS AND LOAD COMBINATIONS

The main types of loads are listed as follows:

1) Dead loads from the weight of building parts and components.

2) Dead loads from storaged and transported components, water filling, etc.

3) Prestressing force loads before and after time-dependent losses.

4) Pressure loads (in accident situation).

5) Test pressure loads.

6) Temperature loads (internal forces originating from temperature differences within structural parts) during operating conditions.

7) Temperature loads from accident, test and construction cases.

8) Static and dynamic pipe rupture loads (reaction and jet impingement forces).

9) Water pressure in flooding situations.

10) Pressure oscillations in the suppression pool.

The above listed elementary loading cases can be combined in quite a number of load combinations (approximately 50) which, of course, implies aggravating circumstances for the design analysis.

The TVO containments are not calculated for seismic loads.

5. PRELIMINARY DESIGN CALCULATIONS

The first phase of the design work was carried out when the layout of the plant was decided. The main structural drawings with the concrete dimensions had to be available at an early stage of the project.

During this period of the design work rather extensive calculations were made but no large computer models were used. The different structural elements (slabs, walls, shells) were analysed by "hand" calculations or by minor time-sharing programmes with conservative approximations regarding boundary conditions etc. For each member only the most important load combinations were considered. With this approach we could judge that the concrete dimensions were sufficient to carry the applied loads with respect to bending compression and shear and that the bedrock stresses were within acceptable limits. As mentioned before, this was the main objective of the preliminary design calculations but we could as well define a first-step arrangement of the prestressing tendons. Moreover, we could judge that the necessary reinforcement areas (for membrane tension, bending tension and shear) were in any case reasonable with respect to available spaces within the concrete members. After we had carried out the preliminary design calculations, our remaining design problems could be listed as follows:

1) To calculate concrete shear and bending stresses with greater accuracy so that the calculations could be presented to scrutinizing authorities.

2) To calculate the necessary reinforcement areas in every section. On the one hand a considerable amount of loads had to be taken into consideration, on the other hand an excessive conservatism would result in important cost increases.

3) To check and possibly adjust the distribution of the prestressing forces.

6. FINAL DESIGN CALCULATIONS

6.1 General considerations

In order to solve the remaining design problems according to the preceding chapter, we judged it necessary to make the final design calculations by means of finite element models (FEM). On the one hand we felt that a model comprising the whole containment would be too difficult to survey and too expensive, on the other hand the models had to embrace a considerable number of concrete members if the FEM-calculations were to be more accurate than the preliminary calculations already made. Moreover, different parts of the containment could be approximated with different conditions of symmetry. On the FEM-programme we had the following special requirements:

1) The radial shear forces had to be listed if it were to be possible to calculate the shear reinforcement areas (if shear reinforcement was necessary). Many FEM-programmes cannot fulfil this requirement.

2) The output should be listed in a user-orientated manner so that the FEM-calculated forces and moments could easily be taken for the stress calculations. The format of the output paper should preferably be the same as the format used for the hand calculations as all the calculations of the project had to be copied and distributed to all the partners involved. The requirement on user-orientated printing of the output data was further underlined by the restricted time available for the design work after the FEM-calculations had been carried out.

   On the contrary we could simplify the FEM-models as the models were primarily made for the reinforcement dimensioning. We judged it adequate to approximate all the members as thin elements. The influence of the member thickness was of minor importance and could be neglected. If the members were to be modelled in transversal direction as well, the calculations would be more expensive and it would be difficult to use the output data directly for the dimensioning of the reinforcement.

   Moreover, the static loads were in most cases dimensioning compared to the dynamic loads. The latter ones could in many cases be approximated as static loads. Therefore the main FEM-models were made only for static analysis.

The programme chosen for the analyses was STRIP STEP S developed by the Swedish company Nordisk ADB.

The containment was divided in two FEM-models, one for the base and one for the roof part of the containment.

6.2 FEM-model of the base part of the containment

The model of the base part comprises the bottom and ring slabs together with the cylindrical and radial walls between the slabs, as well as the lower parts of the inner and outer containment walls and the bedrock foundation (Figure 2). The bedrock is simulated by means of a system of vertical walls coupled to each other and to the containment base and given such properties as to simulate "elastic halfspace" of the bedrock below the containment. Due to the symmetries of the structures and of the loads, the model is limited to include only a 22.5^{o}-segment of the base part. Deviations from the symmetry are analysed by manual calculations.

When modelling the structure (collaboration between the design engineer and the computer engineer) the different members were modelled primarily as thin shell elements. At connections between the base part and the cylindrical walls above the base part we thought it suitable to use rigid links.

The FEM-calculations were carried out in two stages. In the first stage the stiffness matrix was decomposed. The decomposed matrix was saved for later needs (if any). At the same time the different types of loads were analysed but no load combinations were made. The structural behaviour was studied and the results were compared with the results of the preliminary design calculations. The results gave no surprises. The conclusions after the preliminary design calculations were still valid.

Having made this scrutiny (including scrutiny of the FEM-model of the roof part of the containment), we found it suitable to make minor adjustments of the prestressing force distribution. The calculations were then made regarding the influence of the load combinations.

For the base part model we could restrict the number of load combinations to 17 but with the addition of manual corrections. The output data with corrections were used for the reinforcement dimensioning of the different structural members including checks of concrete bending and shear stresses. The reinforcement was dimensioned not only for tension and bending but also for the twisting moment and for the tangential shear. The calculations were made "by hand" and were rather time-consuming.

6.3 FEM-model of the roof part of the containment

The model of the roof part of the containment comprises the roof slab, all the pool walls and the bottom slabs of the fuel pools. The upper part of the outer containment wall is included in this model. Due to the (approximate) symmetries of the structures and of the loads the model was limited to include a 90^{o}-segment of the roof part. Deviations from the symmetry were analysed by manual calculations. The model is shown in Figures 3-5. The structure was modelled according to the same principles as for the base part.

The FEM-calculations were carried out in two stages in the same way as for the base part model. For the roof part model the number of load combinations were of the magnitude of 30. The experience gained from the dimensioning of the base part indicated that these calculations would be time-consuming and difficult to survey for the roof part. For these reasons we decided to make the reinforcement dimensioning by means of computers. At that time there was no such programme available in Sweden. After discussions with the computer engineers of Nordisk ADB we engaged them to develop such a programme based upon the output data from the programme STRIP STEP S. The programme also calculates concrete bending compression stresses and concrete shear stresses with the theory of cracked concrete (stage II).

7. CONCLUSIONS

The experience gained from the projects TVO I and TVO II regarding the use of computers for the design of a reactor containment in concrete can be summarized as follows:

1) The FEM-calculations shall be preceded by rather detailed preliminary design calculations. The design engineer ought to be sure that the concrete dimensions are thick enough before the FEM-models are made.

2) The computer calculation of the FEM-models shall be made step by step. The design engineer ought to control the results at least for the most important loads before any load combinations are made.

3) The decomposed stiffness matrix ought to be saved for possible future demand (i.e. additional loads).

4) The dimensioning of the reinforcement and the checks of concrete bending stresses and shear stresses ought to be made by means of computer programmes linked to the FEM-programme.

5) In most FEM-programmes there are routines for checking the geometry (for example by plots). For a containment analysis it is almost just as important to have routines for checking the loads. This was not carried out for the TVO I and TVO II projects.

   For containments to be designed later on we have asked the computer engineers to attach summation control of the loads. The coordinates of the load resultant are calculated as well. The design engineer can then make an easy check that the loads have been applied correctly or with only small errors (if there are any).

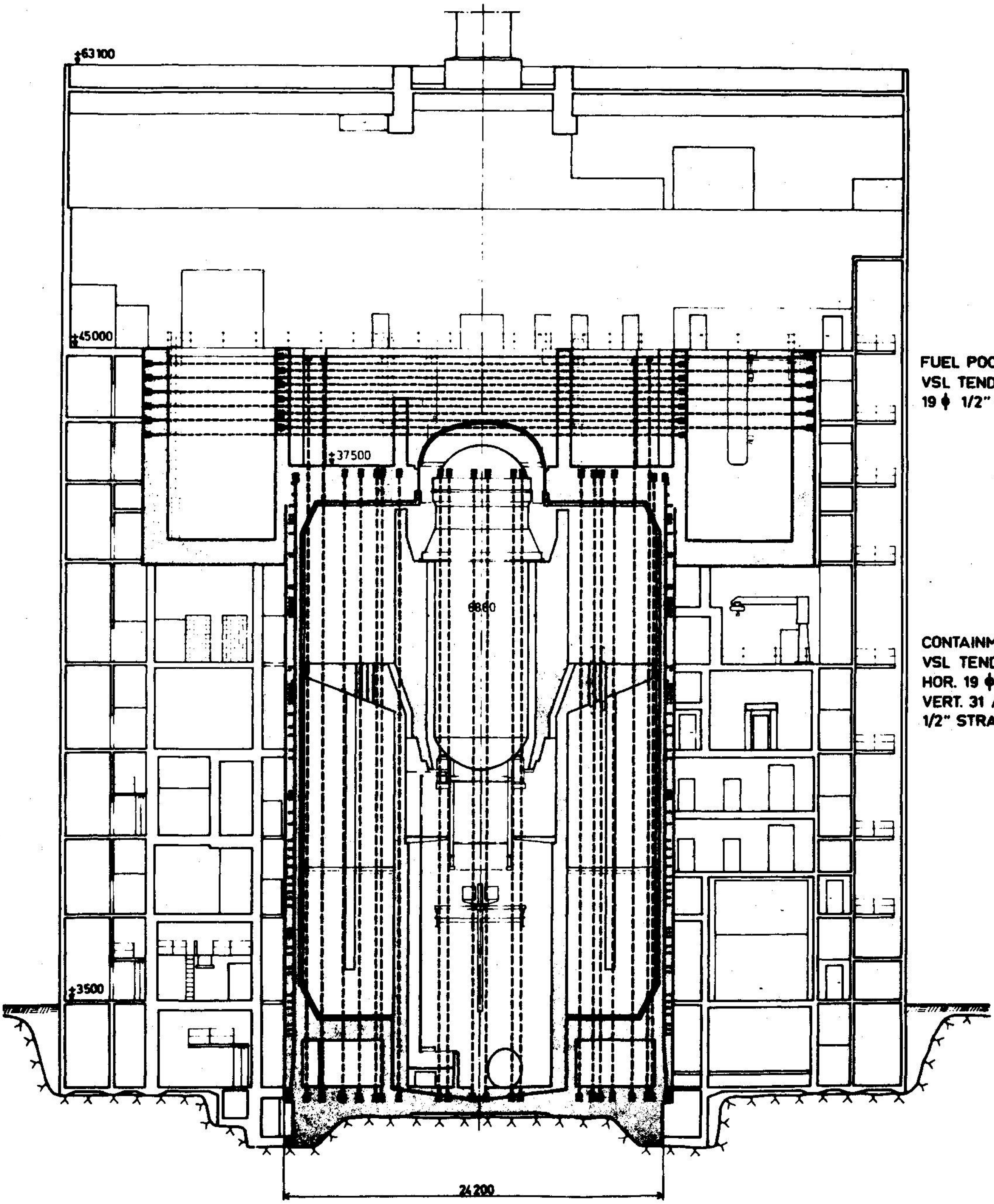
6) Most FEM-programmes print the results loading case after loading case. For calculations with tenths of load combinations, it is convenient to print the results node after node (with all load combinations for one node) as a supplement to the normal output.

7) The computer programme ought to be well known to the computer engineer, who must be competent and allowed to make adjustments and supplements to the programme in order to meet certain demands from the design engineer.

8) The preceding remarks indicate that there must be a close cooperation between the design engineer and the computer engineer. They must talk the same language. For that reason it is suitable if the computer engineer has a structural engineering background (preferably civil engineer). On the other hand, the design engineer must be wholly responsible for the safety of the structure. He must be skilled enough to judge the computer data correctly. If there are any errors in a certain computer programme, he can blame the computer engineers for that, but he cannot get away from the responsibility for the structure.

FIGURE 1

TVO OLKILUOTO NUCLEAR POWER PLANT I
ASEA-ATOM BWR 660 MW_e
REACTOR CONTAINMENT PRESTRESSING



FUEL POOLS:
VSL TENDONS
19 ϕ 1/2" STRANDS

CONTAINMENT VESSEL:
VSL TENDONS
HOR. 19 ϕ 1/2" STRANDS
VERT. 31 AND 19 ϕ
1/2" STRANDS

FIG. 2

TVO I

FEM-MODEL OF THE BASE PART OF THE CONTAINMENT

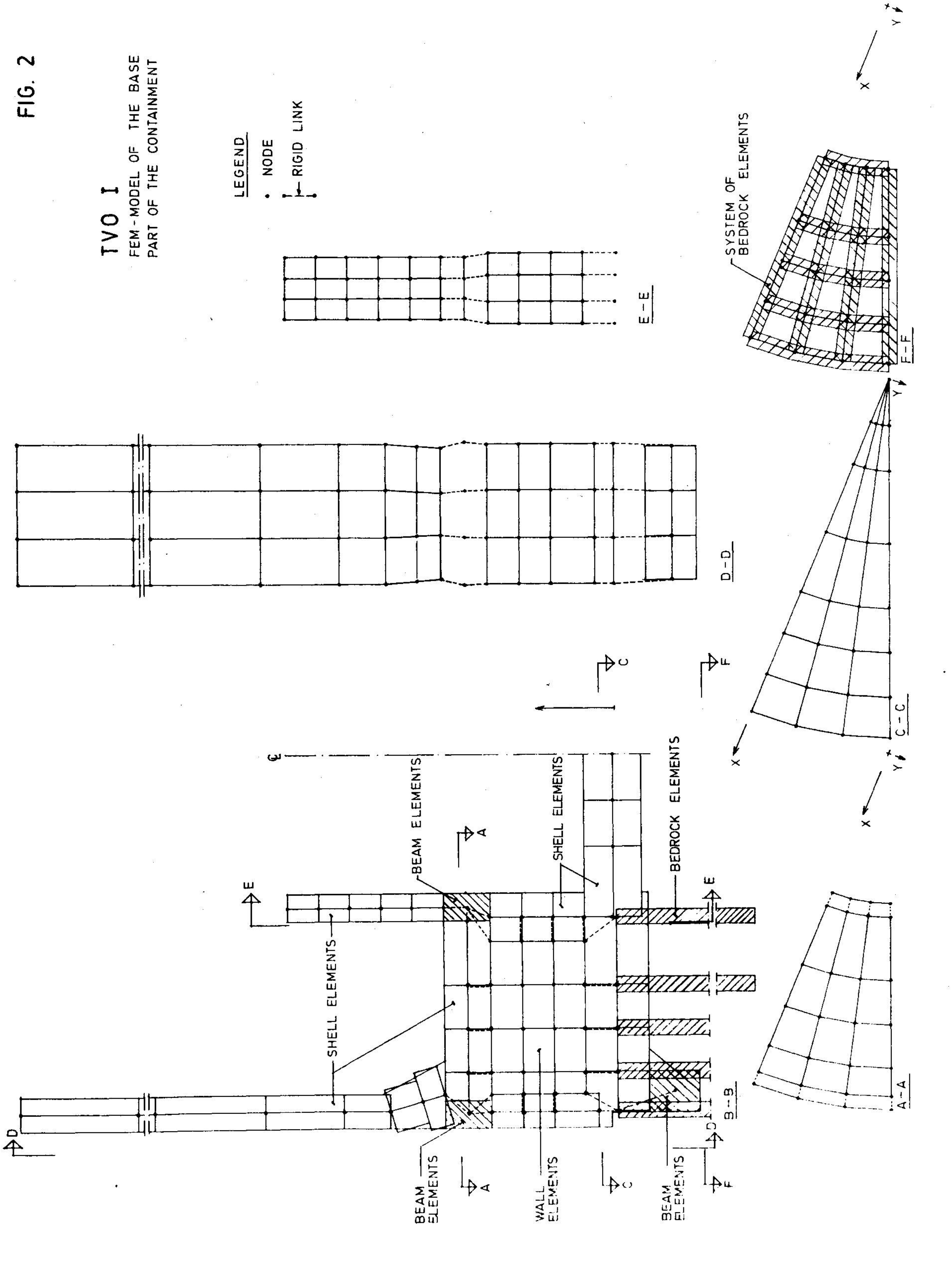
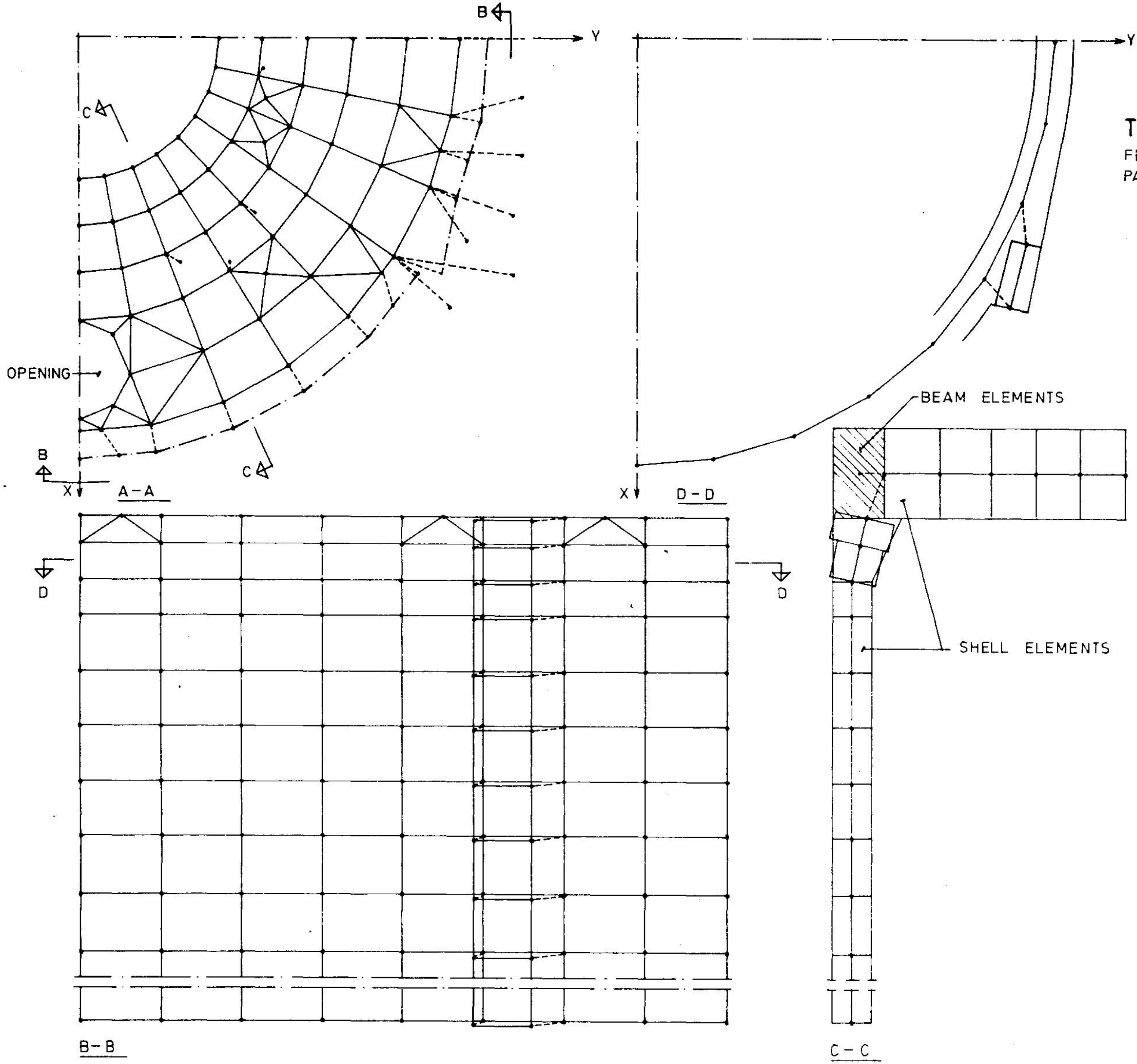


FIG. 3

TVO I

FEM-MODEL OF THE ROOF PART OF THE CONTAINMENT



LEGEND

· NODE

RIGID LINK

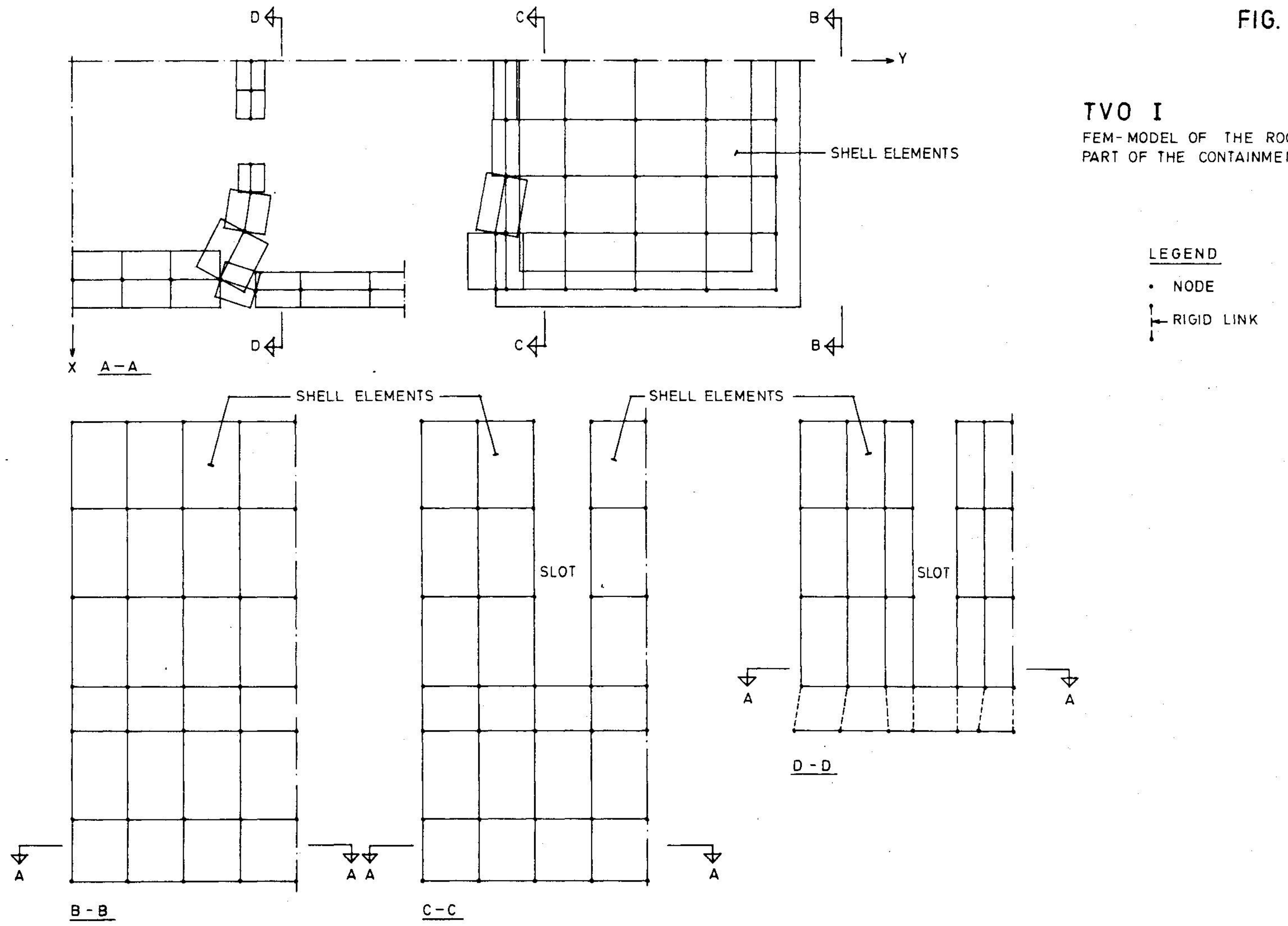


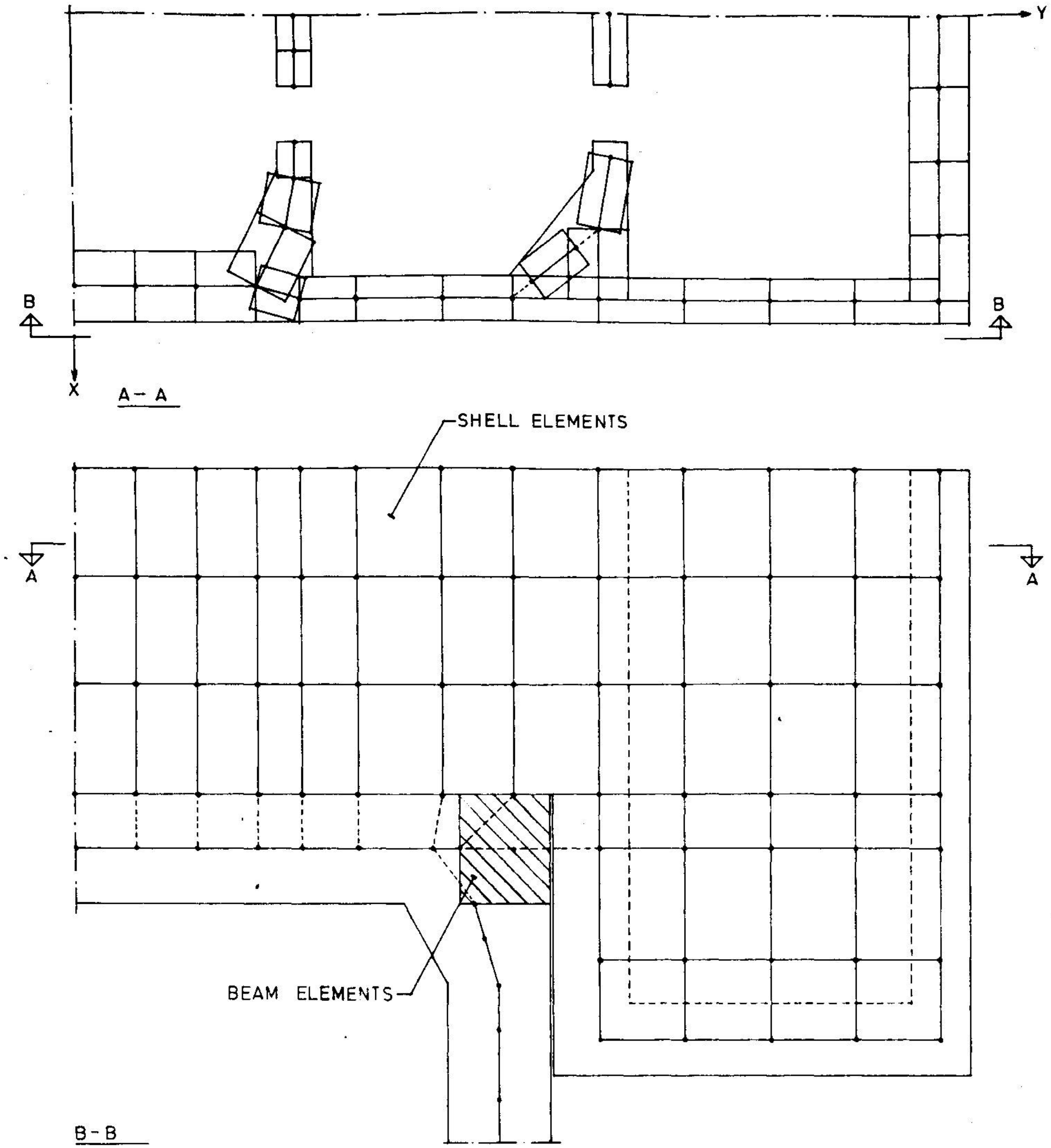
FIG. 4

TVO I
FEM-MODEL OF THE ROOF PART OF THE CONTAINMENT

FIG. 5

TVO I

FEM-MODEL OF THE ROOF PART OF THE CONTAINMENT



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COLLOQUIUM on:
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The Impact of Computer Development on the Art of Concrete Dam Displacement Control (a 15 yr, case-history)

Les consequences du developpement des ordinateurs sur l'art du controle des deplacements des barrages (un historique de 15 ans de travail)

Die Folgen der Rechenmaschinenentwicklung hinsichtlich der Dammverschiebungskontrolle (eine fünfzehnjährige Studienarbeit)

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Summary
The joint work carried out by CRIS (Center for Hydraulic and Structural Research) of ENEL - the National Power Board in Italy - and by ISMES (Experimental Institute for Models and Structures) in the field of rational numerical models for dam displacement forecasting provides excellent illustration of a gradual evolution made possible by the progress, over many years, of both hardware and software. The main steps in this development are recalled, with the aim to bring into focus the interplay between the researcher's intuitions, real phenomena as measured in the field and the world of computers.

Résumé
Le travail réalisé en commun par le CRIS (Centre de Recherches d'Hydraulique et de Structures) de l' ENEL (Autorité Nationale pour l'Energie Electrique d'Italie) et par l' ISMES (Institut Expérimental pour les Modèles et les Structures) de Bergamo, dans le domaine des modèles mathématiques rationnels pour la prévision des déplacements des barrages, fournit un example éclatant de l' évolution graduelle qui a été rendue possible par le progrès continu des ordinateurs et du logiciel. On fait l' historique des étapes successives de ce développement, dans le but de mettre en lumière l' interaction entre l'intuition des chercheurs, les phénomènes réels - tels qu' ils sont measurés en nature - et le monde des ordinateurs.

Zusammenfassung
Die gemeinsame Arbeit der CRIS (Center for Hydraulic and Structural Research) der ENEL (the National Power Board of Italy) mit der ISMES (Experimental Institute for Models and Structures) auf dem Gebiet effizienter numerischer Modelle für die Prognose von Staudammwerschiebungen ist ein hervorragendes Beispiel für die über viele Jahre hindurch kontinuierliche Entwicklung von "Hardware" und "Software". Die Hauptschritte dieser Entwicklung werden in Erinnerung gerufen, um die gegenseitige Einwirkung zwischen der Intuition von Forschern, der Realität und der Computerwelt aufzuzeigen.

1. FOREWORD

"In the last years a great widening of the field of use of automatic computational tools has become apparent in every technical application ... the static analysis of arch dam did not escape this general trend". Such words opened an article by FANELLI and CAPOCCHIA written in 1966. Nowadays - 12 years later - these same words still apply. Indeed, the development of "automatic computational tools" proceeded - and will presumably further proceed - at a breakneck pace, reaching higher and higher landmarks.

What only yesterday was almost a science-fiction dream is today a commonplace and will be tomorrow in the shade of obsolence.

If we keep strictly to the field of structural analysis, which is envisioned by the present Colloquium, we could add that already in the beginning of the Fifties one had forewarning glimpses of those advanced computational methods which were to reach full blossom in the following years, namely only after the introduction of ever more powerful electronic computers.

In this line of development, certain methodological avenues gradually opened up, which were not previously practicable owing to the overwhelming amount of computations to be performed.

A short historical review of the main methods successively used for design and behaviour analysis of arch dams clearly evidences this type of evolution.

Indeed, the first arch dams were analysed by the "independent arches" technique; very simple formules were initially used, such as the "thin pipe", or Mariotte, formula, or the "thick pipe", or Lamé, one. More elaborate "independent arch" formulas, taking into account either rigid or compliant end constraints, were then developed, such as the Guidi [1] and Bresse theory. Some dead-end alleys such as the "active arch" or the "arc plongeant" [2] methods were tried and abandoned, till at last very elaborate statical schemes were conceived: typically, an interconnected arch-cantilever lattice, see the Guidi-Ritter scheme, or a continuous, curved plate or shell (Tölke et al.) [3].

In the same years, a fully codified and organic computational method was put forward by the Bureau of Reclamation: the "Trial Load Method" (TLM) [4], a refinement of the Guidi-Ritter scheme. The TLM lent itself readily to automatic, or semi-automatic, treatment. In fact, at the beginning of the Sixties some "fully" automatic procedures, such as the "Algebraic Load Method" (ALM) [5], [6], the "minimum potential energy method" (CST) [7], [8], the "Modified Ritter method" (MRM) [9], [10] etc., were implemented on the "first generation" of commercially available electronic computers. All of these methods were aimed at taking into account - by means of different approaches - the main structural resources of arch dam, whose shape and size could be varied at will, within limits, by the user. The Vogt theory [11] allowed a simple, albeit approximate, way of representing the foundation compliance. Also the first theoretical developments of the Finite Element Method (F.E.M.), which was to gain such widespread acceptance toward the end of the Sixties, date back to 1956 [12] and to the availability of first-generation computers as well as of first-generation programming languages.

Concurrently with the development of the above-mentioned automatic computational tools, the possibility came to light of endowing each dam with a "tuned" procedure of continuous displacement check-up. The latter was to be based, in principle, upon "a priori" computations giving theoretical estimates of each displacement: these estimates being the sum of a "water level variation" effect and of a "thermal load" effect [13] [14] [15] .

II. CORNERSTONES OF THE CHECK-UP METHOD; FIRST APPLICATIONS BY MANUAL, OR SEMI-AUTOMATIC, MEANS

In 1965 a paper by FANELLI [16] set down the cornerstones of a rational check-up procedure for arch dam displacements. A deterministic correlation was postulated to hold between the external actions (water level variations, thermal variations) and their externally observable structural effects, such as displacements, rotations, unit elongations etc. Basically, the hypothesis of linear-elastic behaviour of the dam material is retained:

$$\varepsilon = \frac{\sigma}{E} + \alpha \, \Delta\vartheta$$

(Hooke's law : E = Young modulus, α = thermal dilatation coefficient, $\Delta\vartheta$= local thermal variation); namely, linear superposition is assumed to hold for separate contributions due to stress level changes and to temperature changes.

The practical implementation of this basic idea needed two main supports:

a) Setting up a proved, automatic computational procedure in order to simulate the structure behaviour under certain loading conditions;
b) a formulation of suitable "unit loading conditions" such that any instantaneous situation could be reasonably well approximated by a linear combination of the unit conditions chosen. In this way, also such external structural effects as the displacements, rotations and so on could be assigned "theoretical" estimates given by linear combinations of "influence coefficients" computed for the corresponding effect under each unit loading condition. These "influence coefficients" would be computed by the automatic computational procedure under a) by using as inputs the unit loading conditions as per b) .

The development of a "unit load conditions" definition technique such as under b) was made necessary, in particular, for thermal effects estimation. Otherwise, any particular thermal situation would need a long series of manual operations, at the end of which a corresponding particular thermal load would have obtained; and this, in turn, would still have to be introduced as input in the procedure under a) in order to get the corresponding structural effect estimate. Such an approach, of course, would have been intolerably money - and time - consuming, to the point of making a continuous check-up practically unfeasible. On the other hand, this "unit thermal load conditions" definition technique (referred to hereafter , for brevity sake , as the "monothermometric technique") achieved the goal of a substantial reduction in the amount of the computations required by each particular thermal situation only at the expense of introducing certain simplifying assumptions (these were later largely relaxed, see further on) . Among these simplifying assumptions were the linearization of thermal distribution inside the arch thickness (fig. 1) (°) as well as the Navier-Stokes hypotesis of plane-section conservation.

(°) Each "unit loading condition" at the arch level had to be transformed in an equivalent linear temperature diagram, which had to conserve the same average ($\bar{\tau}$) as the diagram of the unit thermal load as well as the same barycentric abscissa (this latter condition allowing one to deduce the linearized thermal jump, $\Delta\tau$).

With this "monothermometric technique" many sets of unit loading conditions had still to be introduced into the numerical model; but all of these computations were carried out only once, before actual beginning of the check-up process, which was then reduced to the very simple form of a linear combination of the influence coefficients which were the model output under the different "unit loading" conditions.

The above-defined logical scheme needed a powerful, easy-to-use structural analysis program, in order to allow the building-up of the corresponding unit influence coefficients. Of course, such a program could also be used as a general tool for the static analysis of arch dams subjected to any set of loads.

Around 1958-1960 we had no general-purpose structural analysis program. The decision was taken to develop a specialized code, restricted to arch dam analysis. The outcome of such a decision was the so-called "CST" program, implemented on a UNIVAC USS/90 computer system. The latter consisted of three I/O peripheral units (card reader, card puncher, printer) and a central unit with a magnetic-drum core of 5000 words capacity (each word having 10 characters plus algebraic sign). No auxiliarystorage was available; half of the above-mentioned central memory was taken up by the interpretative program, so that working storage capacity was severely restricted. Because of this, the overall computation was subdivided into 16 successive steps (fig. 2), the punched card being used as the physical support of data and program. Each successive program segment performed a single computation step, after checking and storing the segment instructions as well as the necessary data; at the end of each step the results were printed and card-punched in order to be fed (as an input) to the successive step. (fig. 3). This program was written using "advanced" programming techniques (as compared to the state-of-the-art prevailing at the time) in order to impart to it the greatest flexibility: none of the program steps had to be modified when the shape of the dam to be analysed was changed.

The limitations of the system conditioned the choice of the method of analysis. In fact, linear algebraic systems could be solved by UNIVAC USS 90 only if the number of unknowns was less than 64 (with symmetric matrix of coefficients) or 48 (with unsymmetric matrix). Given this limitation, the method develop (based on the principle of minimum potential energy as already stated) used special "degrees of freedom" which allowed such a reduction in the number of unknowns, at the expense of rather heavy preliminary computations. (°)

The whole program was made up by 12, 000 instructions in interpretative language, that were punched on 4, 000 cards. In addition to this, about 2, 000 cards were needed to record the input data for a medium-size arch/cantilever grid (about 7 arches and 7 cantilevers for half of the dam, which was supposed to be symmetrical).

(°) One may observe in passing that the "modern" approach can take the opposite choice (many unknowns, very simple "degrees of freedom" and scanty preliminary computations) thanks to the enormously increased capacity and speed of present-day computers.

During the period of use of CST program, which covered several years, about 120 hours of computer time were used for 34 complete analysis of arch dams.

By way of example, some results are presented concerning the displacement check-up for a crown target of Isolato arch dam. Fig. 4 shows the chronologic diagram of computed and measured displacements for years 1954-1964; also shown are discrepancies between "forecast" and "observed" values and their frequency distribution. Fig. 5 exemplifies a chart, embodying the influence coefficients computed with CST in the above-described way, by means of which the synthesis of the "theoretical" displacement could be effected graphically, once the water level and the thermometric readings were known.

Till 1969 this early type of displacement check-up was used for only six italian dams, owing to limitations in computer time availability and to the necessity of validating the basic conceptions of this methodology through a suitable period of experimentation.

III. COMING OF AGE OF F.E.M. AND THIRD-GENERATION COMPUTERS

At the beginning of the Seventies, larger and faster computers were made commercially available by IBM, Honeywell, CDC, UNIVAC etc. The capacity of automatic computations was so increased as to change radically working methods in every engineering field. The main innovations brought on by these so-called third-generation machines concerned the hardware as well as the software. The new developments of hardware can be summarized as follows:

a) a fast memory by far exceeding in capacity any previous possibility and having almost limitless expansion potential;
b) several direct-access peripheral units such as : mag-tape, disk, fast paper tape or card reader/puncher, fast printer etc. ;
c) possibility of long-distance access to large "remote" computers by means of "terminals" ;
d) indirect-access units such as interactive graphical systems, plotters, "pencil-follower" tables for coordinate reading, etc.

In the same time the available software became ever more flexible and comprehensive, so as to allow development of computational codes using "programming languages" both simple and accessible to non-specialists. In other words, a deep knowledge of the machine and of its internal language was no longer necessary in order to write a reasonably efficient program.

In this period many computer codes were indeed developed, addressed specifically to the solution of important engineering problems. In particular for the analysis of static and dynamic behaviour of structures, big programs or even "systems" of programs were written based on the F.E.M.

It would be beyond the scope of the present paper to dwell at length on such a widely known method [17] ; suffice it to say that it was and is often used - since the beginnings - also to study the behaviour of dams of the most various types: arch, arch-gravity, gravity (massive or hollow), buttress etc. This new general tool allowed, indeed, a better schematization not only of the structure itself, but also of its foundations; besides, also the effects of local details or singularities could be more faithfully simulated.

Also the implementation of displacement check-up methodology found in these new developments a natural vehicle for advances in basic model approximation flexibility and scope of applications. The starting assumptions were mantained, namely the superposition of separately computed effects for hydrostatic load and for thermal loads, but the techniques for building up "influence functions" was considerably refined, especially so for the "monothermometric coefficients", whose computation was made not only fully automatic, but remarkably more accurate at the same time.

Fig. 6 illustrates the logical flow-chart of the present version of our deterministic displacement-forecasting method [18] [19].

As far as the computation of "monothermometric coefficients" is concerned, the present methodology is based on a F. E.-oriented solution ("TERFUN" program) of the heat-conduction equation :

$$a \nabla^2 \vartheta = \frac{\partial \vartheta}{\partial t}$$

[a = thermal diffusivity coefficient, $\vartheta = \vartheta$ (x, y, z, t) = temperature inside the dam],

with limit conditions $\vartheta = T_j$ at thermometer n° "j" (which can be located either on a dam facing or in the interior), under the simplifying assumption of periodic, sinusoidal variations of ϑ (the period is usually assumed a yearly one). Since a synthesis of the overall temperature distribution is needed starting only from the known thermometric readings, the estimate of the temperature at the generic point (x, y, z) at general time t is assumed to depend on these readings through a relationship of the type:

$$\vartheta(x, y, z, t) = \Sigma_j b_j T_j(t) + \Sigma_j c_j \frac{\partial T_j}{\partial t}$$

which is consistent with the hypothesis of sinusoidal time-variations of T_j ($1 \leq j \leq M$, if M is the total number of installed thermometers).

In this expression, b_j and c_j are spatial distribution coefficients $b_j = b_j(x, y, z)$; $c_j = c_j(x, y, z)$ respectively for unit temperature variation at T_j (and zero variation at any other thermometer) and for unit thermal time-gradient at T_j. Such distribution coefficients are automatically computed by the "TERFUN" program. The thermal diffusivity coefficient a, whose knowledge is necessary as an input data to TERFUN, can be estimated from the thermal time-history of the inner control thermometers using another special F. E. program, named "TERDIF".

Spatial-distributions embodied, for each unit thermometric variation, by coefficient b_j, c_j make up a thermal input for the structural analysis F. E. program "TRITEN"; the output being the above-mentioned "monothermometric" influence coefficients for displacements (or any other structural quantity).

The heavier part of this present version of our methodology is tied to the creation of a 3-D F. E. mesh for the geometric input to TRITEN: this requires a considerable expenditure of man-hours. A possible improvement in this direction lies in the development of new types of F. E., such as the "hyerarchic" family, which can yield very high-precision results, even with relatively coarse meshes, so that the time required for their geometric definition can be substantially reduced.

Fig. 8 shows some results of displacements check-up for the Talvacchia arch-dam crown plumb-line (obtained with this more recent methodology).

The advent of general-purpose F. E. methods allowed us to extend the same check-up methodology also to other dam structural types, e. g. gravity dams. The first results were of particular interest, insofar as they evidenced some, hitherto neglected, components of structural behaviour. Indeed, the study of displacement behaviour of Barbellino gravity dam showed that not only the dam and foundation deformability had to be accounted for, but also the whole "regional tilting" of the impoundment basin under the reservoir water level variations affected the overall observed displacements and had therefore to be included in the theoretical model. Figs.9, 10 show the forecast displacements obtained with, respectively without, such regional tilting effect. It is evident that these methodologies offer a valuable means of assessing possible trends of time - evolutions in the mechanical characteristics either of the concrete or of the foundation rock; such information is of outstanding importance to follow the current state of safety of the dam.

IV. CONCLUSIONS

The dam displacement check-up, which can be taken as a signal instance of in-depth structural analysis - even under its present limitations to the linear - elastic field - underwent, in 15 years time, a remarkable evolution, for which the spectacular improvements in capacity, speed and flexibility of computers (and their ancillary equipment), as well as in software power, were largely the prerequisites. On the other hand, such an influence only made possible operations never before attempted because of their complexity, but could not by itself produce new concepts or lines of approach: the latter are still firmly in the hands of the research people, who were ready to take advantage of the "amplifier" effect brought around by computers in terms of number-crunching power.

If sometimes the sheer availability of such computing power fostered serious consideration of "new" types of analyses never before attempted, this availability was always a necessary, not a sufficient, condition.

Every new approach in the particular field here illustrated was checked against the actual behaviour of real-size structures. This allowed a balanced development and the eventual formulation of an organic proposal for the design of an on-line, real-time, continuous check-up system, based on present - day capabilities of computers (big ones as well as micro) and harmonically integrated in the public utility management . (°)

It would, however, be unfair to leave unmentioned some outstanding unsolved problems :

- management problems : which operative decisions should be taken when the check-up system shows an "abnormal" event? In this field the computer could help in making possible a rational (optimal) choice between the existing alternatives: a very complex problem of optimal strategy choice should in this case be posed, taking into account many parameters (width of tolerance bands, risks etc.) . Unfortunately, many of the quantities involved (e.g. estimates of the economic consequences of accidents) are very difficult to define quantitatively.
- responsibility problems : where should the responsibility for taking operative decisions reside? Public utilities local and top management, public Authorities, public opinion are all involved; in this area, however, the computer cannot give much help (save perhaps in the field of automatic classification, storage and retrieval of information about past experiences).

(°) Let us recall in this context the efforts being deployed within the ENEL organization toward remote power-plant operation.

- technical and organizational problems, such as data archive updating, mathematical model updating; maintenance and updating of check-up systems; links with research organizations, etc.); these could all profit from present-day computer capabilities.
- economic problems (cost and cost/benefit analysis of system features and specifications; evaluation of possible alternatives etc.) Here the computer could help in performing lengthy and complex analyses, but only under condition that all the terms of the question were quantitatively known (or could be parametrized within reasonably limited ranges), which is not always the case.

These problems must be clearly focussed and a serious effort is needed to solve them. Otherwise, the almost limitless possibilities offered by computers cannot be fully exploited: worse than that, a risk is run of creating a delusory sense of safety and/or a conflict of competences and loyalties, leading to greater confusion.

The organizational structure of our technical world and, more generally, of our society is not yet prepared to cope with the impact of computers. Let us take a clear conscience of this "fact of life" and start looking for means and procedures by which these new, powerful allies can be made "compatible" with old, but always worthwhile, goals such as greater safety and well-being for everybody.

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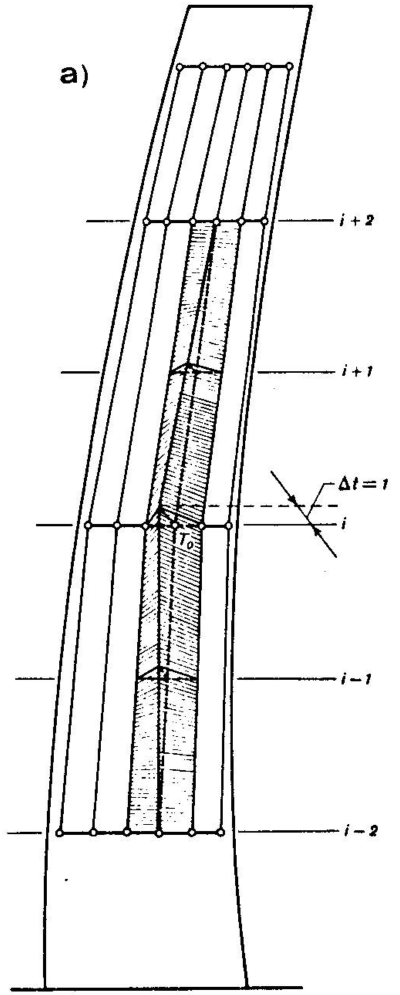
a)



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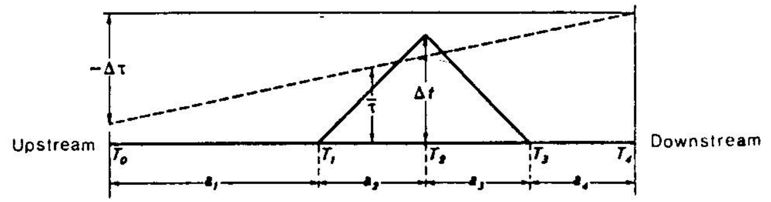


a) Thermal distribution in the crown-cantilever for the unit temperature variation in the thermometer T_0.

b) Average temperature ($\bar{\bar{T}}$) and linear thermal difference between upstream and downstream ($\Delta\tau$) for a variation $\Delta T = 1^{\circ}C$ affecting only the thermometer T_2.

Fig. 1

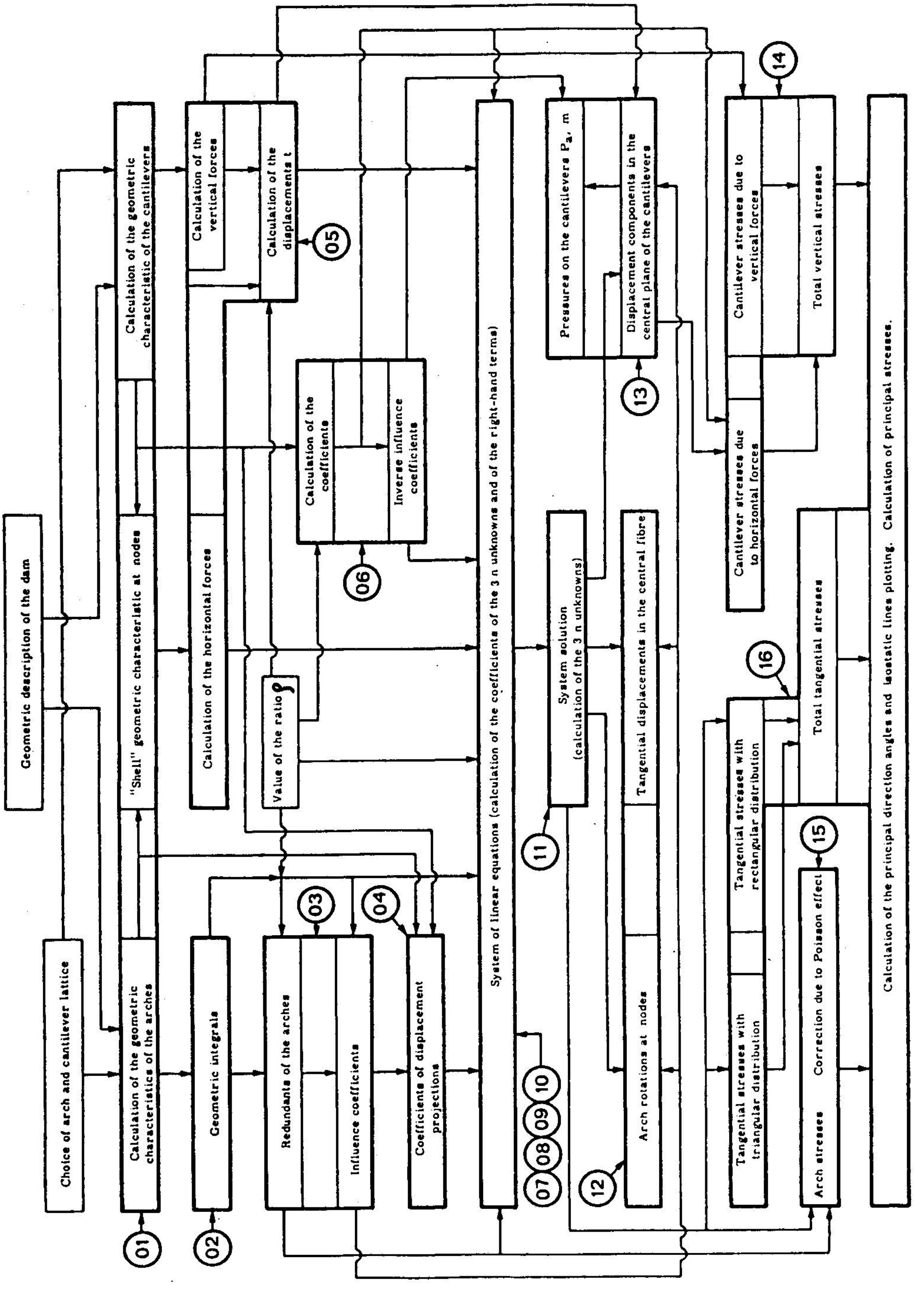


Fig.2 FLOW CHART OF THE PROGRAM (16 STEPS)

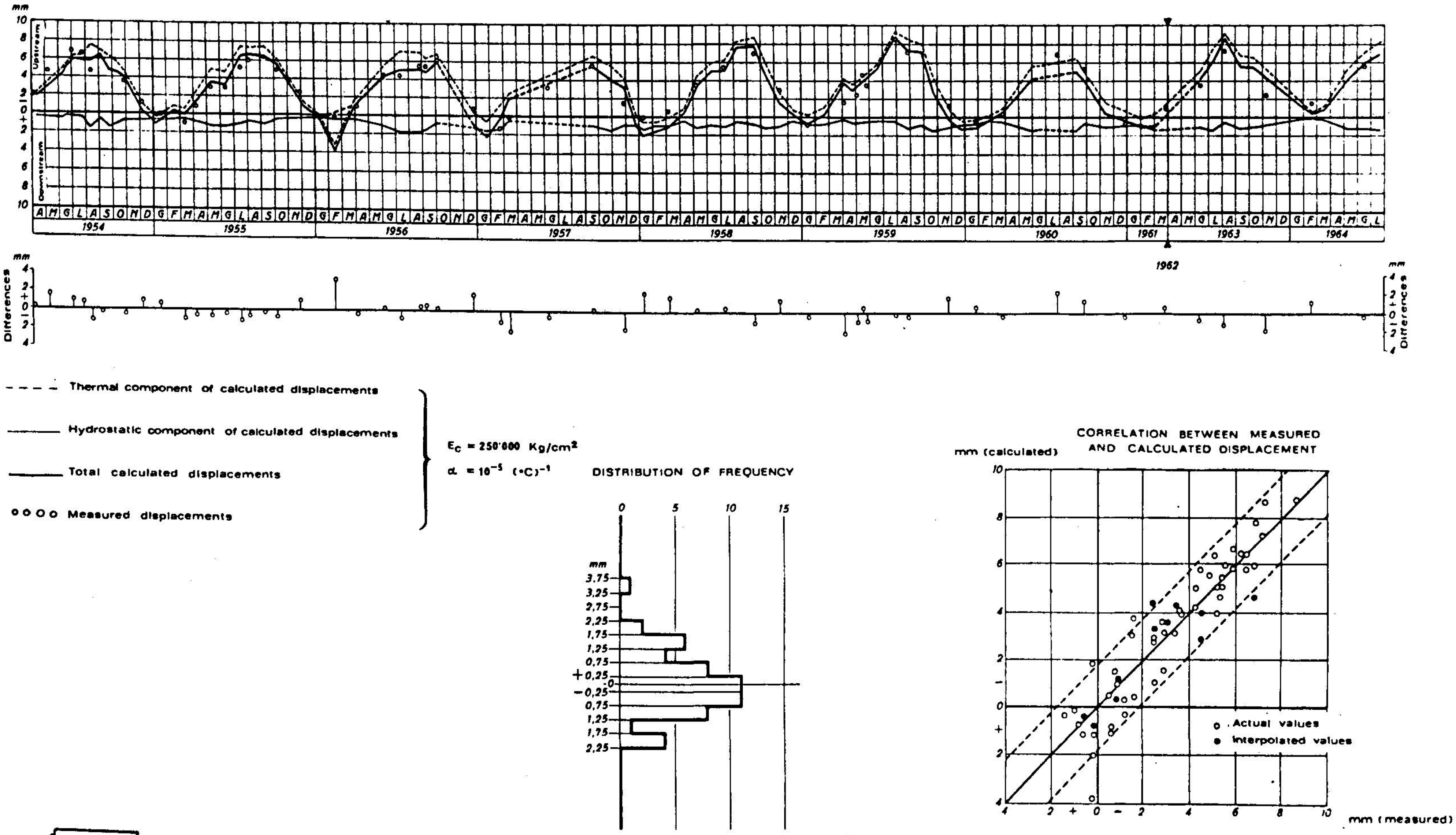
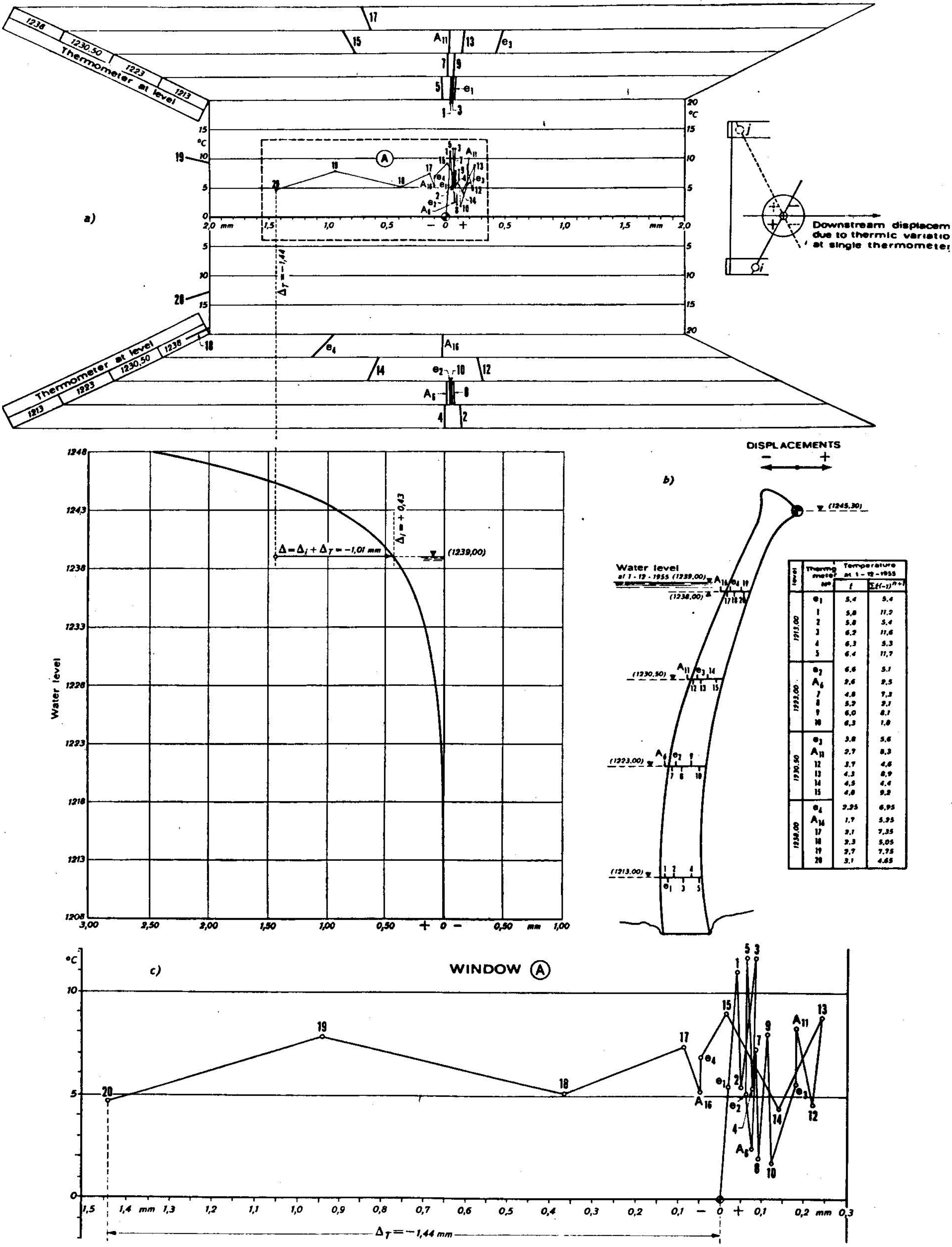


Fig. 3 Isolato arch-dam. Comparison between measured and calculated displacements for the movable staff at the level 1245.30 m. slm (yrs 1954 - 1964).



level	Thermometer Nº	Temperature at 1-12-1955 t	$\Sigma t(-1)^{n+1}$
1213,00	e_1	5,4	5,4
	1	5,8	11,2
	2	5,8	5,4
	3	6,2	11,6
	4	6,3	5,3
	5	6,4	11,7
1223,00	e_2	6,6	5,1
	A_6	2,6	2,5
	7	4,8	7,3
	8	5,2	2,1
	9	6,0	8,1
	10	6,3	1,8
1230,50	e_3	3,8	5,6
	A_{11}	2,7	8,3
	12	3,7	4,6
	13	4,3	8,9
	14	4,5	4,4
	15	4,8	9,2
1238,00	e_4	2,25	6,95
	A_{16}	1,7	5,25
	17	2,1	7,35
	18	2,3	5,05
	19	2,7	7,75
	20	3,1	4,65

Fig. 4
a) Graphic calculation of the theoretic displacement at 1. 12. 1955
b) Crown-cantilever of the Isolato arch-dam and thermal situation at 1. 12. 1955
c) Window A - Graphic calculation of the thermal displacement at 1. 12. 1955.

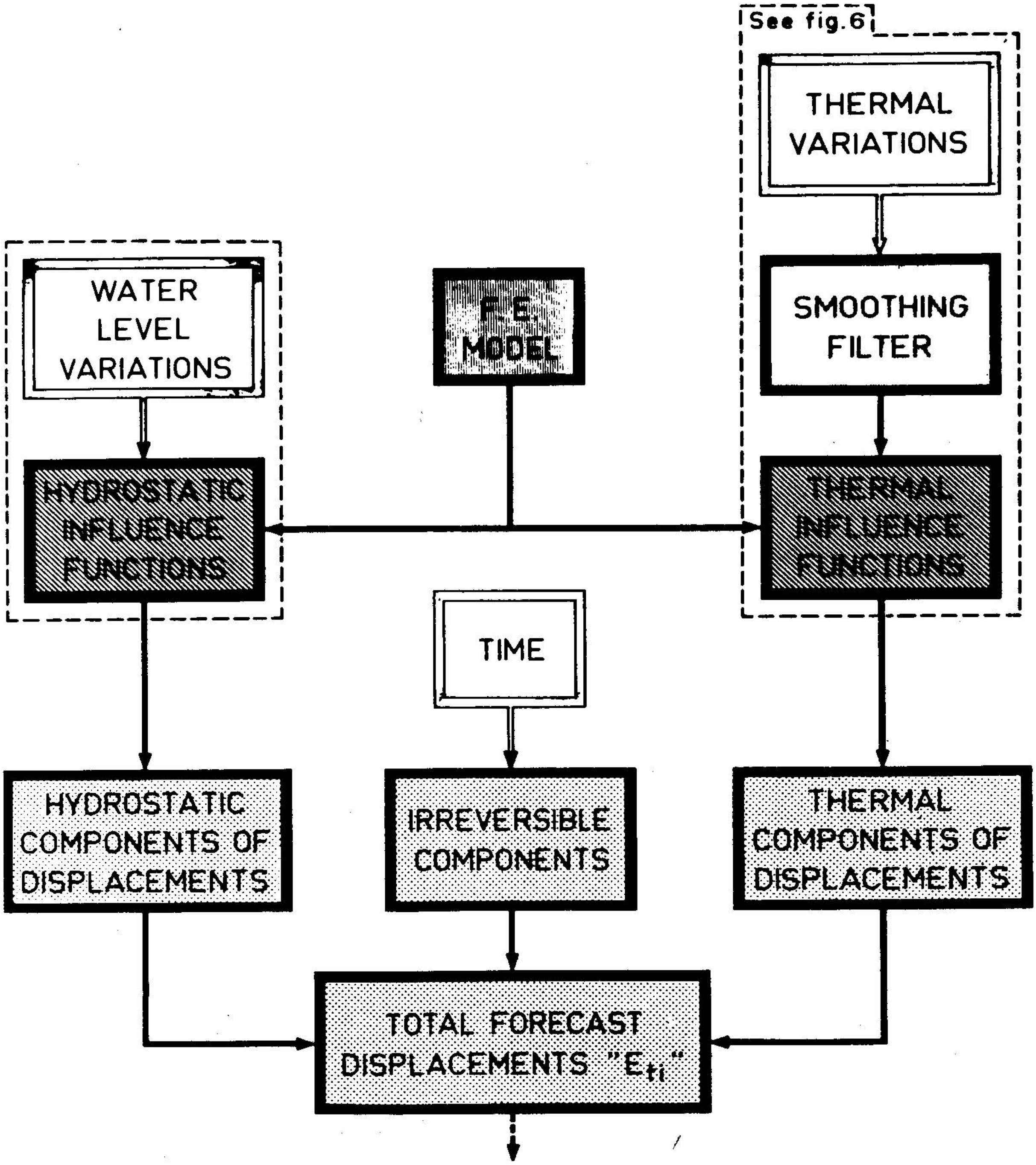
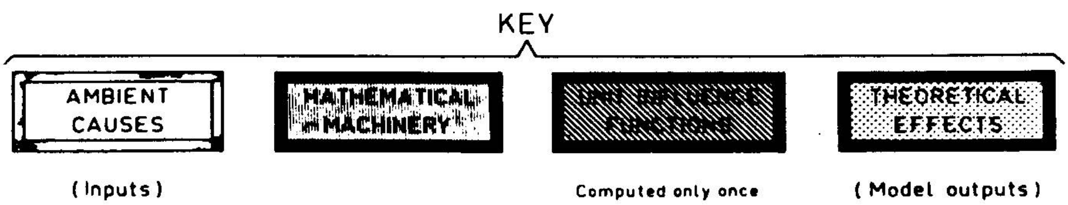


Fig. 5 GENERAL LAYOUT OF THE DETERMINISTIC, F. E. MODEL



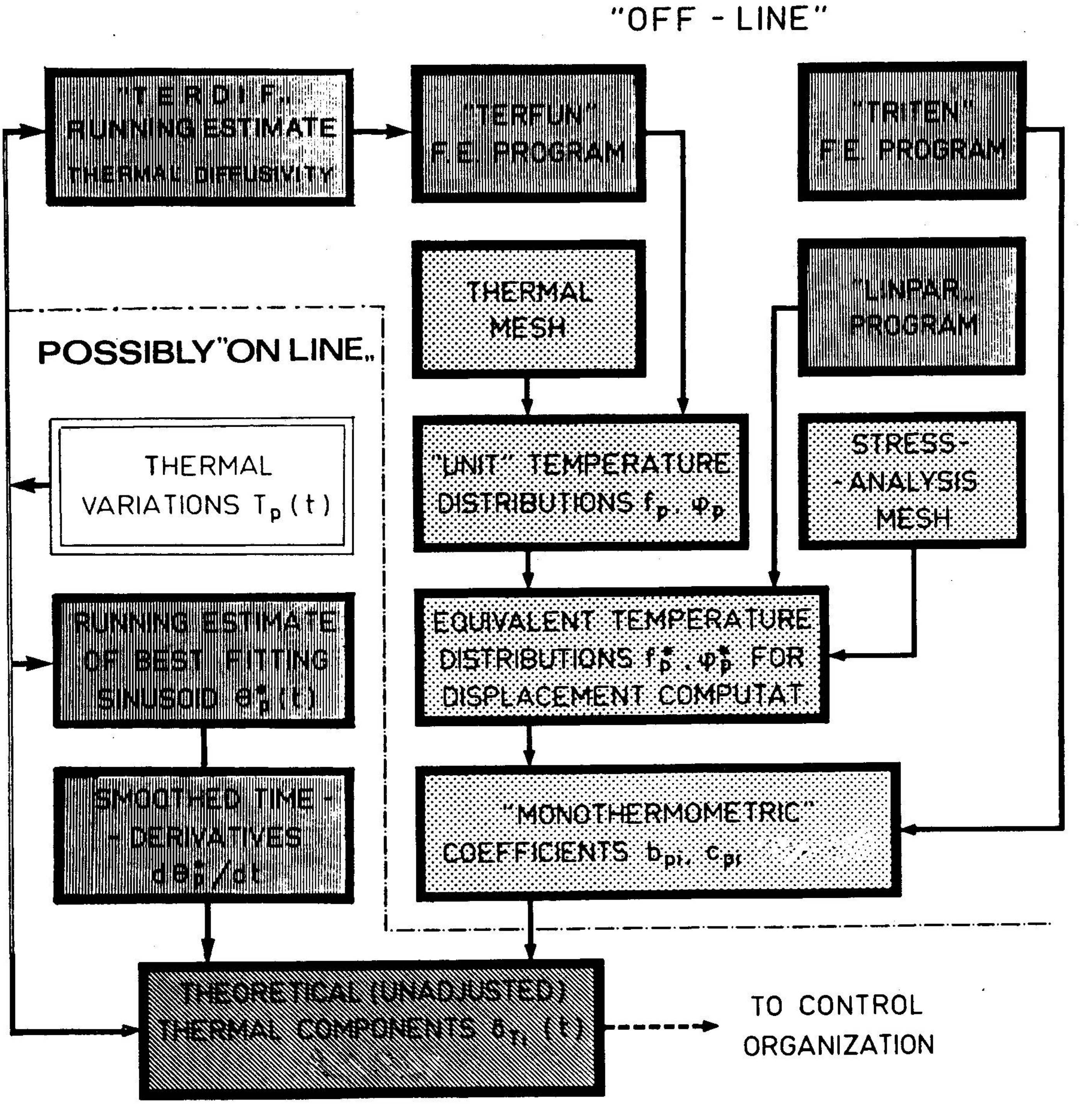
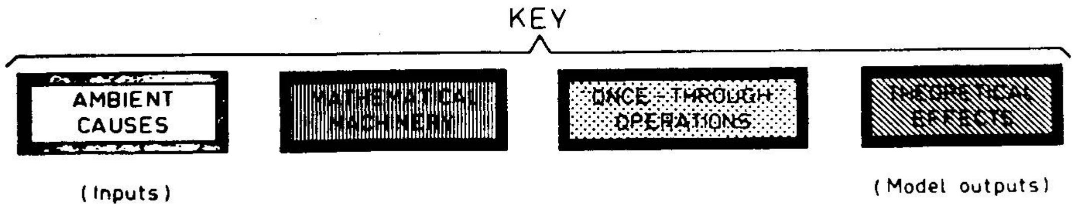


Fig. 6 FLOW - CHART FOR COMPUTATION OF "MONOTHERMOMETRIC COEFFICIENTS"



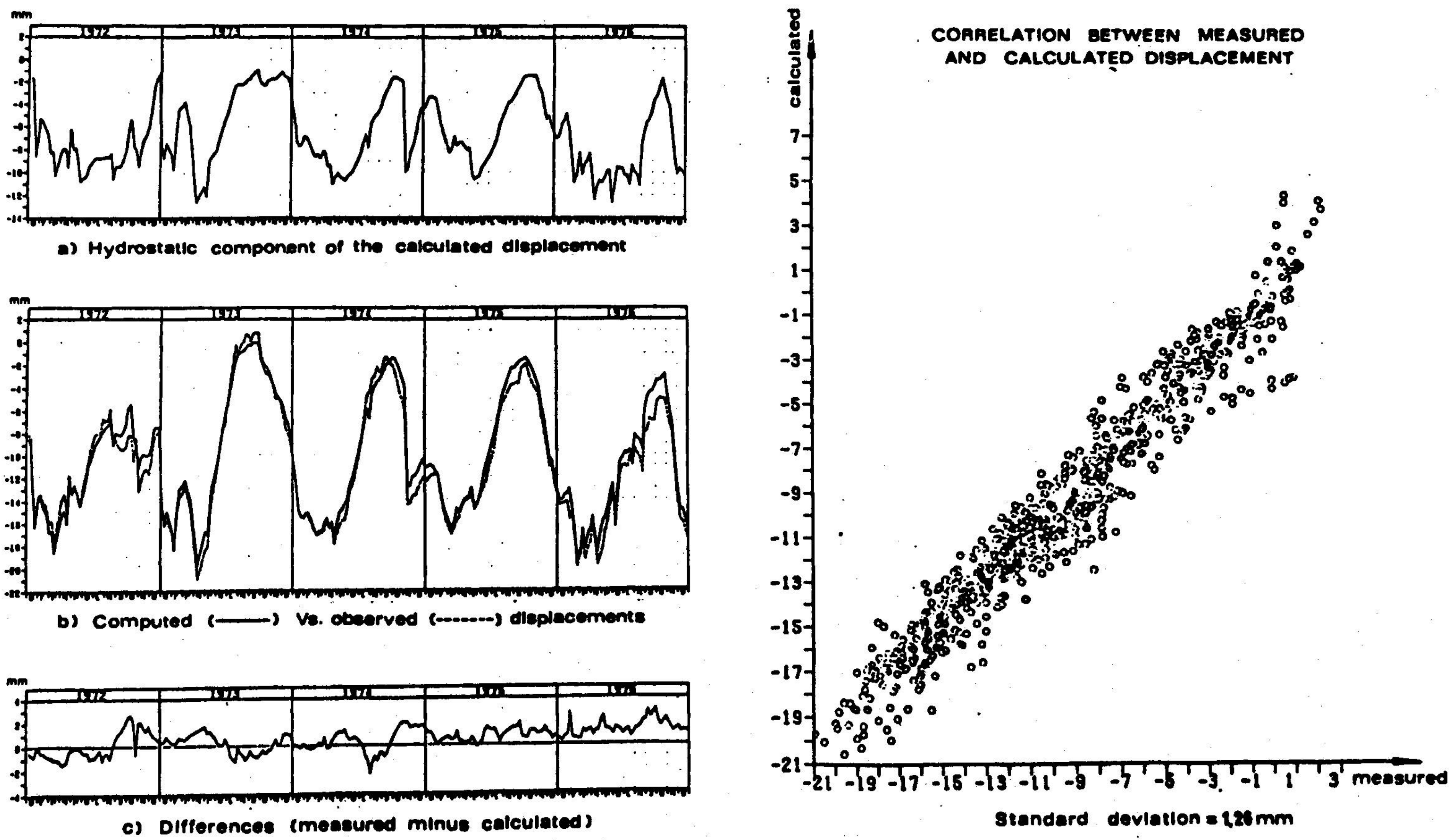
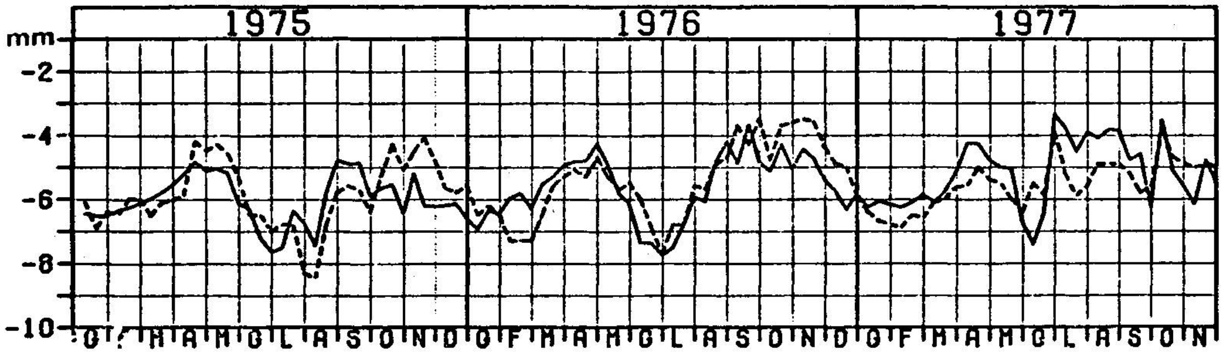
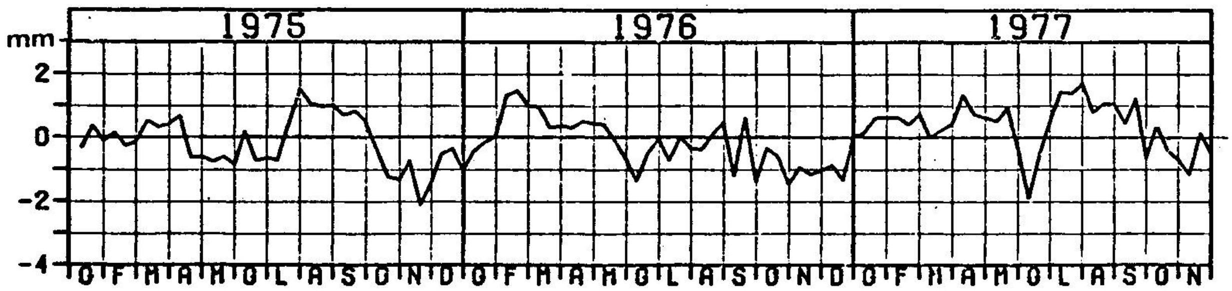


Fig.7 Talvacchia arch-dam. Comparison between measured and calculated displacements at the plumb-line (yrs 1972-1976).

Plumb - line



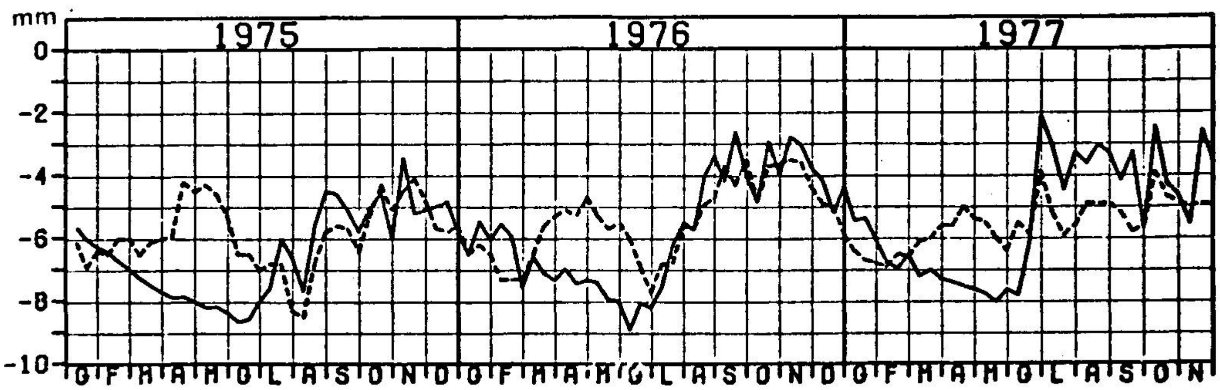
a) Computed (———) vs. observed (------) displacements



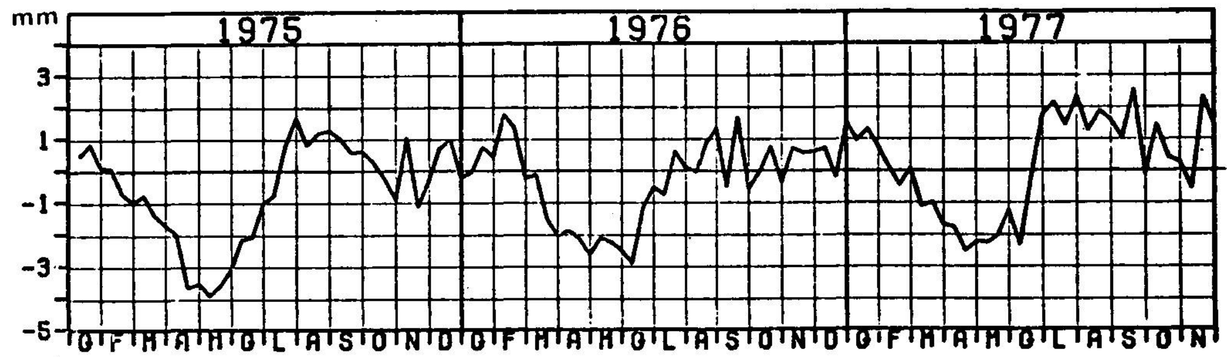
b) Differences

Fig. 8 Displacement forecasting for Barbellino gravity dam with Boussinesq regional tilting included.

Plumb - line



a) Computed (———) vs. observed (------) displacements



b) Differences

Fig.9 Displacement forecasting for Barbellino gravity dam without Boussinesq component.

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SUPPLEMENTARY SESSION

DISCUSSION

September 1, 1978. Afternoon.

Chairman: KLEMENT (Austria)

KLEMENT - We listened to seven papers about "case histories" and first Mr Vos gave us a very interesting view on the building of such an artificial Island. I ask if there are any questions or discussions in this respect.
I think we must have thousands of questions, of course. I want to ask Mr Vos all the loadings which you had. In that way they were decided: you cannot design such a framework from nothing else, so you have to consider a lot of solutions. In what way was this reached ?

VOS - Well, this was not an easy job, but let us first say that the FIP, the Federal International Precontraint, has done some work on concrete structures and I must say that really there was a great work from my colleague from Europe Etudes. About how to cope and how to deal with, the main problem, in this case, was the loading of other construction stages and very important were tolerances too, together with creep and these heavily loaded discs. So, anybody who was thought expert in load case was really asked to work and control it.

DUTERTRE - I would just add one word about this structure. It was designed very conservatively, because it was the first one, and, about finite elements, we did not use them to design it, it was designed with the old stress analysis. But the way which had to be taken into account in the design took three more times , since the structure is on the sea.

KLEMENT - Is there another question or another discussion point to this ? Then, I will ask for the bridge designing programs and their comments or questions.

BLAUWENDRAAD - As regards to the paper concerning creep problems , I have a question. I think that the real problem arise when you design a bridge and take into account the effect of creep at the several stages; you have to adopt figures on the creep you do not know exactly. At least, I think that is the Dutch practice. At the time you predict the behaviour of the structure, the material which will be used is not known yet. What about the feedback in the bridges ? Is there any experience whether the measures may be true or not ?

HAAS - When we do the analysis, we consider the German standards. Unfortunately, there is very little feedback. A bridge was carefully analized but where you wanted to measure the deflections unfortunately the first measurement was made in a very improper way, so we are no longer able to compare the measurements and they are very important in this case, but we have just analyzing a third

bridge of this kind and this bridge will be very well measured.
In a couple of years, I think that we will have some data in order to check our analysis, but this is not a check of our program, because I think our program works well. It is in the range of the models which has done by the German prestressing guidelines. It is a check whether prestressing guidelines are in accordance with the structure or not.

KLEMENT - I have my own opinion on this subject. In the structures, creep and shrinkage are very important for the deformations and maybe for stresses, but not so much for the safety of the structure: actually, the shrinkage and the creeping are sometimes nearly twice as much.

FISKUM - I have followed two bridge measurements: the first 110 m. long continuous; for the second we have 7 years' measurements. In the Netherlands we have another bridge 140 m. long, which has some problems, but I think it is not due to a wrong calculation. It is rather due to other reasons, quite difficult to explain here.

UHERKOVICH - We made a lot of measurements, but we have found that it was not only a problem of the creep, but we have a very wide system of unknown factors and very influencing factors. For instance, we do not know exactly what the bridge weight is, because we do not know what the actual size of the bridge is. The measurements are very expensive and they do not always give satisfactory results. So we know that, for example, this creep calculation and these values are approximately and satisfactorily valid but we cannot take into account them only. This is the reason why our Laboratory is looking for a complete way of calculating bridges.

VOS - I would also like to comment on creep very rapidly.
I think of about 15 years ago - it was in 1965 - when computer came in into our structural engineering. I was in force in the design of a kind of disc-shaped exposition building in the Netherlands. There was a big structure and with a slab projecting from the central support, because we were able,in those days, to compute the flexions due to creep and temperature within 4 or 5 digits, we really did it. The most stupid error I ever made is giving this sheet to the client who, three days ago, asked me a new updated report to give a motivation of how the deflections were done. In my opinion, creep goes back to test from Russia at the beginning of the century, at the 20' or 30', and I think in bridges and so on. If there is a small difference in creep as you explained, this really is not so dangerous.
Of course, when creep comes into stability problems, it really can be of a fairly big danger.

KLEMENT - Is there anything else to say about creep, bridges, or use of computer? Then, I must ask for questions on the impact of computer development on the art of concrete dam displacement control. I have a question on this remark. I would like to know of how many dams are you able to get displacement; moreover, what is your experience in earthquake measurements in Italy.

FANELLI - Well, for the control and check-up of static displacement, we have under control now, with this type of prediction, about 16 or 17 dams and 40 more concrete dams are scheduled to come into stream in the next few years. This is for existing dams - in service dams. Of course, our type of analysis could also be applied to design new dams, but in Italy there are now very few dams which have being designed. In fact, most of the available sites have already been exploited. This is for static displacement check-up. As for the dynamic behaviour, we are pursuing, together with ISMES, full series of in-situ tests to measure actual dynamic characteristics of dams, in order to be able to make a reliable seismic analysis of these dams. Besides this, ENEL, together with a National Authority for the research of new energy, has made up quite a wide national network of accelerographs. There are 170 of them and many of them are located near the dams, but to my knowledge only one or two are installed very close to dams. On some dams we have instruments able to record weak seismic events, but not strong ones. There are now studies to equip important dams with strong accelerographs placed on the dam itself.

KRUISMAN - Do you have any possibility to get the response of the dam for earthquakes with a seismographic network? Are you able to compare theoretical and real values?

FANELLI - Yes, I may mention in this connection an experience we had after one of the bigger seismic events in Friuli, in 1976. We could install a geophonic network on a dam which was very near to the epicenter, and which was also very near to one of those accelerographs. So we were able - on the after shock, not on the main shock - up to the 5th degree of the Richter Scale, to measure the actual response of the dam and compare it with the theoretical response.
The results were extremely satisfactory, and in that case we could also make an evalutation of what the response of the dam would have been to the main shock if it had remained in the elastic range, which can be open to doubt, because in some areas maybe they did not remain in the elastic range, but only in very small restricted areas. So, we have a full set of studies on this topic of the behaviour of dams and if you are interested, I can send you some material on this topic.

KLEMENT - Other questions on this as we are at the very end of our meeting? Are there questions on other important things we had this last day ?

KRUISMAN - I think the organizators of this colloquium have heard a lot of opinions about interface, about computing, about structural engineering, and what is the result of it ? I mean, what are you going to do with it? Are you going to make any suggestions for improvement of this interface ? Because, say, all those papers have been prepared loose from each other; we have now seen and heard them all, and they could not be compared before but now they have to be.

KLEMENT - I think that you will be able, as President of our task group, to give an answer to that.

FANELLI - Naturally, this is the question which poses itself. I am grateful to Dr. Kruisman for posing it from the floor. I am trying now to reply. The task which was set to our task group was to try to formulate clearly the problems and we proposed to IABSE to hold such a colloquium as the present one as a first stage in order to formulate clearer questions for the 1980 IABSE Congress in the hope that in a better occasion, the question of interface between design and computing could be met in a more systematic way and, if possible, with some answers to the more pressing questions. So, what are we going to do now with the results of this colloquium is to try and formulate clearly the main theme and some sub-themes for the special session of 1980 IABSE Congress. This will be announced in due time and, of course, all the participants in this colloquium and also all interested people are invited to submit papers, so that we can reach some definite conclusions after the Vienna Congress. Thi is a partial response, a partial answer to your question, and on the role, I think, that the mission of the first stage of the task group has been accomplished in the sense that we have heard many ideas and many questions and remarks but there seems to be, on the whole, a general consensus about what are the more pressing questions in this business. I know that now, in this moment, at the end of three days sessions, with all the fatigue of these sessions behind us and without having had the time to digest all the information that has been given, it would be quite foolish to try to sum up the discussions and the papers but general remarks maybe can be made. For instance, it seems to me that everybody agrees that computers are to stay with us; there is no return back to a world without computers. However, there is a widely felt uneasiness that the computers, these powerful tools which are offered to us, can often be misused or there are dangers of misuse, either due to a wrong choice of tools in relation to objectives, wrong expectations, wrong or no appreciation of consequences, even addition to computers, inability to check results independently, failure in communication, duplication of efforts and so on. Besides that, everybody seems to agree, or at least, if not everybody, the large majority seems to agree that the computers or, to define better the things, hadware and software, are only tools, are only means, and for the use of these tools, of these means, the designer is the main responsible, if not the sole responsible. Well, where are these dangers of the misuse of the computers coming from ? Partly, they derive from the cahotic development that we have witnessed on the computer work. This development has been quite on an anarchic base, so to speak, and it has not yet been integrated within the communication structure, the communication fabric of our society. This is characteristic of every new technical tool that is introduced. If I may borrow from Dr Pfaffinger's paper, thus coming back to the very first one of our colloquium to close the circle, I may refer to the example of motor-cars. With the advent of motor-cars, at the beginning there was complete freedom : one who purchased a new car could go wherever he wanted. He had no circulation rules to follow, and so on. But nowadays, we know that this is impossible now. We have to exercise not only new skills; it is not sufficient to learn to drive a car. First of all, the car must be chosen in relation to its use. You will never use a sport car if you have to go across the country. New sets of regulations and checks must be added to the fabric of our society, if the input of these new tools is to be a beneficial one and not a harmful one. For instance, with the advent of motor -cars, we had to set up roadworthiness checks, traffic lights, traffic police, car insurance, legal liabilities (of drivers of course, not of cars), and so on. We had also to undertake the education of the users, educations in the new skills of driving, but also in common good sense. For instance, one must not be carried away by his new car, so that he uses it even for going from his home to the newspaper kiosk 50 m. away. All of these aspects can be transposed more or less to the world of computers, and we have heard about them, so I would not repeat them. This is the situation. Our society is not yet copying efficiently with the impact of these new tools. We have to recognize this fact and urge other interested parties to help us in making these new tools compatible with the goals of every time and every society, that is a better life and a safer life for everyone. Thank you.

CLOSING ADDRESS

Prof. M. Fanelli, Chairman of the Colloquium

The aim of the present Colloquium was to bring into the open the main critical questions concerning the correct use and the proper role of computing in structural engineering. It was intended - in close agreement with IABSE, to which our Task Group belongs - that from the outcome of this Colloquium a more precise and detailed formulation of these critical questions should be achieved, on the basis of which to send out an official call for papers for the specialized session to be held at the IABSE Vienna Congress in 1980.

Naturally, the task of synthetizing these questions or subthemes will demand some time. All we have heard and discussed during these last three days must be digested and filtered down to a few essentials. At present, we are still reeling under the impact of many impressions, and we begin also to feel the stress of a very intense work. So, it would be foolish to attempt now to sum up the conclusions of our Colloquium. All that I propose to put forward are a few fleeting impressions about points on which there seems to be a quite wide consensus among the participants.

It seems, first of all, that practically everybody agrees that computers 'are with us to stay': we have passed the 'point of no return', so that to go back to a computerless world is well-nigh impossible. However, it is also widely recognized that these powerful tools are - or can be - often misused (eg. through a wrong choice of means in relation to objectives; wrong expectations; wrong or no appraisal of consequences; addiction to computers; inability to check results independently; failures in communication; duplication of efforts, etc.).

Also, nearly everybody agrees that computers and programmes are only means for the correct use of which the designer is the main responsible (indeed, according to many, the sole true responsible).

The potentiality for misuse of computing derives - partly, at least - from the chaotic development of the computer world, which has not yet been rationally 'integrated' into the communication system of our society.

If I am allowed to take a leaf from Dr Pfaffinger's book (and so to close the circle by going back to the very first paper of our Colloquium), I would like to draw a similitude with what happended with the advent of motorcars. Then, as with the advent of every new technical tool intended to do faster and better what we did formely by less advanced means, it was necessary, but by no means sufficient, to acquire new skills (eg. to learn driving). Many stringent requirements soon became apparent in order that the new advance could be integrated reasonably well into our way of life: the type of car had to be chosen in relation to its use (no sports car for city driving !); new sets of regulations and checks had to be introduced, e.g. 'roadworthiness checks', traffic lights, traffic police, car insurance, legal liability of drivers (not of cars of car makers !) for accidents etc. Also, education of the user, not only in the new skills, but also in plain good sense, had to be sought for (one should not get so carried away by his new car as to use it to go from his home to the newspapers' stand 50 yards away !). If you care to take the trouble, you can find a striking correspondence of every one of the aspects just mentioned with the problems we have been talking about 'Computing in Structural Engineering'.

It seems to me that a main lesson is emerging: we have not yet coped with the task of reconciling, in an orderly way, the enormous potentialities of computers with the general 'cybernetic' (both in Ampere's and Wiener's sense) fabric of our society. The sooner we recognize and face this fact of life, the better is, because only by striving toward such a reconciliation can we hope that structural engineering will use computing to the full advance of people at large, in the sense of promoting a better and safer 'structural environment' for all mankind.

CLOSING ADDRESS

Prof. E. Fumagalli, Director of ISMES

At the close of this Colloquium, 'Interface between Computing and Design in Structural Engineering', as Director of ISMES, I like, also on behalf of our Managing Director, Prof. Alfredo Marazio, to thank all those of you who have so kindly partecipated in this discussion on the most interesting topics in the sector of computer methods. I feel we should also extent our thanks to IABSE for having sponsored this meeting and having lent its weight and prestige to the colloquium. At the same time I should like to thank the Chairman, Prof. Fanelli and the Member of the Task Group, who have made every effort to represent the spirit of this debate with their contributions.

Finally, I think all of us here would wish to thank the Members of ISMES organization who have made every effort towards a successful outcome of these discussions. I should also rightfully mention Dr Pistocchi, who has dealt with the public relations aspects, and especially our kind secretaries who have lent a graceful note as well as their habitual efficiency.

And now that we have come to the end of our work, I trust we shall be able to say that we have enriched our knowledge and understanding of problems in this field. It only remains for me now to thank you once again on behalf of ISMES and wish you "bon voyage" and a hearty good-bye.