VII
Développements nouveaux
Neuere Entwicklungen
New Developments

Leere Seite
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Page vide

VII

A New Automatic Drawing Language

Un nouveau langage automatique pour le dessin

Eine neue automatische Zeichnungssprache

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1. Abstract

Lately labour cost has enormously risen and managers of factories are obliged to adopt automatic production system in which N/C machines play an important part.

We also have an attempt to fabricate steel bridges and frame works automatically with N/C gus-cutters drills and so on. But it is not so easy to supply paper tapes or magnetic tapes in which N/C instructions are packed. These tapes can never produce quickly and infalliably without a computer with large memories and its software.

After thorough investigation we got such conclusion that all informations about each material of which a structure is composed, should be packed into a magnetic tape and that it should be send to the factory.

In the factory they will pick up suitable informations as occasion demands.

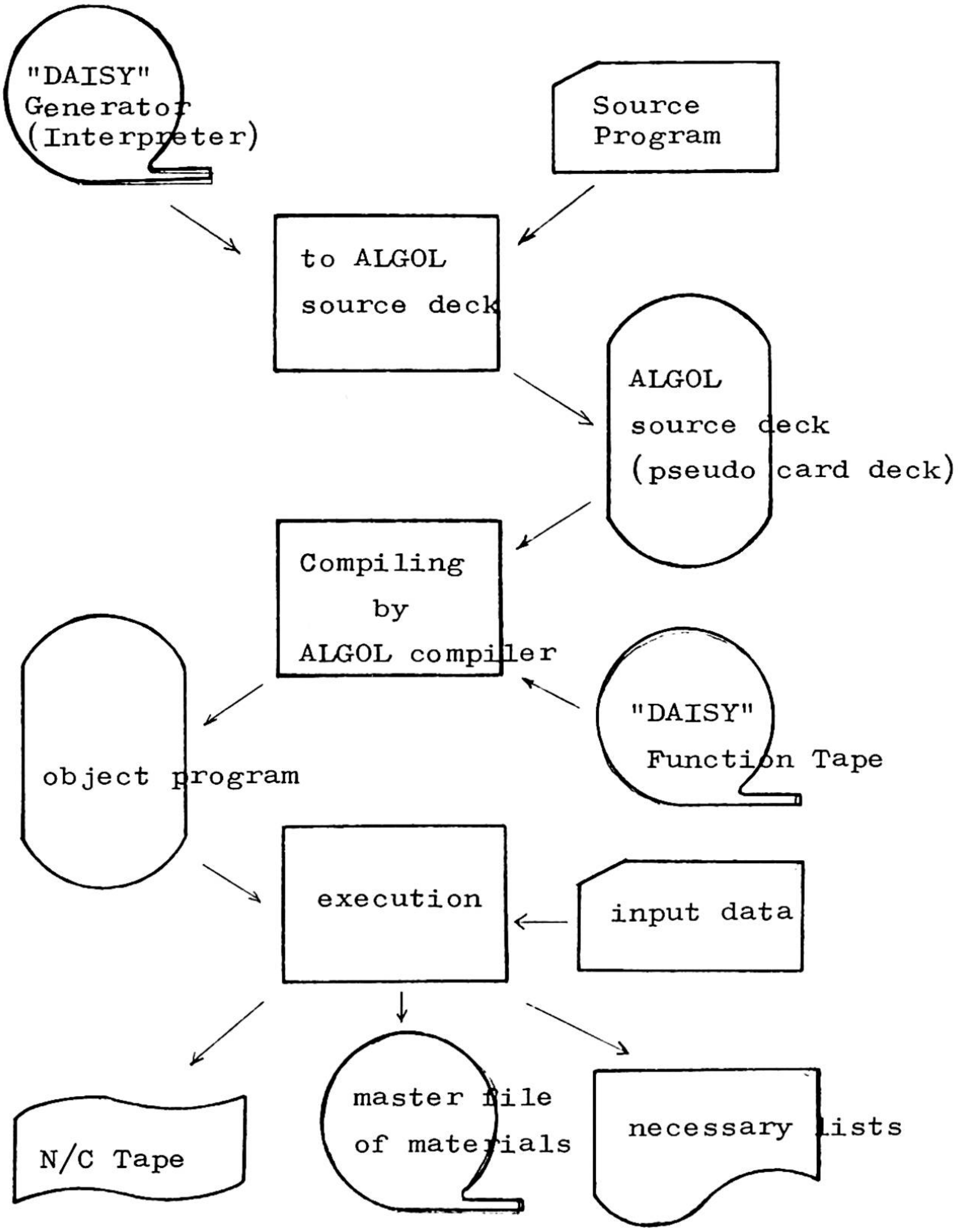
We searched for such a software in vain. At last we decided to make a new language with which we can get not only a automatically drawn plan but also a magnetic tape mentioned above. We began on it in the spring of 1970 and as we brought to completion with difficulty, we now report this paper to you.

This processing system is so called "generator system."

The merits of this method is as follows. As we can code the system programs with a compiler language, we can complete the system within short time and even if a bug is found unfortunately, we can easily repair it. Besides when we are to entrust its maintenance to our successor, we can transfer it to them smoothly. On the other hand the weak point of this method is that process is so complicated that much process time is required to accept necessary informations, and the operator of the computer are apt to commit misoperation. We examined these merits and demerits carefully before we take up this method.

2. Flow of Data

We named this system "DAISY", and source cards which are coded by a programmer are processed as Fig. 1.



4. Features of "DAISY"

As mentioned in abstract the main aim is to make an information tape for fabrication, the function is designed to meet this desire.

N/C languages in the past aims at the fact that a coder can make a cutter location tape easily, copying the plan by hand. So if you begin coding at your desk where there is no plan, you will find it quite difficult. I dare to say it to be impossible. This reason is as follows.

1. It is inconvenient for numerical and logical operation.
2. Components of vector cannot so easily be derived.
3. The concept of system doesn't take how a drafter make up his plan into consideration.

The third item is very important. When a skillful drafter is goint to code auto-drawing program, he must completely change the way of constructing the plan.

We watched carefully skillful drafters working to know how they will get points and how they draw lines on the tracing papers, etc.

We at last knew that they will not separately draw each line but place a piece which they are going to draw on a tracing paper and rotate it 3-dimensionally so that the piece will expose the designed surface and drop the suitable shadow of it on a paper in their heads, and then they begin to draw it. This process in the drafters' head is thoroughly copied to the new language. When you will make a certain plan, you will arrange all the parts that the plan is composed of, on an imaginary tracing paper and rotate them mentioned formally and then if you write "DRAW ALL" at the last line of the coding sheet, you will get the N/C tape desired. Of course general faculty as a N/C language is furnished.

Another conspicuous faculty is that the placed pieces can freely be processed. For example if you want to cut the piece you can do so. You can bare halls of the piece. If desired you can join two pieces to one.

The piece once placed may change its location to wherever you like. But if you move it 3-dimensionally only the shadow of it to the surface in view will be left.

To complete a plan another important component we must not forget is dimensions and dimension lines. Not a little quantity of labour is spent to it when a drafter draws a plan by hand. System is also designed so as to code them with a few statements and the distance between two arrow marks signed to the edges of a dimension line can be automatically calculated.

Summing up the faculty of the "DAISY" we exhibit as follows:

1. Vectors (point, line, curved line, figure, pattern, member) can be used.
   Where member has its thickness.
2. Figures, patterns and members can be change their shapes freely and move them anywhere.
3. Various values can be derived from the vectors.
4. All the information of a member can be entried to the master tape of the job.
5. Dimension lines can be easily made.
6. Another N/C command can be generated.
7. "Macro-Drawing" is possible.

5. Version-Up

As mentioned former paragraph, this system has occupied "generator method." 3-step process is desired before we receive a N/C tape. We are carrying a plan that we will improve the "DAISY" to the compiler. If this plan is achieved remarkable quantity of processor time of the computer will be spared.

At present N/C tape format is not universal and is only able to a certain maker's machine. We must improve the post-processor so that any type of format may be allowed.

6. Conclusion

By occupying "DAISY" system, we succeeded to get automatically drawn plans at relatively cheap cost.

The amount of core memory which this system desires is 196 k bytes and the average processor time for one sheet is about 3 minutes.

In other department of industry it might be impossible to handle data 3-dimensionally, but steel structure is mainly composed of plates and rolled shapes. For this reason we managed to succeed to complete the system in relatively short period of time.

Automatic Designing and Drawing of Structures in JNR

Projet et dessins automatisés de structures à la JNR

Automatisches Entwerfen und Zeichnen von Bauwerken bei der JNR

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I. Introduction

1. Construction and use of a structure and application of an electronic computer for this purpose

Construction of a structure at a given site will involve broadly the following four stages:

a) Overall planning
b) Designing
c) Construction
d) Use

Each of these stages may be subdivided and depending on the contents of these subdivisions, the electronic computer will find versatile applications.

For instance, in the stage a) different proposals are compared including the decision on the advisability of constructing a given structure at a given site. Suppose thereby the factors in planning are adequately chosen. Then it will be possible to formulate some set patterns of making comparison between different plans. If such set patterns were programmed, numerous complicated comparisons would be computerized.

Just as in the stage a), many comparative designs of a similar structure take place in the stage b), too. It is, however, in the stage of final designing that the computer best proves its worth. There is no denying that the computer is found most useful in stress calculations or calculations for selecting the dimensions. When the designing is finished, usually the plotting of a design drawing comes and even this work has come to be automatically executed by the computer in recent years. This work, however, involves extremely intricate problems not encountered in stress calculations and therefore its execution by the computer has to be preceded by most careful preparations.

The stage c) mainly represents movements of concrete objects, so it may seem that here the computer can not play any prominent role. As a matter of fact, however, there are a considerable number of cases where the computer is found useful in practical work step control and automation of construction machines. In the interest of liberating humans from unfavourable work environments, this trend will be promoted at an increased pace.

In the stage d), many examples can be cited of the computer serving for automatic operation of a structure itself like a sluice, but on the other hand a number of cases such as railway bridges may be mentioned where the collection and retrieval of information is computerized for the purpose of reflecting the past experience of maintenance and use in a new design.

2. Concrete example of automatic drawing

In the present paper a report will be made on the automatic drawing, the most unique one of the computer applications mentioned above. In Japan, automatic drawing is attempted not only JNR but also by many others such as the Construction Ministry, general contractors, bridge-builders, ship-builders and design consultants. The description here will, however, center on the practice of automatic drawing in JNR, whose design section pioneered in practical application of this technique and seemingly has the richest experience in Japan.

II. Designing of Structures in JNR

The drawing is a mode of expressing the results of designing. At design offices which deal with numerous bridges or overbridges of similar types, different patterns of drawings are taken depending on the design systems of the structure groups to be treated. Thus it is pre-requisite for discussion on the automation of drafting to have full grasp of a design system.

1. Standardization of structures

In 1972 JNR observes the 100th anniversary of its founding. The JNR policy during this period has been consistently to standardize its structures. It is motivated by consideration of the efficiency of designing, saving of construction cost, facility of maintenance and administration, and interchangeability of structural members in the event of an emergency or for the purpose of improvement.

The first standardization ever made for designing of bridges in JNR was that of wrought iron girders 20 - 70 feet in length; this occurred in the year of 1893. Later iron was superseded by steel and now common steel is steadily yielding to high tension steel in bridge construction. Meantime, in the method of joining the structural members, riveting has been outmoded by welding and in some cases, use of high-tension bolts is beginning to be preferred. At present, for steel bridges alone there are the following standard designs available:

Steel girders - trough girder, I-beam girder, plate girders (deck and through), composite girder, pedestrian overbridge, construction girder.

Truss girders - simple support truss girders (deck and through), three-span continuous truss girder.

Similarly, for concrete bridges, too, the following standard designs have been established:

Reinforced concrete girders - slab, T-girder, H-beam embeded girder.

Reinforced concrete elevated bridge - three-span rigid frame elevated bridge (single track, double track, tangent track, curved track)

Reinforced concrete culvert - simple plate culvert, rigid frame box culvert.

Other standardizations include various abutments, piers, retaining walls, wells, inspection pits, wash stands and platforms.

Now take, for instance, deck plate girders. There are two basic load systems for them: KS-18 and KS-16. For each of these load systems there are 11 different spans ranging from 8.2 m to 46.8 m. Moreover, there are four angles of road intersection to be considered; 90°, 75°, 60°and 45°; and as many restrictions to be imposed about the height of girder. When all these items are taken into consideration, the combinations will be countless. In reality, only the most practical of them are standardized and field jobs are executed utilizing these established standards as far as possible.

As the design conditions become more elaborate to meet the field conditions better, the types of structures to be standardized increase, defeating the original purpose of standardization policy. This is an inevitable contradiction inherent in these groups of structures and there is no simple solution to the difficulty. Under the present circumstances, reconciliation of elaboration and simplification of design conditions based on experience is all we can do.

2. Standardization of design procedure

As stated above, JNR possesses a certain amount of ready design drawing for standardized structures. In the execution of a practical job the local agency in charge of the job execution applies an appropriate standard design as it is or with necessary modifications, depending on the local conditions. If in that case local engineers make such design modifications with personal approaches, the meaning of standardization will be lost. Therefore, the design procedure should be unified as far as possible to avoid personnal differences. Here lies the greatest necessity for standardizing the design procedure.

JNR now holds about 900 standard drawings of steel structures, concrete structures and earthworks, and tabulated calculation results on about 2830 cases of designing abutments and piers. These are put to practical use and according to the survey conducted in 1967 on the state of standard design application, 60 to 95 % of girders, culverts and retaining walls in the local track expansions are designed by application of the standard procedure as it is or with slight amendments, but in the similar jobs executed in suburban areas the rate of application of the standard design is a mere 10 - 60 %. Namely, about 40 - 90 % of girders, culverts and retaining walls executed in suburban areas had to be designed anew as non-standard jobs.

With progressive urbanization, this trend is likely to spread throughout the country. As a countermenasure, it has become necessary to establish a new standardized design procedure anticipating all the particular conditions happening in suburban areas, instead of the conventional practice of setting a large number of conditions in advance and selecting from among them to meet the requirements of a given job.

At present, the criteria for designing the steel structures, concrete structures and earthworks of JNR are established.

These design criteria lay down the basic principle and set rules for designing but they are not any consistent specifications for the procedures of designing various structures. The principle and rules, however, do not remain unchanged forever; they ought to keep step with theoretical evolutions and general advances in technology. On this ground JNR makes a point of reviewing these design criteria every three years.

If such a revision is relatively minor, the design criteria will be little changed. Major changes will be needed in them, however, when, for instance, the allowable stress degree is raised as the result of better materials becoming available or when a new theory on the impact or repeated load has come to be adopted. To cope with such possibilities, there is no alternative but to expedite a new designing by application of an electronic computer using a programmed standardized design procedure.

3. Automation of designing

As mentioned in the preceding section, both the structures themselves and their design procedures have been standardized, but the fact is that many non-standard designs are being made to meet the actual conditions, while there happen occasional needs to make major revisions in the already standardized designs. Here the vital point calling for serious attention of the engineers is how to revise the design procedure; to execute the calculation in accordance with the revised policy is in itself nothing sophisticated. To delegate this job of calculation to the computer, JNR is striving to realize consistent programming of the design procedures. For example, the automatic designing program for PC girders makes possible an automatic execution of the necessary calculations in accordance with the established criteria, when the computer is fed with 21 data input such as the span length, the number of main girders, the type of PC steel rod; and thereby the necessary 20 data such as the ultimate sectional dimensions, the stress degree in design section, and the PC steel consumption will be printed out by the computer. Such programs are already available for structures with a high frequency of use.

III. Automatic Drawing of Structures in JNR

1. Uses of drawing

Structure designing stage is followed by the drafting stage. The drawing of a structure expresses the conception of a structure to be build in terms of design calculations together with a list of its details and materials. The uses of such a drawing are as follows:

a) Reference for the designer in his practical work of designing.
b) Reference for the customer in signing a contract with the contractor.
c) Reference for field engineers in executing the job.
d) Reference for the structure administrator in maintaining the structure.

Naturally, a drawing with such uses is prepared with many descriptive elements to facilitate visualization of the structure to be built.

Take, for instance, a line on the drawing. It can be thick in several degrees; it can be solid, dotted, broken or one (or, two, three)-dot chain, with respective meanings. In the Japanese practice of drawing, so far as the structure design drawings are concerned, the use of colors is as a rule prohibited; and perhaps for reason of an ordinary paper with expansion and shrinkage being employed it is customary to indicate the dimensions by the dimension line and numerals; you never find the dimensions by applying a divider on the drawing and multiplying the measured result by the reduction scale factor. If the dimensions of a vital member cannot be known otherwise, the drafter of the drawing is to blame; in that case you have to consult the design calculation sheet and enter the relevant data together with the dimension line in the drawing.

This is a big difference from the electrical wiring diagram with a different use or from the drafting technique directly related to machine tools.

In recent years, however, at bridge building shops research is under way on automation of precise drawing for cutting steel plates, where by the obtained drawing is linked to a machine tool, which directly cuts the plate in accordance with the drawing. This can not however be compared to a design drawing of a structure; rather it should be regarded as a stage of material processing.

It is for reason of its great bearing on the development of automatic drafting to be described in the next section that the features of a structure drawing are discussed here.

2. Automation of drafting

As stated above, construction of a structure is necessarily preceded by drafting of its drawing. In recent times it rarely happens that ready-made standard drawings as they are meet the practical purposes; more often some revisions have to be made in them or a redesigning is to be made under new conditions and accordingly a new drawing has to be drafted. But variations of standard design naturally retain the original features. Take the case of a deck PC girder, for which it is common to have three drawings containing a plan view, a side elevation view, sectional views, a reinforcing bar layout with numerical data and design conditions. The modes of expression, - for instance, what sections are to be drawn or enter a side elevation view at top and a plan view below the former on the same drawing - are established in set patterns on many standard drawings. Thus main point in the drafting of such a drawing is statement of the difference in the profile or in the reinforcing bar consumption and this is nothing which needs an advanced technical decision-making. Therefore, such a job may be left to the computer and for this purpose the automation of drafting will be promoted so that the engineer may be assigned a more sophisticated duty of decision-making.

A very high level of standardization on structures and their design procedures has been quite favourable for automation of drafting, but as pointed out in the above the structure drawings have three-dimensional complex contents with many descriptive elements and it would be considerably difficult to have them drafted automatically. As to be described in the succeeding section, we began with investigation into what should be expressed and succeeded in perfecting an automatic design system.

3. System for automatic drafting

A block diagram showing the process of automatic drafting as practiced in JNR at present is given in Fig.1.

At 1, drafting program receives additional inputs of specific values, which are then fed to the large-side computer at 2. Thereupon, a magnetic tape 3 is produced and this goes again into a large-size computer, which produces a paper tape 4. This paper tape 4 is fed to the numerical controller 5 located by the side of the drafter 6. Then, the pen of the drafter 6, controlled by NC, traces a required drawing.

From the magnetic tape 3 branches out a graphic display 3' (GD or CRT = cathode ray tube), which serves to save time that would required for checking or debugging the program if for this purpose a drafting were repeated.

Insertion of the stage 3 (MT) instead of direct jump from 2 (large-size computer) to 4 (PT) is made for the following reasons;

1) Since the paper tape is not produced in the program-debugging stage, time can be saved and the computer efficiency can be raised.
2) With MT already available, there is freedom of producing PT at the central of terminal machine with convenient timing.
3) Work duplication necessitated by various troubles in the computer can be held to a minimum.
4) GD makes the debugging more efficient.

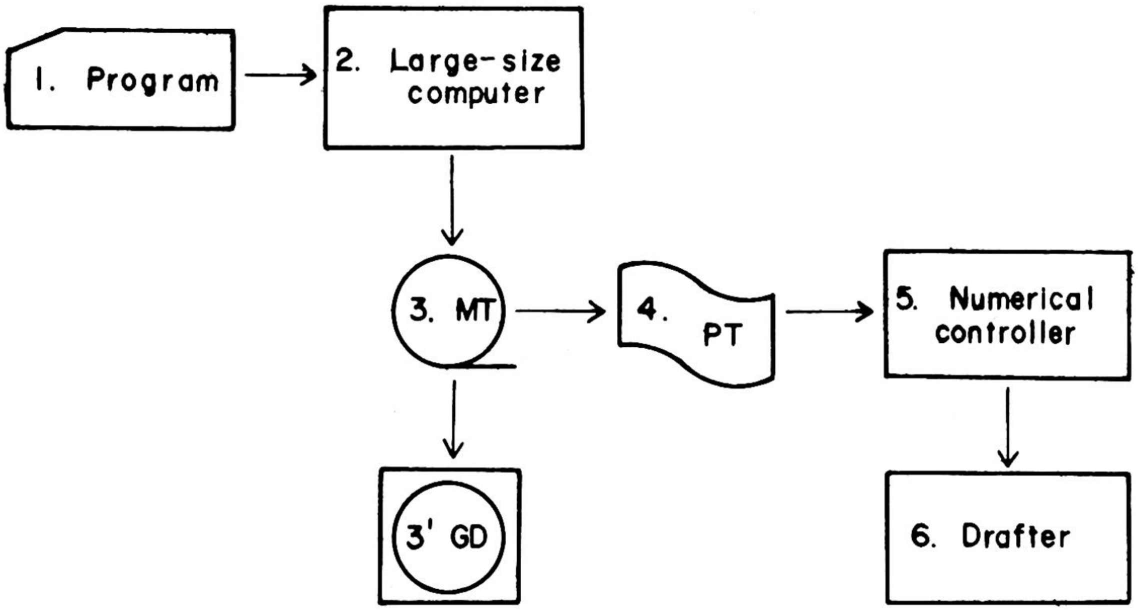


Fig. 1 Block diagram of automatic drafting process

The large-size electronic computer for this purpose is installed at the Railway Technical Research Institute, the model being FACOM-230-60. The drafter, however, is located at the Structure Design Office. Separated by about 30 km, these offices are linked by communication lines. The numerical controller is FANUC-250b, while the automatic drafter is NUMERICON.

At the Structure Design Office, there is also the terminal equipment centered around a computer called FACOM-R, which can be connected for input to or for output from the large-size computer. A similar terminal equipment is provided at the JNR Head Office and the Labour Science Institute, too.

The large-size computer at RTRI, originally intended for research use, has lately been burdened to its limit capacity with the loads originating from the Institute alone. On the other hand, the loads of calculation jobs demanded from the Structure Design Office and from the construction offices in the suburban Tokyo have reached tremendous volumes. In view of this situation it has been decided to install a new computer in the near future in the Shinjuku building which houses these offices: the model to be adopted will be FACOM-230-45 which is similar to the existing machine, ensuring the availability of existing programs.

Photo 1 reproduces the automatic drafter room and Photo 2 the automatic drafter. Figure 2 is a side elevation view, automatically drown, of prestressed concrete girder illustrated as an example.

The automatic drafting programs already developed or being developed by JNR are listed in Table 1.

The scissors crossings represents four switches coupled with two tracks. With the design of each of these switches established, this is no more than a combination of existing standards. As a mat-

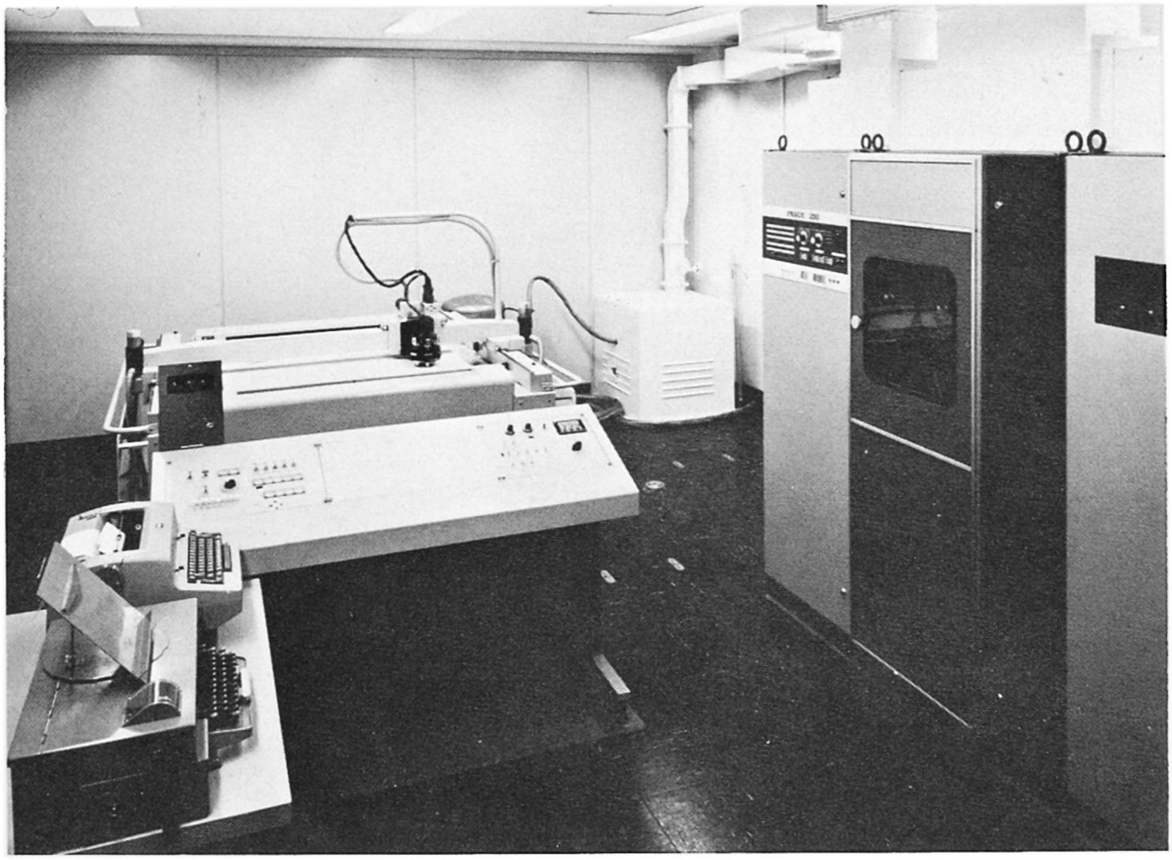


Photo 1. Automatic drafter room. Right: numerical controller; Left background: automatic drafter; Left foreground: control desk and input device

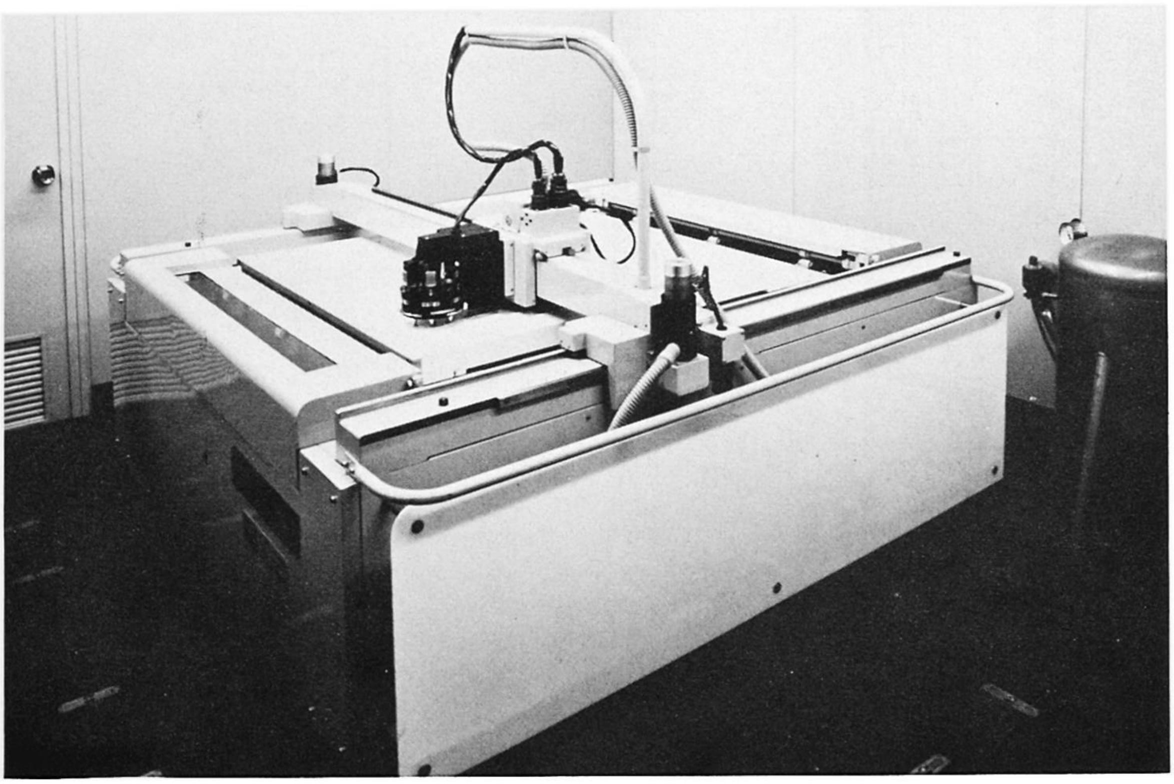
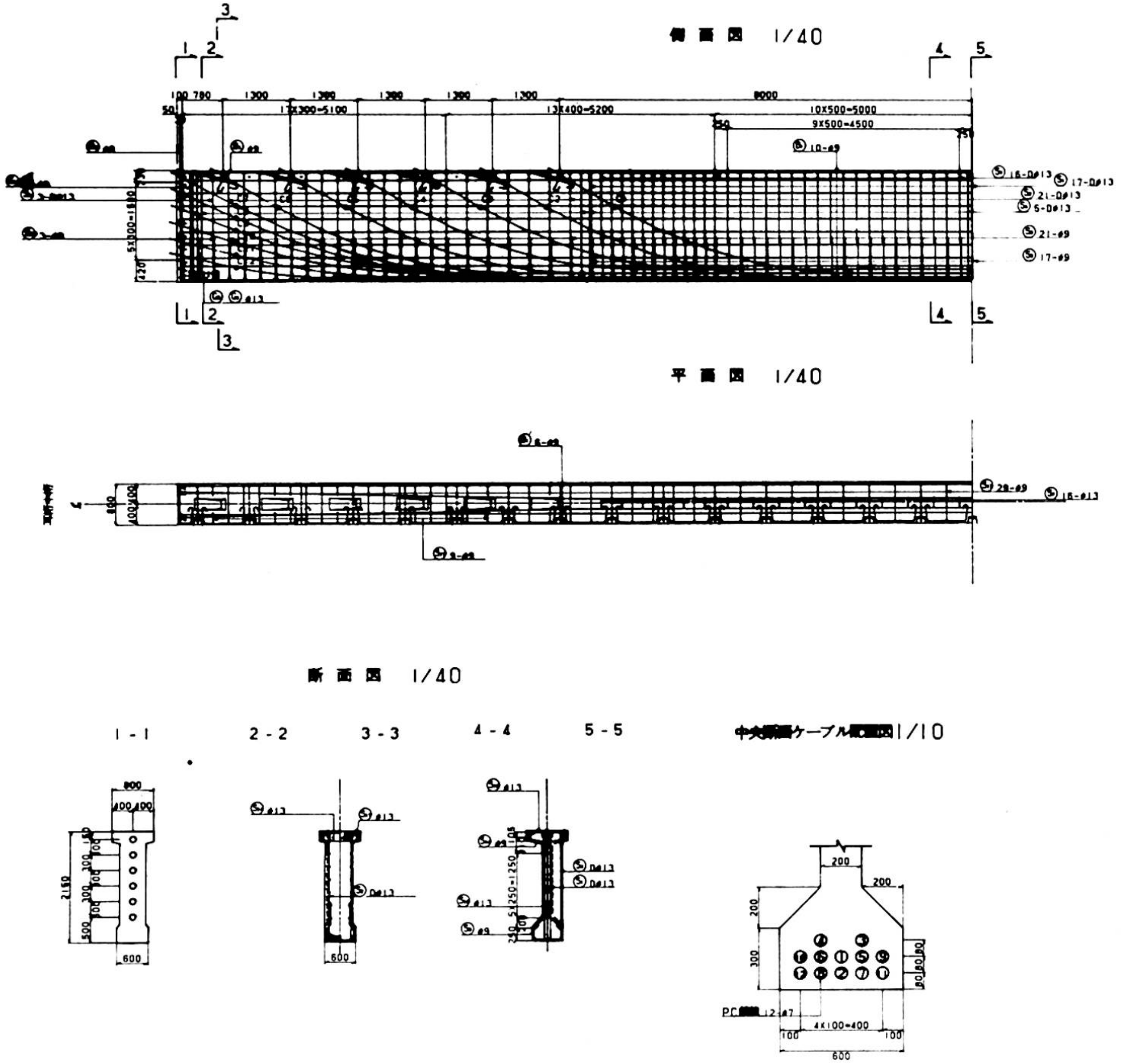
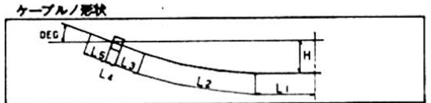


Photo 2. Automatic drafter



ケーブルノ形状



ケーブル NO	L1 (MM)	L2 (MM)	L3 (MM)	合計	L4 (MM)	L5 (MM)	DEG	A	H (MM)
1	2387	4799	1080	8305	120	299	30	0.06298	1917
2	3420	5096	1080	9715	120	299	30	0.05964	1997
3	5275	4506	1080	10980	120	299	30	0.06737	1837
4	6574	4506	1080	12279	120	299	30	0.06734	1837
5	7587	4799	1080	13585	120	299	30	0.06298	1917
6	8887	4799	1080	14885	120	299	30	0.06298	1917
7	[illegible]	5615	1001	15623	120	299	27	0.04853	1920
8	9162	5269	978	15529	120	299	24	0.04448	1620
9	9820	4567	950	15456	120	299	21	0.04303	1240
10	10497	3845	931	15381	120	299	18	0.04300	940
11	10905	3399	914	15338	120	299	15	0.03968	720
12	12329	1954	901	15303	120	299	12	0.05438	420

プレストレッシング

ケーブル NO	[illegible]	[illegible] (KG/MM²)	伸ビ (MM)
1	12	114.1	87.5
2	11	115.6	101.0
3	10	115.7	113.5
4	9	118.4	127.3
5	8	119.8	141.5
6	7	121.3	155.7
7	6	114.6	161.6
8	5	119.9	163.2
9	3	119.8	158.4
10	4	117.4	160.9
11	2	117.3	161.8
12	1	116.3	161.4

Fig.2 Example of automatic drafting - PC 1 - simple girder

ter of fact, however, the length of cross-over differs depending on the track separation, which is variable between 3.6 m and about 5 m.

Table 1 A list of JNR automatic drafting programs (as of Oct. 1971)

Items	Program Name	Developed	Partially being revised	Being developed
Track	Special scissors crossings	●	●	
Steel structures	Deck plate girder (rectangular)	●	●	
	Through " " (")	●	●	
	" " " (skew)	●	●	
	Composite girder (box)	●	●	
Concrete structures	RC box culvert	●		
	PC box girder (single-track, rectangular)	●		
	" (single-track, skew)	●		
	PC box girder (single, double-track, rectungular)			●
	" (single, double-track, skew)			●
	PC I-beam girder (rectangular)	●	●	
	" (skew)	●		
	RC rigid frame elevated bridge			●
	Wing parapet	●		
	Graphical representation of FEM calculation results	●		
Building	RC multi-layer rigid frame structure layout	●		

Switches, too, are different with respect to the angle, direction, right/left proportion and so on. If all conceivable combinations were taken, their number would be astronomical and it is out of the question to have all the possibilities calculated in advance. Thus the alternative is to work out calculation programs for all possible conbinations; and using them as the occasion requires, perform numerical calculations and, based on the results, make the machine automatically trace diagrams showing a general plan, skeleton, slack, bend of rail, boring position in rail, etc.

4. Programming system

Now the composition of the program for automatic drafting is to be described.

The automatic drafting can be programmed by FORTRAN, but FORTRAN cannot express the pen movement for drafting and for this purpose the subroutine DRF/1 has been developed.

In 1967 when JNR took to development of automatic drafting, CDC-G-20 was the only computer available; and matching this machine, at first DRF1L was developed. Subsequently, as the machine changed to 230-60, several corresponding modifications were made, producing DRF2L. However, this subroute was quite primitive in its control functions and accordingly the drafting program work turned out considerably bothering; besides, the adaptability to a model change of the computer was lacking. Thereupon, based on the idea of DRF2L and on the experience in the actual development of drafting program, a new DRF/1 was developed. The new programing system therefore has been composed as indicated in Fig. 3 in such manner as to permit use of DRF2L - based programs as they are.

Generally speaking, FORTRAN and ALGOL may be regarded as language for numerical calculation. For the purpose of working out the drafting program, it is common practice to provide basic, universal subroutines with functions of coordinate calculation and graphical representation, and, with the aid of these subroutines, work out a drafting program.

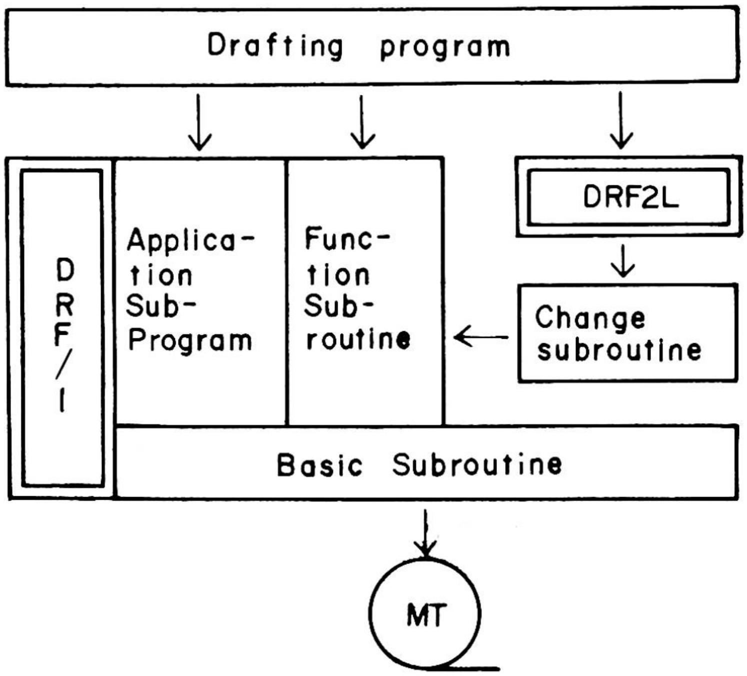


Fig.3 Drafting programming system

Since these subroutines are intimately correlated their adequacy will largely determine the efficiency of programming work. With this in mind, utmost attention has been paid to the following points in the course of developing DRF/1:

1) DRF/1 is to be worked out as a subrouting package, so that various changes or additions in the functions may be executed easily in the form of individual revisions or addition of subroutines.

2) Automatic drafting is desirably to be linked to automatic designing. For this purpose, FORTRAN most commonly employed in JNR for design calculations is adopted as the language for DRF/1. Therefore, the subroutines for various coordinate calculations can be utilized for details calculations in the design stage.

3) In the drafting program which makes coordinate expression of figures, enormous volumes of coordinate calculations are involved. With this in mind, a large number of subroutines for coordinate calculations have been worked out to facilitate the program formulation.

4) A check routine for error message is to be attached to each of these subroutines to help debugging, because the drafting program is subject not only to errors in the use of instructions for coordinate calculations or drafting as well as to grammatical errors in common calculations. Incidentally, as mentioned in the preceding section, GD is utilized for debugging.

As illistrated in Fig.3, DRF/1 is composed of Basic Subroutine, Function Subroutine and Application Sub-program.

Basic Subroutine changes the pen shift data, etc., as calculated in Function Subroutine into a form readable by the controller of the automatic drafter, said data being written into the magnetic or paper tape, i.e., the medium of input to the controller. This part is directly linked to the automatic drafter and accordingly a change of this part will permit connection to the controller of any other model. Basic subroutine for DRF/1 comprises 14 subroutines for controlling the vertical motion of the pen and the drawing speed.

The drafting program is formulated mainly using Function Subroutine, and Basic Subroutine is used only in Function Subroutine.

Function Subroutine consists functionally of the following four parts:

1) Control subroutine

This is a command to start or end the drafting process.

2) Figure-modifying subroutine

This is a command for selecting the drawing paper size, the reduction scale, the revolution or displacement of a figure.

3) Coordinate calculation subroutine

A figure is split into dots, lines and segments for treatment and the necessary coordinates are calculated using this subroutine.

4) Figure-tracing subroutine

This is a subroutine for tracing the defined lines, segments and characters.

Application Sub-program is a sub-program which represents, using Function Subroutine, specific figures often occurring on a drawing, - for instance, arrow marks or dimension lines - in the form of a single subroutine.

Aided by Function Subroutine and Application Sub-program, the programmer works out a drafting program. At present about 70 subroutines of this kind are in use by JNR and, depending on the need more of them will be added in future.

5. Future problems

As understood from the above statement, automatic drafting is nothing easy to realize. Even the establishment of a drafting programming system as illustrated in Fig.3 has been a tremendous work and it will be still harder to work out a drafting program with application of this system. Therefore, the greater the variety of drawings that can be obtained using this hard-earned program, the better. In other words, the drafting program ought to be as universal as possible. For this purpose it would be desirable that a structure be standardized as far as possible and, even after minor modifications necessitated by the field condition, it as a whole conform to the standard pattern. Another important thing is how cleverly to standardize the figures in a drawing and how to maintain many affective Application Subprograms.

Increased applications of the computer for structure design calculations have promoted a movement to review the existing design specifications with assumed use of the computer technology, while at the same time practical application of automatic drafting has cast doubts about the established customs agreed upon in the conventional drafting method. Thus, as mentioned in III, 1, from the standpoint of drafting various customs have been submitted to reviewing. They are, however, products of long traditions and, to avoid possible chaos caused by sudden change: under a policy of gradual transition revisions were started with relatively insignificant ones. It is certainly a great improvement that too meticulous expression of unnecessary details simply by the force of habit has been considerably eliminated through these efforts.

In quest of a simpler method for formulating a drafting program than the above-mentioned automatic drafting, JNR is now developing LADD (Language for Automatic Design and Drawing), which is to be a specialized language for automatic drafting. Still under development, the whole thing cannot be introduced here, but its idea is briefly as follows.

LADD is intended for experts engaged mainly in automatic design and drawing of structures; without any more advanced training than experience in FORTRAN programming, in which most of them are trained, they will be easily able to use LADD. This is a language with geometrical concepts which will facilitate the formulation of a drafting program.

The figures around which this language is built are patters which occur repeatedly or combine to constitute larger patterns in a hierarchy.

A pre-compiler processor serves for LADD processing. At first a LADD-program is rewritten into a corresponding FORTRAN-program by the LADD processor. The FORTRAN-program thus obtained is next converted to a machine language for execution by the FORTRAN compiler. Upon completion of LADD, the formulation of a drafting program will be still easier than now.

IV. Concluding Remark

As described above, in the fast-moving age of technical innovations JNR has energetically pushed the automation of structure design and drawing as one phase in management streamlining and modernization and these strenous efforts are steadily bearing fruit. It is a recurring question in the application of computer where the boundary should be set between the territory of humans and that of machines, but this continues to be a difficult problem to solve. Because theoretically the problem may be as simple as to couple the merits of human ability with those of computer capacity, thereby enhancing the efficiency of human thinking to a maximum, but when it comes to realization of this theory, the problem turns out far more complicated. Nevertheless, we are determined to strive undaunted toward our set aim.

Meanwhile, standardization of structures and automation of their design and drawing are feared to result in a downgrading of technical ability amont engineers. Even in the past a tendency has been noted fundamental errors being committed in the stage of its application after a standard design drawing has been established. This will not mean, however, that the policy of design standardization should be reversed. Hence the problem is how to prevent commission of such errors. It seems that the future engineers should try more to learn what is the essential thing to understand in applying the achievements in automatic design and drawing than to acquire the knowledge the past engineers were required to possess for each design job they had to work out for themselves. A new system of knowledge, which is still absent, ought to be established as soon as possible.

Synopsis

In line with the policy of modernizing its management, JNR is striving to relieve its personnel of jobs at which humans are not so good or jobs which might as well be done by machines as by humans and, instead, to assign them with more sophisticated jobs which can be executed by only humans. Automatic design and drawing of structures is one example of achievements from these efforts. Practice of automatic drafting in JNR, the process of its development and future problems are discussed here.

The "Total" System for Design and Fabrication of Steel Structures by Means of Electronic Digital Computer

Le système "Total" pour le projet et la fabrication de structures en acier à l'aide de calculatrices électroniques

Das System "Total" beim Entwurf und bei der Fabrikation von Stahlbauten mittels Elektronenrechnern

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I. Total system

Many automated design and drafting systems for steel structures have been recently developed. In addition to that, the introduction of automated fabrication methods has been made possible with progress of N/C machines. But, we can not expect an economic effect because of the large variety of members of steel structures, if the part programs are prepared for all these members. We have our opinion that it becomes possible to automate the fabrication and to rationalize extensively the production system by adopting the total system in combination of automated design and drafting systems and fabrication system making use of an electronic computer and N/C machines.

Now, we are developing our total system. By this system, we can not only automate the design, drafting and fabrication, but also abolish the templete shop and marking-off works.

Merits given by this system are as follows:

Saving of labor,

Improvement of quality,

Shortening of manufacturing process,

Decrease of error, and

Cost down.

II. Automated design system

It is possible to design automatically various type of bridges and other steel structures.
We will explain automated design of simple composite girder already accomplished. The calculation method of composite girder are clear, but the programs have to be available for various pattern of bridges. Automated design of composite girder consists of basic design and detail design.

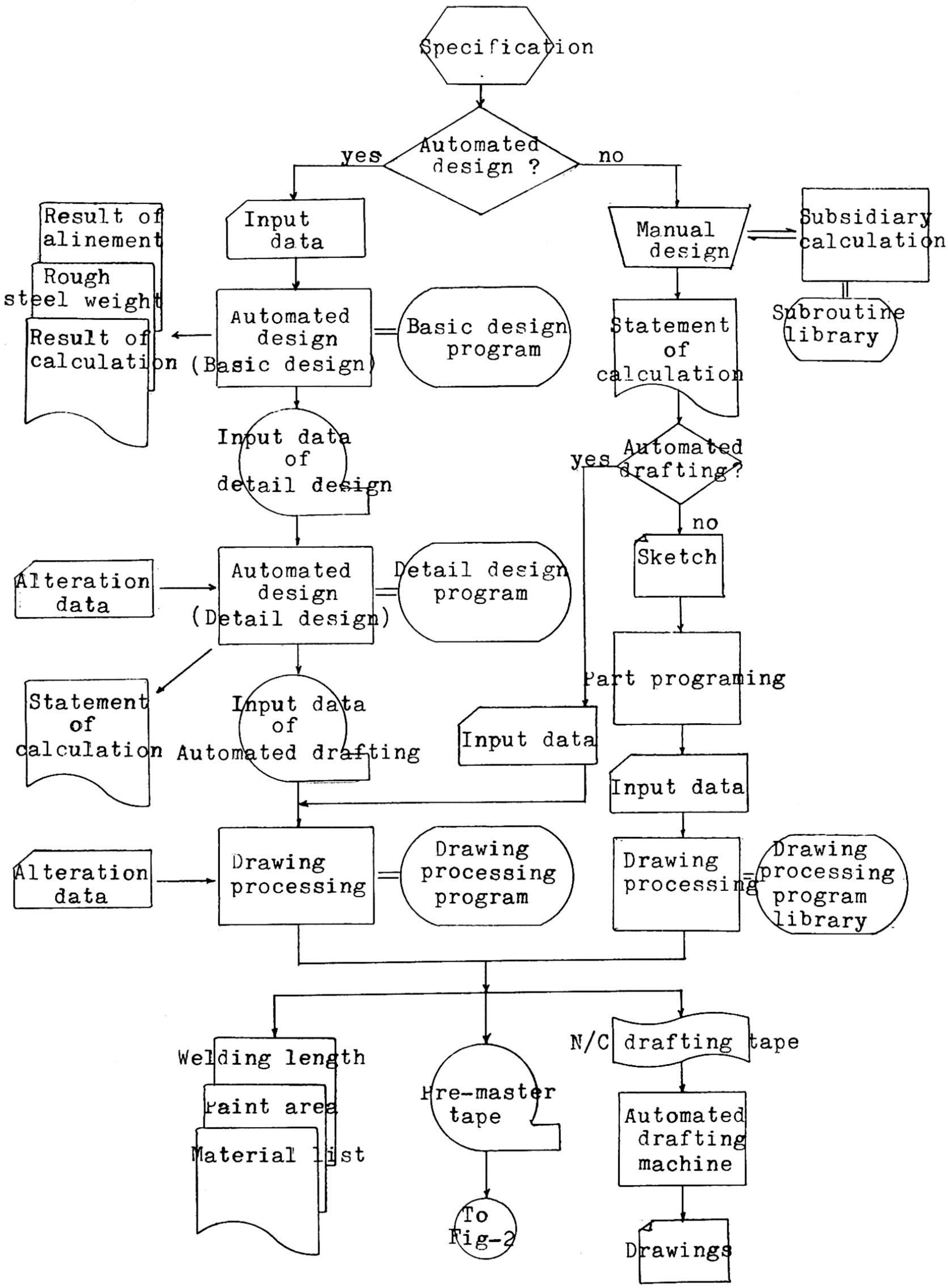

Fig.1 System flow chart of total system (1)

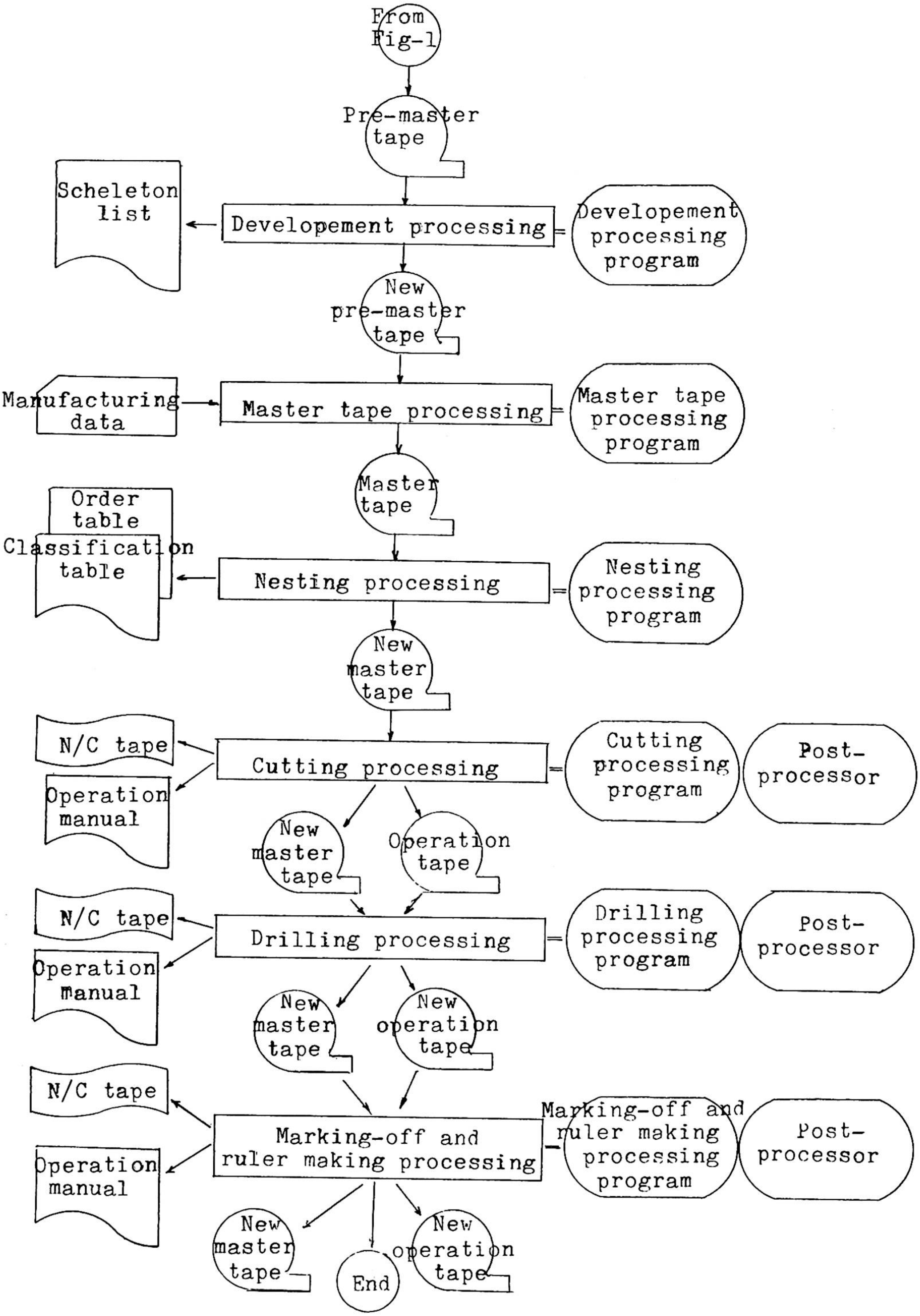

Fig.2 System flow chart of total system (2)

1. The basic design of composite girder

In accordance with the specification the most appropriate arrangement of structure is determined and preliminary calculation of the detail design is executed. The contents of this program are as follows:

Determination of arrangement type of main girder,

Determination of the optimum number of main girder,

Determination of interval between main girders and cantilever length of slab,

Determination of formation of cross section, haunch height and thickness of slab,

Arrangement of cross beams, sway bracings and lateral bracings,

Determination of the optimum height of girder, arrangement of sections and splices,

Estimation of rough steel weight.

2. Detail design of composite girder

In this process, details of structure are determined and calculated by use of the results of the calculation of the basic design. This system consists of alinement calculation program, structural analysis program, slab program, main girder program and cross beam and lateral bracing program.
Each program is processed sequentially being controlled by information in disk files.

III. Automated drafting system

N/C drafter's input tapes, pre-master tapes and material lists are offered by electronic drawing processing, using the parameters determined in the above system. The part program should be made for the complicated and particular structure to which the above system can't be applied.

1. Drawing language

We have uniquely developed the programing language for drafting. This language basis on ALGOL language, and consists of geometric and drawing variables in addition to ALGOL's reserved words.

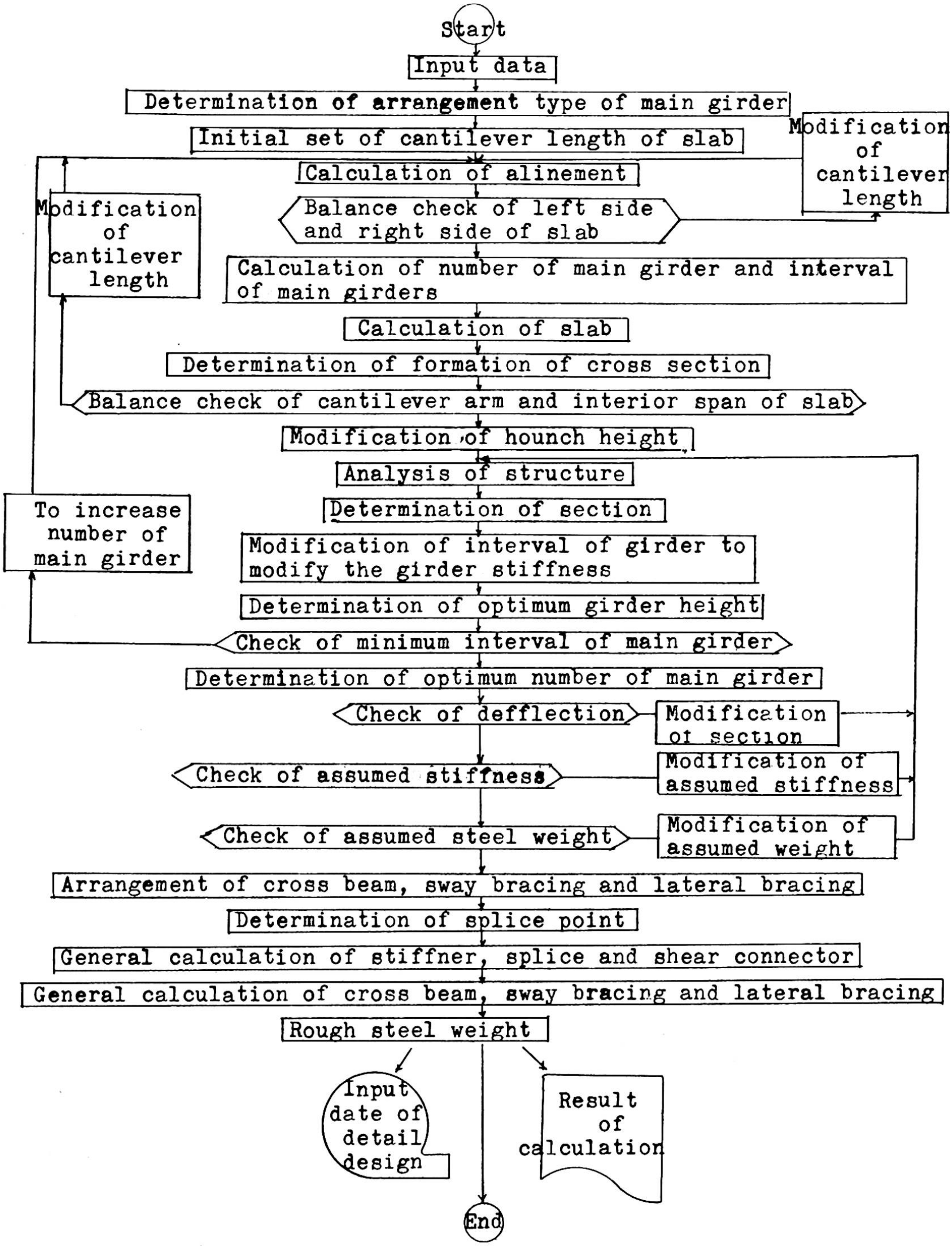

Fig.3 General flow chart of the basic design

In this language system, the patterns and members of a structure are directly processed, and the structure is built up by the electronic processing.

2. Master tape

Master tapes are equivalent to drawings, and consist of all basic information for shop fabrication. The contents of master tapes are separated to two types of data. The one is a set of member data, that is, figures or properties of each member, and the other is what means the relations between members.
The member data are as follows:

Original number of data,

Arrangement of member,

Number of members,

Dimension of member,

Quality of material,

Outline of elements,

Kind, number and co-ordinate of holes,

Bending, twisting, beleveling data,

Mark,

etc.

Relational data are link keys that indicate the connections of members, and form a "tree structure".

3. Automated drafting system of composite girder

This system consists of six programs, that is, for main girder, cross beam, sway bracing, lateral bracing, material and pre-master making.
In this paper only the general flow chart of main girder program is shown in Fig. 4.

IV. Automated fabrication system

Adopting this total system will help us to make N/C tape, and make the automatic fabrication possible. Furthermore, it will be possible to abolish the templete shop and marking-off works as auxiliary processes of production. Our plan is dealing with so-called pre-process, that is, cutting, drilling, marking-off and rule-making process. Consequently, the member assembling and welding process is out of our objects.

1. Development of full size drawing and addition of manufacturing data

We make a master tape from a pre-master tape by this electronic processing.
Practical measure is calculated automatically, modifying data in consideration of deformation of members due to dead load and developing the scheleton.

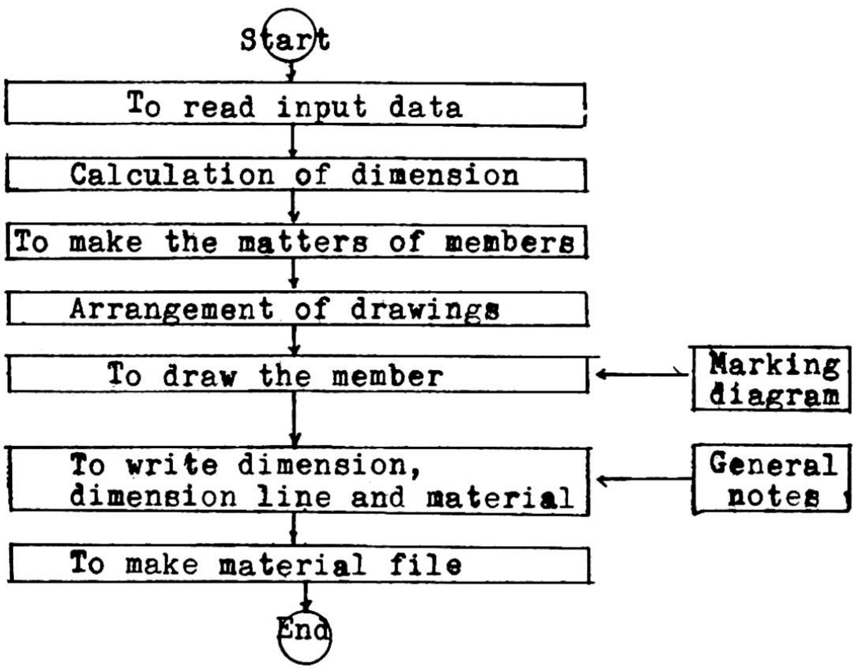

Fig.4 General flow chart of main girder program in automated drafting system

A person in charge of fabrication has to make data concerning the clearance for cutting, shrinkage and planing, and the deformation in accordance with fabrication condition, after examining drawings.

2. Nesting processing

The classification table showing qualities and thickness of material is make automatically by use of data in master tape, and the order table is done after determining optimum cutting stock with minimum loss.

3. Cutting processing

By use of data in a master tape, the form to be cut and sequence of cutting are determined in consideration of material deformation due to heat, and CL data is made. And then, using the CL data, N/C tape having a format according to each N/C machines is made by use of post-processor. It is possible to make N/C-gas-cutter draw lines by use of paint instead of gas.

4. Drilling processing

In order to drill holes for joint, this processing supply the determination of drilling position, automatic drilling and automatic cleaning up of dust after drilling.

5. Marking-off and rule-making processing

At this step, marking-off and making templetes are processed automatically, when it is needed to draw post-marking-off because of deformation due to gas-cutting and welded joint.

Objects of rule making processing are post-marking-off, planing and so on.

Summary

We are developing our total system, from design to fabrication of steel structures making use of an electronic computer and N/C machines. This total system consists of automated design system, automated drafting system and automated fabrication system.
By combination of these three systems, it is possible to fabricate the steel structure automatically without templete shop operation.

The Automation of Detailed Drawings by the Lucid System

L'automatisation de dessins de détails à l'aide du système Lucid

Automatisierung von Detailzeichnungen nach dem Lucid-Verfahren

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1. INTRODUCTION

Although engineers have used computers since the mid-fifties to assist them to design structures, it was only in the early sixties that they began to consider the possibility of using them to produce detailed drawings. As time progressed more and more firms began to explore this possibility and by 1965 there were probably about ten firms in Great Britain beginning to develop their own automated systems. From 1964-1969 the author was Chief Structural Development Engineer of a firm developing one of these systems and quite naturally during that period he concentrated solely on his own company's requirements. Similarly persons in other firms developing automated systems were primarily concerned with their own requirements, and no doubt each firm thought that its own method was the best solution. This is a perfectly normal human reaction in which a person's loyalty is to his own employer, and it results in a competitive situation which is highly desirable in the pioneering days of any enterprise, but which also has its disadvantages.

In January 1970 however the author took up his present appointment and was then able to review the problem of the automation of structural engineering drawings on a wider scale than had been possible when employed by an individual company. Consequently in conjunction with Professor G.C. Brock, Head of the Department of Civil Engineering, it was decided to study the problems of automating the production of construction information, and to assess what contribution the Department could make to the needs of the country as a whole. As the study progressed it soon became apparent, that if methods of automating the production of detailed drawings and bar bending schedules were left entirely to individual firms, then the following undesirable features would result:

a) The limitation of capital, and the dispersion of the available specialists in this area of work would probably mean that no system developed by an individual firm would reach its full potential

b) Since no individual firm has sufficient repetition of certain types of work to make it economic to develop automated techniques for these items, then the methods would be restricted only to the most commonly occurring structural elements.

c) Contractors would have to continually educate their staff to understand and interpret the many differing methods of presenting construction information which were being developed. This proliferation of different systems would be particularly unfortunate since it was only in 1968 that a Standard Method of Detailing was recommended(1).

d) Due to the high development cost of this type of work many of the smaller firms would not be able to afford this cost and consequently would not benefit from these new techniques

e) Although not true of all methods, some methods of automation would be such that the cost of interpreting the information on site would increase, and in some cases would also cause more construction materials to be used, resulting in a higher total cost to the client.

It also became apparent that many of these undesirable features could be eliminated, if a large proportion of our construction industry would combine to work together to produce an agreed system. Such a method could, of course only be produced provided there was co-operation from members of the industry almost on a national scale. Thus, although the need for co-operation seemed clear, the method whereby it might be achieved, and whether there was sufficient interest, had yet to be established.

2. PROPOSED METHOD OF CO-OPERATION

During the early part of 1970, discussions on this subject continued and a plan whereby the suggested co-operation might be achieved began to take shape. If the method of automation which was developed was to include a wide variety of structural elements such as bases, pile-caps, columns, beams, slabs, walls, staircases, retaining walls, culverts and subways, it was estimated that the cost of developing and introducing such a scheme might be of the order of £200,000. If this work was to be carried out by a full-time staff, and even if a hundred firms could be persuaded to join in the venture, this would still mean a contribution of £2,000 per member and it was concluded that many small firms would not be able to afford this expense. This method also had the disadvantage that the members would perhaps not feel directly responsible for the development of the system with the consequence that the sense of involvement and mutual co-operation would be lost. It therefore became more and more apparent that the most likely chance of success was to involve the eventual users in creating the system right from the outset, and to make the actual financial contribution from users as small as possible. This could be achieved if the organisation only had a small full-time staff, provided that members themselves would give expert assistance from their staff to develop the system as the need arose. On this basis it was calculated that the subscription rate per member firm could be reduced to only £100 per annum, provided sufficient firms would join in the venture.

It was therefore proposed to adopt the latter scheme and to examine the possibility of forming an organisation jointly between Loughborough University of Technology and members of the Construction Industry to develop techniques to reduce the cost of producing and interpreting construction information. The process of producing construction information would probably be based on the use of standard drawings of various structural components, in conjunction with printed information which, if desired, could be produced by a computer. It was from the initial letters of this process Loughborough University Computerised Information and Drawings that the organisation derived its name LUCID. It was estimated that the work could probably be accomplished in about three years.

The proposed organisational structure of LUCID was that the full-time staff would be directed by the author who would be responsible to a Steering Committee comprised of contributing members. This committee would be assisted by a Technical Advisory Committee, and Working Parties would be formed from the members to carry out and report on specific aspects of the organisation's work. The co-ordination of this work would be one of the tasks of the permanent staff who would also be responsible for preparing and disseminating information and ensuring that any feedback from members was brought to the attention of the relevant part of the organisation.

Discussions of these plans at a meeting in July 1970 with representatives from about fifteen firms indicated that the main principles which have been outlined were generally acceptable, and the interest was sufficiently encouraging to proceed further. By October 1970, after further discussion about fifty firms had nominally agreed to support the idea, and it was therefore decided to attempt to implement the plan.

3. LUCID'S DEVELOPMENT PLAN

Although the development of LUCID is a continuous operation nevertheless its proposed activities can be broadly divided into seven main stages. The stages are not of course separate and overlap considerably in time and content but they are in approximate chronological starting order. The broad titles of the stages are now given so that the overall plan can be appreciated, and later each of the early stages will be expanded. The seven stages are as follows:

a) The setting up of the LUCID organisation

b) The creation of standard drawings

c) The introduction of standard drawings into members' offices

d) Feasibility study of the combined use of computers and standard drawings

e) Development of computer programs for use with standard drawings

f) Introduction of computer programs into members' offices

g) Servicing and updating LUCID techniques

The individual stages of the development plan are now discussed in more detail.

4. SETTING UP OF THE LUCID ORGANISATION

The first step in assessing the interest in the industry was to publicise LUCID's existence, and towards the end of October 1970 a small brochure was sent out to a selected number of firms in the construction industry briefly outlining the aims of the organisation and inviting them to join. Since then no further publicity has been sent out and LUCID already has over 100 member firms.

These members represent all types of firm within the construction industry and include government departments, contractors, consultants, city and county authorities, reinforcement suppliers, structural steelwork designers and fabricators, as well as several computer manufacturers. The task of setting up an efficient organisation to co-ordinate the efforts of such a large number of firms is of course extremely important and took a great deal of time, effort and planning, but it would be inappropriate in this paper to go into too many details of this aspect. It is however worth noting that since LUCID sends out a considerable amount of information to its members such as Technical Reports, User Manuals, Bulletins, Newsletters and Questionnaires, it has set up, with assistance from the University, the facilities to prepare and print all its own documents. A typographical designer was also employed to assist in the design of a consistent house-style for the whole range of its documents.

Our two main committees, namely the Steering Committee and Technical Advisory Committee, have both been formed and LUCID is fortunate to have serving on these committees some of the country's most eminent engineers. In addition to this members have been extremely generous in offering the services of many of their senior staff to assist with working parties, and over 200 engineers have offered to serve in this capacity as the need arises.

During the first year the permanent staff has gradually been increased and has included the services of an engineer financed by the University, a second engineer who was engaged in July and a part-time secretary. While the administrative side of LUCID was being organised the technical side was also progressing and this aspect is reported in the next section.

5. THE CREATION OF STANDARD DRAWINGS

In the initial LUCID publicity brochure it was stated that the process of producing construction information would probably be based on the use of standard drawings of various structural components in conjunction with printed information which would, if desired, be produced by computer. This statement was based on the author's knowledge of the economics of the various techniques which have been employed by various organisations, coupled with the desire to develop a method which would not necessarily be completely dependent on the use of computers. Nevertheless the first task undertaken was to review all known methods of automation to see which, if any, could be recommended as the method which should be used by LUCID. As a result of this study the initial assumption remained unaltered, and it is significant that a report (2) produced by a Working Group of the 'Sub-Committee on the Application of Computers in Structural Engineering', which was established by the Department of the Environment, basically reached the same conclusion.

There are however many ways in which standard drawings can be used in conjunction with printed information, with considerably differing end results, and this whole subject was therefore studied in depth. It should be made quite clear at this point that the use of the phrase 'standard drawings' does not mean employing structural members of standard dimensions. Indeed it should be an essential part of the specification of any automated procedure that it shall not place any restriction on an engineer's choice of type of structure or individual member, but that once he has made this decision it should assist him to produce his detailed drawings more cheaply.

The first task therefore which was undertaken was to study the way in which standard drawings would be used so as to be able to set out a specification of their requirements. However before formulating a specification it was necessary to examine the whole range of structural drawings to see which might be included. It was not considered the task of LUCID to standardise structures, and the choice of structure, its layout and structural form must be determined by the designers to best suit their client's requirements, and it is no part of a communication system to influence this choice. Consequently, it was considered unlikely that layout and general arrangement drawings could be standardised and these are best produced by whatever process the designer feels appropriate. It was assumed that these would generally be scale drawings, and at the present time LUCID will not assist in their production process.

It was however concluded that the production of detailed drawings could be automated and it was convenient and effective to subdivide these into the basic elements of which structures are composed. Consequently the items for which it was decided that standard drawings could be produced are the detailed drawings for individual structural elements such as bases, pile-caps, columns, beams, slabs, walls, staircases, retaining walls, culverts or indeed any others which occur frequently. However, drawings even for an individual element are composed of separate diagrams such as plans, elevations and sections and it is therefore these items which are the basic standard details of a particular structural element.

It follows therefore that whatever process of automating the production of detailed drawings is used, the system must have available a series of preconceived pictorial arrangements. For convenience these will be called standard drawings but only the layout is standard not the dimensions, and they should not impose restrictions on an engineer's choice. There is no necessity to restrict in any way the number of preconceived pictures though it would only be sensible and economical to produce those which recur regularly.

Once having stated the area in which standard drawings could be applied a specification of their requirements was set out and this is now summarised:

5.1 Summary of a Recommended Specification for Standard Drawings

5.11 Standard Drawings. Standard drawings should be of individual structural elements and each drawing should include all the relevant plans, elevations, sections and written information relevant to that element on a single sheet.

5.12 Scale. A standard drawing need not be to scale but there should be a sufficient choice of standards available to eliminate any possibility of misinterpretation.

5.13 Size. A standard drawing should generally be A4 size but A3 size is acceptable if slightly modified to allow easy single folding into an A4 folder.

5.14 Reinforcement Details. Where a standard drawing shows reinforcement, the detailing method, positioning, and quantities of bars shown should conform as closely as possible to the equivalent scale drawing that the standard would replace. Starter bars from adjacent elements must be indicated.

5.15 Concrete Dimensions. Concrete outlines and dimensions should be shown to facilitate checking and shutter erection and to compensate for the use of not-to-scale drawings.

5.16 Material. Standard drawings should be produced on translucent material which will readily accept ink, pencil or typewritten additions and should be such that, even after any additions or alterations to the linework, high-quality copies can be obtained from dyeline machines or any other type of copying machine commonly used in design offices.

5.17 Recommended Layout. For preference the finished drawing inclusive of the written information should be basically the same whether the written information is produced manually, by small computer, via a terminal or using a fast line printer. The form of linework and written information should follow the recommended Standard Method of Detailing(1).

5.18 Future Developments. The standards should be devised to allow the use of any equipment which may be manufactured in the reasonably forseeable future. In particular, the technique should be easily adaptable to the use of plotters and visual display tubes when their use becomes economically viable.

5.2 DEVELOPMENT OF A SYSTEM TO SATISFY THE SPECIFICATION

The specification basically requires that the final drawing of an element contains all the relevant plans, elevations and sections while also allowing the written information to be produced by hand, or by a small computer, via a terminal, or on a large computer with a fast line printer. All this information should be on one sheet of paper and an example of the finished quality envisaged is given in Figure 1.

However, although the final drawing is comprised of two distinct but associated parts namely the picture and the textual information it is vital to subdivide the automation process into two stages. The first of these is to find a method of producing the picture and the second how to combine the text with the picture. The reasons for this are numerous but clearly if one can manage to develop a single method of producing the picture which will easily allow a user to change at will his method of producing the text this gives a very flexible system indeed.

After examining all known systems it was concluded that no existing system in use completely satisfied the specification but that a method known as the 'overlay technique' offered the greatest potential, provided it could be developed to achieve a considerably higher line quality than had previously been obtained, and that it could be coupled with a flexible computer system.

It is perhaps important at this stage to explain briefly what the overlay technique is and why its use was considered necessary. Initially we will only be concerned with the picture part of the drawing and will deal with the text later.

5.21 Method of Producing Linework of Standard Drawings.

One stated requirement of standard drawings was that although they could be not to scale they should not be misleading. For preference therefore every section, plan and elevation should closely resemble its scale equivalent. Suppose therefore that standard drawings are required for all reasonable alternative variations of square and rectangular column bases supporting either square or rectangular columns. From Figure 1 it can be seen that the total drawing comprises three separate details; a plan, a section through the base, and a column section. To ensure that the range of standard drawings will include a close representation of any possible base that may be designed it would be necessary for there to be, say, *four* alternative plans, *three* alternative base sections, and up to *ten* column sections to allow for the commonly used steel arrangements in square or rectangular columns. If all these possible alternative drawings are pre-printed ready for immediate use the number required would be 4 x 3 x 10 = 120. Thus pre-printing complete drawings is impractical and uneconomic, especially for more complex elements. However, returning to the problem of column bases again, instead of pre-printing complete drawings suppose that each of the alternative plans, base sections and column sections were each pre-printed in its correct position onto separate sheets of thin transparent material. In all we would then have 3 + 4 + 10 = 17 separate transparent sheets, each with just one detail on it. When a particular column base configuration is required, the sheet with the most appropriate plan is selected, together with the most appropriate base section and column section. When these three transparent sheets are superimposed we then have the basic linework for the complete drawing. If these three transparent sheets together with a suitable translucent light sensitive material are passed through a dyeline machine and the light sensitive material is developed, we have the required complete drawing on

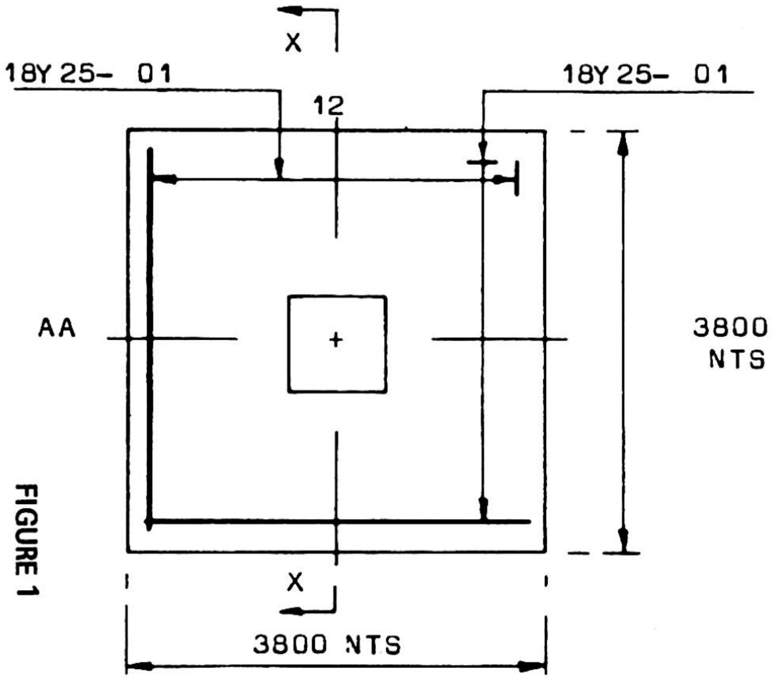
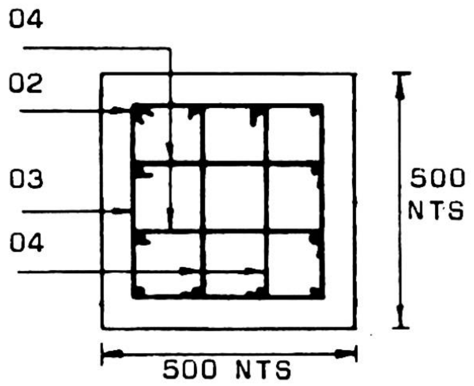
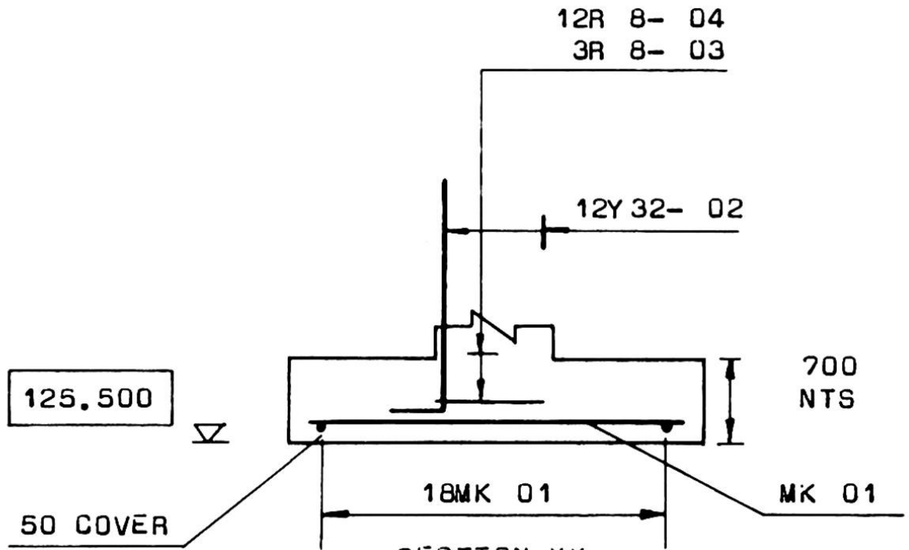


FIGURE 1



ENLARGED COLUMN DETAIL

BASE SECTION 2   COLUMN SECTION 3



SECTION XX

NOTE COVER TO COLUMN STARTER BARS 50

A.N.OTHER AND PARTNERS GREAT GEORGE STREET LONDON S.W.1	
CONGRESS CENTRE RAI EUROPAPLEIN,AMSTERDAM THE NETHERLANDS	
LUCID DEMONSTRATION COLUMN BASE 24 GRID LINES AA-12	
JOB No 30615	DRAWING No 107

one sheet of translucent material. The three transparent sheets of details would then be stored away for further use. Since the complete drawing is made up by 'overlaying' transparent sheets of the individual details of the complete picture it has been named by the author the 'overlay technique'. This paragraph describes the principle of the overlay technique but while the principle is simple, to develop it into an effective working system required a long and tedious research programme.

In the example just described 120 different complete drawings can be made from only 17 overlays, and the attraction of using overlays therefore is that by holding on transparent sheets only the basic individual details of which the total picture is composed, the number of configurations which can be produced is enormous. Any particular combination needed is produced only when it is required and consequently large quantities of pre-printed drawings do not have to be held in stock. The effectiveness of the system becomes even more marked as the number of details on a drawing increases. Thus in the case of beams, if the complete drawing consists of an elevation and three sections (therefore requiring four overlays) merely by having 20 alternative elevations and say 40 different sections then 20 x 40 x 40 x 40 = 1,280,000 alternative beam drawings can be obtained from only 60 basic individual details. While it would be completely uneconomic and unworkable to have 1,280,000 standard pre-printed drawings for beams it is extremely cheap and practical to hold only 60 details.

To operate such a procedure however it is necessary to have a technique whereby it is possible to overlay three or four sheets of details and produce the final composite picture without appreciable loss of quality. It is also necessary to be able to assemble the individual details on the final drawing within close dimensional tolerances both to superimpose individual details, and to ensure a later match with the textual information probably produced by a computer. An additional requirement is that the system must not be expensive to manufacture and the total process must be easy and cheap to operate.

Research into the technique was undertaken both at Loughborough University by Mr. C.A. Yardley and the author, and at the Cement and Concrete Association by Dr. M.R. Hollington. The work was tedious, time consuming and often frustrating but after about six months work the two teams eventually found a suitable transparent material, and a method of printing onto it, and a simple method of maintaining registration between the individual overlays. It may be thought that any transparent material would be suitable but this is not so, since it must be cheap, extremely translucent, durable, not discolour with time, be dimensionally stable in varying humidity and rapidly changing temperature, as well as having a surface which would accept and maintain a printed image.

The overlays which have been produced are on A4 size sheets of transparent material 0.05 mm thick and the relevant details are printed onto this material by a silk-screen printing process. In order to obtain registration between the overlays a thin metal strip with three studs was manufactured and each overlay has three holes punched in the filing margin at the same spacing as the studs. The simple procedure of containing all the sheets within a similarly perforated translucent cover was found to maintain registration and handling through a dyeline machine. To obtain initial correct registration, the back sheet of the cover is fitted on the studs followed in succession by the various overlays and the light sensitive material on which the composite picture is to be produced. Finally the top sheet of the cover is folded over. The package is then lifted from the studs and run through a dyeline machine. Completely satisfactory quality and registration are obtained by this simple and rapid process. It also has the advantage that overlays can be easily assembled by clerical staff and the linework in Figure 1 was produced in less than two minutes using a normal dyeline machine. After use, the overlays are merely filed away until required again. One set of overlays has been used over 1000 times, and they are still in excellent condition.

Quite apart from the fact that the drawing was produced in two minutes compared with about an hour by a draughtsman, the cost of producing the drawing is merely the labour cost of two minutes of a clerk's time plus three pence for a sheet of the highest quality light sensitive paper. The cost of producing this drawing was considerably less than 10% of that when produced by a draughtsman. It should further be noted that this is a very simple drawing, and while the time and cost of producing a drawing by a draughtsman increases with its complexity, the time and cost of producing a drawing by the overlay system is constant and quite irrespective of the complexity of the individual details.

Having found a satisfactory method by which the linework of standard drawings could be achieved, attention was then directed towards combining the picture and text .

5.22 Combination of Pictorial and Written Information

It was envisaged that the textual information could be produced in three basic ways:

a) By hand

b) Using a computer print unit which would allow single sheet feed

c) By a computer print unit using continuous stationery

With method a) there is no difficulty and the linework is produced on a negative which will accept ink or pencil and a draughtsman writes in the appropriate text. When method b) is used again the linework is produced on a negative and this is positioned in the computer print unit, and the computer program is so devised to write up the drawing in the appropriate places. If method c) is used the continuous stationery may be either a diazo or opaque paper. In both cases the text is printed in the required format by a computer onto a plain sheet of paper. If the paper is diazo this sheet is registered with the linework overlays and either a negative or prints can be obtained using a dyleine machine. If the paper is opaque the sheet is registered with the overlays and a copying machine such as a Rank Xerox is used. All these methods have been used successfully to produce drawings which give high quality prints.

5.23 Cost of Producing Overlays

The cost of printing A4 size overlays is less than £0.1 each. Assuming therefore that each different structural element requires an average of 30 overlays then overlays for all the following structural elements; bases, pile-caps, columns, beams, slabs, walls, staircases, retaining walls, culverts and subways can be manufactured for less than £30. Every design office will be issued with sets of overlays and since their use only requires equipment which already exists in a normal design office a cheaper way of producing the linework of drawings is difficult to imagine.

5.24 Advantages of Using Overlays

The main advantages of the overlay system where it is applicable are as follows:

a) Drawings can be produced rapidly, by clerical staff, for less than 10% of those produced by draughtsmen
b) Since the drawings are similar to those produced by draughtsmen, site staff do not require any additional training to interpret the drawings
c) The linework is produced without the necessity of a computer and it is always of a constant and high quality
d) The text may be added by hand, or by the use of a small computer, terminals, or large computers with high speed printers, thereby giving the user considerable flexibility when using the system
e) Since the linework is produced on a translucent material additions and alterations can easily be made by a draughtsman
f) Because the picture can be produced quite separately from the text, and pictures are an *international language*, the system can be used by people of *any nationality* provided the method of detailing is acceptable

The 'overlay technique' has been accepted by the Technical Advisory Committee as the method whereby the LUCID organisation will produce its drawings, and a report[3] describing the extensive development work on it has been issued to members.

5.3 Formation of Working Parties on Standard Details

Once the overall method of producing drawings had been approved, the Technical Advisory Committee began to form working parties to define precisely the concrete outlines and reinforcement details which would be required for each structural element. The first working party to be set up was one on beam-column intersections. Clearly the policy and layout at these positions must be determined before either the beam or column working parties can begin their work. This report has now been prepared and will shortly be sent to the Technical Advisory Committee for their comments. Working parties have also been formed to make recommendations for reinforced concrete bases, pile-caps, staircases, retaining walls, culverts and subways, and to study the problem of detailing structural steelwork. The working parties on columns, beams, slabs and walls will begin their work once the policy on beam-column intersections has been decided.

Once the details of a particular element have been agreed by the Technical Advisory Committee these will be circulated to members for their comments. After any amendment, overlays based on these details will then be produced together with User Manuals, and these will be distributed to selected members for field trials. After a suitable trial period any changes required will be made and the final version of the particular overlays will be produced. This process for various structural elements will probably continue for a period of about a year.

6. INTRODUCTION OF STANDARD DRAWINGS INTO MEMBERS' OFFICES

The exact details of how this will be done will be discussed in great detail with members prior to their introduction. A working party will be set up in the near future to study and make recommendations on both the training programme and method of implementation. The training program will obviously be on a very large scale, and the most appropriate educational methods must be devised, and carefully planned. In addition the method of operation of the techniques in offices, and any organisational changes which may be necessary, will also have to be studied in great detail. To speculate at this stage on what the working party will recommend would be pointless, but the author is conscious that the problem of introducing automated methods into design offices must not be underrated.

7. COMBINED USE OF COMPUTERS AND STANDARD DRAWINGS

Although the computer feasibility study is listed as stage four in the development plan several aspects of it have already been carried out. The first part of this study was whether small computers, terminals and computers with fast line printers could be used successfully with standard drawings. To assist in this exercise a specification of a program related to the design of square column bases was prepared[4]. This specification sets out the relevant calculations and flow charts, and includes references to the standard drawings which had been devised for this task. The requirement of the program, starting with the column load, ground pressure and column size as data, was to calculate the necessary base size and thickness, and the base and column reinforcement details, and to print three documents. The first of these documents was the calculations and their answers; the second a schedule listing the bending dimensions of the reinforcing bars in accordance with British Standard 4466[5]; and the third was to list the most appropriate combination of overlays to form the picture, and to write up the complete textual information on the picture formed from the listed combination of overlays. Figure 1 is an example of the solution for one particular set of data.

Besides writing programs of our own which were perfectly satisfactory, the specification was also sent out to various manufacturers of small computers, and they were invited to write programs to demonstrate the effectiveness of their machines. Not only has this opportunity been taken up and satisfactorily achieved by seven firms, but three of them have also loaned computers to LUCID as a consequence. The question as to whether computers can be successfully used with standard drawings has therefore already been proved. The second stage of this feasibility study,on which information is already being collected,is to examine the economics of using computers in conjunction with standard drawings. Since our members vary in size from the government's Department of the Environment to small consultants with less than twenty staff then clearly there is no single answer to this problem. We will however endeavour to make recommendations for various sizes of offices so that each member may judge his own position more clearly.

This last item ends the progress that has been made to date,and while it would be premature to anticipate the outcome of the feasibility study,it should be noted that stages given in the development plan allow for its successful outcome and even anticipate that a servicing and updating process will be necessary when the development work is concluded.

8. CONCLUDING REMARKS

LUCID as an organisation is of course still in its infancy but the progress that has been made in less than one year is very encouraging. So far, an extremely cheap method of producing the linework of drawings has been developed and working parties are now formulating the actual details which will make up these drawings. A successful method has also been found to combine the pictorial and textual information whether this is produced by hand or by a variety of computer configurations. As a result of this, the form of drawing produced by this automated method is entirely consistent with the traditional recommended Standard Method of Detailing. Finally and perhaps most important, members of our construction industry have shown, since over one hundred firms have joined together in this venture, that although they are often in competition with each other, this does not preclude them from co-operating on a matter of such national importance.

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SUMMARY

This paper describes how LUCID, an organisation formed by Loughborough University and over 100 firms in the British Construction Industry, has developed a general technique whereby the linework of drawings can be produced for less than 10% of the normal cost. It is shown that text may be added to the linework, by hand or by computer, and that the finished drawing is of the highest quality. The cost and method of operation are discussed and the progress on applying the method to numerous structural elements is reported.

GENESYS
(GENeral Engineering SYStem)

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1. History of GENESYS

In 1967 the Ministry of Public Building and Works, now the Department of the Environment, set up a working party to examine suitable computer programs which were available to structural engineers to see if any could be used and made more widely available in the construction industry by improvement, translation for a wider range of computers, or simply by making them better known. The working party proposed that the greatest need was to establish some order out of the chaos that at present exists. The working party found that the main obstacles preventing engineers from using computers were that

a) Current programs are not readily exchanged between computer owners.

b) There are many different methods of preparing data for the programs that do exist which increases unnecessarily the errors in data preparation.

c) There is much duplication in certain areas but notable gaps particularly for programs which do simple design operations.

The first report of the working party maintained that future wastage of programming effort could be much reduced by housing a system which could file any new program in its library for use by all. The system is now called The GENeral Engineering SYStem, or GENESYS.

Professor Peter Morice of Southampton University led a separate team of experts who evaluated the GENESYS proposal. They compared GENESYS with many other systems, including ICES and PLAN, that it was thought might be adopted to satisfy the same specification. Representatives of the construction industry, consulting engineers, contractors, central and local government bodies and also computer manufacturers and bureaux, were all approached in order to find out if there would be sufficient support for the system. The industry was strongly in favour of the proposal. They had recognised a need for such a system and GENESYS appeared to fulfill their requirements.

Clearly the order of cost put such a project outside the reach of individual engineering consultants and computer firms, who might perhaps have ventured the capital. Computer firms, however, could not have been expected to preserve the machine independent characteristics of the project. So, towards the middle of 1969 the Ministry of Public Building and Works secured the financial authority to give Alcock, Shearing and Partners the contract to write the system and placed several other contracts elsewhere to produce the initial set of subsystems (which are the problem-solving programs). GENESYS is now in the final stages of development and will be released in the spring of 1972.

2. What is GENESYS?

The Basic System, GENESYS can be used only through the medium of the basic system which controls all the input and output operations and also the storage of programs and data. This is the computer software which will be issued to computing installations in the form of magnetic tapes or disc packs together with detailed operating instructions. As the basic system retains control of everything that goes on inside the computer the term "subsystem" is used in preference to program for the set of instructions written to solve particular problems.

GENTRAN. This is the language in which all subsystems are written. GENTRAN is an extension of FORTRAN and it follows that many existing FORTRAN programs will need only minor alterations to turn them into GENESYS subsystems.

GENESYS is machine independent. It can run on a large variety of computers.

ICL 1900 series
ICL System 4
IBM 360 series
Honeywell 200 series
CDC 6600

The minimum size of main store is 32K of 24 bit words or the equivalent. Backing store: four magnetic tapes or one disc.

The three main features of GENESYS are that :-

1. It is a library of programs which can be run on a large variety of computers,

2. the sequence of programs used in the calculation is under the control of the User,

3. the input data required by these programs falls into a standard pattern.

The library of programs will be increased by subsystems written by GENESYS users. The GENESYS Centre, of which more will be said later, will operate a brokerage service whereby authors of subsystems will be able to lease or sell them to other GENESYS users.

The information for a GENESYS subsystem consists of tables and commands. The tables contain the data and the commands instruct the system what is to be done with the data. By using commands the engineer is able to direct the sequence of calculations carried out by the computer. By this means he does not lose control of his design and consequently he has more confidence in using a computer for calculations.

It has long been a recognised fact, that, when using a computer most faults occur due to incorrectly presented data. It is also true that these data errors occur because an engineer has to learn a new set of rules for data preparation every time he operates a different program. GENESYS goes a long way towards eliminating these faults by using a standard form of data input for all subsystems. An engineer may be designing highways and then change to designing buildings or some such operation. The same rules will apply and he will not notice any difference in the manner in which he prepares his data.

The first five subsystems which will be issued by the GENESYS Centre will be :-

(i) FRAME-ANALYSIS/1. This subsystem enables an engineer to describe a loaded skeletal frame for elastic analysis. The skeletal frame may be a space frame, a grid loaded normal to its plane, or a plane frame loaded in its own plane. No limits are imposed by the subsystem on the number of joints, members, etc. The size of the problem able to be solved by FRAME-ANALYSIS/1 is only limited by the size of computer being used.

(ii) HIGHWAY/1. This subsystem has been developed from Chapter 6 of the British Integrated Program System for Highway Design. The engineer is able to obtain the following information.

- (a) Setting out schedules for alignments, paving limits and slope stakes.
- (b) Details of cross-sections including the location of substrata where they intersect the cut surface.
- (c) Tabulation of volumes of cut and fill calculated separately for each material in the geological model.
- (d) Volumes of cut and fill in regions where one road in an interchange overlaps another.
- (e) Outcrop areas of the various geological materials on the formation surface of the road.
- (f) Soil stripping and seeding quantities.

(iii) BRIDGE. This subsystem carries out a linear elastic analysis of bridges where transverse load distribution need not be considered.

(iv) RC-BUILDING/1. This subsystem allows a user to analyse, design and detail a Reinforced Concrete building of beam and slab construction. The building may be irregular in plan or elevation and may include rectangular and skewed sections.

(v) SLAB-BRIDGE/1. This subsystem is described in the Appendix.

The GENESYS Centre

No matter how good GENESYS may be, merit along will not secure its acceptance by engineers. Only by first class presentation and adequate supporting services is it possible to overcome the initial inertia which always exists when a radical innovation of this kind is provided. The Government considered it essential that there should be a body of of engineers who, conversant with every aspect of GENESYS, would be able to maintain improve and develop it; and that this organisation should be available to all users of GENESYS to provide any maintenance, advisory or consultancy service that they need. It was also considered equally essential that a full information service be available and that educational facilities be provided by courses, lectures and demonstrations for both the programmers who write for GENESYS and the engineers who would use the existing Subsystems. The GENESYS Centre has been established at the University of Technology, Loughborough, Leicestershire, in order to market the system and to provide the services which have been described above.

The brokerage service has been mentioned before and this will also operate from the Centre. It is a fundamental feature of the concept of a national system that a library of compatible programs will grow and be accessible to all users, but it is neither practical or desirable that the GENESYS Centre should be solely responsible for the production. It is a prime aim of GENESYS to encourage engineers to convert existing programs and to write new ones in the system format and to make these programs available to others who wish to use them. Insofar as may be practical, a library of such programs will be formed at the Centre whence they will be supplied to engineers on whatever terms may have been agreed to by their authors. Thus the Centre is the central focus of GENESYS affairs and the forum within which consultation and co-operation will develop and flourish.

4. GENESYS International

Everything that has been mentioned so far concerns the United Kingdom. What about overseas? A flood of enquiries from all over the world has proved to the GENESYS Centre that there are great possibilities for GENESYS overseas. However, the position abroad must be different from that in the United Kingdom. Links with the GENESYS Centre must be reduced to a minimum. An organisation in Loughborough cannot provide an effective back-up service to customers in foreign countries and would have to devolve this function to a suitable agency. Ideally there should be a Centre in each country similar to the Centre in the United Kingdom. This Centre will deal with all the daily support necessary and will act as a link between the overseas user and Loughborough.

It is planned to operate an international brokerage service similar in function to the service operating in the United Kingdom. This will encourage users to market their programs overseas and, what is more important, encourage international technological co-operation using a standard system, a standard language and standard documentation.

APPENDIX

SLAB-BRIDGE/1

This is based directly upon the program known as Design and Analysis Program Package BECP/1. This package, sponsored by the Department of the Environment, analyses and aids the design of reinforced concrete slab bridge structures (flat slab/grillage combinations) and is based on the Finite Element thin plate and beam bending program BAPS, developed at University College, Swansea under Professor O.C. Zienkiewicz.

To enable BAPS to be more readily used for slab bridge design problems, separate programs were developed which together with BAPS made up an integrated suite of programs in the package BECP/1. This suite of programs has now been extensively used by bridge designers but nevertheless has recently required considerable alterations to make it easier to operate. SLAB-BRIDGE/1 is a further development which utilises successfully the GENESYS system to enable the engineer to solve a complex problem simply and efficiently.

SLAB-BRIDGE/1 carries out a finite element analysis of concrete bridge slab (or slab and beam) structures. The finite element mesh and co-ordinate data can be automatically generated for most regular or semi regular plan shaped bridge structures by setting up a regular grid system over the structure. The structure can have any arbitrary support conditions including elastic bearings while settlements of these supports can also be considered. All standard bridge loadings (B.S. 153 HA and HB loadings) can be easily applied.

The results of the analysis are related to a finite element mesh and are in the form of nodal displacements, nodal reactions and the moment field Mx, My and Mxy and the principle values M_1, M_2 and θ at the centre of each plate bending triangular element or Ma, Mb and T for each beam element.

Any of these results can be printed when required and displacements and moments for only selected nodes or elements need be obtained. Additionally, for structures containing plate elements only, nodal averaged values of moments can be obtained. Results of previous load cases can be combined and factored. The plate bending moment fields for all loading cases (including ones formed by combining previous results) can be processed to obtain critical design moments of resistance of compressive reinforcement in specified directions in the top and bottom of the slab.

By using the GENESYS system SLAB-BRIDGE/1 is able to have the following features.

1. The package is not machine tied and can run on any machine with a GENTRAN compiler without any conversion being necessary.

2. The user no longer has to handle separate programs with the added troubles of data transfer between these programs.

3. The subsystem will be extended to analyse prestressed concrete slabs and additional post analysis processing packages and this will be readily accomplished by the inclusion of appropriate commands within

SLAB-BRIDGE/1.

4. All data is in free format with the facilities of generation of repetitive data and the inclusion of variables in data.

5. The user can easily add to the package as his requirements demand.

Apart from its obvious flexibility as a GENESYS Subsystem, SLAB-BRIDGE/1 has some attractive features not available in the earlier programs. The finite element mesh is set up by the user specifying grid lines. The mesh is generated and numbered by the computer; a regular mesh if a regular shape is specified and an arbitrary mesh if the deck is of an arbitrary shape. (See figures 1 and 2). Line loads can be easily specified and H.B. vehicle loads are generated automatically. By storing details of the finite element mesh of the deck and details of the loadings, the structure can be re-analysed with altered support conditions and using new loadings or with the original loadings. Additional load case results can be generated by combining previous load case results factored by chosen values. By processing the load case results the command REINFORCE will generate the appropriate design moments of resistance table.

The subsystem contains a series of eight separate commands. These commands must be used in a logical sequence, but all the commands need not be utilised for each problem and the sequence of using the commands is not strictly set.

The eight commands can take a number of different forms but are based on the keywords;

MAKE -	Constructs the deck of the bridge.
ASSEMBLE -	Adds supports to the deck to produce a bridge.
FORM -	Produces loading cases for the bridge deck.
APPLY -	Applies loading cases to the bridge and produces solutions.
PRINT -	Prints out selected results from the solutions.
CREATE -	Produces solutions for new loadings by factoring and combining results of existing solutions of applied loadings.
REINFORCE -	Prints the envelope of tensile steel design moments for selected load cases.

Examples of how some of the commands may appear are shown below.

```
MAKE DECK 'DECK/1' OF SHAPE 'RECTANGULAR' ...
USING MESH 'FINE' WITH SLAB PATCHES 'PATCH' ...
OF MATERIAL PROPERTIES 'ORTHO'
```

or

```
MAKE DECK 'DECK/1' OF SHAPE 'ARB' ...
USING MESH 'FINE' HAVING BEAMS 'TYPE A' OF ...
MATERIAL PROPERTIES 'BEAM-MATERIALS'
```

An example of the FORM command could be :

FORM LOAD CAGE 'LOADING-1' FROM LOAD 'DEAD'

The words between the quotes e.g. 'DECK/1' refer to the tables.

The tables follow the same rules as for all GENESYS subsystems which give the engineer the opportunity to choose his own table title and to choose the units that he wishes to work in.

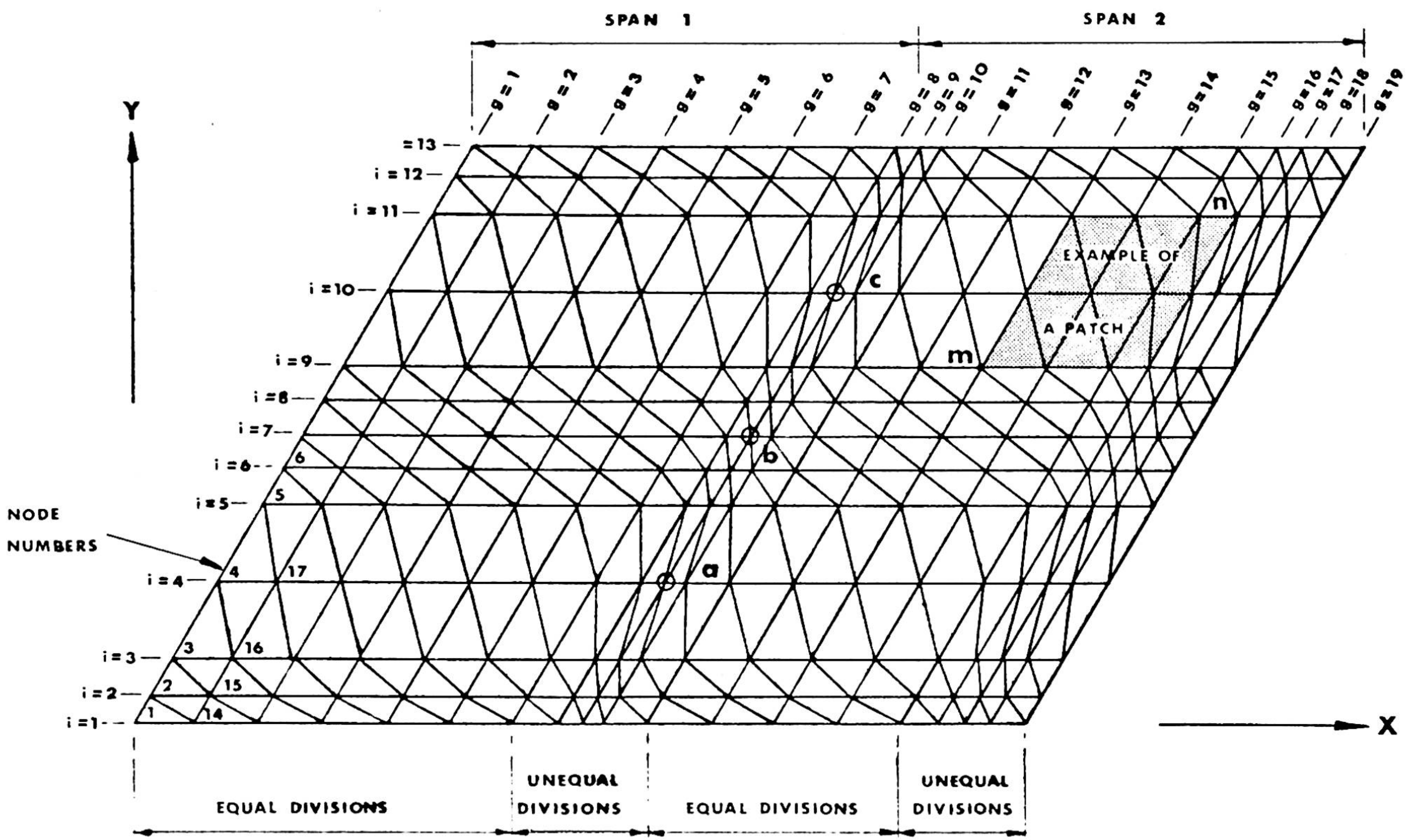

FIG 1 EXAMPLE OF DECK SPECIFICATION

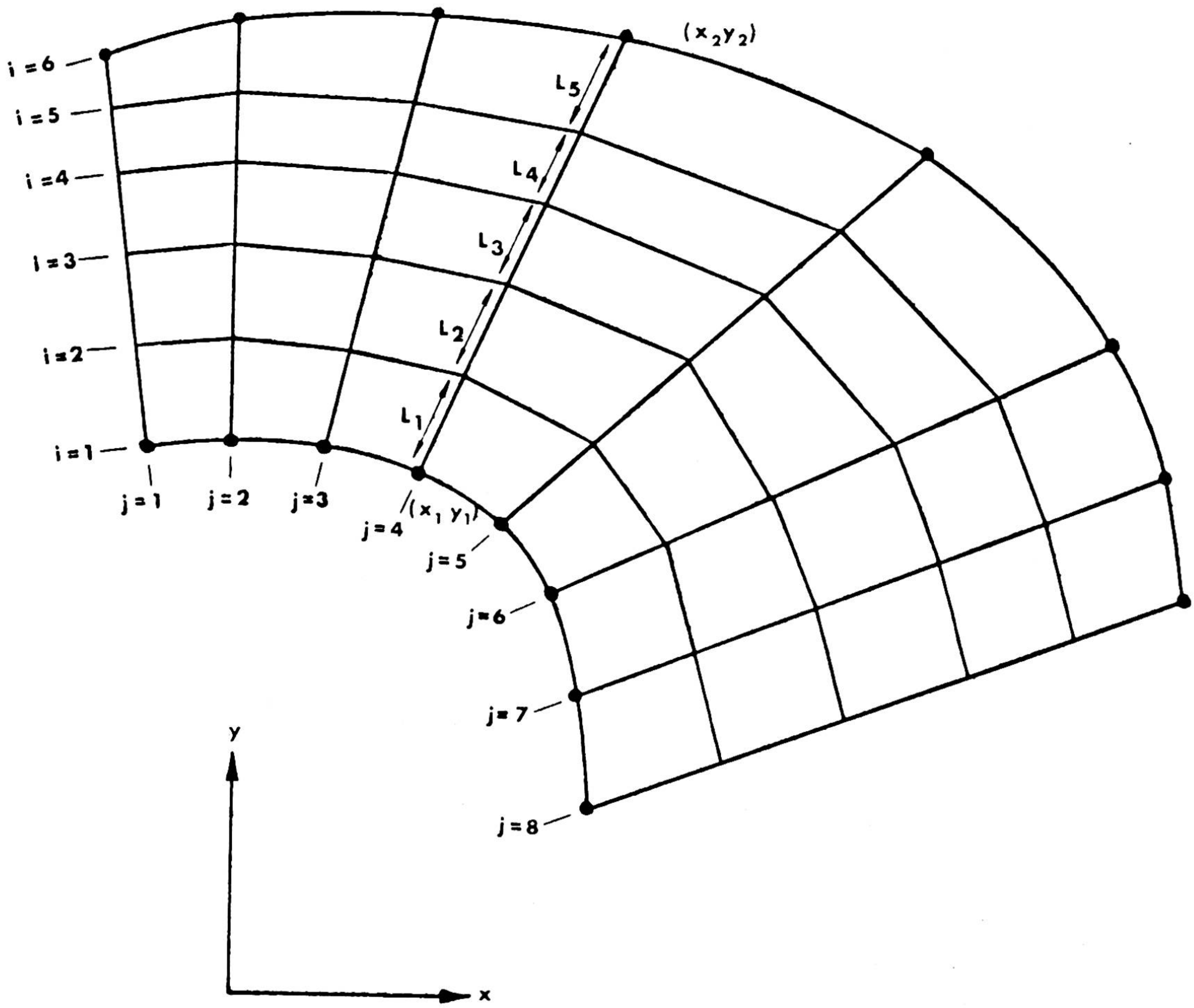

FIG 2 ABITRARY MESH GENERATOR

Acknowledgements

GENESYS has depended on the labour of a great many people and the author wishes to thank the Department of the Environment and in particular the Highway Engineering Computer Branch who together with Messrs. R. Travers Morgan and Partners have cooperated to produce SLAB-BRIDGE/1. The appendix is based on the subsystem manuals prepared by Messrs. R. Travers Morgan and Partners.

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Summary

In 1967 the Ministry of Public Building and Works set up a working party to study the ways of making computer programs more readily available to engineers. The working party reported that much wastage of programming effort could be reduced by a system which would file any new program in its library for use by all. This system is now called the GENeral Engineering SYStem, or GENESYS.

The main featuresof GENESYS are that it is machine independent, it has a large library of programs, by using commands the sequence of programs used in the calculations is under the control of the user and input data falls under a standard pattern. GENESYS will operate on the ICL 1900 series, IBM 360 series, ICL System 4, Honeywell 200 and CDC 6600.

Human Reactions to Automated Design of Concrete Building Structures

Réactions humaines en face de l'automatisation du projet de structures en béton

Menschliche Reaktionen gegenüber automatisierten Entwürfen von Massivbauten

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1. INTRODUCTION

The introduction of computer techniques into almost any human activity seems to generate hostility. This is particularly evident when the computer replaces traditional skill and requires those who practised that skill to modify well-established work patterns. This paper is concerned with the effects of a computer program that carries out the full design, analysis and detailing of the key elements in a reinforced concrete building structure. The program deals therefore with only a part of the structural design which in itself is only part of the total design involved in a construction project. Despite its relatively limited application, it is probably true to say that this program impinges upon a wider variety of people than any other program currently used in the construction industry in Britain. Although the program is constantly being refined and developed it has now been operational for over three years and has been applied to more than 100 projects. In the author's opinion this type of program has wide application throughout the whole design field and it is hoped therefore that the experience described in this paper will be of interest and encouragement to those developing methods of a similar nature.

2. TRADITIONAL DESIGN PROCESS

The traditional design and detailing of reinforced concrete is essentially a product of pencil, paper and slide rule. The engineer selects an arrangement of slabs, beams and columns which in his opinion best suits the particular building and he then chooses the size of each of these elements by a combination of experience and simple calculation. This information is recorded on general arrangement drawings, (framing plans and sections), which are distributed to the other members of the design team to form the basis of their own design work. Apart from this formal distribution of structural information, there is normally an informal flow of freehand sketches used to define those details which are of common interest to other designers. During the process of information exchange, it is to be expected that modifications will be required to the original structural concept to satisfy other needs. Ideally this should be a self-contained process at the end of which would emerge final agreed general arrangement drawings which would form the basis for the final design and detailing operations. Unfortunately, life is not like that and our traditional methods are invariably bedevilled by the need to produce working information for site whilst at the same time attempting to modify the basic concept to suit non-structural requirements.

The final design and detailing process is to a very large extent standardized by virtue of Building Regulations, Codes of Practice and nationally accepted conventions. Despite standardization this part of the work accounts for roughly two-thirds of the time and manpower used in a structural design office. The processes are completely routine and consist of the preparation of detailed calculations to assess bending moments, shear forces, reactions, deflections and hence areas and positions of all main and secondary reinforcement for each single structural element. With these calculations as a guide, the detailer then prepares true-to-scale drawings indicating the shape of each member and the location of each reinforcing rod. The rods themselves are detailed item by item on separate reinforcement schedules from which the weight of reinforcement is calculated for cost purposes.

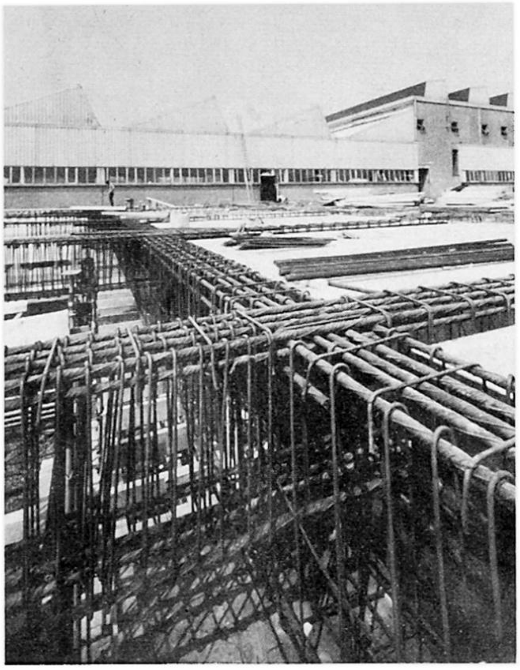
Most members of the construction team regard the end products of this traditional method (that is detailed calculations, true-to-scale drawings and standard reinforcement schedules) as sacred cows which, if they were to be replaced, could only be by means of automated facsimiles. Since the ABCONS program replaces all these traditional items by standard computer print-out, there is initial hostility from all quarters.

3. THE ABCONS PROGRAM

The traditional process described above may be thought of in terms of:-

- basic decisions that ought to be taken by an engineer
- consequential decisions that may be delegated to a computer

With a little care it is possible to extract the basic decisions in such a form that



ABCONS DESIGNED & DETAILED FACTORY REINFORCEMENT they can be made at a very early stage, following which the computer is given complete freedom to develop all the consequential decisions and to record them in a form best suited to the needs of each individual user. What this means in practice is that preliminary calculations and general arrangement drawings proceed in traditional manner using pencil, paper and slide rule following which design data such as loads, stresses and reinforcement patterns are determined by the design engineer and recorded on appropriate data sheets. Once these data sheets have been completed, the design engineer has no further commitment other than recording subsequent structural alterations. From the design data sheets, punch cards are prepared and the computer proceeds to analyse the structure as a whole and each structural element individually. It then calculates the diameter and bending dimensions for each reinforcing bar to ensure maximum economy consistent with the requirements of appropriate regulations, standards and codes of practice. Output includes:-

- comprehensive calculations suitable for submission to local authorities

- reinforcement schedules (one for each element)

- precise fixing instructions for each reinforcing bar

- quantities (with summaries) of formwork, concrete and reinforcement

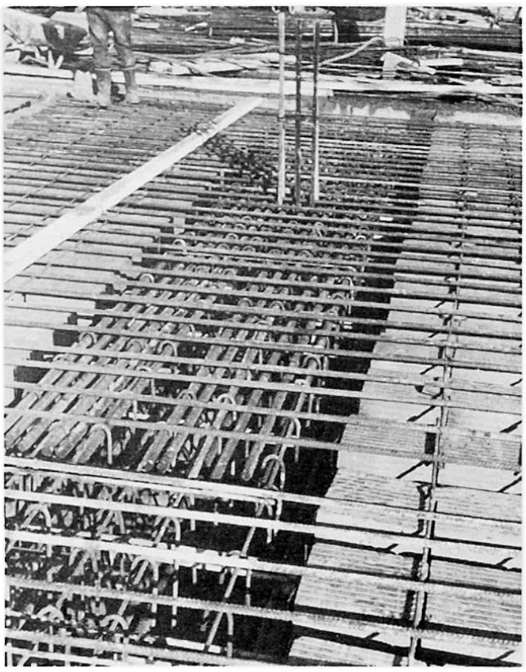
A feature of particular interest to site management and reinforcement fixers is that each structural element is described and detailed on a separate sheet identified by a comprehensive indexing system. This is a great help to scientific planning and enables the reinforcement for each element to be individually bundled and delivered to its appropriate location during construction.

After the initial computer run the design engineer uses the output as a basis for agreeing refinements and alterations to structural members and he records all such decisions on the appropriate output sheet. The ABCONS system includes a second up-dated computer run at about the time the contractor is appointed which incorporates all revisions. Since the computer is only able to deal with structural matters, items of non-structural concrete such as architectural nibs, chases, holes etc. must be superimposed manually and are recorded on output sheets in one of a number of simple ways.

Checking what may be thousands of sheets of computer output is neither attractive nor practical and an alternative procedure has therefore been developed. When completing the data input sheets, the design engineer records his estimate of the main reinforcement required for each individual element. The computer then compares this estimate with its own calculated value and draws attention to any significant discrepancy. A final overall check is obtained by comparing the total quantities of concrete, reinforcement and formwork calculated by computer with those measured by the Quantity Surveyor. These quick and simple checks have proved far more reliable than any known system of checking traditional designs.

4. REACTIONS - DESIGN ENGINEER

The reactions of the design engineer during trial runs of the program were so disturbing as to require a reappraisal of how the system should operate. The original intention was for the design engineer to be given a simple manual instructing him how to complete the data input forms. In the event this was unworkable because of the underlying hostility of human beings to automated systems. It quickly became apparent that the design engineer wanted to do battle with the system and was anxious to discover and exploit all its weaknesses. He went out of his way to mis-read instructions and was always looking for design combinations that were unacceptable to the computer. After this initial experience it was felt that no amount of education could guarantee a smooth transition from traditional to automated methods for the older, experienced design engineer, although these same techniques might be acceptable to a young, inexperienced man. To overcome the problem it was decided to introduce an ABCONS systems engineer who would act as the interface between the design engineer and the computer. ABCONS is now therefore built up around systems engineers who are experienced structural designers specially trained to understand and to be in sympathy with ABCONS procedures. By introducing a systems engineer, the problem of the design engineer has been solved and he now readily accepts the system and is appreciative of its benefits. In particular he enjoys the concentration of "real engineering" into a period of a few days as opposed to spreading it out over several and he is thankful to be relieved of the customary tedious checking activities. The greatest benefit, however, is the elimination of the detailing process which is a nightmare in most offices because sufficient good and reliable detailers no longer exist. The bane of most engineers' lives is to supervise an extensive detailing operation carried out by inexperienced students or disgruntled junior engineers.



ABCONS DESIGNED & DETAILED SHOP REINFORCEMENT

During the design development stage, the design engineer discovers substantial benefits from having clearly referenced calculations and construction details for each individual element available to him. With this information he is able to agree and record all necessary modifications, confident in the knowledge that his data are reliable and that his decisions will be properly incorporated in the final design taking account of all the consequential effects of those decisions.

Despite these acknowledged benefits, the inate hostility is readily evident. The slightest flaw in the program, or a breakdown of the computer, will produce the most unreasonable outburst from a man who spends the rest of his life happily accepting the failings of his own and allied professions.

5. REACTIONS - CHECKING AUTHORITY

During the development of the program it was thought that checking authorities would be reluctant to accept computer output in place of conventional calculations since the output does not provide the step-by-step working of ordinary arithmetic. The computer gives only the loading patterns and the final critical bending moments, shear forces and deflections together with moments of resistance, reinforcement areas, etc. In practice, however, local authorities have received these calculations with enthusiasm since the information given is all they require. Few checking authorities will attempt to unravel another man's arithmetic and they therefore welcome a system that provides readily referenced data in respect of each single structural element.

6. REACTIONS - ARCHITECT

If you ask an architect, he is most unhappy about the system since he is essentially visually orientated and therefore mistrusts information supplied in alpha-numeric form. If, on the other hand, you do not consult the architect, he is unaware that you are using the system since in practice he is concerned only with general arrangement drawings and those items of non-structural concrete which can best be defined on freehand sketches. Indirectly, the architect benefits because framing plans can be drawn up at a far earlier stage in the process whilst alterations can be accommodated up until a far later date.

7. REACTIONS - SERVICES ENGINEER

Like the architect the services engineer benefits from an earlier release of general arrangement drawings but in particular he finds that it is much easier to agree positions, sizes and details of holes for services because the structural engineer has all necessary data at his fingertips.

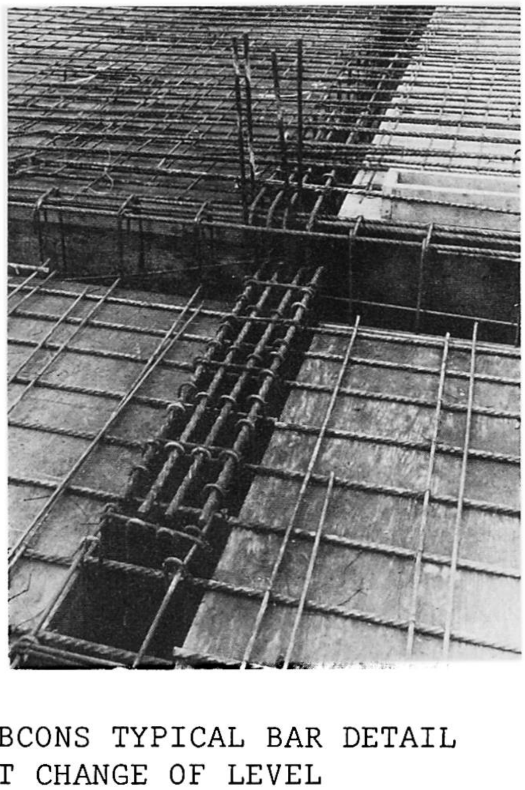
8. REACTIONS - QUANTITY SURVEYOR

Although the ABCONS system produces both individual and summarised quantities for reinforcement, concrete and formwork, this is of relatively limited value to the man measuring the job. Just as the form of an animal depends upon the nature of its bone structure so does the measurement of a building derive from the proportions of its structure and therefore the starting point for any building measurement is the structure itself. Thus, although all the quantities are available they are only used by the Quantity Surveyor as a check on his arithmetic. Similarly his measurement provides a valuable check on the computer output.

9. REACTIONS - GENERAL CONTRACTOR

General contractors are not renowned for progressive attitudes and they therefore generally start by being suspicious of anything that replaces conventional drawings. Their typical reaction is to assume that any such system makes life easier for the designer at the expense of the contractor. Unfortunately contractors speak with many voices and have their own communications problems so that by the end of most jobs it is quite common to find those who have been directly involved with the system reacting favourably whilst others cling to their original misgivings. In fact the planning engineer has available to him a great deal of valuable information that previously did not exist. He can therefore program work and order materials confident in the knowledge that his information is reliable.

Clearly a favourable reaction from the main contractor is essential to the success of any automated design system but it cannot be assumed that he will always appreciate the benefits that flow from it. Occasionally a contractor lacks the management skills necessary to utilize available information and sometimes he is unwilling to deviate in any way from traditional procedures. Our experience is that progressive contractors, willing to co-operate, gradually recognize the unsatisfactory nature of the existing information system based upon drawings. When this fact is established, they begin to appreciate the full benefits of automation



ABCONS TYPICAL BAR DETAIL AT CHANGE OF LEVEL

10. REACTIONS - REINFORCEMENT SUPPLIER

Of all those concerned, the reinforcement supplier is the one person who suffers by the system. Previously all similar bars in similar structural members were bundled together and delivered in large unsorted piles for site to unravel. With the ABCONS system, the reinforcement for each element is bundled separately thus eliminating the sorting process on site. Although this adds additional labours to the reinforcement suppliers' work, he does not complain but accepts the system without fuss. The reinforcement schedules appear in a form to which he is accustomed and it may not be immediately apparent to him that their requirements differ from traditional ones.

11. REACTIONS - REINFORCEMENT FIXER

At first sight, the reinforcement fixer could be expected to be least happy with the system. He is not a highly educated man and he has trained himself to work from traditional drawings and conventional bar schedules so it would not be surprising to find him hostile to sheaves of computer output. In fact the reinforcement fixer has proved without exception to be an enthusiast for the system which was devised to suit his needs. Although he uses traditional drawings, he cannot in fact use them directly since his work involves climbing ladders and contending with wind and rain. In practice therefore he has carried up the ladder a piece of paper or notebook with a simple shorthand notation describing the position of each rod. The ABCONS system merely prints out that shorthand notation in a standardized form which can be readily understood by any experienced bar fixer. Since each output form relates only to one structural element, the fixer can take it with him on to the job and it matters little if it gets wet or torn since after the element is complete, the piece of paper is disposable. In practice the reinforcement fixer does not throw the sheet away since it gives him an accurate record of the weight of reinforcement fixed, which forms the basis of his weekly pay packet. Most reinforcement fixers find it difficult to calculate weights of reinforcement and they appreciate a system which gives them, for the first time, a reliable check upon their employer's arithmetic.

12. CONCLUSIONS

This paper describes a variety of reactions and it is therefore perhaps inappropriate to attempt to draw conclusions. It may however be of interest to note that we were unable to find any solution to the interface problem between a traditional design engineer and the computer system. Since we could not solve the problem we had to eliminate it by introducing a computer systems engineer. Those most affected by the system liked it best, whereas those remote from it tended to be more critical. Finally, if the system fails everyone is only too willing to mistrust an alien technology.

13. REFERENCE

Field experience of replacing conventional detailed drawings — J. Seifert, The Concrete Society Symposium, Bristol, December 1970

14. SUMMARY

This paper describes an automated process for design, analysis and detailing of concrete building structures. It highlights the difference between this system and traditional methods and describes individual reactions of those most involved both within the design team and on site.

Les vérifications essentielles dans le calcul des tours de réfrigération à tirage naturel et l'importance de la conception dans leur sécurité

Die wichtigsten Ergebnisse in der Berechnung in Naturzug-Kühltürmen und die Wichtigkeit des Sicherheitsentwurfes

The Essential Verifications in the Analysis of Cooling Towers with Natural Draught and the Importance of the Safety Concept

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Les tours à tirage naturel sont de gigantesques voiles minces de révolution, libres à leur sommet, et reposant à leur base sur le sol de fondation par l'intermédiaire d'un réseau de poteaux croisés. Leurs dimensions ne cessent de croître avec la puissance des centrales électriques.

De nombreuses études ont été dernièrement publiées sur ces structures. Mais cette somme de recherches ne forme pas encore un ensemble cohérent. Soumis par ailleurs à des contraintes économiques, comment le projeteur peut-il être guidé dans la recherche de la solution optimale ?

Que penser des différentes méthodes de calcul ? Quelles vérifications essentielles effectuer sous les effets du vent ? Quelle est l'importance de la forme du méridien sur la résistance statique et sur la stabilité de forme ? Telles sont les questions auxquelles nous voulons essayer de répondre.

A - VERIFICATION DE L'EQUILIBRE STATIQUE.

Deux catégories d'efforts sont considérées dans une telle structure (fig. 1) :

- Les tensions de membrane N_x, $N_{x\theta}$, N_θ

- Les moments internes de la coque M_x, $M_{x\theta}$, M_θ

Ces efforts sont calculés en principe à partir de la théorie générale des coques de révolution, qui fait intervenir leur rigidité propre de flexion. Mais les dimensions et les proportions des tours, ainsi que leur finesse, sont telles que la prise en compte de cette rigidité de flexion n'a pratiquement aucune influence sur les valeurs des tensions de membrane. La théorie générale est donc utilisée essentiellement pour calculer les moments de flexion dans l'épaisseur de la coque.

En fait l'équilibre statique de ces tours est essentiellement un équilibre de membrane .

Ceci entraine des conséquences très importantes :

1 . Les tensions de membrane peuvent être calculées à partir de la théorie des membranes de révolution . Les déformations dûes à ces tensions peuvent en être déduites directement .

2 . Cet équilibre de membrane étant isostatique , les tensions ne peuvent donner lieu à aucune " adaptation " . Cela signifie que l'état limite ultime de cet équilibre est atteint dès qu'une des tensions est égale , en un seul point de la coque , à la valeur maximale que peut supporter le matériau .

Les vérifications essentielles à l'équilibre statique sont donc les suivantes :

- Dans la tour elle-même , sous l'action simultanée du vent extrême et du poids propre , la traction des aciers méridiens doit rester en tout point inférieure à leur limite élastique et la contrainte du béton tendu (supposé non fissuré) soit rester inférieure à sa résistance à la traction , de manière à éviter une fissuration horizontale généralisée sur une grande surface , qui amplifierait considérablement les déformations (cette dernière condition peut être déterminante pour définir les épaisseurs dans la partie inférieure de la coque) .

- Au niveau des fondations , aucune traction ne pouvant exister entre la semelle et le sol , les compressions dûes au poids propre doivent être en tout point supérieures aux tractions dûes au vent extrême . Ceci impose pour la tour et sa fondation un poids minimal , donc un volume minimal de béton (Cet état limite au niveau des fondations n'a aucun rapport avec ce qu'on appelle habituellement la stabilité au renversement d'une structure rigide . En particulier , la notion de noyau central de la fondation n'a aucun sens ici) .

De plus , pour pouvoir résister aux composantes radiales des efforts de soulèvement , la semelle doit obligatoirement constituer un anneau circulaire continu, buté latéralement sur le terrain par l'une au moins de ses faces .

3 . Toutes ces vérifications doivent évidemment être faites à partir des tensions de membrane N_x correctement calculées , ce qui signifie en particulier , la prise en compte de la forme exacte du méridien . On sait,par exemple , que l'assimilation d'un cône toroïde à un hyperboloïde fait considérablement sous-estimer les tractions méridiennes dans les zones inférieures et au niveau des fondations (fig. 2)

On peut montrer en effet que la courbure du méridien et la loi de variation de cette courbure le long du méridien jouent un rôle essentiel dans l'équilibre de membrane (1) .

Ceci s'explique simplement par le fait qu'en chaque point , une part de la pression du vent est équilibrée directement par la composante normale des tensions méridiennes $\frac{N_x}{R_x}$; tout se passe donc comme si la tour était soumise à des efforts du vent réduits , égaux à chaque niveau à $p - \frac{N_x}{R_x}$.

Ce terme minorateur des efforts $\frac{N_x}{R_x}$ n'est efficace qu'à partir du niveau où N_x a atteint des valeurs non négligeables. La courbure du méridien n'est donc pas utile au sommet de la tour . Les cônes toroïdes sont donc une mauvaise solution puisqu'ils localisent une forte courbure dans une zone restreinte près du sommet.La courbure du méridien n'est pas utile non plus à la base de la tour , car les forces qui y agissent ont alors un bras de levier trop faible par rapport à la base . Il existe donc des formes optimales qui peuvent amener des réductions considérables des efforts et donc augmenter à coût égal , la sécurité .

B – VERIFICATION DE LA STABILITE DE FORME .

Si la rigidité de membrane est suffisante pour justifier un équilibre statique, il n'en est pas de même de la stabilité de forme . Celle-ci requiert une rigidité de flexion de la coque .

En effet , la région de la tour supportant les surpressions du vent doit résister au cloquage sous l'action des compressions circulaires $N\theta$ dûes à ces surpressions .

Il est toujours nécessaire de faire le calcul de cette sécurité car elle varie considérablement avec les proportions et les formes des tours . On peut montrer en effet que dans les tours cylindriques , la pression critique est de la forme :

$$P_c = 0.4\,E\,\frac{e}{H}\frac{e}{R\theta}\sqrt{\frac{e}{R\theta}}$$

soit , avec les dimensions actuelles , environ 200 Kg/m2 . Cette valeur serait très insuffisante mais, dans les tours à double courbure , elle dépasse souvent 1000 Kg/m2 . Il est donc clair que la courbure du méridien joue un rôle considérable , en particulier en diminuant les déformations radiales .

Si le calcul est fait avec les hypothèses classiques d'un matériau homogène élastique , la sécurité des tours apparait en général considérable (encore faut-il tenir compte de la souplesse du sol et des poteaux supportant la coque)

Mais l'analyse de ce calcul montre que cette sécurité est liée à l'énergie de déformation de flexion circulaire $(M\theta)$ de la coque : une chute de la rigidité de flexion entrainerait un abaissement brutal de la pression critique .

Donc , s'il est nécessaire de faire le calcul dans les hypothèses classiques dont on vient de parler , il faut aussi s'assurer que ces hypothèses sont effectivement réalisées , c'est à dire que les courbures d'ovalisation $\rho\theta$ sous le vent extrême ne produisent pas la fissuration de la coque en flexion .

En effet , la double nappe d'armatures circulaires souvent préconisée n'est qu'un remède illusoire contre une chute de rigidité de flexion ; même de forts pourcentages d'acier , bien supérieurs à ceux utilisés actuellement , ne peuvent éviter cette conséquence : les sections fissurées sont 5 à 10 fois moins rigides que les sections homogènes .

La seule solution correcte de cet important problème réside dans une conception d'ensemble de la structure , qui minore les contraintes de flexion .

L'importance de cette conception d'ensemble peut être comprise en analysant les points suivants :

1. Les déformations sont imposées par l'équilibre de membrane .

La minoration des tensions , que nous avons étudiée plus haut, est donc un premier but à atteindre .

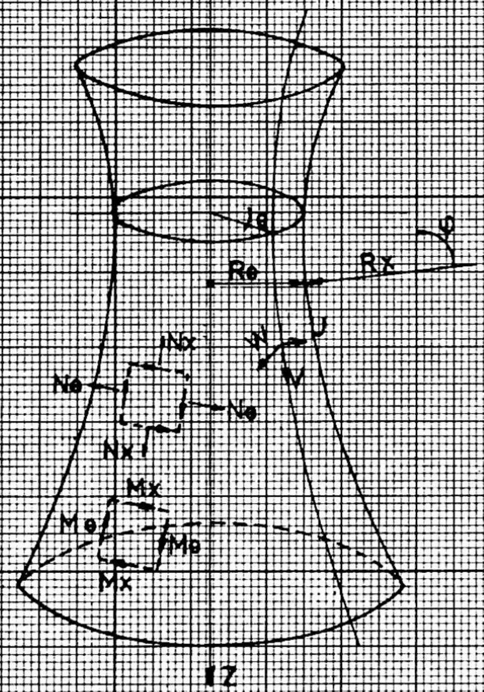

fig:1

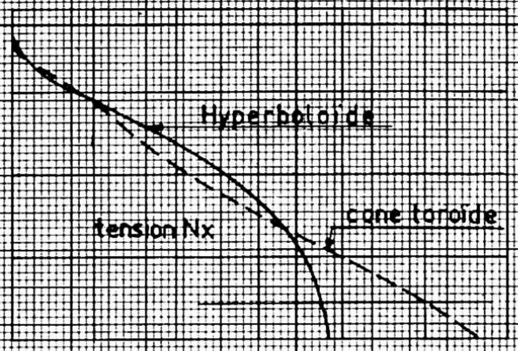

fig:2

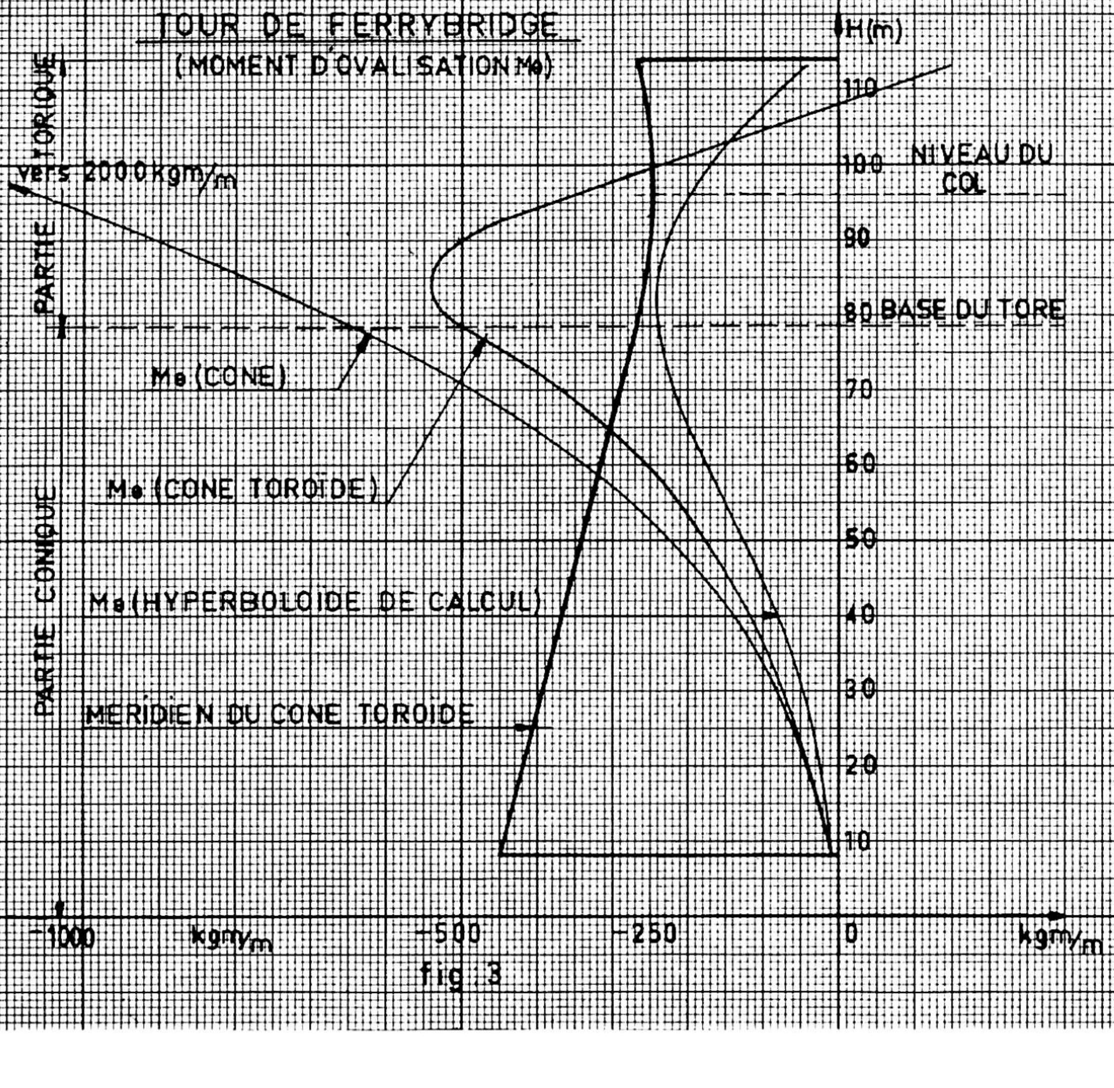

fig 3

2. Ces **déformations de** membrane ε_x ; $\varepsilon_{x\theta}$; ε_θ sont inversement proportionnelles aux épaisseurs ; donc les déplacements u , v , w de chaque point de la coque , et par conséquent les courbures ρ_θ . Mais les contraintes de flexion d'ovalisation sont , elles , proportionnelles aux épaisseurs : ($\sigma_{F\theta} = \rho_\theta \cdot \frac{Ee}{2}$.) . Une augmentation générale de toutes les épaisseurs dans un même rapport ne peut donc diminuer les contraintes de flexion ; au contraire , la souplesse du sol et des poteaux restant les mêmes , cette façon de faire augmente les contraintes . Notons aussi que dans ce cas , les tassements différentiels (dûs à l'hétérogénéité du sol) augmentant proportionnellement aux épaisseurs , les contraintes correspondantes de flexion augmentent elles-mêmes proportionnellement aux carrés des épaisseurs .

3. La solution parfois proposée d'augmenter localement les épaisseurs aux niveaux où les moments d'ovalisation sont maximaux ne peut qu'augmenter les contraintes de flexion à ces niveaux (puisqu'elles sont proportionnelles à l'épaisseur locale)

   Pour les mêmes raisons , la création d'une ceinture extérieure au sommet des tours est une cause de fissuration de la partie supérieure de la coque .

4. Une première solution consiste donc à répartir judicieusement les épaisseurs , en particulier en affinant la partie haute de la tour , et en épaississant les niveaux inférieurs , où les tensions méridiennes sont très élevées .

5. La forme du méridien a une influence considérable sur la valeur maximale des déplacements normaux et des courbures de flexion ρ_θ dans la coque .

   On peut le comprendre aisément en calculant ces courbures avec quelques approximations permettant de mettre en évidence le rôle essentiel de la courbure du méridien .

   Nous avons les relations suivantes :

   Entre les tensions et les déformations :

$$\varepsilon_x = \frac{N_x - \nu N_\theta}{Ee} \qquad \varepsilon_{x\theta} = \frac{2(1+\nu)N_{x\theta}}{Ee} \qquad \varepsilon_\theta = \frac{N_\theta - \nu N_x}{Ee}$$

   Entre les déformations et les déplacements :

$$\varepsilon_x = \frac{dv}{dx} + \frac{W}{R_x} \qquad \varepsilon_{x\theta} = \frac{du}{dx} + \frac{dv}{ds} - \frac{u\cos\varphi}{R_\theta} \qquad \varepsilon_\theta = \frac{du}{ds} - \frac{W\sin\varphi}{R_\theta} - \frac{v\cos\varphi}{R_\theta}$$

   En faisant $\sin\varphi \simeq 1$, $\cos\varphi \simeq 0$, et en négligeant ε_θ et $\varepsilon_{x\theta}$ devant ε_x il vient :

$$\varepsilon_\theta = \frac{du}{R_\theta d\theta} - \frac{W}{R_\theta} = 0 \implies W = \frac{du}{d\theta} \implies \frac{dw}{dx} = \frac{d^2u}{dx\,d\theta}$$

$$\varepsilon_{\theta x} = \frac{du}{dx} + \frac{dv}{ds} = 0 \implies \frac{d^2u}{dx\,d\theta} = -\frac{d^2v}{R_\theta d\theta^2} \implies \frac{dw}{dx} = -\frac{d^2v}{R_\theta d\theta^2}$$

$$\implies W = -\int_0^x \frac{1}{R_\theta}\,\frac{d^2v}{d\theta^2}\,dx$$

$$\varepsilon_x = \frac{dv}{dx} + \frac{W}{R_x} \implies V = \int_0^x \left(\varepsilon_x - \frac{W}{R_x}\right) dx$$

soit , pour chaque terme de la série de cosinus de la pression du vent :

$$W_n = n^2 \cos n\theta \int_0^x \frac{1}{R_\theta}\,dx \int_0^x \left(\varepsilon_{xn} - \frac{W_n}{R_x}\right) dx$$

(avec origine à la base de la tour)

Les courbures de flexion ρ_θ sont obtenues à partir de w :

$$\rho_\theta \simeq \frac{d^2w}{ds^2} + \frac{W}{R_\theta^2}$$

ou pour chaque terme de la série de Fourier :

$$\left|\rho_{\theta n}\right| = (n^2-1)\frac{W_n}{R_\theta^2} = n^2(n^2-1)\frac{\cos n\theta}{R_\theta^2}\int_0^x \frac{1}{R_\theta}\,dx \int_0^x \left(\varepsilon_{xn} - \frac{W_n}{R_x}\right) dx$$

On peut donc imaginer la détermination des déplacements w au moyen d'un calcul itératif depuis la base (où w est connu à partir de la souplesse des poteaux) jusqu'au sommet de la tour , l'intégrale double de $\left(\varepsilon_x - \frac{W}{R_x}\right)$ étant faite à chaque niveau avec les valeurs de w précédemment calculées .

Cette présentation des calculs permet de dégager des conclusions importantes :

a) Dans une tour troncônique , R_x étant infini , les valeurs de W et de M_θ croissent de plus en plus vite jusqu'au sommet .

b) Si dans les tours à double courbure , les moments d'ovalisation M_θ passent par un maximum dans la région du col , ce n'est pas parce que cette zone " attire " les moments , mais au contraire parce qu'elle stoppe leur croissance : le terme $\frac{w}{R_x}$ y est beaucoup plus grand que ε_x (qui lui-même diminue) .

c) Dans les cônes toroïdes , $\frac{w}{R_x}$ est nul sur toute la surface tronconique, et , jusqu'au tore , les moments M_θ ne cessent de croître de plus en plus vite , comme dans les tours troncôniques . Puis l'action inhibitrice de la courbure du méridien se fait sentir brutalement , mais trop tardivement alors que les courbures ρ_θ et les moments correspondants M_θ ont déjà atteint des valeurs très élevées .

Sur la figure 3 , où sont représentées les valeurs de M_θ dans 3 hypothèses - Tronc de cône , Cône toroïde , Hyperboloïde - on observe parfaitement le changement brutal de pente et de courbure du diagramme des M_θ au niveau du tore (pour le cas du cône toroïde)

Ainsi , un raisonnement analogue à celui que nous avons fait pour les tensions de membrane , montre que le terme minorateur des déformations méridiennes ε_x ,soit $\frac{w}{R_x}$, n'est efficace qu'à partir du niveau où w atteint des valeurs non négligeables La courbure du méridien n'est donc pas utile à la base de la tour . Mais elle ne l' est pas non plus au sommet si les déplacements w ont pris des valeurs déjà trop grandes . Il existe donc ici aussi des formes optimales .

Il est certain que la sécurité vis à vis de la stabilité de forme dépend de l'absence de fissuration généralisée sous le vent extrême .

Or , comme on le voit sur la figure , les moments dans le cône-toroïde sont plus du double de ceux d'un hyperboloïde de mêmes dimensions . Ils créaient , le 1er novembre 1965 , des contraintes de 20 Kg/ cm2 environ dans la région du col . Donc , même si les tours de Ferrybridge avaient été calculées et armées correctement , elles auraient présenté un danger de cloquage sous des vents extrêmes .

C - <u>CONCLUSION</u> .

Il est important de noter que , dans de telles structures , la solution la plus sûre n'est pas nécessairement celle qui correspond à l'emploi de la plus grande quantité de matière . La sécurité maximale se trouve ici dans un choix judicieux des formes et un balancement des épaisseurs .

On a dit quelquefois qu'avec le béton armé , l'architecture était morte,parce que tout était possible . On voit que ces tours , au contraire , nous confrontent avec de tels problèmes que nous sommes contraints d'utiliser notre matériau avec une certaine ruse , comme nos prédécesseurs durent le faire pour la pierre .

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RÉSUMÉ.

Les tours de réfrigération sont des coques dans lesquelles l'équilibre de membrane est fondamental , mais aussi dont la rigidité de flexion interne doit être suffisante pour assurer la stabilité de forme . Cependant , il faut que les déformations restent faibles et que la coque soit choisie la plus fine possible si l'on ne veut pas que cette rigidité de flexion soit brisée par une fissuration généralisée dans les déformations imposées par l'équilibre de membrane . La forme du méridien joue un rôle primordial dans la minimalisation des déformations sous le vent . Les cônes toroïdes , souvent utilisés jusqu'ici, doivent être éliminés de nos projets .

Théorie des équivalences
Fondements et applications au calcul des dalles et des coques

Theorie der Gleichwertigkeiten
Grundlagen und Anwendung bei der Berechnung von Platten und Schalen

Theory of the Equivalences
Fundamentals and Applications of the Calculation of Plates and Shells

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La Théorie des équivalences constitue une approche générale permettant de substituer à l'étude d'un corps chargé celle d'un corps fictif plus accessible au calcul. Ce corps fictif peut avoir des caractéristiques et des lois de comportement qui n'ont aucun sens physique. Cette théorie, que nous développons depuis une dizaine d'années, s'applique à tous les problèmes qui dérivent d'un champ et se prête aisément au traitement sur ordinateur. Elle nous a permis d'aborder avec succès l'étude de divers ouvrages d'art complexes. L'équivalence peut être réalisée entre un corps continu et un corps discret, entre deux corps continus ou entre deux corps discrets. Dans la présente étude nous illustrerons, en particulier, son application au calcul des dalles et des coques.

I) EXPOSE GENERAL.

Considérons un corps déformable soumis à un chargement dérivant d'un potentiel $\emptyset$. Le potentiel total π du système (corps + charges), supposé conservatif, s'écrit :

$$\pi = \int_V U_0 \, dV + \emptyset \qquad (1)$$

où U_0 est la densité d'énergie de déformation dans le corps chargé.

La résolution de ce système revient à rechercher un champ de déformation compatible avec les liaisons du corps et minimisant la fonction π ($\delta\pi = 0$).

Soit un deuxième corps occupant le même espace V que le premier et soumis au même chargement. Désignons par U'_0 sa densité d'énergie de déformation. Le potentiel total π' du système s'écrit alors :

$$\pi' = \int_V U'_0 \, dV + \emptyset \qquad (2)$$

De même, la résolution de ce système revient à trouver le champ de déformation minimisant π' ($\delta\pi' = 0$).

Supposons qu'on ait ($\delta\pi = \delta\pi'$). Dans ce cas, les deux systèmes sont dits <u>équivalents</u>. Ils admettent le même champ de déformation. En effet, tout champ de déformation minimisant π minimise aussi π' ($\delta\pi = \delta\pi' = 0$).

Il y a, en particulier, équivalence si les deux densités d'énergie de déformation sont égales ($U_0 = U'_0$) ou si elles diffèrent d'une certaine quantité u_o ($U_o = U'_o + u_o$) telle que l'on ait

$$\delta \int_V u_o \, dV = 0 \tag{3}$$

Dans ce cas on a aussi ($\delta\pi = \delta\pi'$).

II) PROBLEMES DE CONTRAINTES PLANES

Soit une plaque mince chargée dans son plan moyen (x_1, x_2) et d'épaisseur h (fig.1). Le tenseur de déformation e_{ij} est donné en fonction des composantes u_1 et u_2 du déplacement u par la relation :

$$e_{ij} = \frac{1}{2} \left(\frac{\delta u_i}{\delta x_j} + \frac{\delta u_j}{\delta x_i} \right) \tag{4}$$

La densité d'énergie de déformation U_o par unité de surface s'écrit, d'après la théorie de l'élasticité :

$$U_o = \frac{Eh}{2(1-\nu^2)} \left[(e_{11})^2 + (e_{22})^2 + 2\nu e_{11} e_{22} + 2(1-\nu)(e_{12})^2 \right] \tag{5}$$

On peut trouver des corps équivalents formés par des modèles élémentaires supposés suffisamment petits pour que le tenseur e_{ij} reste uniforme au sein de chacun d'eux. Les modèles envisagés sont constitués par des éléments travaillant à la flexion composée. Calculons leur énergie de déformation.

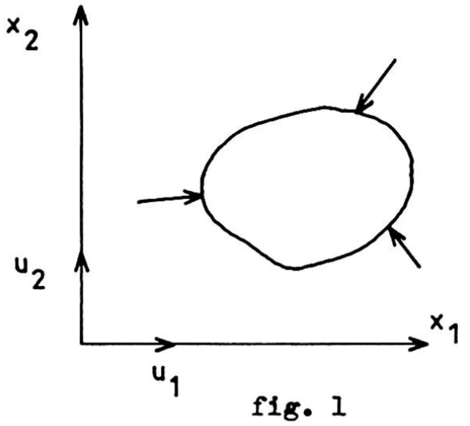

fig. 1

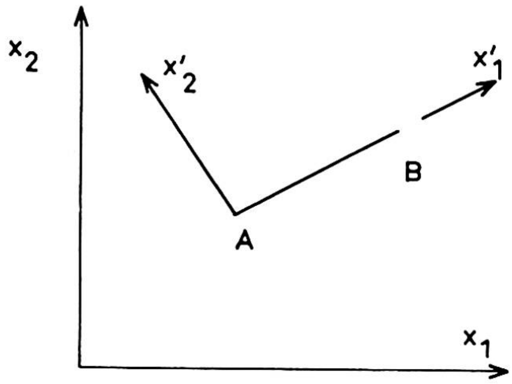

fig. 2

Soit une barre AB de faible longueur l. On lui associe un système de référence propre x'_k tel que x'_1 soit dirigé suivant AB (fig.2). Désignons par u'_k la composante de $\vec{u}$ suivant x'_k et par α_i les cosinus directeurs de x'_1 (ou de AB). On a alors :

$$u'_1 = \alpha_i \, u_i \tag{6}$$

L'allongement unitaire ε de AB s'écrit donc :

$$\varepsilon = \frac{\delta u'_1}{\delta x'_1} = \alpha_i \frac{\delta u_i}{\delta x_j} \frac{\delta x'_j}{\delta x'_1} = \alpha_i \alpha_j \frac{\delta u_i}{\delta x_j} = \alpha_i \alpha_j \, e_{ij} \tag{7}$$

De même, en désignant par β_i les cosinus directeurs de x'_2, on a :

$$u'_2 = \beta_i \, u_i \tag{8}$$

$$\frac{\delta u'_2}{\delta x'_1} = \beta_i \frac{\delta u_i}{\delta x_j} \frac{\delta x_j}{\delta x'_1} = \beta_i \alpha_j \frac{\delta u_i}{\delta x_j} \qquad (9)$$

L'effort normal N dans AB est évidemment

$$N = E\,S\,.\,\varepsilon = ES.\alpha_i\,\alpha_j\,e_{ij} \qquad (10)$$

et l'énergie de déformation correspondante est donc :

$$W' = \frac{1}{2} N\,.\,\varepsilon l = \frac{1}{2} E\,S\,l\,\varepsilon^2 = \frac{1}{2}\rho(\alpha_i\alpha_j\,e_{ij})^2 \qquad (\rho = E.S.l) \quad (11)$$

Supposons qu'on applique à l'extrémité libre B une force transversale F et que l'extrémité A subisse une certaine rotation θ (fig.3). La longueur l étant petite, le déplacement BB' s'écrit, compte tenu de la relation (11)

$$BB' = \Delta + l\,\theta = \frac{\delta u'_2}{\delta x'_1}\,l = \beta_i\alpha_j \frac{\delta u_i}{\delta x_j}\,l \qquad (12)$$

D'après les lois de la résistance des matériaux, on a

$$F = \frac{3EI}{l^3}\,\Delta \qquad (13)$$

L'énergie de déformation correspondante s'écrit :

$$W'' = \frac{1}{2} F.\Delta = \frac{3}{2}\,\frac{EI}{l}\,\left(\frac{\Delta}{l}\right)^2 = \frac{1}{2}\,\eta\left(\frac{\Delta}{l}\right)^2 \qquad \left(\eta = 3\,\frac{EI}{l}\right) \quad (14)$$

Si au noeud A, supposé rigide, aboutissent divers éléments (AB, AC, .. AN), l'équilibre des moments pris par rapport à A donne (fig.3)

$$(l.F)_{AB} + (l.F)_{AC} + \ldots. + (l.F)_{AN} = 0 \qquad (15)$$

En combinant les diverses équations (12) à (15), écrites pour l'ensemble des éléments (AB, AC, ..., AN), il est possible d'éliminer la rotation θ du noeud A et d'avoir l'expression (14) de W" en fonction des $\frac{\delta u_i}{\delta x_j}$ uniquement.

$$W'' = \frac{1}{2}\,\eta\left(\frac{\Delta}{l}\right)^2 = f\left(\frac{\delta u_i}{\delta x_j}\right) \qquad (16)$$

Divers modèles équivalents sont possibles. Examinons en particulier les modèles rectangulaires et en losange.

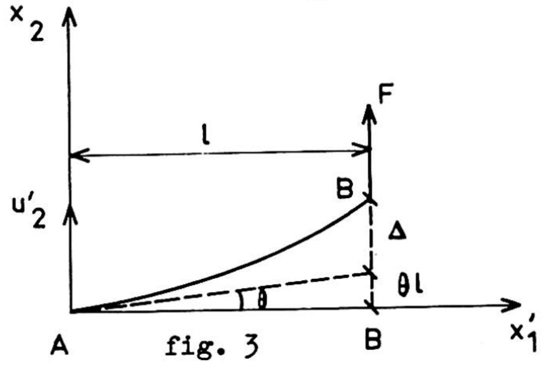

fig. 3

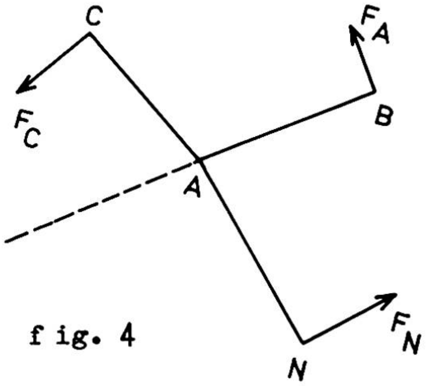

fig. 4

a) <u>Modèle rectangulaire</u> : Les noeuds situés aux sommets du rectangle ABCD sont articulés et le centre H est un noeud rigide (fig.5). En conséquence, les barres situées sur le contour ABCD travaillent uniquement à la compression. Par contre, les éléments diagonaux (HA, HB, HC et HD) travaillent à la flexion composée.

Désignons par A l'élément de surface délimité par le contour ABCD et par W l'énergie de déformation susceptible d'emmagasiner. Exprimons la condition d'équivalence en écrivant l'égalité des énergies de déformation. On a des équations (5), (11) et (14) :

$$W = AU_0 = W'_{AB} + W'_{BC} + W'_{CD} + W'_{DA} + W''_{HA} + W''_{HB} + W''_{HC} + W''_{HD} \tag{17}$$

Remplaçons chaque terme par sa valeur en fonction des termes $\frac{\delta u_i}{\delta x_j}$ ou des e_{ij}. On trouve par identification :

$$\left.\begin{aligned} \rho_{AB} &= \frac{Ah}{2}(2\lambda+\mu-\mu\,\mathrm{cotg}^2\alpha) & \rho_{AC} &= \frac{Ah}{2}(2\lambda+\mu-\mu\,\mathrm{tg}^2\alpha) \\ \rho_{HA} &= \frac{Ah}{4\sin^2\alpha\cos^2\alpha} & \eta_{HA} &= \frac{Ah(\mu-\lambda)}{4\sin^2\alpha\cos^2\alpha} \end{aligned}\right\} \tag{18}$$

b) <u>Modèle en losange</u> : Les quatre sommets du losange ABCD sont articulés et le centre H est, par contre, un noeud rigide (fig.6). En suivant le même raisonnement que précédemment, on trouve tout calcul fait

$$\left.\begin{aligned} \rho_{AD} &= \frac{Ah\lambda}{4\sin^2\alpha\cos^2\alpha} & \rho_{HA} &= \frac{Ah}{2}(\lambda+2\mu-\mu\,\mathrm{cotg}^2\alpha) \\ \rho_{HD} &= \frac{Ah}{2}\left[\lambda+2\mu-\lambda\,\mathrm{tg}^2\alpha\right] & \frac{\eta_{HB}\,\eta_{HD}}{\eta_{HB}+\eta_{HD}} &= \frac{Ah(\mu-\lambda)}{2} \end{aligned}\right\} \tag{19}$$

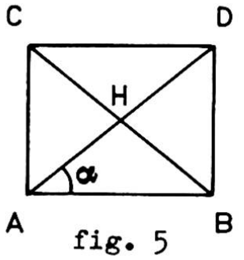

fig. 5

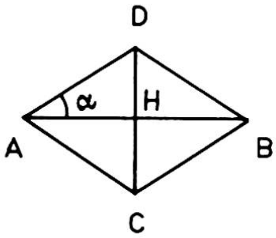

fig. 6

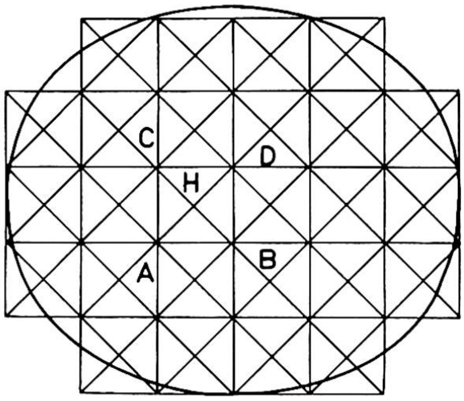

fig. 7

On voit des équations (18) et (19) que les paramètres η de divers éléments diagonaux s'annulent si $\lambda = \mu$. Dans ce cas, les modèles précités deviennent à barres articulées. On peut démontrer que cette propriété est tout à fait générale.

La figure (7) donne un schéma de maillage possible.

III) ETUDE DES DALLES.

Considérons une dalle d'épaisseur variable $h(x, y)$ dont le feuillet moyen est contenu dans le plan (x, y). L'énergie de déformation emmagasinée par unité de surface est donnée par la relation :

$$U_o = \frac{1}{2} D \left\{ \left(\frac{\delta^2 w}{\delta x^2} + \frac{\delta^2 w}{\delta y^2}\right)^2 - 2(1 - \nu) \left[\frac{\delta^2 w}{\delta x^2} \frac{\delta^2 w}{\delta x \delta y} - \left(\frac{\delta^2 w}{\delta x \delta y}\right)^2 \right] \right\} \tag{20}$$

où w est la flèche prise par la dalle et D est sa rigidité :

$$D = \frac{Eh^3}{12(1 - \nu^2)}$$

On se propose de trouver un système équivalent constitué par des poutres travaillant à la flexion et à la torsion. L'étude générale des éléments triangulaires permet d'obtenir divers types d'éléments équivalents et qui sont intéressants pour les applications.

Soit un élément triangulaire constitué par les trois poutres ij, jk, ki (fig.8). L'énergie de déformation W_{ij} emmagasinée par une de ces poutres ij s'écrit dans son système de référence propre (X, Y) :

$$W_{ij} = \frac{1}{2} (EIl)_{ij} \left(\frac{\delta^2 w}{\delta Y^2}\right)^2 + \frac{1}{2} (\mu Jl)_{ij} \left(\frac{\delta^2 w}{\delta X \delta Y}\right)^2 \tag{21}$$

où EI et μJ caractérisent respectivement les rigidités à la flexion et à la torsion des barres considérées.

On a évidemment des expressions similaires pour les deux autres poutres jk et ki. On peut réduire toutes ces expressions, par un changement de variable, au système de référence général (x, y). En désignant par A la surface du triangle ijk, l'égalité des potentiels s'écrit :

$$W_{ij} + W_{jk} + W_{ki} = AU_o \tag{22}$$

Par identification, on trouve tout calcul fait :

$$(EIl)_{ij} = A \ (1 + \nu) \frac{\cos \varphi}{\sin\alpha \sin \gamma} \tag{23}$$

$$(\mu Jl)_{ij} = (EIl)_{ij} + 4\nu A \frac{\cos 2 \varphi}{\sin 2\alpha \sin 2 \gamma} \tag{24}$$

Pour les autres poutres on obtient des expressions similaires.

a) Approximation de la théorie des dalles : La théorie classique des dalles admet que le feuillet moyen est inextensible. Ceci revient à considérer que la courbure totale de la déformée de ce feuillet, qui se produit par flexion après chargement, reste quasi nulle. Par conséquent :

$$u_o = \frac{D}{2} \left[\frac{\delta^2 w}{\delta x^2} \cdot \frac{\delta^2 w}{\delta y^2} - \left(\frac{\delta^2 w}{\delta x \delta y}\right)^2 \right] \not\approx 0 \tag{25}$$

La densité d'énergie de déformation U'_o dans le corps équivalent peut donc s'écrire :

$$U'_o = U_o \neq \frac{D}{2}\left[\left(\frac{\delta^2 w}{\delta x^2}\right)^2 + \left(\frac{\delta^2 w}{\delta y^2}\right)^2 + 2\left(\frac{\delta^2 w}{\delta x \, \delta y}\right)^2 \right] \qquad (26)$$

L'expression précitée de U'_o devient une égalité parfaite si le coefficient de Poisson est nul ou si la dalle est parfaitement encastrée sur son contour. En effet, S étant la surface de la dalle, on peut démontrer aisément que dans ce dernier cas

$$\delta \int_S u_o \, dS = 0 \qquad (27)$$

Ce développement est à rapprocher de l'équation (3).

b) <u>Modèle équivalent formé par un grillage orthogonal de poutres</u> :

En adoptant l'expression (26) de U'_o, on peut obtenir un modèle équivalent constitué par un grillage orthogonal de poutres dont l'utilisation est très commode dans la pratique (fig.9). Déterminons les caractéristiques des divers éléments (ij, kl, ...). Par analogie avec l'équation (22), on a :

$$AU'_o = W_{ij} + W_{kl} \qquad (28)$$

où A est la surface du rectangle (ij x kl). En remplaçant chaque terme par sa valeur, on obtient :

$$AD\left[\left(\frac{\delta^2 w}{\delta x^2}\right)^2 + \left(\frac{\delta^2 w}{\delta y^2}\right)^2 + 2\left(\frac{\delta^2 w}{\delta x \, \delta y}\right)^2 \right] =$$

$$(EIl)_{ij} \left(\frac{\delta^2 w}{\delta x^2}\right)^2 + \left[(\mu Jl)_{ij} + (\mu Jl)_{kl} \right] \left(\frac{\delta^2 w}{\delta x \delta y}\right)^2 + (EIl)_{kl} \left(\frac{\delta^2 w}{\delta y^2}\right)^2$$

Par identification on trouve immédiatement

$$(EIl)_{ij} = (EIl)_{kl} = \frac{1}{2}\left[(\mu Jl)_{ij} + (\mu Jl)_{kl} \right] = AD \qquad (29)$$

On voit que les rigidités à la flexion du modèle équivalent sont bien définies. Par contre, les rigidités à la torsion sont arbitraires pourvu que la condition (29) soit réalisée.

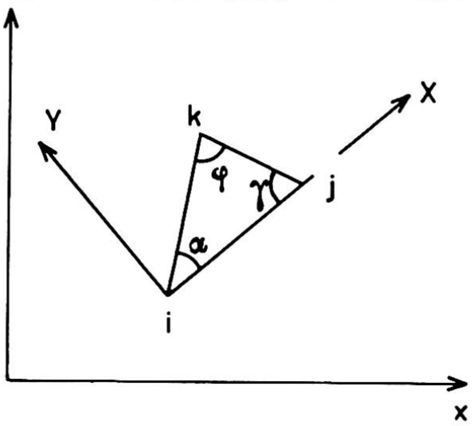

fig.8

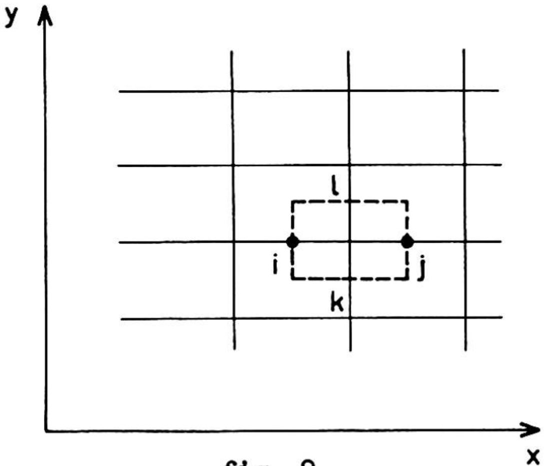

fig. 9

IV) COQUES ET VOILES MINCES.

La densité d'énergie de déformation d'une coque par unité de surface s'obtient en additionnant celle d'une plaque, équation (5), avec celle d'une dalle, équation (20). En conséquence, tout type de modèle équivalent valable simultanément pour un problème d'élasticité plane et de dalle l'est aussi pour les coques et les voiles minces.

V) COMPATIBILITE DES MODELES.

La juxtaposition de divers modèles autour d'un même noeud courant, ou sur le contour, doit satisfaire en ce noeud aux conditions d'équilibre exprimées en fonction du tenseur de déformation e_{ij}. Si ces conditions ne sont pas réalisées, les modèles sont dits incompatibles entre eux.

VI) APPLICATIONS.

La Direction des Ouvrages d'Art de la S.N.C.F. a confié au C.E.B.T.P. l'étude théorique et expérimentale des deux ponts-dalles :

- Passage Supérieur de Saint-Pol-sur-Ternoise
- Passage Inférieur de Hautepierre

Les figures (10-a, b, c et 11-a, b, c) illustrent les résultats numériques donnés par la Théorie des équivalences et ceux obtenus par mesures sur modèles réduits dans le cas d'un chargement uniforme.

VII) CONCLUSIONS.

La théorie que nous proposons constitue une approche générale pouvant servir de cadre à diverses méthodes particulières et justifier parfois leur utilisation. Son application nous a permis de retrouver, d'une manière relativement simple, certains résultats déjà connus et d'en établir de nouveaux. Le traitement d'un corps continu par équivalence avec un corps discret est très intéressant dans la pratique. C'est un procédé d'une utilisation aisée et qui s'adapte parfaitement aux ordinateurs.

Il est à noter que cette théorie reste valable que le milieu soit isotrope ou anisotrope, à comportement linéaire ou non linéaire.

RESUME

Dans la présente étude nous examinons les fondements de la Théorie des équivalences ainsi que son application aux problèmes de contraintes planes, de dalles ou de coques. Une étude comparative est donnée concernant deux ponts-dalles.

P.S. Il est à rappeler que les coefficients d'élasticité E et de Poisson sont reliés aux coefficients de Lamé λ et μ par les relations :

$$\lambda = \frac{\nu E}{(1 - 2\nu)(1+\nu)} \qquad \mu = \frac{E}{2(1 + \nu)}$$

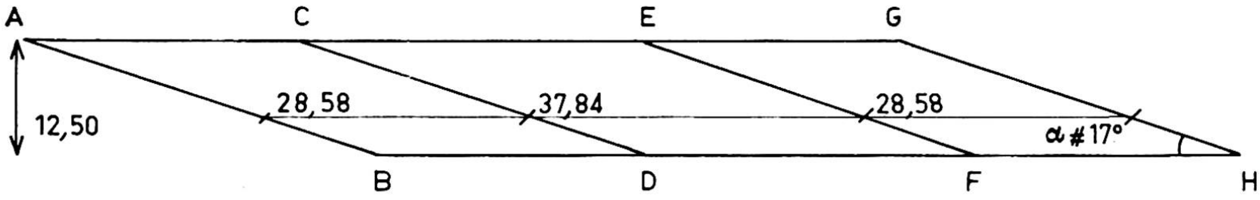

fig. 10a – Pas. sup. Saint Pol sur Ternoise.

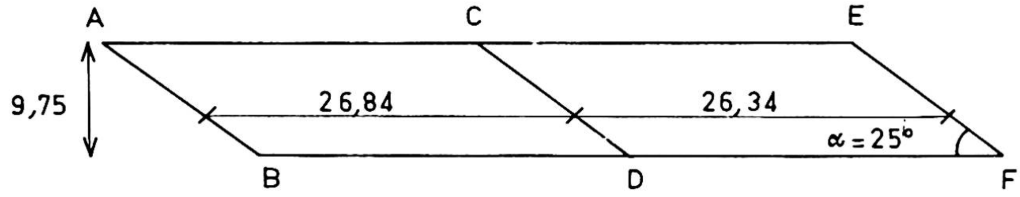

fig. 11a – Pas. sup. Hautepierre.

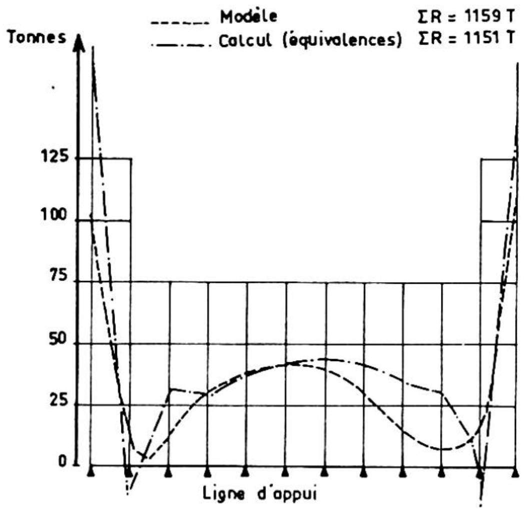

fig. 10 b – P. S. St. Pol sur Ternoise
Réactions des appuis intermédiaires.

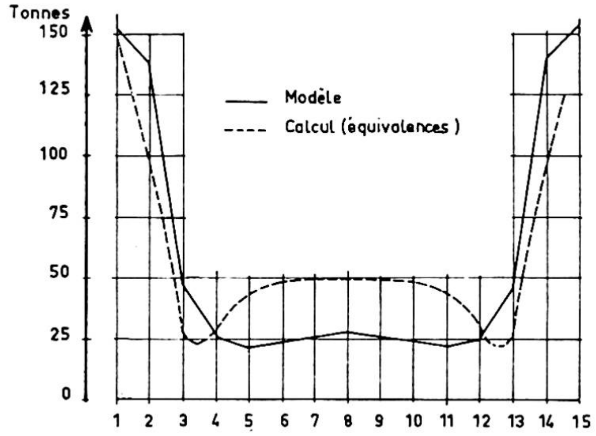

fig. 11 b – P.I. Hautepierre
Réaction de l'appui CD

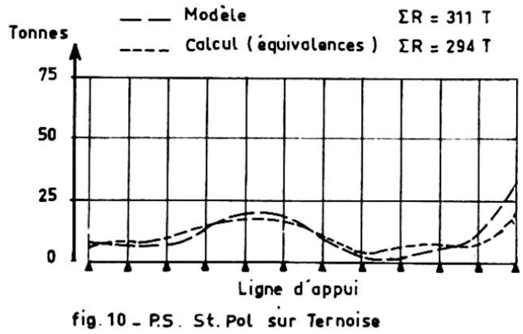

fig. 10 – P.S. St. Pol sur Ternoise
Réactions des appuis extrêmes AB.

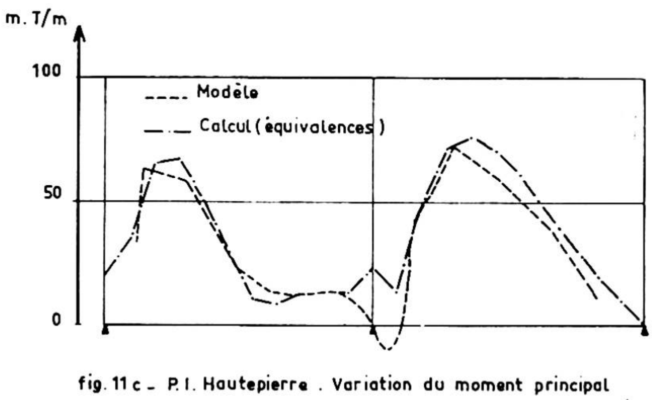

fig. 11 c – P.I. Hautepierre. Variation du moment principal maximal parallèlement sur CD.

Minimum Weight Design of Frameworks

Projet de minimalisation de poids de charpentes

Entwurf einer Gewichtsminimalisierung bei Fachwerken

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1. Introduction

Full stress design is commonly adopted for design of frameworks. For a given set of joint locations j, and a system of self equillibrating loads, a framework of most general topology can have a total number of members $m = j(j-1)/2$. This, in general will be statically indeterminate. A feasible full stress design may or may not exist for a statically indeterminate structure [1,2]. However, full stress design of an indeterminate framework is feasible through geometrically controlled prestress [3,4]. Any such fully stressed structure may have its weight less than some determinate forms but not necessarily lower than all the feasible determinate forms of the indeterminate framework [5,6].

Least weight design for a fixed load condition is known to be statically determinate [7,8,9,10,11]. Hence the problem of Minimum weight design of a framework reduces to the identification of the least weight statically determinate form, out of the several feasible determinate forms. Present methods [8,10,11] of identifying the least weight statically determinate form use, either special mathematical tools or certain theorms of plastic design. In the present work, a direct method using wellknown principles of structural analysis is proposed.

2. Full Stress Design With Prestress

An initial lack of fit in a statically indeterminate system will induce prestress in the system. The forces in a statically indeterminate framework are given by

$$\{F\} = \{F^{\circ}\} + [f] \{X\} \quad \ldots . (1)$$

in which $\{F\}$ = element force vector; $\{F^{\circ}\}$ = element force vector of a determinate structure; $[f]$ = influence coefficient matrix associated with redundant forces; and $\{X\}$ = vector of redundant member forces.

If the members are to develop preassigned stresses at working load, the compatibility of deformations requires

$$\{f\}^T [L] \{\sigma\} + E \{\lambda\} = 0 \qquad \ldots . (2)$$

in which $[L]$ = diagonal matrix of member lengths; $\{\sigma\}$ = vector of member stresses; E = youngs modulus; and $\{\lambda\}$ = vector of initial lack of fit in redundant members. Preassigned stresses at working load of a statically admissible force system, automatically fixes the sizes of the members of the framework. It is seen from Eq. (2) that for a given set of member stresses, $\{\lambda\}$ is unique. Thus the member forces in no load condition (only prestress condition) are fixed and can be determined. The prestress in any member is given by

$$\sigma_{Pi} = \frac{F_{Pi}}{a_i} \qquad \ldots . \quad (3)$$

in which σ_{Pi} = prestress in the ith member due to initial lack of fit; F_{Pi} = force in the ith member in prestress condition; and a_i = cross sectional area of the ith member.

The design will be an acceptable one, if the stresses under no load condition are also within the allowable limits i.e.

$$\sigma_{Pi} \leqslant \sigma_{ai} \qquad (i = 1, 2, \ldots, m) \qquad \ldots (4)$$

in which σ_{ai} = permissible stress in ith member.

3. Optimal Statically Determinate Form

Prestress changes the datum level of a member capacity and in the presence of prestress the effective capacity of the member is

$$F_{ei} = F_i - F_{Pi} \qquad \ldots .. \quad (5)$$

in which F_{ei} = effective capacity of the ith member at working load; and F_i = capacity of the ith member at full stress. The effective capacity of a member is increased by the presence of a compensatory type of initial prestress. Therefore, the efficiency of a member in transferring the external loads to the supports may be represented by a nondimensional factor ρ_i given by

$$\rho_i = \frac{F_{ei}}{F_i} = 1 - \frac{F_{Pi}}{F_i} \qquad \ldots .. (6)$$

in which ρ_i is defined as efficiency factor for the ith member. The efficiency factor can be as high as 2 for equal permissible stresses in tension and compression. The position of the member in the framework and the loading system are reflected in the efficiency factor. Therefore, the efficiency factor ρ is a direct indication of the effective utility of a member for the particular load condition. The optimal statically determinate form can be obtained by eliminating the required members of least efficiency. Application of this simple logic is illustrated through examples.

4. Illustrative Examples

Example 1: It is required to find the optimal statically determinate form corresponding to an indeterminate framework shown in Fig. 1(a). The detailed calculations for member efficiency factors are given in Table 1. Fig. 1(b) shows the variation of the volume (or weight) of truss for full stress design, with respect to force in redundant member at working load. The choice of the redundant member has no effect on the nature of this curve. The dotted lines in Fig. 1(b) indicate that the prestress in some members exceed the allowable values. The Kink points correspond to statically determinate forms of the system. The optimal topology will not get affected by adopting different allowable stresses in tension and compression. The optimal statically determinate form (point b) is obtained by removing the least efficient member 4.

Table 1. - Computations of Example 1

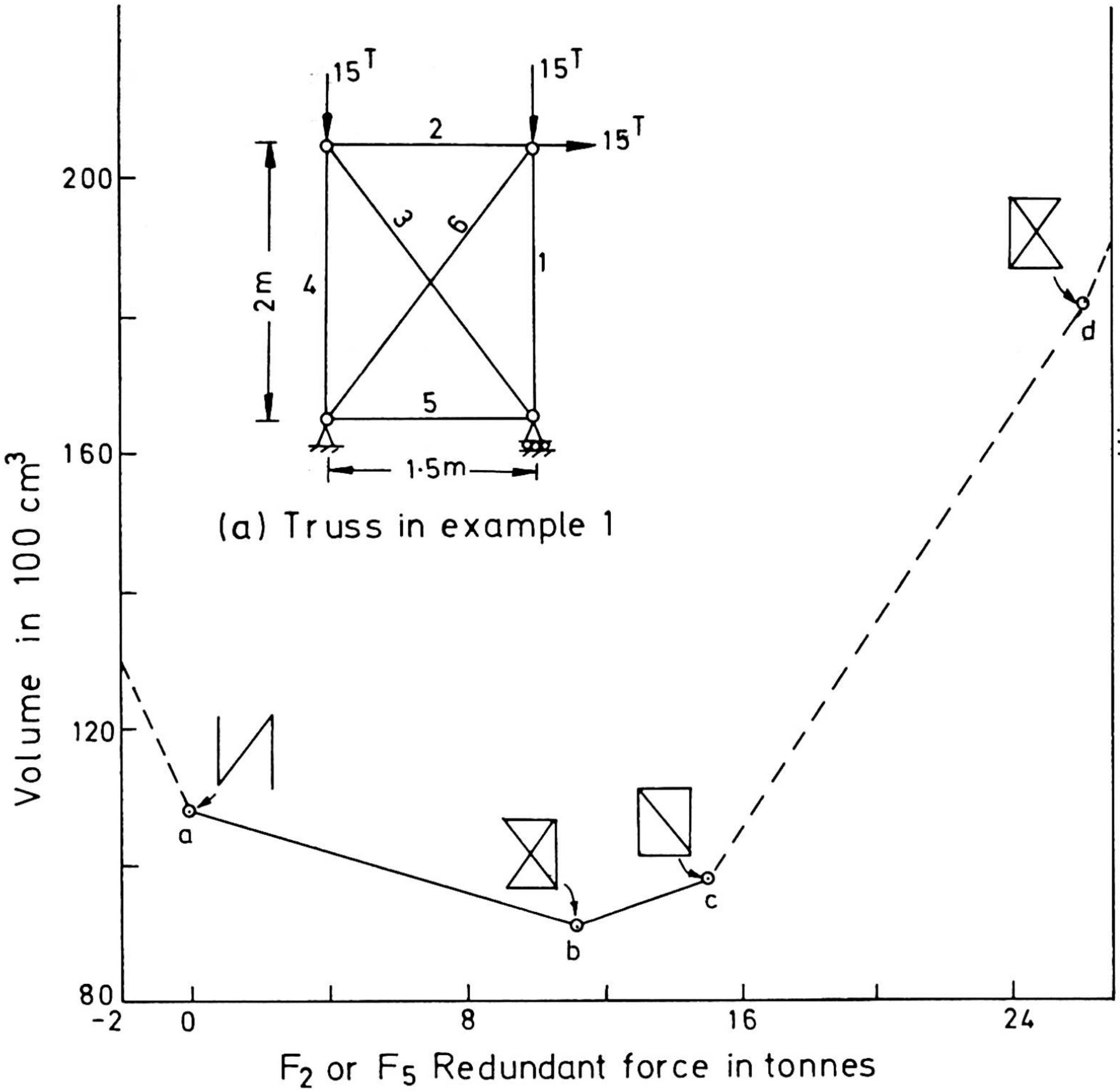
Member	F_i°	f_{i1}	F_i	a_i	F_{Pi}	σ_{Pi}	σ_{ai}	ρ_i
1	2	3	4	5	6	7	8	9
1	-15.0	-0.80	-27.00	18.00	-1.354	-0.075	-1.50	0.950
2	15.0	-0.60	6.00	4.00	-1.016	-0.254	-1.50	1.169
3	-25.0	1.00	-10.00	6.67	1.693	0.254	1.50	1.169
4	5.0	-0.80	-7.00	4.67	-1.354	-0.290	-1.50	0.806*
5	15.0	-0.60	6.00	4.00	-1.016	-0.254	-1.50	1.169
6		1.00	15.00	10.00	1.693	0.169	1.50	0.887

F_i° = force in ith member due to external loading when the redundant members are removed; f_{ij} = force in ith member due to unit tensile force in the jth redundant member when the other redundant members and external loads are removed.

Units: Force in Tonnes; area in sq. cm.; and stress in Tonnes per sq. cm.

* Least efficient member.

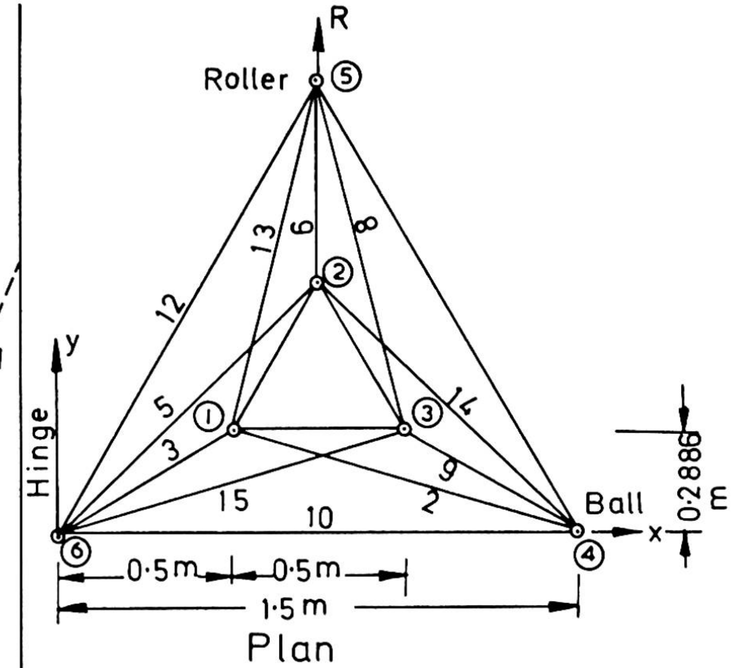
Example 2: It is required to find the optimum design of the space framework loaded as shown in Fig. 2. The calculations for the efficiency factors are listed in Table 2.



(a) Truss in example 1

(b) Weight of truss Vs redundant force

FIG. 1 ILLUSTRATIVE EXAMPLE 1



Joint i	Loading in tonnes P_x	P_y	P_z
1	0	0	-10
2	2	0	-10
3	0	0	-10

Nodes 1,2,3 are at a height of 2·5 m above the plane 4,5,6

FIG. 2 SPACE TRUSS-EXAMPLE-2

Table 2. - Computations of Example 2

Member	F^o_i	F_i	a_i	F_{Pi}	σ_{Pi}	σ_{ai}	ρ_i
1	2	3	4	5	6	7	8
1	-1.33	-0.22	0.150	0.063	0.419	1.50	1.279
2	0.00	-2.00	1.333	-0.114	-0.085	-1.50	0.943
3	-10.26	-4.58	3.052	0.222	0.072	1.50	1.048
4	-1.33	0.88	0.588	0.066	0.112	1.50	0.926*
5	3.61	-0.39	0.260	-0.118	-0.455	-1.50	0.696*
6	-13.68	-6.10	4.069	0.166	0.041	1.50	1.027
7	-3.33	-1.12	0.745	0.031	0.042	1.50	1.028
8	3.61	-0.39	0.259	-0.056	-0.217	-1.50	0.855*
9	-13.68	-8.00	5.333	0.161	0.030	1.50	1.020
10	1.78	2.15	1.432	0.021	0.015	1.50	0.990
11	1.78	2.52	1.677	0.010	0.006	1.50	0.996
12	2.44	3.18	2.122	0.022	0.010	1.50	0.993
13	..	-4.0	2.667	-0.118	-0.044	-1.50	0.970
14	..	-4.0	2.667	-0.056	-0.021	-1.50	0.986
15	..	-2.0	1.333	-0.114	-0.085	-1.50	0.943

Units: Force in Tonnes; area in sq. cm.; and stress in Tonnes per sq. cm.

* Least efficient members for elimination.

Removal of the least efficient members 5, 8 and 4 will result in a statically unstable situation. So only members 5 and 8 are omitted in the first instance. The method is repeated with the reduced framework having one order indeterminacy. The calculations for member efficiency factors are not listed here. The optimal statically determinate form obtained by removing the members 2 and 15 having the same lowest efficiency factor is shown in Fig. 3(a).

5. Conclusions

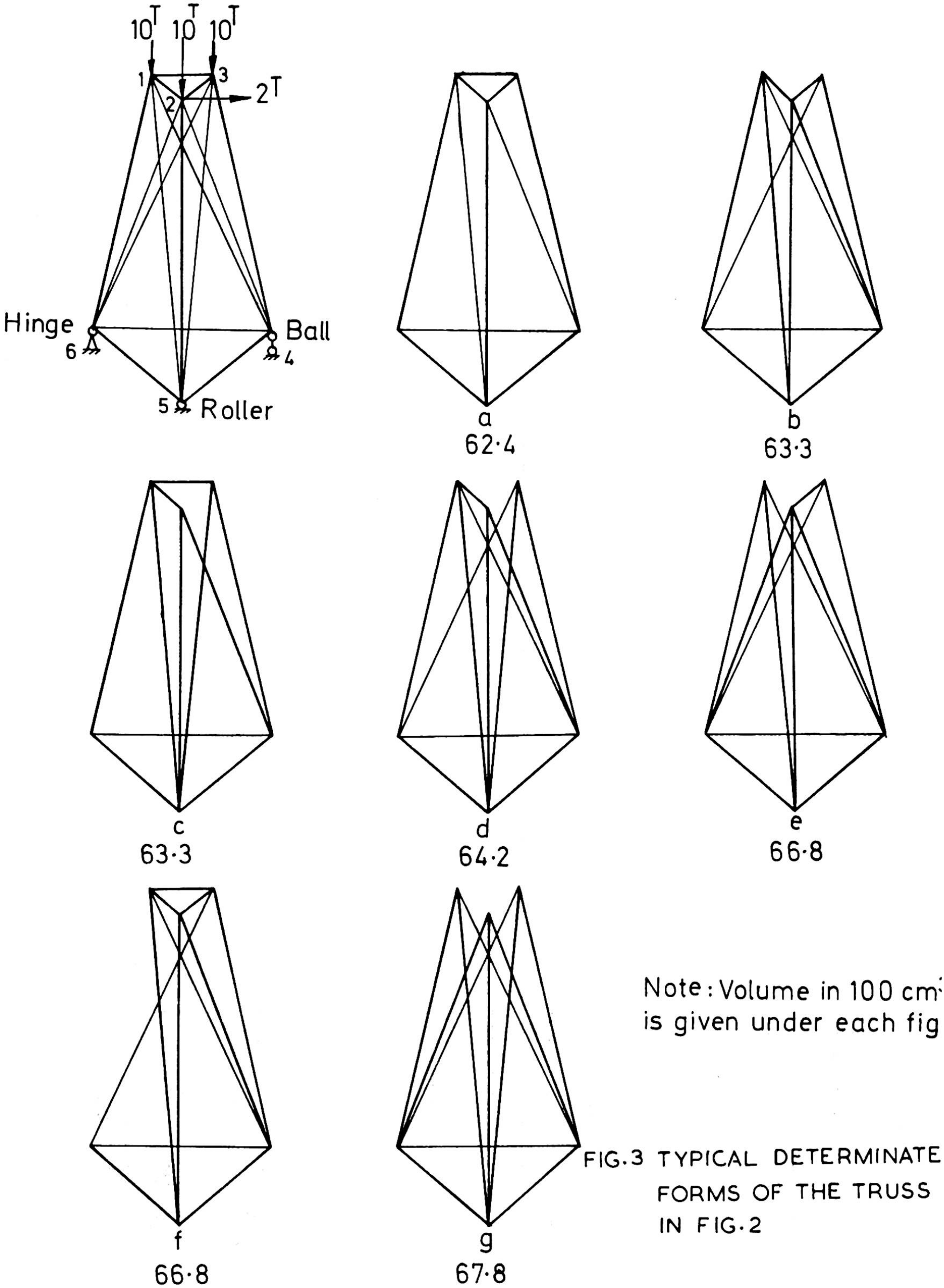
1. The optimal statically determinate form can be obtained using the following sequence of operations:

   (a) If the order of indeterminacy of the framework is n, then n members having the lowest efficiency must be eliminated

   (b) If two or more members have the same efficiency factor, they must be treated as a unit in the member elimination process. This may lead to the elimination of a joint.

   (c) If the operation (a) leads to a statically unstable system, remove less than n members so that the resulting system is stable.

   (d) As a consequence of (b) and (c) some times the reduced framework obtained will be indeterminate of reduced order;



Note: Volume in 100 cm
is given under each fig

FIG.3 TYPICAL DETERMINATE FORMS OF THE TRUSS IN FIG.2

in which case the method is to be repeated starting with the reduced framework.

2. The process of eliminating members of least efficiency to get the optimal statically determinate form corresponding to an indeterminate framework was applied to several other examples successfully.

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Summary

The problem of finding the minimum weight topological form for a given loading from a general configuration which includes all possible member locations with respect to the given joint locations, is solved by using the concept of efficiency of a member. Relative efficiency factors of members are generated by introducing virtual prestress into the system. The optimal topology which is statically determinate is obtained by eliminating members with least efficiency.

Some Results in the Optimization of Tall Building Systems

Quelques résultats dans l'optimisation des systèmes pour bâtiments élevés

Einige Resultate in der Systemoptimalisierung von Hochhäusern

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INTRODUCTION

With the advent of high-speed digital computers, a significant amount of work has been accomplished in structural optimization [1] [2] [3] [4] [5] [6]. Presently available techniques enable the economical proportions or other parameters of structural systems with specified topology to be determined for buildings of moderate size and complexity under given loading schemes.

However, at the next higher level of building optimization, namely the selection of an overall optimal building system from among many candidate systems and building topologies, much remains to be done. The paper, which is an attempt in this direction, describes an approach to the optimization of topology and structural systems for tall buildings and focuses on obtaining realistic trends for possible use in design practice.

THE PROBLEM

This study is concerned with frameworks that are, or can be considered in groups of units that are, rectangular in plan. It is assumed that the usable space at each level is constant.

The structural system consists of the following component sub-systems: 1) floors, 2) framing for gravity loads, 3) framing for wind loads and 4) cladding.

1. *Floor Systems.* Candidate floor systems [7] in Fig. 1 are used in the study and are designed for a live load of 70 psf. An additional one way concrete slab derived from CRSI designs [8] is also included.

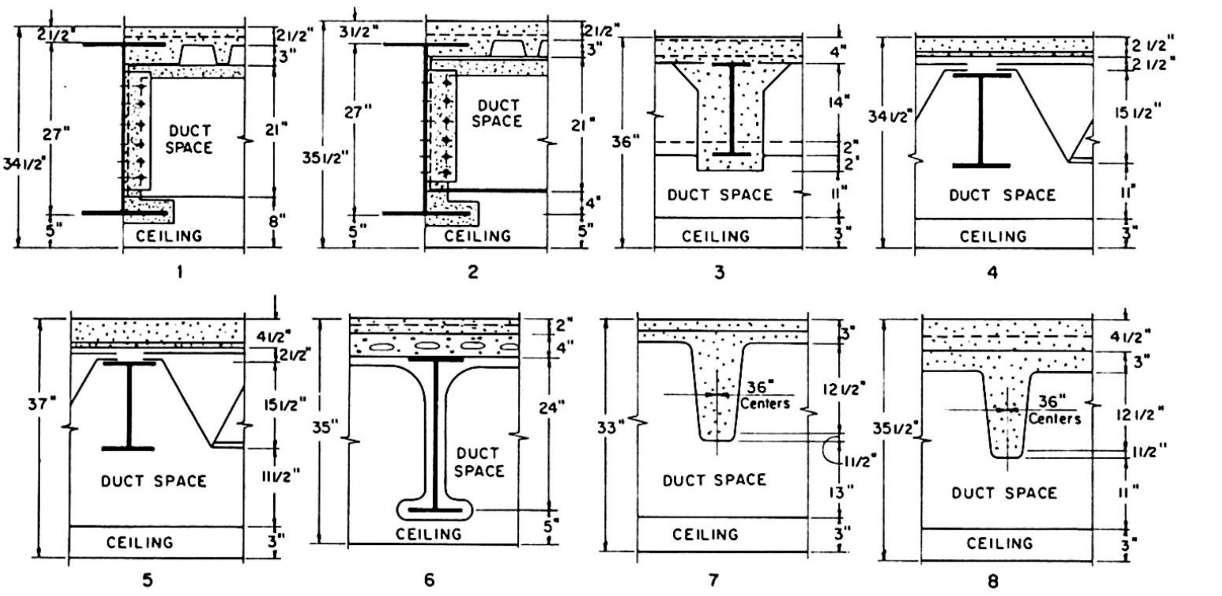

Fig. 1 Candidate floor systems (after [7])

2. *Framing for Gravity Loads* consists of beams and columns of either structural steel or reinforced concrete and their combinations. Their design conforms to current practice, i.e. the elastic design based on AISC 1963 [9] is used for structural steel and the ultimate strength design based on ACI Code 318-63 [10] is used for reinforced concrete. Pertinent design parameters include $f_s = 36$ ksi, $f_y = 60$ ksi, $f'_c = 4$ ksi, $E_s = 29{,}000$ ksi and $E_c = 3{,}000$ ksi.

3. *Framing for Wind Loads.* Three types of framing are considered: a) *conventional unbraced frames*, rigidly connected in both directions; b) *tube unbraced frames* with moment resisting exterior frames; and c) *braced frames* having two lines of cross-bracing in each direction. Drift limitation is taken as H/200, where H is the total height of the building.

4. *Cladding.* Two types of exterior walls are considered, i.e. metal curtain wall and masonry, for which average unit costs and weights are determined.

Regression analysis of the available floor data enables functional relations for the unit weight, W_i, and cost, C_i, of the typical floor systems to be evaluated in terms of the bay dimensions B and D. Thus, for a typical floor system:

$$W_i = f_i(B,D) \qquad C_i = g_i(W_i) \tag{1}$$

A regression analysis for the weight and cost of the *structural frame* is not possible and initial designs are necessary. Columns are assumed in a regular pattern, with spacings of B and D in the width, W, and length, L, directions, respectively. Columns and girders are designed at control sections of H/4 for dead and wind load combinations. The column properties are assumed to vary linearly between the quarter points.

For such a variety of systems and dimensions to choose from, a means of identification is required: systems that transmit vertical load to the frame (floor and wall systems) are identified by their unit weight, W_i, while structural frames, columns and girders are identified by their cross-sectional area, A_i.

With the above premises, the optimization problem can be stated as follows: *Given*: a required floor area of a building, a set of candidate structural components characterized by an appropriate parameter (W_i or A_i), a set of design rules and pertinent cost data, *find*: the structural topology (bay width and length and building height), the structural framing, type and material, and the system components, *such that*: the total cost of the structure is a minimum and *subject to*: all functional requirements of the current codes of practice and appropriate dimensional constraints.

THE MATHEMATICAL MODEL

The mathematical model used in the optimization process is illustrated by the flow diagram in Fig. 2. The program consists of four main operations: 1) a preliminary design of the column and girders on the basis of vertical loads is first performed; 2) this design is then revised with regard to the wind forces; 3) drift requirements are next satisfied and finally 4) a system optimization is performed to select systems from among the competing alternatives. Results of the optimization then become feedback input for the next iteration.

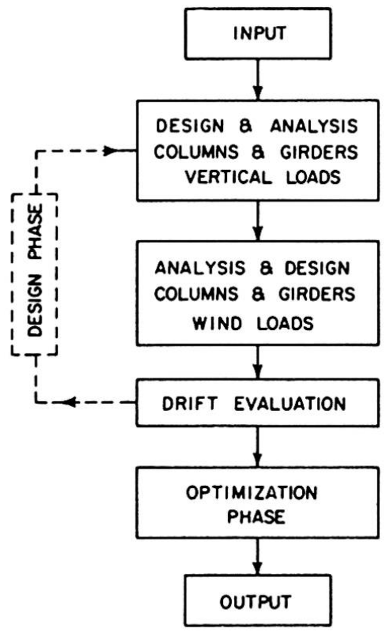

Fig. 2 - General Flow Diagram

Preliminary Design. Vertical loads are assembled from the data on the various individual floor systems, the wall system, and the spandrels. Live loads are uniformly applied in all bays. Axial forces in the columns for these loadings are determined at the quarter heights, neglecting all effects of continuity. The effect of the column and girder weights is added to these thrusts.

Typically, the weight of a composite floor system may be represented as

$$W_i' = \Sigma R_i W_i \tag{2}$$

where W_i, the weight of a composite floor system, consists of contributions from all the compônent floor systems, W_i, and their corresponding participation ratios, R_j.

The participation ratio, R_i, defines the relative contribution of a structural component to the total configuration. While $0 \leqslant R_i \leqslant 1$ is a continuous variable, during the process of optimization it converges to either 0 or to 1, indicating that component i does or does not participate in the optimal solution.

Wind Analysis. The following standard assumptions are adopted: a) points of inflection occur at mid point of columns and girders; b) at any elevation the vertical wind shear at all columns is identical and c) the increment of wind between mid story heights at any level is neglected. Wind thrusts in the columns are computed, assuming that the structure behaves as a càntilever beam. Column bending is computed assuming "portal action". Girders and columns are proportioned in accordance with the pertinent ACI and AISC codes, using the story height as an effective length.

Drift Analysis. The total drift is approximated by the sum of its components, the cantilever and portal drifts.

The cantilever drift is calculated by:

$$\Delta_c = SH^3/6(REI)_t \tag{3}$$

where S is the total wind force on the building, H is the building height and $(REI)_t$ is the transformed stiffness at the building base, allowing for the participating ratios of the component elements.

The portal drift in the direction B over the height, h, of the story n is derived from:

$$\Delta_n = [h^3/\Sigma(REI)_c + h^2 B/\Sigma(REI)_g]\, S_n/12 \tag{4}$$

where S_n is the total shear at the level of the n^{-th} story, indices g and c refer to girders and columns, respectively, and the summations extend over elements of the n^{-th} story. A similar expression applies to the drift in the direction D by replacing B with D in eq. (4).

The value of Δ_n is computed at each quarter point of the building height. A linear variation of the relative drifts is assumed between quarter points and the drift at the top is assumed to be twice the value at the first quarter-point from the top. Then the portal drift, Δ_p, can be expressed in terms of the quarter-points drifts, Δ_1, Δ_2, Δ_3, and Δ_4, as:

$$\Delta_p = (2\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4/2)n/4 \tag{5}$$

The total drift ,Δ, becomes:

$$\Delta = \Delta_c + \Delta_p \tag{6}$$

If the drift limitation (H/200) is exceeded, a subroutine is provided to increment columns and girders in an optimal way. The sequence of column and girder design and drift analysis is repeated in a sufficient number of iterations.

Optimization. In this study, the total structural cost, C, is chosen as the merit function to be minimized. The merit function is expressed as:

$$C = \Sigma R_i W_i C_i \tag{7}$$

where W_i is the weight (area) parameter of the i-th component and R_i and C_i are its corresponding participation ratio and cost, respectively. All costs are Pittsburgh, U.S.A., costs converted to a 1970 base.

The formulation of the optimization problem involves 117 variables, (including the bay dimensions B and D, and all candidate structural component parameters) and 202 equality and inequality constraints. The behavioural constraints represent the column design, the assemblage of floor and spandrel weights and vertical columns loads. The side constraints limit the values of participation ratios, bay dimensions and column and girder sizes. The merit function and the constraints are highly nonlinear. Search and penalty function techniques did not prove successful for the nonlinear programming problem on hand, due to the difficulty of enforcing all of the constraints in the presence of a large number of variables. Instead, a cutting plane technique [11] was adopted wherein both the merit function and constraints were linearized by a Taylor's series expansion. The resulting linearized form was solved by a linear programming technique, "Optima" [12].

SOME OPTIMIZATION RESULTS

On the basis of the mathematical model described, a large number of optimal solutions have been investigated. These fall into two groups: a) certain ratios of the building dimensions are held constant and the volume of the building is allowed to vary: b) the total floor area is held constant while the pertinent building dimensions are allowed to vary. Buildings of 12, 20 and 32 stories and with height to least plan dimension ratios of 2 or 4 are selected in the first group; whereas buildings of 50,000, 500,000 and 5,000,000 sq. ft. floor area are chosen for the second group. In all optimization studies reported, the story height is taken as 11 ft.

In general, the program tends to select reinforced concrete frames supporting steel floor systems. The cost of connecting such systems imposes constraints to overrule this combination. Masonry walls are most often chosen. Whenever only floors in ref. [7] are available, the program chooses the steel joist floor system for steel frame designs and the waffle floor for concrete frame designs. The CRSI floor systems tends to be selected when available and for minimum 15 ft. bay dimensions. On larger bay dimensions, this conclusion does not hold.

Optimal bay dimensions range between 10 and 20 ft. However, since no value is assigned to the advantage of open space, this conclusion is only of relative validity. Generally, the total structural cost increases only slightly as the bay dimensions increase from 15 ft. to 40 ft.

Reinforced concrete frames prove advantageous for bay dimensions of 15 and 40 ft., for both conventional and tube unbraced frames. Apparent advantages decrease as the bay dimension increases. On the other hand, the total structural cost appears to be relatively insensitive to the frame material, as illustrated by Figs. 3,4. These show the total cost ratio of a steel frame structure to a reinforced concrete frame structure with various floor areas for unbraced and braced frames, respectively. In general, braced frame structures are slightly more economical, in steel than in concrete, whereas for unbraced frame structures, the economics of steel over concrete depends on the building aspect ratio, W/H. Moderate changes in unit costs would, probably alter these results. However, the trend of the cost ratio (structural steel to concrete) increasing as the building becomes more slender would remain valid.

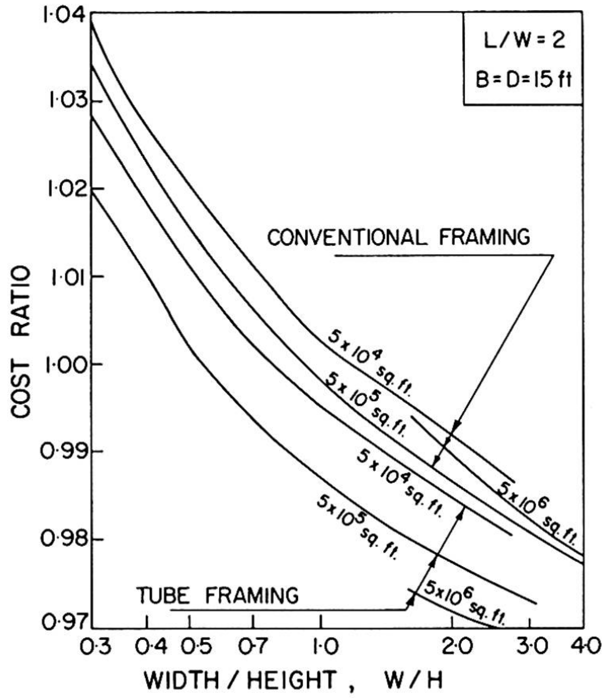

Fig. 3 - Total Cost Ratio of Steel to Reinforced Concrete Unbraced Frame Structures for Various Floor Areas

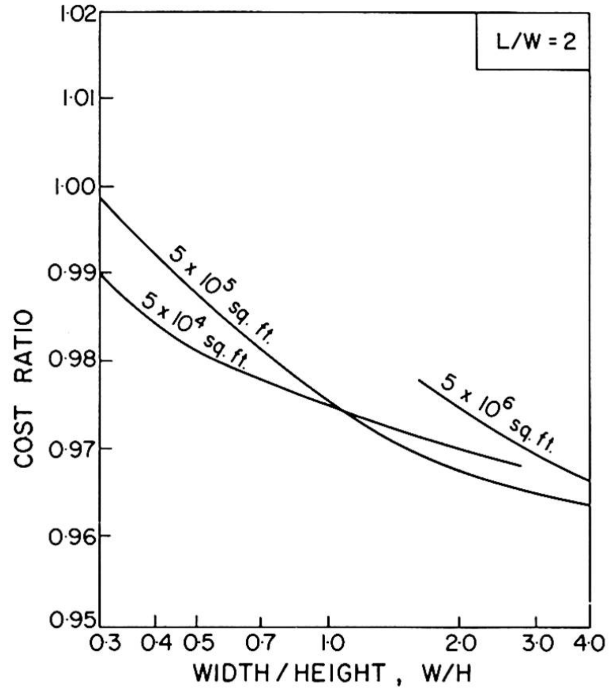

Fig. 4 - Total Cost Ratio of Steel to Reinforced Concrete Braced Frame Structures for Various Floor Area

The type of frame plays a much more significant role in the overall optimization. Unbraced and braced frames are compared in Figs. 5,6,7, for various required floor areas for bays of at least 15 x 15 sq. ft. Consistently, braced frames prove to be more economical even for low rise structures. Also it is seen that a cost premium must generally be paid for more slender buildings. From these data, one might further infer that one large structure is more economical than a series of small buildings of equivalent total area.

Certain numerical solutions of this study related to the land cost effects could prove useful in an early decision-making process. Assuming first that land costs and constraints are not considered, Figs. 5,6,7, indicate that the optimal number of stories for buildings of 50,000, 500,000, and 5,000,000 sq. ft. of required floor space is approximately four to five, six to nine, and ten to thirteen, respectively. Thus, squatty buildings are more economical where there is no relation between structure cost and the land requirements of the building.

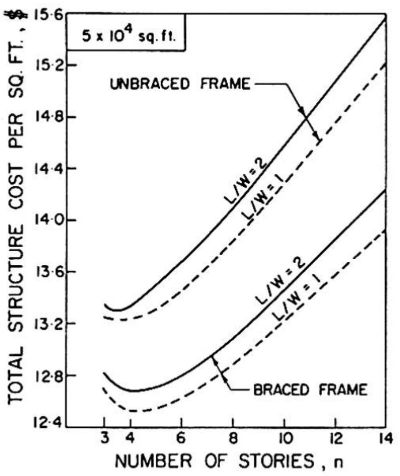

Fig. 5 - Total Structural Cost for 5×10^4 sq.ft. Total Floor Area

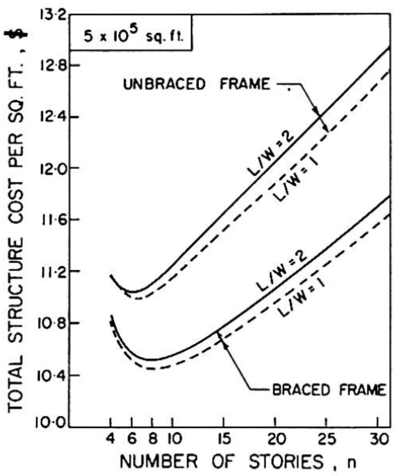

Fig. 6 - Total Structural Cost for 5×10^5 sq.ft. Total Floor Area

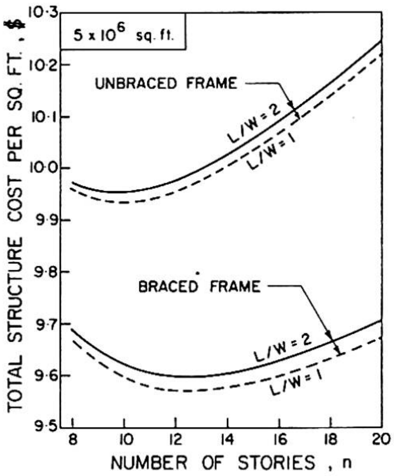

Fig. 7 - Total Structural Cost for 5×10^6 sq.ft. Total Floor Area

Alternatively, if land becomes a constraint, then plan dimensions as close as possible to the plan dimensions at optimum height produce the most economical structure. Fig. 8 shows that this condition prevails since, for constant floor area, unit cost increases with height.

When land costs play a governing role, they can significantly modify the optimum heights. For example, the effect of land costs of two, twenty and \$200/sq. ft. has been investigated. Fig. 9 shows the total costs of the structure including land cost for a 500,000 sq. ft. floor area requirement. Similar curves can be developed for other area needs. As a result, the most economical heights now increase over the no cost land heights, the increase becoming larger as land becomes more expensive. Table 1 illustrates these results.

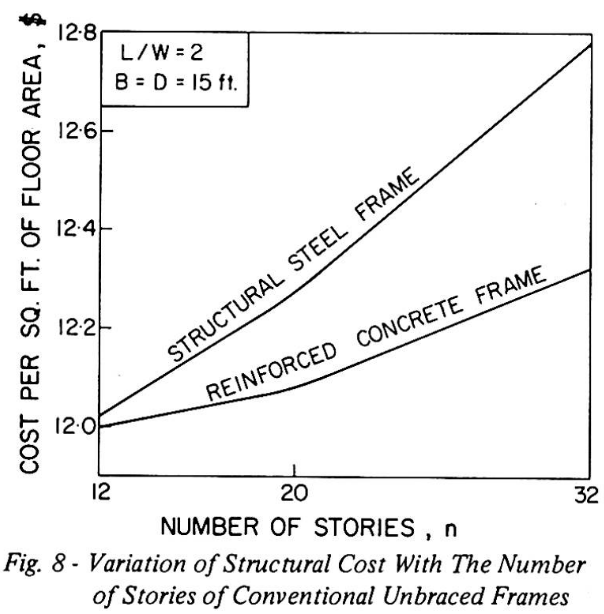

Fig. 8 - Variation of Structural Cost With The Number of Stories of Conventional Unbraced Frames

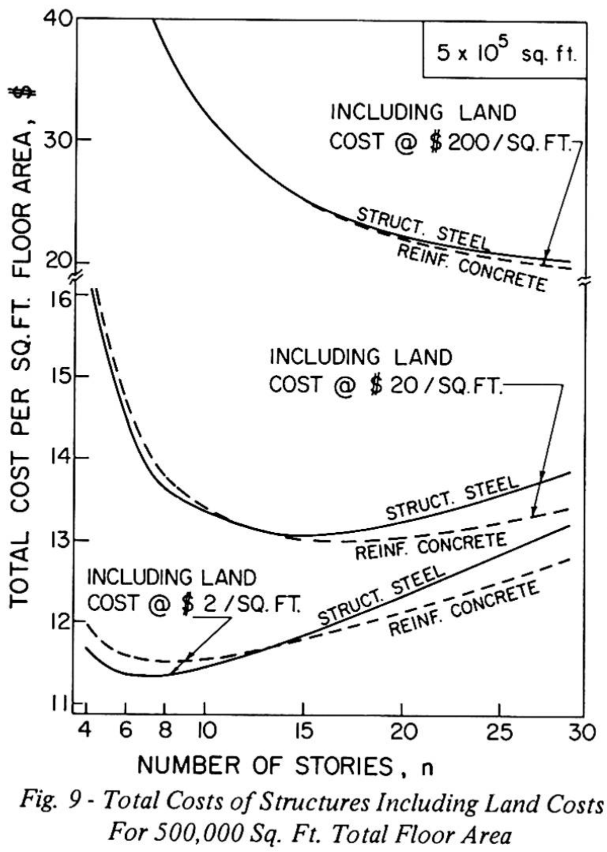

Fig. 9 - Total Costs of Structures Including Land Costs For 500,000 Sq. Ft. Total Floor Area

TABLE 1

OPTIMAL BUILDING HEIGHTS

Total Floor Area (sq.ft.)	Optimal number of storys for land cost of:			
	\$ 0/sq.ft.	\$ 2/sq.ft.	\$ 20/sq.ft.	\$ 200/sq.ft.
5×10^4	4–5	4–5	8–10	> 14
5×10^5	6–9	7–10	15–18	> 30
5×10^6	10–13	10–15	20	> 20

CONCLUSIONS

The present study is an attempt at a comprehensive building system optimization. Its efficiency is related to the successful development of a computer based cost-minimizing procedure for selecting a set of subsystems and topology parameters.

A number of factors such as the foundation, electrical, mechanical and architectural subsystems, have intentionally been excluded. While the economic trends will somewhat be altered by these factors, most of the present results and conclusions will remain essentially valid.

Some design trends are noted from a large number of studies based on the procedures developed. Use of this information may lead to designs that are much closer to the optimum than by intuitive judgement and experience, particularly in the conceptual or preliminary stage of planning. All trends must be tempered, of course, by the limitation of the model and the specific cost data adopted. Land cost may play a major role as a decision variable, can be included in the model and its effects can be evaluated.

Still more comprehensive optimization programs can be attempted, wherein all major building systems can be considered. The success of the present program establishes the precedent for such a bold approach to building systems optimization.

ACKNOWLEDGEMENT

The study is based on a Ph.D. dissertation prepared by the first author in the Department of Civil Engineering, University of Pittsburgh, Pittsburgh, Pa. under the joint supervision of the other authors.

The use of the computing facilities of the U.S. Steel Research Laboratories, Monroeville, Pa. and the University of Pittsburgh, Pittsburgh, Pa., are gratefully acknowledged.

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NOTATION

A_i	= cross sectional area of component i.
B	= bay width.
C	= total cost of the building.
C_i	= cost of the i-th system component.
D	= bay length.
E_c, E_s	= young moduli for concrete and steel, respectively.
EI	= flexural stiffness of a structural component.
f_s	= yield stress for structural steel.
f_y	= yield stress for reinforcing steel.
f_c	= cylinder strength of concrete.
H	= building height.
h	= story height.
L	= building length.
n	= number of stories of the building.
R_i	= participation ratio of component i.
S	= total wind force on the building.
S_n	= total shear at story n level.
W	= building width.
W_i	= unit weight of the i-th system component.
Δ	= total drift of the building.
Δ_c	= cantilever drift of the building.
Δ_n	= contribution of story n to the portal drift of the building.
Δ_p	= portal drift of the building.

SUMMARY

The possibility of using programming techniques for the optimization of realistic structural building systems is explored. The object is to determine the bay dimensions the framing type and the system components, such that the total cost of the structure be minimized and that the functional requirements of the current codes of practice be satisfied. Some typical results of the optimization process and trend of optimal solutions are briefly discussed.

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New Developments in Dutch Steel Bridge Building

Nouveaux développements dans la construction des ponts en acier dans les Pays-Bas

Neue Entwicklungen im Stahlbrückenbau in den Niederlanden

ir. E. YPEIJ
Chief Engineer
Directie Bruggen
Rijkswaterstaat, Netherland

In order to overcome the influences of the ever increasing wages on the total costs of the construction of steel bridges various changes have been adopted in the design taking into account the following principles:

1. Increasing the number of identical items to be used in the bridges.
2. Minimising the number of items with which a bridge should be built.

These changes can be illustrated with the following examples.

Steel bridges with a light-weight concrete deck.

Across the new Scheldt-Rhine canal and the Amsterdam-Rhine canal a number of new bridges have to be built.

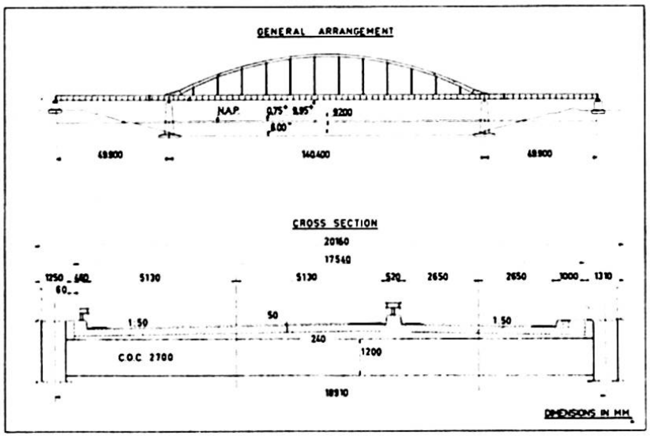

figure 1.

Various designs have been made for these canal crossings, as well in concrete as in steel. After considering the designs, both from technical and economical points of view, the "bowstring" type has been chosen for the main-girders. This also, because such a type of bridge requires a small constructional height (the distance between the upper side of the bridge-deck and the under side of the steel structure) which is of great importance for the approaches. The bridge-deck is a composite construction, consisting of a light-weight concrete deck acting in co-operation with the cross-girders. The deck has not any stringers. The cross-girders are placed 2.70 metres c.o.c. and are I beams provided with studs on the top flange and having only provisions at the ends for the connection to the main-girders.

The use of light-weight concrete for the bridge-deck has given a weight saving of appr.700 tons in the bridge-deck which in turn gave an additional saving of 120 tons in steel material for the main-girders.

The first bridge completed in this type of construction is the bridge across the Scheldt-Rhine canal near Tholen (figure 1.). The light-weight aggregate used for the concrete consists of "Korlin" a product of the D.S.M. (Dutch States Mines) having a specific grain weight of 1.19 to 1.23 and a water absorption of 3%.

Since only the heavy aggregate in the concrete was replaced by the light-weight aggregate the specific weight of the concrete in the deck was in average 1780 kgf/m3. The cube strength after 28 days was in average 350 kgf/cm2.

Push out tests.

In order to investigate the carrying capacity of the studs in light-weight concrete, special tests have been performed. The test specimen consisted of a steel plate with a thickness of 30 mm steel quality Fe 37 and light-weight concrete blocks connected to the plate at each side with the use of 4 studs (figure 4). Special precautions were taken to reduce secondary influences as much as possible.

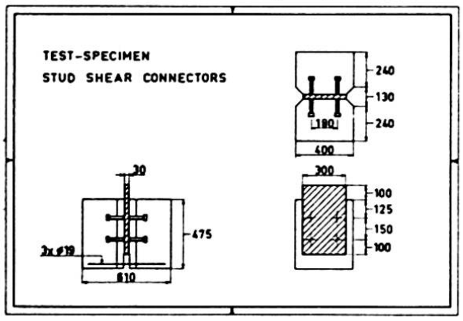

figure 2.

The following test specimen were used:

Specimen number	stud diameter in mm's	stud length in mm's	spiral around studs
A1 A2 and A3	22	110	yes
B1 B2 and B3	22	132	yes
C1 C2 and C3	22	154	yes
D1 D2 and D3	22	110	no
E1 E2 and E3	22	132	no
F1 F2 and F3	22	154	no
G1 G2 and G3	20	100	no
H1 H2 and H3	20	120	no
I1 I2 and I3	20	140	no

The aggregate used in the light-weight concrete was a mixture of sand and Berwilit, an expanded slate product. The specific weight of the concrete was in average 1760 kgf/m3, the cube strength in average 345 kgf/cm2 and the cleave strength in average 26 kgf/cm2.

The material used for the studs was Fe 37.

The results of the tests are given in table 1.

A comparison of the test results with the design values of the allowable loads on studs in normal concrete is given in table 2. In this table, S means the safety factor of the allowable load according to the mentioned standards as compared to the ultimate load as found in the test.

	COMPARISON OF TEST-RESULTS WITH DESIGN VALUES AS GIVEN IN FOREIGN STANDARDS.					
Test-specimen	$P_{0,1}$ per stud (t)	$P_{ultimate}$ per stud (t)	Design values in tons			
			Austrian standard		British standard CP-117:2	
			Bridges	S	Bridges	S
A	14,9(13,2)	20 (19,3)	4,53	4,4	-	
B.	14,9(14,0)	19,3(19,0)	4,53	4,2	-	
C	13,4(13)	19,5(18,4)	4,53	4,1	-	
D	9,1(8,3)	15,1(14,5)	3,60	4,2	3,70	4,1
E	8,6(7,3)	14,8(14,3)	3,60	4,1	-	
F	7,9(7,5)	13,3(11,7)	3,60	3,7	-	
G	6,8(6,2)	14,5(14,4)	2,95	4,9	2,95	4,9
H	7,1(6,9)	13,3(11,8)	2,95	4,4	-	
I	7,7(7,0)	12,1(10,8)	2,95	4,1	-	

•BETWEEN BRACKETS THE LOWEST VALUE PER SERIE.

table 2.

For the calculation of the allowable loads, the values for the cube strength as given in table 1 are taken into account. The used formulae as given in the provisional Austrian Standard (1) are:

studs with spirals $P_{allow.} = 50\ d^2 \sqrt{K_{28}}$;

studs without spirals $P_{allow.} = 40\ d^2 \sqrt{K_{28}}$

in these formulae K_{28} means the cube strength after 28 days.

These formulae are deduced from the allowable loads per stud as given in the A.A.S.H.O.(2) and are based on a safety factor of 4 against the ultimate load. It can be seen that this required safety factor is also found in the tests with the light-weight concrete and furthermore that this safety factor is decreasing when the length of the stud is increased.

RESULTS PUSH-OUT TESTS

Test-specimen	Stud diameter (mm)	Stud length (mm)	Cube-strength K_{28}	$P_{0,1}$* (t)	$P_{0,1}$ aver. (t)	P ultimate (t)	P_{ult} aver. (t)	$\frac{P_{ult}}{P_{0,1}}$
A_1 (S)	ø 22	110		128		156		
A_2 (S)	"	"		125	119,7	169	160	1,34
A_3 (S)	"	"		106		155		
B_1 (S)	"	132	350	112,6		152,5		
B_2 (S)	"	"	kgf/cm2	127	119,7	156,5	155	1,30
B_3 (S)	"	"		119,5		156		
C_1 (S)	"	154		104		151		
C_2 (S)	"	"		107	107	147,5	148	1,38
C_3 (S)	"	"		110		147		
D_1	ø 22	110		74		123,5		
D_2	"	"		77	72,5	123	120,8	1,67
D_3	"	"		66,5		116		
E_1	"	132	345	76		116		
E_2	"	"	kgf/cm2	72	68,8	126	118,8	1,74
E_3	"	"		58,5		114,5		
F_1	"	154		60		114		
F_2	"	"		69	63,5	112	106,6	1,68
F_3	"	"		61,5		94		
G_1	ø 20	100		57		115,5		
G_2	"	"		50,1	54,7	116	116,2	2,14
G_3	"	"	340	56,4		117		
H_1	"	120	kgf/cm2	58		107		
H_2	"	"		56,5	56,5	95	104,8	1,86
H_3	"	"		55,2		112,5		
I_1	"	140		56,2		108		
I_2	"	"		60,5	61,7	86,5	97,0	1,58
I_3	"	"		68,0		96,5		

(S) = Stud with spiral reinforcement.

*$P_{0,1}$ is the required load to receive a permanent displacement of 0,1 mm in the connection after unloading.

table 1.

Tests on cover-plated beams.(5)

In order to receive an economical use of the material it is a normal practise to increase the strength of beams, there where required.This is mostly done by placing additional flange plates to the base flanges of the beam. These additional flange plates are mostly welded to the base flanges.

Fatigue tests of beams constructed in this way show that failure nearly always appears in the area adjacent to the end of the welds. This is due to the fact that in this area the strain concentrations appear and that this zone is heat affected by the welding. The tests have shown also that the effect of special end details of the additional flange on the fatigue strength is rather small and does not justify their additional costs.

To improve the fatigue strength, the use of high strength bolts in co-operation with the fillets welds has been considered.

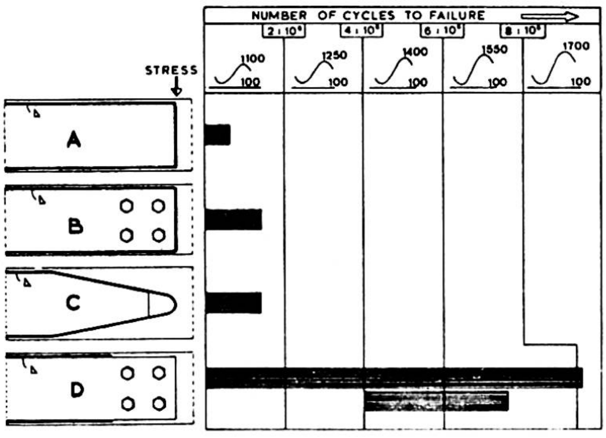

figure 3.

In fig.3 the details of the end connection of the additional flange plate of the tested beams are given as well as the number of cycles to failure and the magnitude of the maximum and minimum stress in the tested beam at the location of the end connection. The tested beams themselves had a length of 2.9 metres and were reinforced with additional flange plates over the middle 1.04 metres. The concentrated load was placed in the centre of the beam. The load cycles had a frequency of 5 Hz and were chosen in such a way that the maximum stress in the beam near the end of the additional plate reached a value of 1100 kgf/cm2 and the minimum stress a value of 100 kgf/

cm2. In case the beam did not show any sign of failure after two million cycles, the maximum stress was raised with 150 kgf/cm2 to 1250 kgf/cm2 and so on. In all cases the minimum stress was 100 kgf/cm2.
The beams of the shape D were made in twofold.
The testing of the second beam of the shape D started at a maximum stress of 1400 kgf/cm2 due to the fact that the first beam of this shape did not show any damages after more than two million cycles at the stress cycle of 1550 - 100 kgf/cm2.
The beams were IPE 40, the additional flanges have been cut to size with a cutting torch from a steel sheet of quality Fe 37.
The welds have been made with a Habilis 4mm electrode (ISO : E443 - T45).
The quality of the high strength bolts was 10K with nuts 8 G
(10K : R_m = 100 kgf/mm2 $R_{0,2}$ = 90 kgf/mm2)(8G : HB = 353 kgf/mm2).
The diameter of the bolts was 20 mm, the pre-tension 17,1 tonf.
Neither the beam nor the plates did get any surface treatment.
In all cases crakcs did occur near the welds with the exception of one of the beams D where the cracks started at the first bolt row.

First test type D:

1100 - 100 kgf/cm2	2×10^6 cycles	no crack
1250 - 100 kgf/cm2	2×10^6 cycles	no crack
1400 - 100 kgf/cm2	2×10^6 cycles	no crack
1550 - 100 kgf/cm2	3.3×10^6 cycles	no crack
1700 - 100 kgf/cm2	1.4×10^4 cycles	crack

Second test type D:

1400 - 100 kgf/cm2	2×10^6 cycles	no crack
1500 - 100 kgf/cm2	1.6×10^6 cycles	crack

Although the number of tests made is not large enough to give a definite conclusion regarding the fatigue strength and the loading conditions in practice mostly differ from the type of loading used in these tests, it can be said that the correct use of high strength bolts in combination with welding gives a great improvement.

Steel bridges with steel decks.

A system of standardising the steel deck has been adopted in which the cross-girders are not directly welded to the steel deck. By doing this, the deck plate is only partly used as top flange for the cross-girders and therefore a slight increase in the quantity of steel is required.
The standard items of the steel deck consist of a steel plate with a width of 2.40 metres and 4 trough type stringers welded to this plate, the stringers being placed 0.60 metres c.o.c.
The cross-girders are I beams, the deck plate is supported by the cross-girders using small vertical supporting strips between the stringers and the cross-girders.
The following advantages are obtained:

1. Standardisation of the dimensions (length and width) of the deck sections composed of a steel plate and stringers. The standardisation is practically independent of the length and the width of the bridge.
2. Simplification of the shape of the cross-girders (I beams), through which the fabrication can be done economically, particularly when using the possibilities that modern welding techniques can offer.
3. The division in standardised elements has a favourable influence on the transportation of the units and the assembling of the bridge.

The general arrangement of a bridge built up with standardised elements and the sequence of assembling is given in figure 4.

With this type of construction, consideration must be given to the horizontal displacements between the deck plate and the top flange of the cross-girder that occur as a result of the live loads.
The cause fluctuating deformations in the trough-type stringers in the points A, B and C (fig.5), which arises the question what the influence is with regard to fatigue.
Furthermore the influence of the reaction of the cross-girders towards the stringers with regard to the buckling of the webs of the stringers must be checked. To investigate these effects, tests have been carried out in which the boundary conditions were practically in accordance with those of the bridge-deck (6).

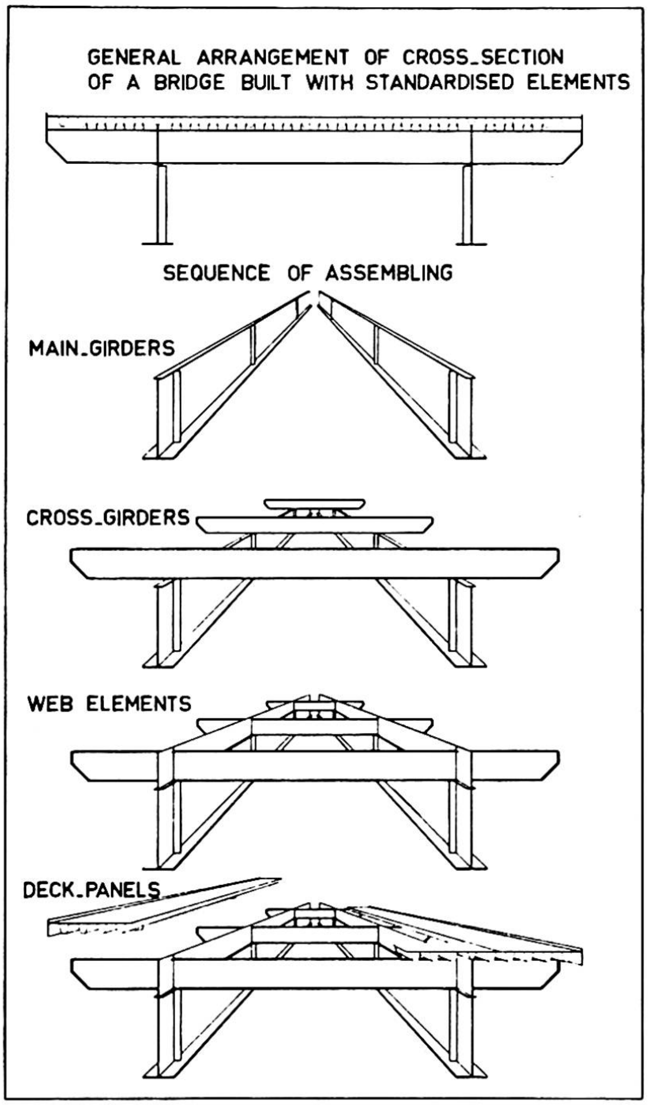

figure 4.

<u>Tests with regards to the horizontal displacements.</u>

The set-up of the tests were such that two connections as given in fig.5 could be tested simultaneously. The results of the tests are given in table 3.

Test specimen no.1 received first a relative displacement from 0.3 mm to 1.2 mm with a frequency of 6 Hz.After 2.080.000 cycles no damage was noticed. After that the displacement was increased from 0.3 mm to 1.7 mm with another 4.019.000 cycles.
For the recorded cracks see table 3.
Test specimen no.2 received first a relative displacement from 0.3 mm to 1.2 mm after that from 0.3 mm to 1.7 mm and finally from 0.3 mm to 2.0 mm.
Test specimen no.3 received a relative displacement from 0.3 to 3.0mm.
As can be seen from table 3 the number of cycles that has been obtained is very large. It has been noticed that the cracks grew very slowly and that at the end of the tests the cracks at point A, that occured first, were not through and through.
Also could be concluded that the cracks did not have any influence on the carrying capacity of the steel deck.

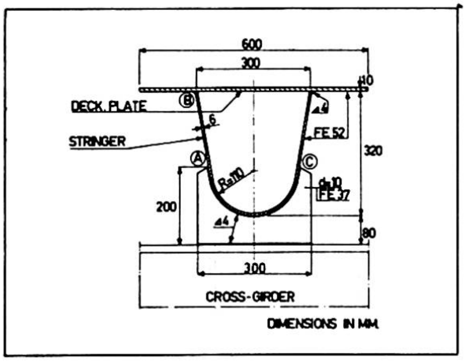

figure 5.

The relative displacement between bridge-deck and cross-girders in a bridge construction as given in fig.4 are the following:

a. Due to a normal rush hour traffic loading 0.5 mm;

b. Due to a very dense and heavy traffic loading 1.5 mm.

Tests with regard to the bearing capacity.

The loads were applied symmetrical to the axis of the trough-type stringer as well as eccentric. Also an extreme horizontal displacement between deck and cross-girder of 3.0 mm has been taken into account.
From these tests it was found that the static ultimate strength was about 60 tons, whilst with a load of 30 tons plastic deformations developed.
A small eccentricity of the load had no influence on the strength of the connection.
The maximum force in this bearing construction as a result of the loading conditions prescribed in the Dutch Standards is 16 tons, which means that the safety factor against failing is sufficient large.

Test-specimen	cycles amplitude mm	cycles number	Recorded cracks
1	0,3 - 1,2	0 2.080.000	none
	0,3 - 1,7	0 445.000 1.760.000 4.019.000	 one box point A other box point A end of test, after that: weld B not through and through.
2	0,3 - 1,2	0 2.017.000	 none
	0,3 - 1,7	0 1.180.000 3.000.000	 one box point A, two cracks the same box, weld point B, through and through
	0,3 - 2,0	0 580.000 7.100.000	 other box point A end of the test
3	0,3 - 3,0	0 200.000 280.000 1.000.000 1.450.000 1.500.000 2.040.000	 one box point A other box point A first crack through and through second " " " " third crack point C end of test, after that: weld B, not through and through

table 3.

Various bridges adopting the described system are under construction, for instance the bridges across the new Scheldt-Rhine canal at the Kreekrakdam.
Figure 6 gives some major information of one of the bridges at this location.

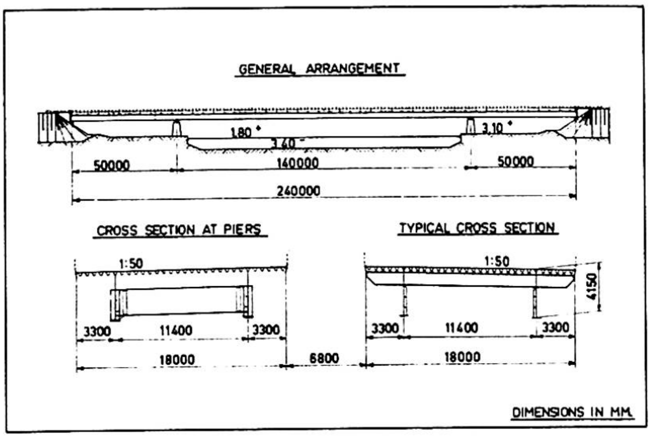

figure 6.

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(2) A.A.S.H.O. Design specification for Highway Bridges.
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(5) Fatigue strength of cover plated beams.
Report 6-70-12 Stevin Laboratory Technical University Delft.
(6) Onderzoek van de verbinding langsligger-dwarsdrager voor een ontworpen stalen brug over de IJssel bij Deventer.
Rapport no.6-69-6 Stevin Laboratory Technical University Delft.

Summary.

In this contribution a description is given of the application of a composite construction in light-weight concrete and the results of push out test of studs in light-weight concrete. Also the results of tests with regard to the connection of additional flange plates to beams by using high strength bolts in co-operation with fillet welds. In the end a description is given of a standardised system for bridges with a steel deck and the tests made for this system.

VII

Steel Brick Buildings

Structures en acier et maçonnerie

Bauten aus Stahl und Mauerwerk

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GENERALITIES

Any transformation in the process of production which leads to a reduction in overall costs compared to quality, may be defined as the industrialization of building. This industrialization, which, in time, permits a decrease in costs, does not dismiss the validity of existing building schemes or their construction processes, although upon a rational, objective examination these could well prove to be theoretically unacceptable. Should the opposite occur, however, the application of industrialization would be limited to a possible razionalization of the existing building process (Industrialization within Building), avoiding its complete transformation (Industrialization of Building). In the first instance, even despite the rational use of existing production plants for existing schemes, it might well prove impossible to invert the present rising curve in the cost of building in time.

Moreover, it is obviously irrational to attempt the industrialization of one single aspect of the building process, for this, in time, will bring about a lack of equilibrium within the organization of the process itself. The most advanced part of the process will refuse those modifications which might result in the total transformation of its plants, while the less advanced part will be unable to organize itself autonomously. For a basic examination of the validity of any industrial process, therefore, it is necessary to formulate objective, general criteria, independent of existing schemes, referred and referrable to the process of production as a whole, and to each of its individual parts, for only by respecting both of these it will be able to guarantee an inversion of the curve of overall expenditure in the future.

THE CRITERIA OF INDUSTRIALIZATION

Because of the need for integrality and generality, these criteria must be formulated independent of the individual process of building, the only presupposition being the verification that the qualities of the building conform to the needs of its users. Moreover, if we take the final, unique aim of the process to be that of meeting man's requirements, and the aim of the criteria that of valuation in the reduction of the cost of the building, we shall see that the criteria themselves must be connected in such a way as to form a "system".

Since, however, the individual application of some of those criteria used in order to evaluate the transformation of the building process as a whole, may well give a negative result, whereas the result they give may well be positive when the complete "system" of criteria is used, even if the system is only being used in the evaluation of the transformation of a certain part of the process; it is consequently easy to see how the order in which these criteria are presented is completely irrelevant. In fact, in the list which follows, the placing of the criteria is arbitrary.

a) The speed of production. The most noted and commonly accepted criterion is that of growth in the speed of production, usually dependent upon progress in the field of technology. The growth of automisation, the introduction of processes of moulding, injecting, die-casting and mass-production, reduces production time.

b) Degree of integrality. This criterion may be expressed via the percentage of sub-processes which the transformation affects, or the relationship between the reduction of the cost of the transformed parts of the process and the cost of the building.

c) The amount of pre-planning. A reduction in the amount of pre-planning necessary in the elementary components of building, brings about a reduction in the range of the family of components in a measure corresponding to the reduction of the parameters (which are variable), with the result that the complete elimination of any kind of pre-planning would lead us to an elementary component with no specific function whatsoever. This criterion favours those processes which require the production of functionally simple elementary components capable of becoming functionally more complex during the succeeding phases of assembly, and frowns on those processes which require the production of complex elementary components such as "beams", "pillars", and "flooring".

d) Modular coordination. Every simplification in the assembly process of the various components of a building reduces cost. The space modular coordination of the components themselves is therefore particularly important, as it sub-divides the total volume of a building in cubes with modulated corners. It should, however, be congenial with the geometry of the construction process.

e) Standardization. As far as the volume of a building is modulated it is advisable to reduce the number of different types of component.

f) Reduction in weight. A reduction in weight (gross and net) leads to a reduction in costs, at the input and output level of the sub-process, with both direct and indirect results. Although sometimes weight has several points in its favour, e.g. thermic inertia,sound insulation, its reduction has been revealed as a positive factor in the evolution of all sectors of production (aeronautic, automobilistic, etc.), and even in the building sector as demonstrated by the gradual reduction of the weight of buildings through the centuries.
g) Duration of efficiency ("Functional space"). An increase in the duration of the efficiency of a building, or rather of the period in which it satisfies man's needs, by supplying a valid environment in which he may work (industrial building) or live (domestic building), is equal to a reduction in its cost. To this end, it is essential to be able to arrive with ease at any given point of the building, via a "functional space", so that new plants may be installed or obsolete ones changed.
h) Social compatibility. The effect a process of transformation has upon society may well condition its adoption. When, for example, it results in overall benefits, yet damages some operator or other in the process, it may provoke the operator to curb it on a social, economic or political level.

It has already been seen that if a technologically industrialized sector is percentually predominant, it may prevent the beneficial transformation of the entire process, should this transformation lead to its own elimination.

EXISTING BUILDING SCHEMES

The afore-mentioned criteria of industrialization can be translated into numerical indexes and thus also form a "system".

Research into, and the definition of this system, are undoubtedly conceptually and operationally important to the objective analysis of the evolution of existing production processes, both from the point of view of total transformation (new process) or of partial transformation (rationalization).

When it comes to total transformation, completely freed from tradition, the Building Industry finds itself at a distinct disadvantage compared to the new industries (aeronautical, automobilistic), for throughout the ages it has been an expression of man's desire for a form of habitation, and has consequently been structuralized by the traditions which have been passed down from craftsman to craftsman.

But even if, for the moment, it could seem simply a utopistic solution to be verified only in the distant future, some kind of hypothesis is nevertheless important as an indication of the direction which the transformation of present day processes should take from an economic point of view. By this we mean that critical analysis can and must tell us beforehand if existing processes of construction are theoretically susceptible to industrialization or not, or whether, in time, they will have to be either partly or wholly abandoned. With regard to those schemes which, theoretically, seem beyond industrialization, the fact that they may well have reached a high level of efficiency in the production of some intermediary component is of no importance whatsoever. Thus if, for example a reduction of costs in time in the typological scheme for "steel buildings", should require the elimination of the sub-process production of beams, the existence of sizeable factories, producing beams should not form an obstacle to that reduction. The same argument holds good for the "reinforced concrete building" scheme, and factories should be prevented from mass-producing pre-cast "columns", "beams" and "flooring" should it be known that these products prevent the building industry from placing itself upon a curve of decreasing costs in time.

RESEARCH PROGRAMMES AND CONCEPTUAL EXPERIMENTS

While waiting for the above-mentioned numerical index system to become available, incorporated in a general theory of industrialization, it has seemed opportune to assume that existing building schemes are doomed to remain on a growing curve of costs in time.

In the face of this limited hypothesis, one wonders if there cannot exist new construction schemes, which, largely satisfying these criteria, could succeed in placing themselves on a decreasing curve of costs in time. Should such schemes exist, they would help to guide actual building research programmes towards solutions possibly completely different from those already in existence.

As an exclusively theoretical treatment of the problems which have come to light would be far too abstract, the National Council for Research has decided, in its "Programme for the Industrialization of Building", to give concrete form to the answer to these problems via the construction of a model whose only aim is to affirm the existence and theoretical validity of that answer, by demonstrating the position of its prototype on a curve of decreasing costs in time, and not its position in the field of existing economic competitivity.

CRITICAL DESCRIPTION OF THE EXPERIMENTAL MODEL

The experimental model, described with the criteria listed in par.2 in mind, is a building which has been realized using the elementary component illustrated in the following figures, and which has significantly been called a "steel brick". Its space-modular dimensions are 2M x 2M x M when M=30 cm, and it is made of sheet metal. The possibility of mass-production by cold-forming enables a single press to produce almost a building a day. As the bricks may be used, joined together, both for the horizontal roofing and for the vertical walls, the installation of a mass-production plant even for a small number of buildings is justifiable. The space modulation is strictly congenial both to the parallelepiped geometry and to pressing, and involves the modulation of useful surfaces, which may be covered by four types of pannelling (perimeter walling, both external and internal; flooring and roofing) with modular characteristics of unification and mass-production similar to those of the brick, and extended throughout the entire building. It is obvious that the dividing-walls, the doors and the windows all follow the same modulation.

The brick does not have a specific, autonomous function; that is to say it is not a "beam" or a "pillar"; and in fact the term "brick" itself indicates the absence of any kind of pre-planning. The lightness of the building represents notable progress in weight reduction. The remarkable rigidity of the reticular structure is evident from the views of the experimental building.

The functional space within the walls and flooring offers a simple solution to the problem of the housing of a variety of installations, and guarantees their continual modernization. The remaining space, together with the internal and external wall coverings, constitutes a valid instrument of thermic and acoustic insulation, in tune with the modern conception of a building as a comfortable container of installations.

The fact that it is unnecessary to define the particular material which will be used to realize the various parts of the building - without excluding any sector of present industrial activity - is an indication of social compatibility of such a typology ("Open Industrialization through Components"). The architectural flexibility of the system does not create tensions within the planning sector, and may be varied to meet the demands of the user. The experimental building has also been tested in order to verify its degree of safety. The following figures illustrate the experimental study.

SUMMARY

Building is the only industrial sector which presents growing costs in time.

The theoretical criteria of Industrialization (standardization, repetition, lightness, mass-production, coordination), although theoretically quoted and affirmed, are not, in fact, respected by present building schemes. In order to obtain a decreasing curve of costs in time new systems must be devised, and new processes of construction evolved which really respect the theoretical criteria. It is the aim of the model realized in steel "bricks" at the CESUN in Naples to theoretically demonstrate this fact.

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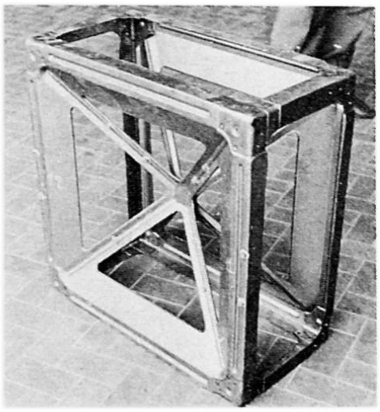
(1) The first studies in "steel brick building" were carried out in 1966 by the author of this article, prof.Pagano, with the collaboration of Carlo Funel and Alfredo Sbriziolo. A.Giliberti, F.M.Mazzolani, L.Morrica, N.Palumbo and S.Terracciano also collaborated.

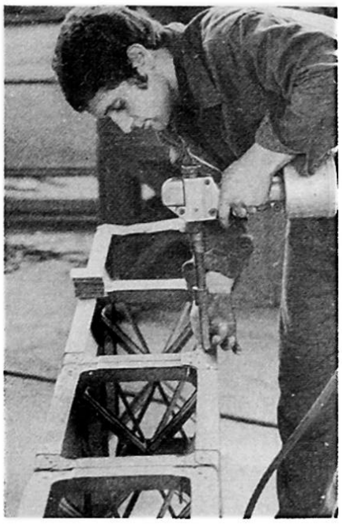
Research into "steel brick building" is actually being continued by prof.Pagano with the financial aid of the C.N.R. - Programma per l'Industrializzazione dell'Edilizia, at the CESUN (Centro Studi per l'Edilizia).

CONSTRUCTION TECHNOLOGY IN
STEEL BRICK BUILDING



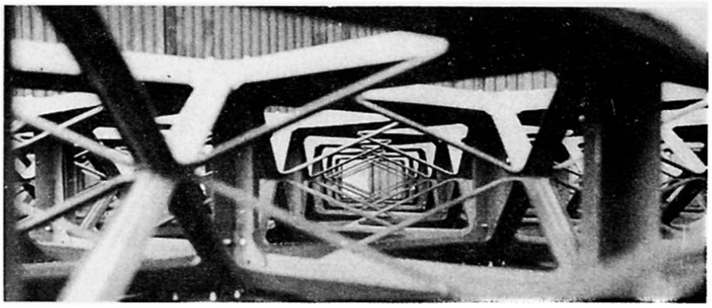
The steel brick:
2MxSMxM M=30 cm

1



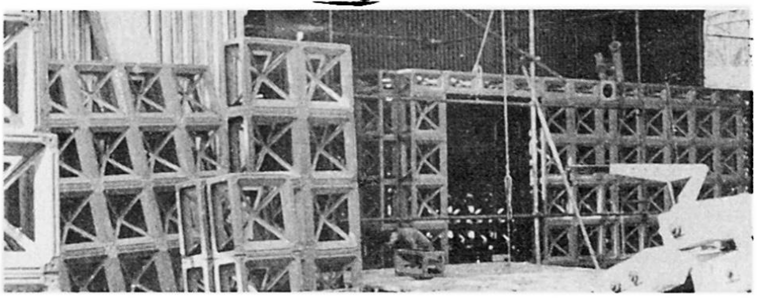
2

Assembly procedure using fastening rivets.



3

Functional space for installations inside roofing and walls.

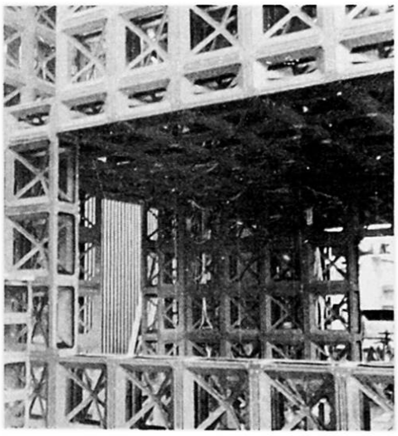


4

Walls and roofing ready for assembly.

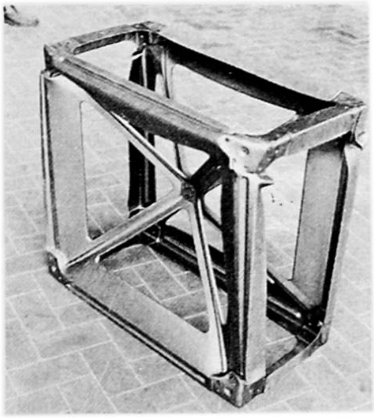
Some detailed views of the building.

5



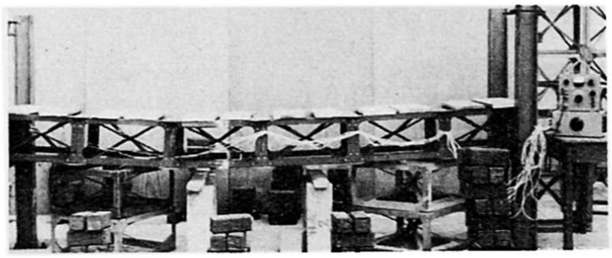
6

TEST RESULTS



7

Beam flexional test.



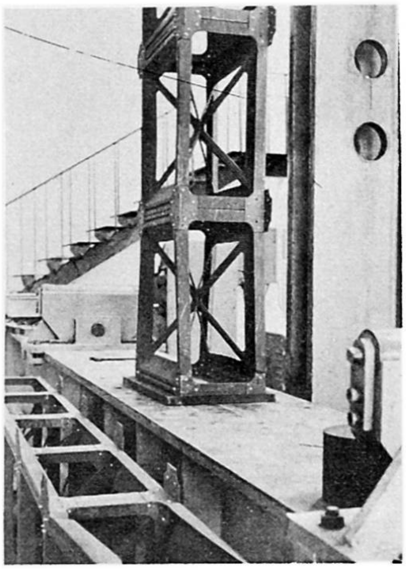
8

The collapse of the corners of a prototype of the brick as a result of a compression test.

Column test

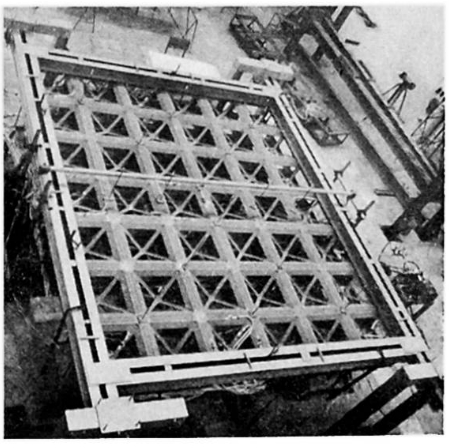
9

Detail of previous view



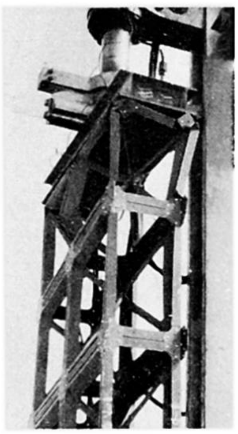
10

Roofing test



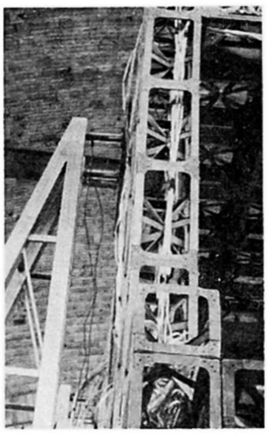
11

Double column test



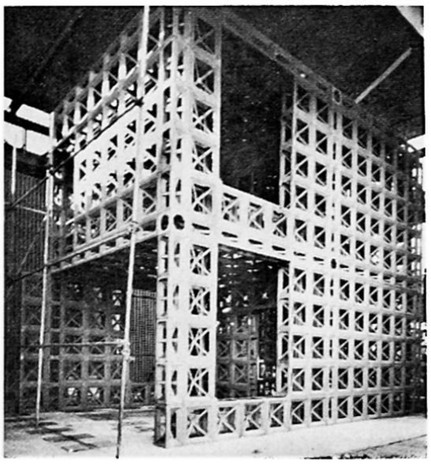
12

The whole construc tion subjected to horizontal forces



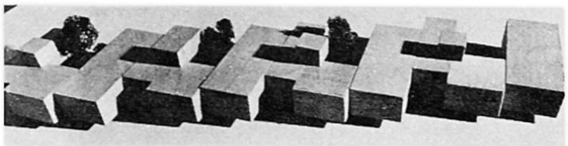
13

ARCHITECTURAL COMPOSITIONS



14

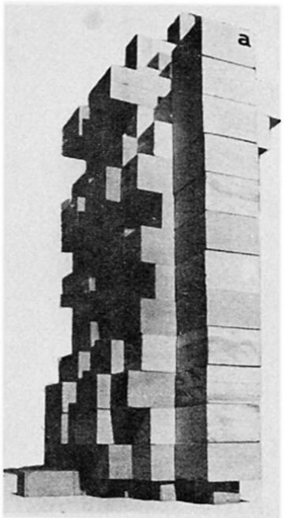
View of the steel brick building prototype.



15

View showing the flexibility of the construction system.

The system permits construction both one-floor and multi-floor buildings.
The system does not limit architectural and volumetric construction in any way whatsoever.



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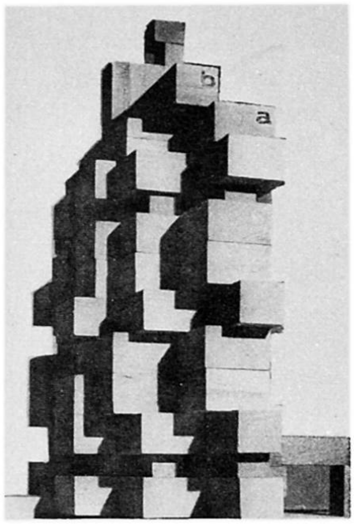

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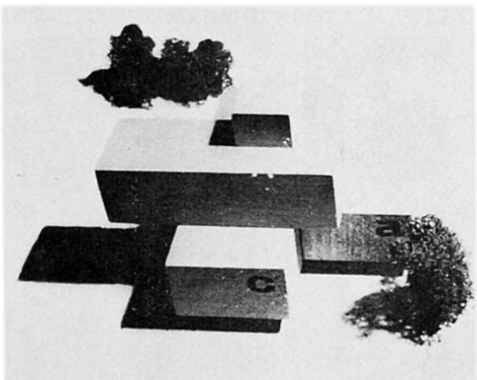

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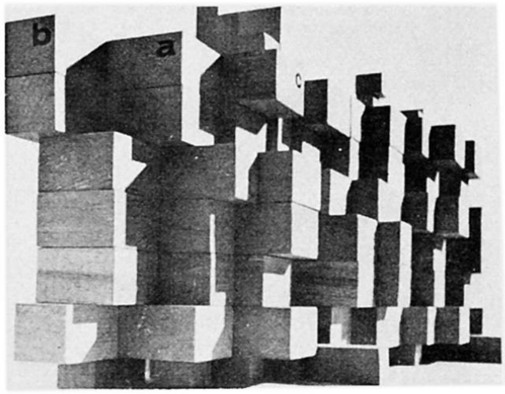

19

VII

Neue Entwicklung von Paralleldrahtseilen für Schrägseil- und Spannbandbrücken

New Developments of Cables with Parallel Wires for Cable-Stayed and Suspended Deck Bridges

Développement récent des câbles à brins parallèles pour la construction de ponts haubanés et à tablier sur câble

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1.) Einführung

Bei dem Wettbewerb "Große Beltbrücke" wurde im Jahre 1966 erstmalig eine Schrägseilbrücke in Spannbeton für eine kombinierte Straßen- und Eisenbahnbrücke mit einer Spannweite von 350 m vorgeschlagen und prämiiert (Bild 1).

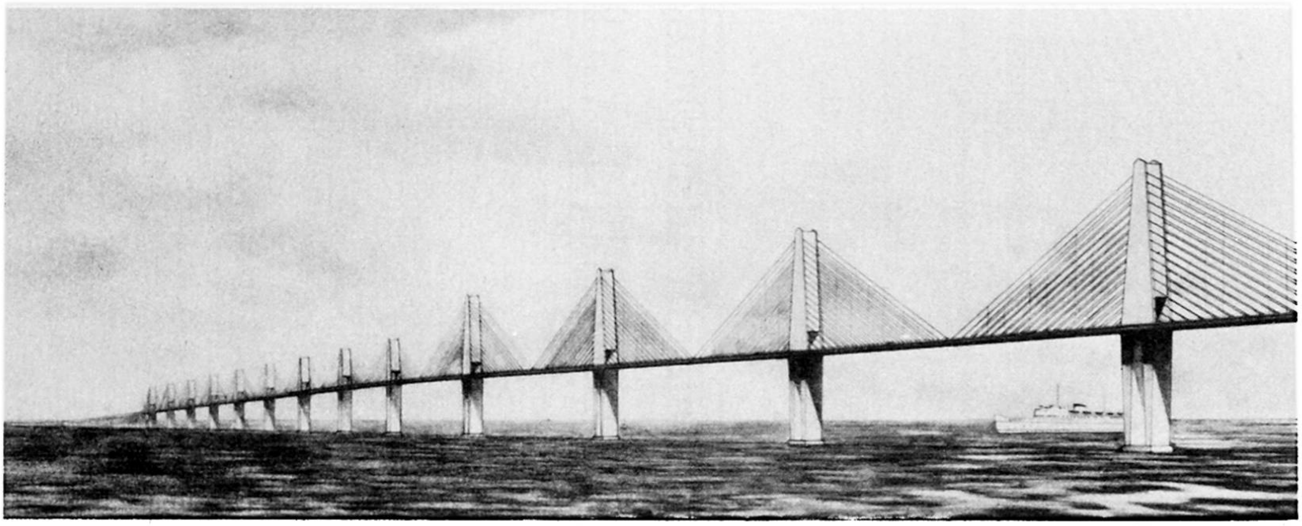

Bild 1: Entwurf "Große Beltbrücke"
Schrägseilbrücke in Spannbeton für eine kombinierte Straßen- und Eisenbahnbrücke

Im vergangenen Jahr 1971 wurde eine solche Brücke von 150 m Spannweite über den Main bei Hoechst erstmalig ausgeführt (Bild 2).

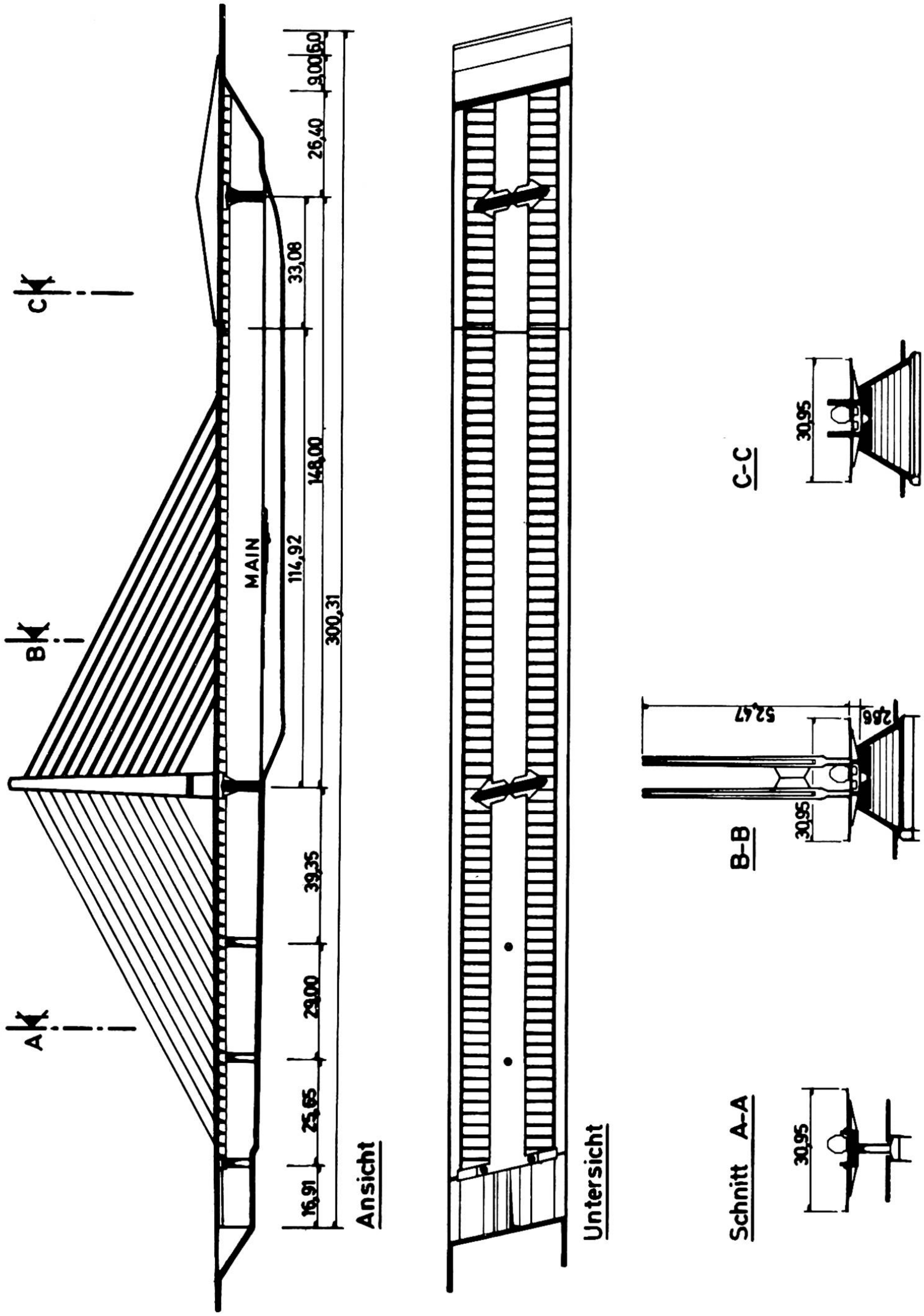

Bild 2: Schrägseilbrücke über den Main bei Hoechst

Da diese Brücke nur einen Doppelpylon aufweist, also einseitig ist, würde die Spannweite bei symmetrischer Ausführung ca.250 m betragen.

Für die Wahl der Spannbetonbrücke an Stelle der ursprünglich geplanten Schrägseilbrücke aus Stahl, waren für den Bauherrn - die Farbwerke Hoechst - drei Gründe maßgebend:

a) Der wesentlich bessere Korrosionsschutz der Konstruktion, insbesondere der Seile in dem extrem aggressiven Klima der Chemischen Werke Hoechst,

b) die wesentlich geringeren Bau- und Unterhaltungskosten,

c) die wesentlich kürzere Zeit von knapp zwei Jahren für Planung und Ausführung bis zur Betriebsübergabe.

Eine wesentliche Grundlage für diese Vorteile wurde durch die Entwicklung des DYWIDAG-Paralleldrahtseils geschaffen.

2.) Das DYWIDAG-Paralleldrahtseil

2.1 Beschreibung

Aufbau und Abmessungen des DYWIDAG-Paralleldrahtseiles zeigt Bild 3. Im Querschnitt besteht das DYWIDAG-Paralleldrahtseil aus

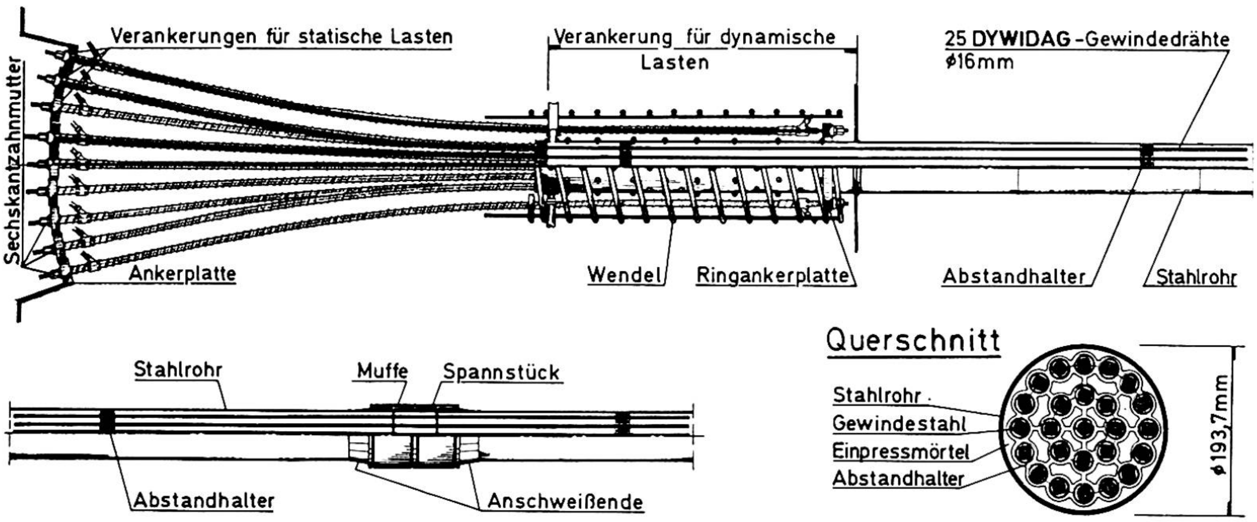

Bild 3: Aufbau und Abmessungen des DYWIDAG-Paralleldrahtseiles

25 Gewindedrähten Nenndurchmesser 16 mm der Stahlgüte St 135/150, die durch Abstandhalter aus Polyäthylen geordnet werden. Über dieses Bündel ist ein Stahlrohr mit den Abmessungen 193,7 x 5,4mm Stahlgüte St 35 geschoben. Im Verankerungsbereich schließt an das Stahlrohr das Endstück an, welches im hinteren, später einbetonierten Bereich, Nieten zur Verbesserung des Verbundes aufweist. Am Ende ist das Endstück mit der Führungskappe abgeschlos-

sen, von der aus die Drähte einzeln verrohrt bis zu ihrer Endverankerung weitergeführt werden. Die Endverankerung wird als Plattenverankerung ausgeführt.

Die Hohlräume zwischen den Gewindedrähten und dem Stahlrohr sind mit Zementmörtel verpreßt. Die vom Stahlrohr abgegebenen Kräfte werden durch Verbundwirkung zwischen Stahlrohr und Beton auf den Konstruktionsbeton abgegeben, wo sie durch eine Ringankerplatte, in der 4 Gewindedrähte Nenndurchmesser 16 mm Stahlgüte St 135/150 verankert sind, aufgenommen werden.

Die Abstandhalter sind so konstruiert, daß die Längsbeweglichkeit der Einzeldrähte voll gewährleistet ist. Man erreicht dadurch, daß die Gewindedrähte einzeln nacheinander mit einer leichten Spannpresse gespannt werden können, ohne daß bei diesem Spannvorgang ein Nachbardraht beeinflußt wird. Jeder einzelne Draht verhält sich deshalb wie ein Einzelspannglied.

Im Montagezustand muß das Stahlrohr an einer Stelle durch eine lösbare Muffenverbindung unterbrochen sein, um Überbeanspruchungen während der Bauzustände zu vermeiden. Die Muffenverbindung baut sich aus zwei Anschweißenden, der Muffe und dem Spannstück auf, welches jeweils ein Links- und ein Rechtsgewinde besitzt, so daß es möglich wird, Längendifferenzen vor dem Zusammenschrauben der Stahlrohre auszugleichen.

2.2 Tragwirkung der Verankerung des DYWIDAG-Paralleldrahtseiles.

Das DYWIDAG-Paralleldrahtseil zeichnet sich durch eine Verankerung aus, die dieselbe Schwingungsfestigkeit wie der ungestossene Einzeldraht aufweist. Diese Eigenschaft wird nachstehend erläutert.

Bild 4 zeigt eine Schemadarstellung der Verankerung. In der Zeichnung sind zwei Verankerungsbereiche dargestellt: a) die Endverankerung, bestehend aus Platten und b) die Zwischenverankerung, bestehend aus dem Endstück und der Ringankerplatte.

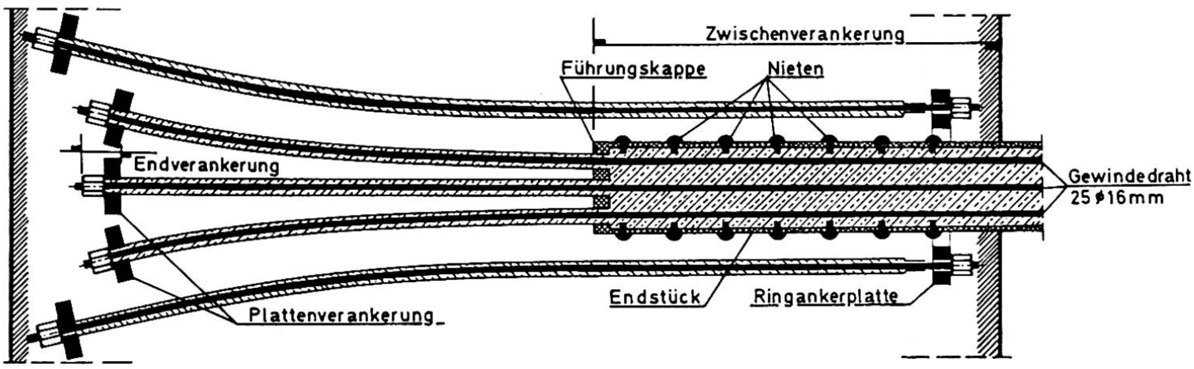

Bild 4: Übersicht über die Verankerung.

Wie im Abschnitt 2.1 bereits erläutert, ist während des Bauzustandes das Stahlumhüllungsrohr an einer Stelle getrennt. Die Kräfte (g), die während dieses Zustandes auf die einzelnen Gewindedrähte aufgebracht werden, zeigen deshalb keine Krafteinwirkung auf das stählerne Hüllrohr. Sie werden über die Endverankerung abgeleitet, da in diesem Zustand noch kein Verbund zwischen den Gewindedrähten und dem Bauwerksbeton besteht (s.Bild 5).

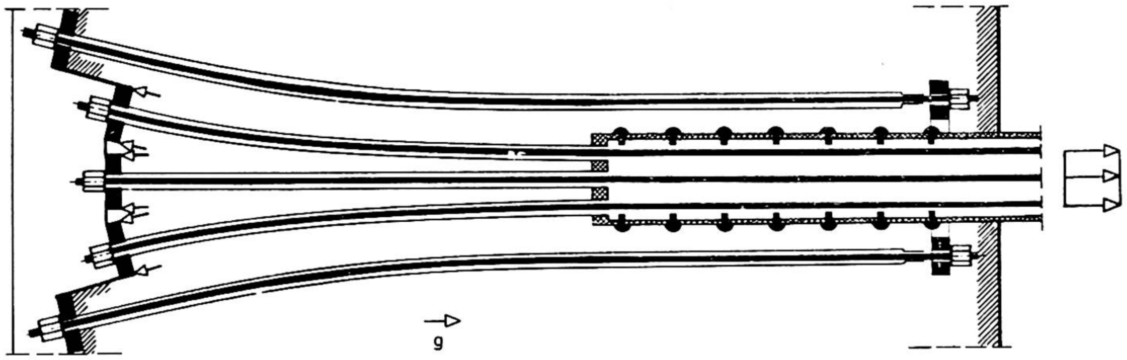

Bild 5: Bauzustand. Die Lasten "g" werden nur über die Endverankerung abgetragen.

Wird nun das stählerne Hüllrohr kraftschlüssig verbunden und mit Zementmörtel verpreßt, so ändert sich zunächst nichts an dem Kraftzustand. Wirken jedoch zusätzliche Lasten z.B. aus Verkehr (p) auf das Paralleldrahtseil, werden diese von den einzelnen Gewindedrähten und dem stählernen Hüllrohr gemeinsam getragen (Bild 6).

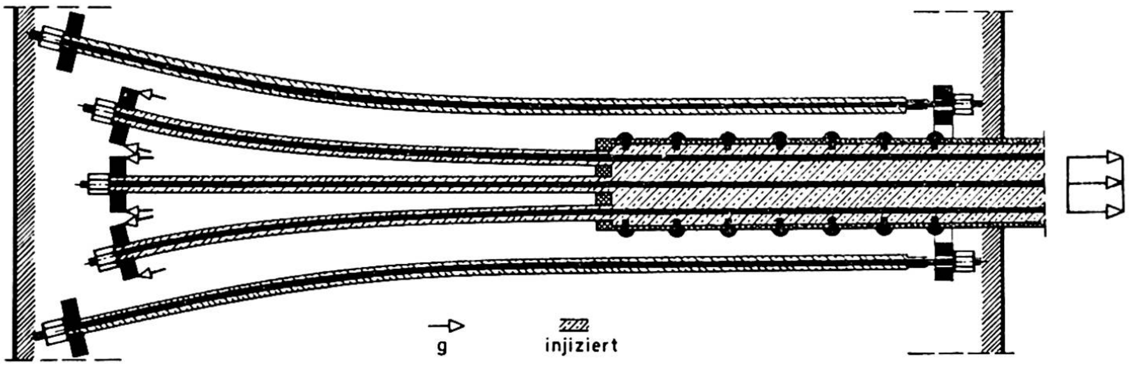

Bild 6: Beanspruchung der Verankerung nach dem Verpressen aus Eigengewichtslasten "g"

Die zusätzlichen Lasten (p) verankern sich aus Verträglichkeitsgründen im Bereich des Endstücks über Haftung und werden über die Ringankerplatte in den Konstruktionsbeton des Bauwerks abgeleitet (Bild 7).

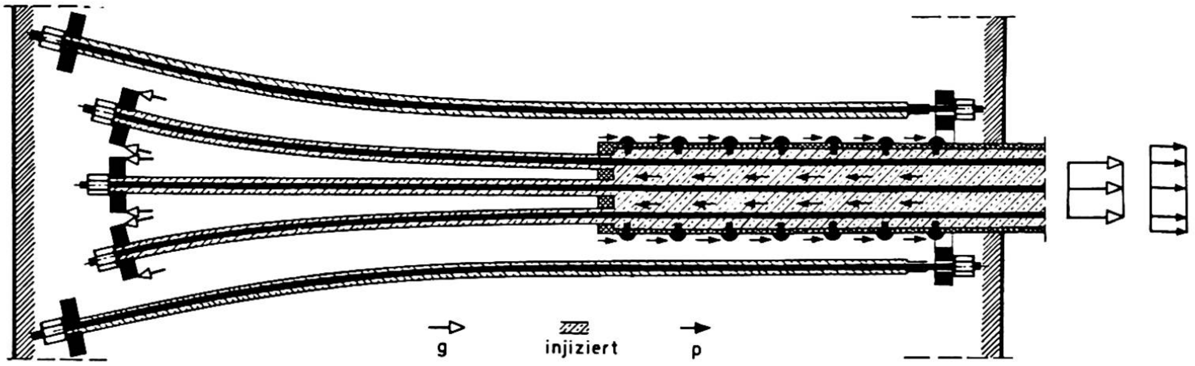

Bild 7: Beanspruchung der Verankerung durch Eigengewicht "g" und Verkehrslasten "p".

Die Aufteilung der Verankerung in eine Endverankerung über Mutter und Ankerplatte und eine Haftverankerung bewirkt, daß die dynamisch wirkenden Kräfte allmählich und ohne Spannungsspitzen in das Bauwerk eingeleitet werden, so daß die Schwingfestigkeit der Gewindedrähte nicht herabgesetzt wird. Die Endverankerung erhält nur statische Belastung. Es ist deshalb möglich, den Gewindedraht mit der Schwingungsfestigkeit des ungestoßenen Drahtes und das Hüllrohr mit der Schwingungsfestigkeit des verwendeten Rohmaterials bezw. der Schweißnähte anzusetzen.

In einem an der Technischen Universität München durchgeführten Großversuch, bei dem zwei Verankerungskörper 4 1/2 Mill. Lastwechsel unterzogen wurden, wurde die Richtigkeit dieser Überlegungen nachgeprüft. Es zeigte sich, daß die Entlastung der eigentlichen Seilverankerung von den schwingenden Kräften vollständig war und daß der Verbund zwischen den Gewindedrähten, dem Hüllrohr und dem Beton des Pylons bezw. des Fahrbahnträgers durch diese Beanspruchung nicht beeinträchtigt wurde.

Die Kräfte im Hüllrohr können so einreguliert werden, daß eine optimale Ausnutzung des Materials unter den zu erwartenden statischen und dynamischen Beanspruchungen eintritt. Diese Einstellung der Kräfte erfolgt über die Spannmuffe.

3.3.4 Der Korrosionsschutz des DYWIDAG-Paralleldrahtdrahtseiles

Der Korrosionsschutzdes DYWIDAG-Paralleldrahtseiles wird durch die Ummantelung der Gewindedrähte mit Zementmörtel und durch das stählerne Hüllrohr gebildet. Die Gewindedrähte sind durch Abstandhalter so geführt, daß sie weder die Wandungen des Stahlrohres noch sich untereinander berühren. Dadurch ist sichergestellt, daß beim Verpressen jeder Hohlraum im Bündel mit Zementmörtel sicher ausgefüllt wird. Die Außenflächen des Stahlrohres werden mit im Stahlbau üblichen Anstrichsystemen vor Korrosion geschützt.

Während des Bauzustandes werden die Gewindedrähte durch das Hüllrohr, welches an den Enden dicht verschlossen werden kann, vor Rost geschützt. Die Wirksamkeit dieser Maßnahmen wurde auf der Baustelle der Mainbrücke Hoechst durch Versuche überprüft.

3.4 Verhalten des DYWIDAG-Paralleldrahtseiles gegenüber Windschwingungen

Eine weitere wichtige Eigenschaft des beschriebenen Seils liegt in seinem Verhalten gegenüber den durch Wind erregten Flatterschwingungen. Durch das nicht vollelastische Verhalten des Injektionsmörtels besitzt das DYWIDAG-Paralleldrahtseil eine innere Dämpfung, welche die Amplituden der Flatterschwingungen wesentlich vermindert.

4. Zusammenfassung und Ausblick.

Mit der Entwicklung des DYWIDAG-Paralleldrahtseiles ist nicht nur für den Spannbetonbau, sondern auch für den Stahlbau eine neue und fortschrittliche Konstruktionsmöglichkeit entstanden. Gegenüber den konventionellen Seilen hat das DYWIDAG-Paralleldrahtseil folgende Vorteile:

1. Volle Ausnützbarkeit der Schwingweite des Drahtmaterials durch Beseitigung der Schwächung der Schwingweite in der Verankerung.
2. Einwandfreier Korrosionsschutz der hochwertigen Stähle durch Einbetonieren, wodurch die gefürchteten Drahtbrüche durch Spannungskorrosion zuverlässig vermieden werden.
3. Genaue und einfache Eintragung der Seilkräft durch Spannen des einzelnen Drahtes mittels einer leichten Spannpresse.
4. Wegfall der Seilreckung und des Schlupfes in der Verankerung und der daraus resultierenden Notwendigkeit einer Möglichkeit zum Nachstellen des Seiles.
5. Einfache Montage des Seiles auf der Baustelle.
6. Günstiges Verhalten gegenüber den durch Wind erregten Schwingungen.

Dieses neuentwickelte Konstruktionsglied ist die Grundlage für neue Möglichkeiten im Großbrückenbau. Außer der genannten Schrägseilbrücke können mit dem Prinzip der Spannbandbrücke mit aufgesattelten Kastenträgern wesentlich größere Spannweiten und Schlankheiten von Balkenträgern als bisher erreicht werden.

Zusammenfassung

Das DYWIDAG-Paralleldrahtseil bietet für den Spannbetonbau und den Stahlbau eine neue und fortschrittliche Konstruktionsmöglichkeit. Vorteile sind die volle Ausnützbarkeit des Drahtmaterials auch in der Verankerung, der einwandfreie Korrosionsschutz der hochwertigen Stähle, wodurch die gefürchteten Drahtbrüche durch Spannungskorrosion vermieden werden, sowie die einfache und genaue Regulierbarkeit der Spannkräfte.

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Frei eingespannte zylindrische Turmbauwerke aus glasfaserverstärktem Kunststoff (GFK)

Cylindrical Built-in Tower of Fiber Glass Reinforced Synthetic Material

Constructions en forme de tour cylindrique en matière synthétique renforcée fibres de verre

KLAUS DÖRNEN
Dr.-Ing.
Marl, BRD

1. Einleitung

Technische und wirtschaftliche Gesichtspunkte sind durchweg für die Wahl eines Baustoffes bestimmend. Die gebräuchlichen Baustoffe werden nach physikalischen Festigkeitseigenschaften, Verarbeitbarkeit, Alterungs- und Korrosionsbeständigkeit, Widerstandsfähigkeit gegen Hitze und Feuer, konstruktive Durchbildungsmöglichkeiten sowie nach Transport- und Montagegegebenheiten beurteilt und eingesetzt. Im neuzeitlichen Bauwesen, vornehmlich im Industriebau, können neben den angeführten Eigenschaften noch weitere Forderungen gestellt werden, wie z.B. Isolierfähigkeit gegen Wärme oder Elektrizität, Beständigkeit gegen chemische Angriffe, hohes Tragvermögen bei gleichzeitiger Lichtdurchlässigkeit, geringes spezifisches Gewicht und dgl. mehr. Kunststoffe zeichnen sich durch letztgenannte Eigenschaften bevorzugt aus und können ihre Eigenarten berücksichtigend sinnvoll angewandt, als Isolierstoffe, Verbindungselemente, flächenhafte Verkleidungen und als tragende Konstruktionsglieder vorteilhaft eingesetzt werden. Noch ist ihre Anwendung und ihr Gebrauch verhältnismäßig wenig bekannt, während der Markt schon eine bedeutende Anzahl von Kunststoffen liefert. Das Marktangebot wächst ständig und die Wahl wird schwieriger, da mit den Vorteilen auch eine Reihe von Nachteilen verbunden ist, die zu erkennen und abzuwägen Fachkenntnisse erfordern. Da das einzelne Kunststoffmaterial für Tragkonstruktionen, und hierzu gehört vornehmlich der glasfaserverstärkte Kunststoff (GFK), nicht den universalen Charakter, wie z.B. Stahl, besitzt, müssen zum technisch einwandfreien und werkstoffgerechten Einsatz jedem Verwendungszweck gründliche Untersuchungen mit praxisnahen Versuchen vorangehen.

2. Glasfaserverstärkter Kunststoff (GFK)

Bei höher beanspruchten Belastungsfällen bekommt das Reaktionsharz durch eingefügte Glasbewehrung die eigentlichen Tragfestigkeiten. Zur Glasfaserverstärkung stehen Matten, Gewebe und Rovings zur Verfügung. Die mechanischen Güteeigenschaften des GFK, wie Festigkeit und Steifigkeit, werden maßgeblich vom Verhältnis Glasfaser/Reaktionsharz und durch die Orientierung der Fasern zur Beanspruchungsrichtung beeinflußt. Je nach Glasfaseranteil können hierbei Zugfestigkeiten erreicht werden, wie sie für Stahl bekannt sind. Vom reinen Harz ausgehend, ist mit wachsendem Glasgehalt des GFK eine beträchtliche Steigerung der Biege-, Zug-, Schlagfestigkeiten und der Verformungs-Moduln verbunden. Nach den "Vorläufigen Richtlinien zur Kennwertbestimmung....." lassen sich die GFK-Werkstoffeigen-

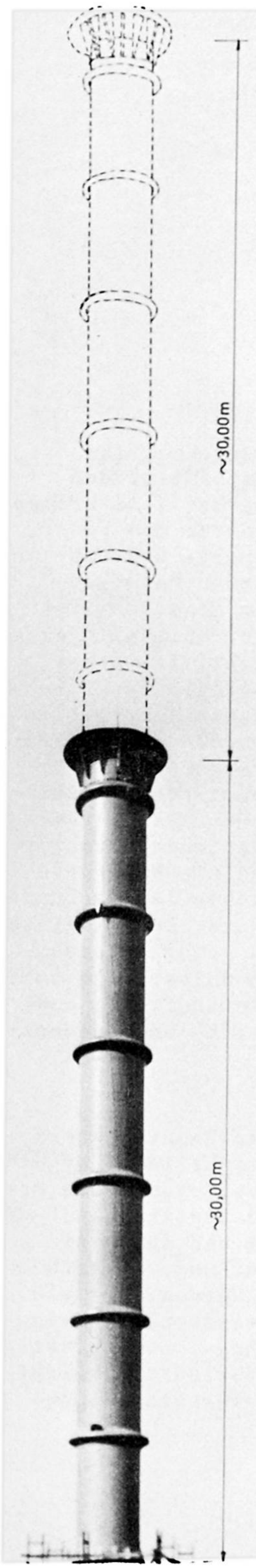


Bild 1: Montierter GFK-Ansaugkamin

schaften hinreichend genau festlegen.

Für bauliche Konstruktionen wirddie Form regellos verteilter, auf 3 bis 5 cm Länge geschnittener Glasfasern am häufigsten angewandt. Diese Matten aus Glasfaserstücken werden für die Bemessung nach Quadratmetergewicht bestimmt und aus größeren Bändern oder Matteneinheiten nach Bedarf herausgeschnitten. Die Matte als stabilisierendes Element im Reaktionsharzbett leitet die Beanspruchung von Faser zu Faser über Schub- und Haftfestigkeit des Reaktionsharzes weiter. Infolge der regellosen Faseranordnung weisen diese Matten-Laminate keine richtungsabhängigen mechanischen Eigenschaften auf. Festigkeit und Steifigkeit sind in allen Richtungen der Beanspruchungsebene praktisch gleich; der Verbundwerkstoff ist nahezu isotrop.

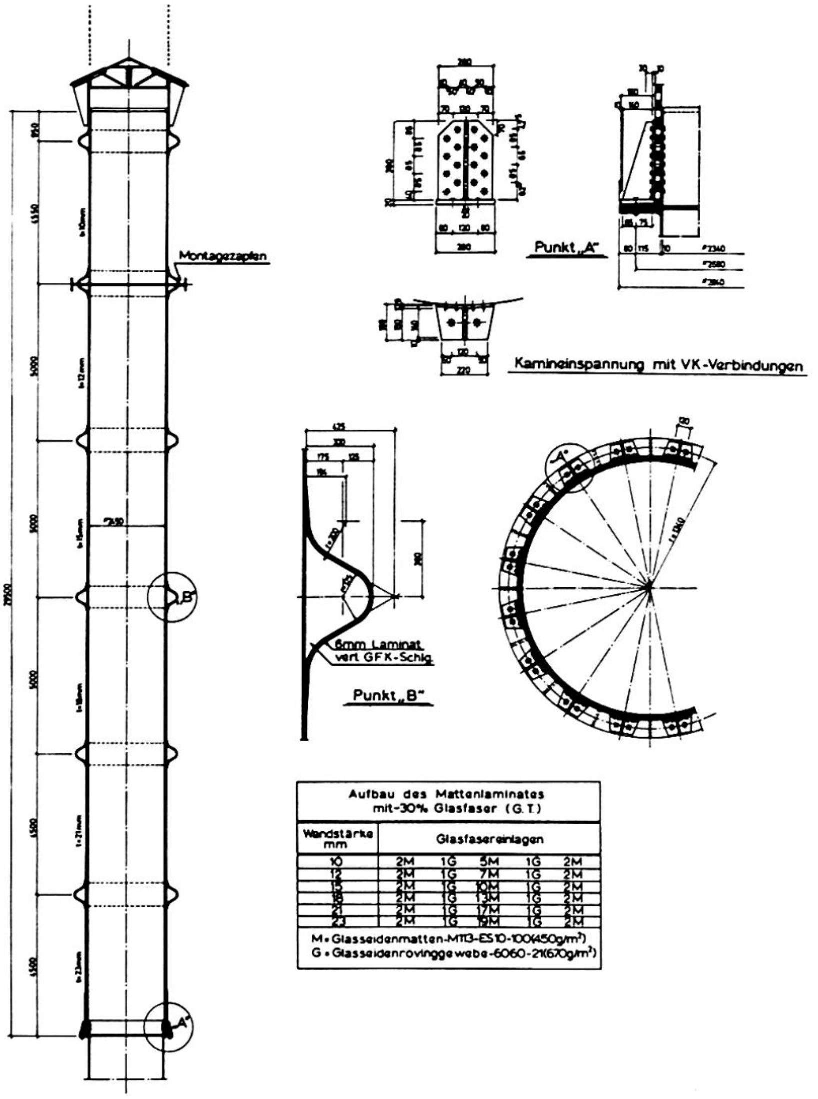
Für optimale Ausnutzung des GFK-Werkstoffes sollte sich die Bewehrungsorientierung der Glasfasern mit der Beanspruchungsrichtung decken. Statt Glasfasermatten werden orthogonale grobmaschige Gewebe oder direkt Rovingsstränge einlaminiert. Der mit Gewebe oder Rovings versehene GFK ist anisotrop und bei senkrecht aufeinander stehenden unterschiedlichen Steifigkeiten, wie z.B. beim Wickellaminat, speziell orthotrop.

Zur Herstellung von GFK-Laminaten bedient man sich unter Verwendung von Matten und Geweben meist des Handauflegeverfahrens und bei richtungsorientierter Glasfaserbewehrung vornehmlich bei zylindrischen Körpern des maschinellen Wickelverfahrens. Während die erstgenannte Herstellungsart sich stark lohnabhängig zeigt, verlangt die zweite durchweg einen höheren Glasfasergehalt, der den Materialpreis des GFK stark beeinflußt. Welche Fertigungsart die größtmögliche Wirtschaftlichkeit bietet, kann letztlich erst nach endgültiger Bemessung bestimmt werden. Für beide Laminate müssen daher Festigkeits- und Stabilitätskennwerte vorliegen.

3. Konstruktive Durchbildung eines GFK-Ansaugkamines

Als Ausführungsbeispiel für turmartige Bauwerke sei hier ein in Abb. 1 wiedergegebener Ansaugkamin mit ∅ 2,50 m angeführt, der zunächst auf 30 m, im späteren Ausbauzustand auf 60 m Höhe zu errichten ist. Der erste 30 m-Abschnitt wurde ohne Werkstatt- und Montagestöße in ganzer Länge fertiggestellt, was bei derartigen Abmessungen praktisch nur im Handauflegeverfahren und dann zweckmäßig mit Matten-Laminaten zu bewerkstelligen war. Konstruktive Details und Aufbau der Glasfasereinlagen im Laminat sind aus der Darstellung Abb. 2 im einzelnen zu entnehmen.

Im Erscheinungsbild des GFK-Kamines fallen



Aufbau des Mattenlaminates mit-30% Glasfaser (G.T.)					
Wandstärke mm	Glasfasereinlagen				
10	2M	1G	5M	1G	2M
12	2M	1G	7M	1G	2M
15	2M	1G	10M	1G	2M
18	2M	1G	13M	1G	2M
21	2M	1G	17M	1G	2M
23	2M	1G	19M	1G	2M

M = Glasseidenmatten-M113-ES10-100(450g/m²)
G = Glasseidenrovinggewebe-6060-21(670g/m²)

Konstruktion des GFK-Ansaugkamines,Abb. 2

die außen angeordneten Ringsteifen auf, die neben der Biege-, vorrangig zur Torsionssteifigkeit, als Hohlkörper ausgebildet sind. Sie haben mehrfachen Zweck zu erfüllen, so z.B. bei Biegebeanspruchung des Kaminschaftes den Kreisquerschnitt zu stabilisieren, die ganze Zylinderröhre in Beulabschnitte zu unterteilen.

Zu ihrer Herstellung wurden vorgefertigte GFK-Schalen auf den bereits fertiggestellten Zylindermantel gelegt, anschließend über- und anlaminiert. Innerhalb der Ringbreiten werden die unterschiedlichen Manteldicken zwischen den Beulfeldern ausgeglichen.

Die am unteren Ende vorgenommene Kamineinspannung ist mit vorgespannten Klebeverbindungen (VK-Verbindungen) vorgenommen worden. Hierbei ist der GFK-Kaminmantel durch die Spannkraft einer Vielzahl von HV-Schrauben zwischen stählernen Ringteilen unter Zwischenschaltung von Klebstoff (Vestopal 400®)kraftschlüssig eingeklemmt. Als Vorteile dieser bewährten Verbindungsart kann angeführt werden:

a) die stählernen Haftflächen sind korrosionsgeschützt;
b) Herstellungs- und Montageungenauigkeiten lassen sich durch Klebstoff, der u.U. gemagert werden kann, ausgleichen;
c) durch die Schraubenvorspannung erhält die Klebverbindung hohe statische und vor allem dynamische Schubfestigkeit;
d) durch flächenhafte Krafteinleitung treten nur geringe Spannungsspitzen im Werkstoff GFK auf.

Der Klebanschluß ist derart konstruiert und bemessen, daß er bei späterer Aufstockung ohne Änderung als Montagestoß verwendbar ist.

4. Statische Berechnung und Schwingungsverhalten

Das zu untersuchende statische System ist eine freistehende, unten eingespannte Röhre von zunächst 30 m, in späterer Ausbaustufe 60 m Höhe. Beide Systemhöhen waren getrennt voneinander derart zu berechnen und zu bemessen, daß zur Aufstockung der erste 30 m-Abschnitt auf eine später gefertigte Röhre gesetzt und mit ihr biegesteif verbunden werden kann. Als Werkstoff wurde ein GFK-Matten-Laminat (30 % Glasfaseranteil GT) mit Vestopal 150® bestimmt.

Von der Werkstoffwahl und Konstruktion her ist dieses turmartige Bauwerk in die Reihe der Leichtbau-Schornsteine einzustufen. Für die Belastungsannahmen war neben der DIN 1056 für freistehende Massiv-Schornsteine auch die im Entwurf vorliegende DIN E 4133 (Jan.70) für Stahlschornsteine mit heranzuziehen. An Belastungen waren zu berücksichtigen:

a) Eigengewicht, 10 mm Eisansatz, Wind, 100 mm WS Unterdruck;
b) Temperaturdifferenz im GFK-Zylinderschaft und innerhalb der Zylinderwandung;
c) Zusatzspannungen durch Kaminbiegung infolge Wind und Temperatur;
d) Kaminschiefstellung durch Einwirkungen des untertägigen Bergbaues.

Die nach endgültiger Bemessung sich ergebenden GFK-Mantelspannungen für 30 m und 60 m Höhe sind mit den endgültig gemessenen Wanddicken in den jeweiligen Einspannquerschnitten auf Tabelle 3 zusammengestellt.

Zur Beurteilung des Schwingungsverhaltens mußte der Eigenfrequenz des Kamines besondere Aufmerksamkeit gewidmet werden. Für die Eigenfrequenz eines dünnwandigen Rohr-Balkens unter Berücksichtigung von Zusatzgewichten, wie Kaminhaube und Ringsteifen, gilt:

$$f_e = \frac{0.561}{l_2} \cdot R \cdot \sqrt{\frac{E g}{2 \cdot \gamma}} \cdot \sqrt{\frac{G_E/4}{G_E/4 + G_H}}$$

	30m hoch t-23mm [kp/cm²]	60m hoch t 39mm [kp/cm²]
Eigengewicht, Eisansatz	- 4,31	- 6,1
Unterdruck	- 0,54	- 0,54
Windbelastung	±108,0	±187,1
Temperaturdifferenz in der Röhre	± 21,8	± 21,8
Temperaturdifferenz in der Wandung	± 49,5	± 49,5
Spannungen II. Ordnung	± 3,7	± 15,9
max. σ	- 187,85	- 280,94

GFK-Mantelspannungen an den Einspannstellen, Abb. 3

Für 30 m und 60 m Höhe wurden die Schwingzahlen für Kurzzeit-E-Modul mit rund 100.000 kp/cm² und für Langzeit-E-Modul reichlich geschätzt mit 70.000 kp/cm² zunächst angenommen. Die rechnerisch ermittelten Frequenzen ergeben sich zu:

	Kurzzeit $E_K \sim 100.000\ kp/cm^2$	Langzeit $E_L \sim 70.000\ kp/cm^2$
fe (30 m) Hz	1,211	1,013
fe (60 m) Hz	0,328	0,275

Mehrfache exakte Schwingungsmessungen am aufgestellten 30 m-Kamin decken sich mit der errechneten Frequenz von 1,21 Hz. Diese Schwingungsmessungen werden in größeren Zeitabständen wiederholt werden. - Von der gerechneten und gemessenen Eigenfrequenz gegenüber einem kritischen Bereich von 3 bis 5 Hz her ist ein Aufschaukeln dieses Kamines durch Karmann-Wirbel nicht zu befürchten, - Zur Festlegung der zulässigen Spannungen im GFK-Kamin-Mantel mußten vorerst die kritischen Beulspannungen ermittelt werden.

5. Beulversuche axial gedrückter GFK-Kreiszylinderschalen

Die Bemessung dünnwandiger Kreiszylinderschalen setzt die Kenntnis der Beulsteifigkeit voraus. Seit langem liegen für derartige Schalen mit isotropen Werkstoffen zahlreiche Versuchs- und Untersuchungsergebnisse sowie Berechnungsmethoden vor, wobei die letzteren konstante E-Moduln, Querkontraktion und nur Spannungen unterhalb der Proportionalitätsgrenze voraussetzten. Da zwischen Theorie und Versuchsergebnissen ständig große Abweichungen auftraten, wurde die klassische Beulspannungsgleichung schon frühzeitig durch verschiedenartige Korrekturfaktoren, wie z.B. durch immer vorhandene Störungen infolge Vorbeulens, berichtigt:

$$\sigma_{krit.\,B} = \frac{1}{\sqrt{3(1-\mu^2)} \cdot \sqrt{1+R/100\cdot t}} \cdot E \cdot t/R$$

Diese und ähnliche Formeln können bei der gewissermaßen heterogenen Struktur des Werkstoffes GFK nicht in Ansatz gebracht werden, da je nach Laminataufbau erhebliche Eigenschaftsunterschiede in Längs- und Umlaufrichtung vorliegen und diese ohnehin von Beanspruchungsrichtung, -höhe und Temperatur Abhängigkeit zeigen. Um die Anisotropie durch unterschiedlich große E-Moduln im Laminataufbau zu erfassen, wurde die einfache Beulspannungsgleichung z.B. erweitert:

$$\sigma_{krit.\,B} = \frac{1}{\sqrt{3(1-\mu^2)}} \cdot E_x \cdot \sqrt{\frac{E_y}{E_x}} \cdot t/R$$

Da die hiernach errechneten Beulspannungen sich wenig mit den experimentell ermittelten deckten, wurde versucht, durch einen Beulkoeffizienten $c = f\ (R/t)$ eine Angleichung beider Ergebnisse zu erzielen:

$$\sigma_{krit.\,B} = c\, E_x \cdot \sqrt{\frac{E_y}{E_x}} \cdot t/R = c\,\sqrt{E_x \cdot E_y} \cdot t/R$$

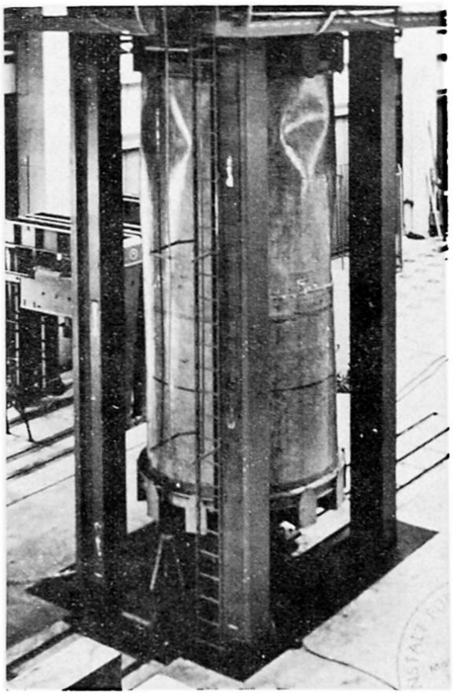
Auch dieser Versuch zeigte sich als nicht zuverlässig genug, so daß letztlich zur Ermittlung einer allgemein gültigen GFK-Beulspannungslinie für verschiedenartige Laminate Versuchsreihen vorgenommen werden mußten, wobei die Versuchskörper in möglichst großen Abmessungen hergestellt werden sollten. Diese Versuche erwiesen sich als dringend notwendig, da abgesehen von Einzelversuchen eine zusammenhängende Beulspannungslinie in einem größeren R/t-Bereich für diesen Werkstoff bisher nicht vorliegt.

Zur Herstellung größerer GFK-Zylinderröhren bedient man sich des Handauflege-, meist jedoch des Wickelverfahrens. Durch den Herstellungsvorgang bedingt ist der Laminataufbau durch Anordnung der Glasfaserbewehrung sehr unterschiedlich. Beim Handauflegeverfahren kommen, wie bereits erwähnt, durchweg Matten und Gewebe zur Anwendung, wobei mit höherem Mattenanteil gegenüber Geweben der GFK-Werkstoff zunehmend isotrope Festigkeitseigenschaften in der Schalenebene annimmt. Das Wickelverfahren sieht dagegen mehr Rovings zum eigentlichen Aufwickeln mit zwischengelegten Geweben vor, so daß die Glasfasern starke Richtungsorientierung aufweisen. Zur Erhöhung der Beulsteifigkeit axial gedrückter Zylinderschalen muß der Glasanteil in Längs- gegenüber Umlaufrichtung überwiegen. - Um beide genannten Herstellungsverfahren wirtschaftlich bemessen zu können, wurden zur Versuchsdurchführung die Laminate mit Vestopal 150® festgelegt:

a) Mattenlaminat mit rund 30 %igem Glasfaseranteil GT für Handauflegeverfahren (12 Prüfkörper);
b) Wickellaminat mit Glasfaserorientierung längs zu umlaufend (l : u) wie 2 : 1 mit rund 50 %igem Glasfaseranteil GT (hierzu Vergleichsuntersuchungen mit Orientierungsänderung 1 : 1 und 1 : 2 mit 14 Prüfkörpern).

Diese insgesamt 26 Prüfkörper mit vorwiegend Ø 1400 mm und Prüfkörperhöhe von 2800 mm wurden mit großer Sorgfalt und Genauigkeit hergestellt. Die unterschiedlichsten Wanddicken t mußten gleichmäßig, vor allem beim Wickellaminat, derart aufgebaut werden, daß unter Einhalten des vorgegebenen Glasgewichtsanteiles und festgelegter Faserorientierung sich auch die Festigkeitseigenschaften in Längs- und Umlaufrichtung nicht änderten. Diese Abstimmungsschwierigkeiten scheiden beim Mattenlaminat praktisch aus. Um die angestrebten Werkstoffestigkeiten zu erzielen, wurde auf entsprechende Aushärtung des Polyesterharzes streng geachtet und, soweit notwendig, mit Tempern nachgeholfen.

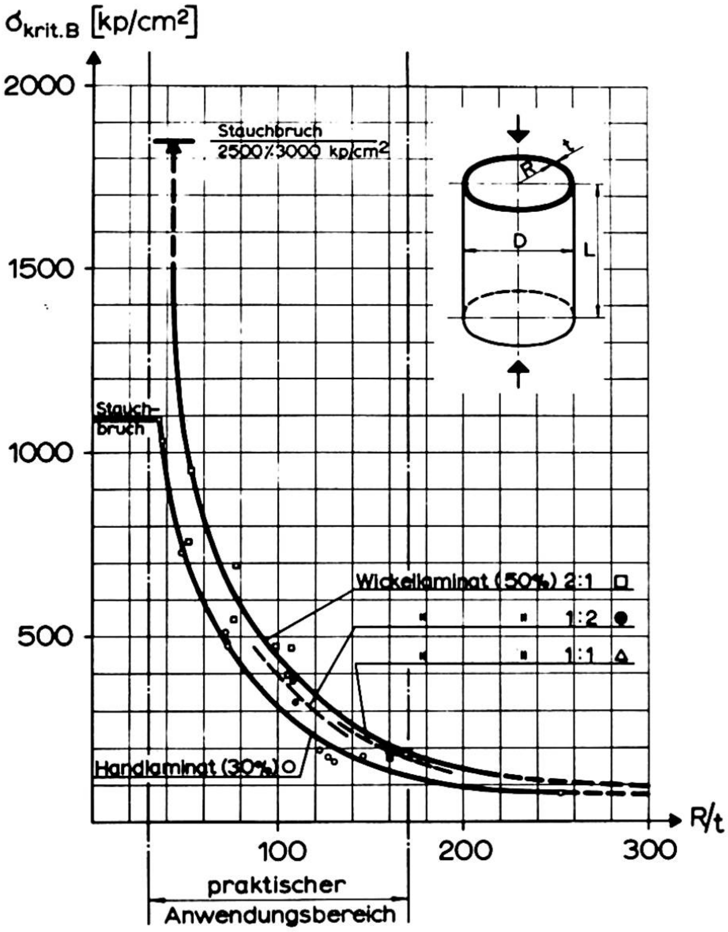
Die Belastungsversuche wurden im Institut für Stahl, Holz und Steine der TU Karlsruhe durchgeführt. Zur gleichmäßigen Lasteinleitung waren zuvor die Zylinderendquerschnitte planparallel abgedreht und diese zur Vermeidung von Randstörungen mit Holzschotten ausgesteift. Die zentrische Druckeinleitung wurde über Meßdehnungsstreifen kontrolliert, die in halber Zylinderhöhe jeweilig innerhalb und außerhalb der Ringviertelspunkte angebracht waren. Mit dieser Meßanordnung konnten während der stufenförmigen Belastung gleichzeitig die Verformungsänderungen und somit die E-Moduln und Querkontraktion bis in den Beulbereich exakt bestimmt werden. Auf der Abb. 4 ist z.B. die Versuchsanordnung an einem 5 m hohen GFK-Zylinder mit ∅ 2,50 m und Manteldicke t = 5 mm (R/t = 253) zu sehen.



Ausgebeulter GFK-Prüfkörper ∅ 2,45 m, H=5,0m, t=5mm unter zentrischer Drucklast, Abb. 4

6. Versuchsergebnisse

Die umfangreichen Ergebnisse dieser Stabilitätsversuche sind in ausführlichen Prüfberichten der TU Karlsruhe festgehalten, von denen hier nur auf die wesentlichsten kurz eingegangen wird. Die zur Beurteilung erforderlichen Angaben der Festigkeits-, Belastungs- und Verformungsuntersuchungen an den GFK-Kreiszylinderschalen sind in Tabellenform auf der Abb.5 für Mattenlaminat und auf Abb. 6 für Wickellaminat zusammengestellt. Im Rahmen von Stabilitätsuntersuchungen liegt hier eine überraschend geringe Streuung der Versuchsergebnisse vor, so daß mit großer Zuverlässigkeit die zugehörigen kritischen Beulspannungslinien auf der Abb.7 zur sicheren Dimensionierung aufgetragen werden konnten.



Kritische Beulspannungslinien für Matten- und Wickellaminat aus GFK, Abb. 7

Bei den dünnwandigen Prüfzylindern bis etwa R/t > 70 zeigten sich bis zur Durchschlagslast ausgeprägte Beulmuster. Mit größeren Wanddicken, d.h. kleiner werdenden R/t-Werten traten während des Beuleinfallens infolge örtlicher Überschreitung der Laminatfestigkeit Scher-

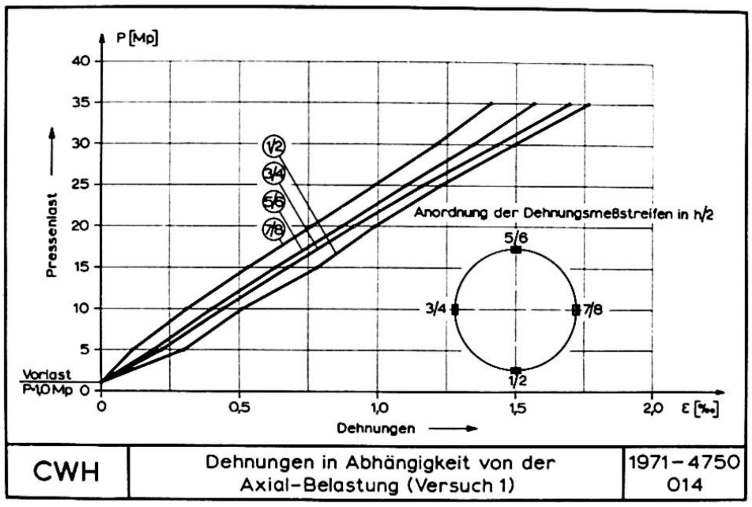
Prüfkörper Nr.	Radius/Höhe [mm]	t^m [mm]	R/t^m	Glasgehalt DIN E53395 % n.G.	σ_z DIN 53455 [kp/cm²]	Quer-kontraktion DIN E53395	Biege-E-Modul DIN 53457 [kp/cm²]	Druck-E-Modul [kp/cm²]	P_B [Mp]	krit. σ_B^m [kp/cm²]
1	700/2818	4,8	146	28,6	1330	0,365	82200	104.100	37,5	178
2	700/2800	5,5	127	26,0	1105	0,348	77400	94.000	42,5	176
3	700/2805	5,4	130	29,4	1435	0,381	76600	87.500	40,0	168
4	700/2800	9,8	71,5	30,6	1550	0,344	97400	108.600	225,0	522
5	700/2810	9,6	73,0	30,1	1220	0,332	97.700	105.200	200,0	474
6	700/2810	9,7	72,3	31,9	1480	0,344	97.000	101.100	206,0	484
7	700/2800	14,7	47,7	30,7	1502	0,351	97.800	94.000	470,0	727
8	700/2804	18,7	37,5	30,6	1213	0,361	102200	101.100	850,0	1030
9	700/2808	19,6	35,7	31,0	1090	0,360	111.100	182.000	940,0	1090
I	1225/4997	4,85	253					104.000	29,0	78
II	1231/5015	10,1	122					88.000	155,0	197

Abb. 5 Beulergebnis von GFK-Kreiszylinderschalen mit Mattenlaminat 1971–4750 013

Prüfkörper Nr.	t^m [mm]	R/t^m	Verstärkung L/U	Glasgehalt DIN E53395 % n.G.	σ_z DIN 53455 [kp/cm²] axial	Biege-E-Modul DIN 53457 [kp/cm²] axial	Biege-E-Modul DIN 53457 [kp/cm²] radial	Druck-E-Modul [kp/cm²]	P_B [Mp]	krit. σ_B^m [kp/cm²]
1	4,23	169						176.000	34,0	182
2	4,59	170	1,7:1	50,1	3276	106.900	109.300	206.000	33,5	190
3	4,29	160						189.000	33,0	174
4	6,68	107						211.000	130,5	468
5	7,05	105	2,1:1	54,7	3112	111.000	120.000	200.000	112,0	399
6	6,66	99						214.000	132,0	473
7	9,18	76	2,2:1	58,7	4032	153.300	119.500	228.000	222,0	546
8	9,30	77						228.000	285,0	693
9	13,52	52	2,3:1	54,6	3494	148.400	95.600	213.000	455,0	759
10	13,42	53						217.000	566,0	951
11	4,44	160						175.000	38,0	193
12	4,62	166	1:1,2	50,0	2540	102.600	160.600	155.000	37,0	181
13	8,08	102						154.000	155,5	438
14	8,13	104						149.000	161,0	448
15	7,40	108	1:2,6	45,8	2444	106.400	104.900	160.000	125,0	382
16	6,73	109						169.000	96,0	324

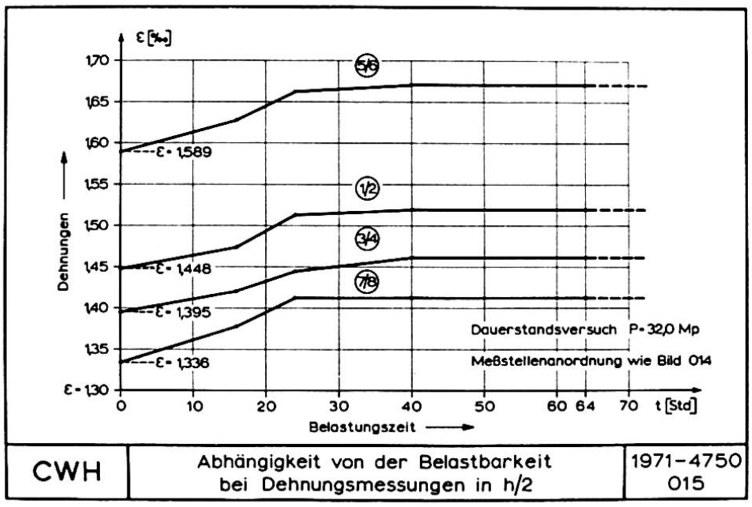
Abb. 6 Beulergebnis von GFK-Kreiszylinderschalen mit Wickellaminat Radius / Höhe = 700 / 2800 [mm] 1972–4750 017

brüche ein. Mit Aufbringen der Belastung verliefen die Stauchungen über die Zylinderhöhe sowie die über die Meßstreifen ermittelten Dehnungen bis zur kritischen Beulbelastung (krit. $\sigma_B \sim 700$ kp/cm^2) absolut gradlinig (Abb.8). Auch bei mehrmaliger Belastung bis zur Ausbeulung zeigte sich ein elastisches Verhalten und bestätigte erneut das außergewöhnlich große Erholungsverhalten des GFK-Werkstoffes.



Typisches Lastdehnungsverhalten eines GFK-Prüfzylinders, Abb. 8

Nachdem bisher die Beulspannungen durch langsame Steigerung, aber insgesamt kurzzeitige Belastung gefunden wurden, sollte auch das Langzeitverhalten unter konstanter Last beurteilt werden. Hierzu war der dünnwandige Prüfzylinder Nr. 3 mit 32 Mp, das bedeutet 80 % der kritischen Beullast, in ausgebeultem Zustand über 64 Stunden hinweg belastet worden. Das aufgetragene Belastungs-/Verformungs-Diagramm (Abb.9) zeigt nach den ersten 16 Belastungsstunden eine zusätzliche Längszusammenstauchung von 7,4 % und bleibt von da ab bis zum Belastungsende nach fast 3 Tagen praktisch unverändert.



Dauerbeanspruchung eines GFK-Zylinders mit 32 Mp, 80 % der kritischen Beullast, Abb. 9

Nach Aufstellung wird die eingespannte Kaminröhre überaus vorwiegend durch Windbelastung dynamisch beansprucht werden. Durch einen Dauerschwellbiegeversuch an einem Prüfzylinder mit $\sigma_u / \sigma_o = \pm 0{,}1$ konnten nach 424 x 10^3 Lastwechseln (33 Lastwechsel/Minute) keinerlei Schäden festgestellt werden. GFK-Prüfstreifen aus dem Druck-, Zug- und neutralen Bereich unmittelbar neben der Einspannstelle des Prüfkörpers nach Versuchsabbruch entnommen, zeigten keine Abweichung von den vorher bestimmten Null-Werten. - Nach diesen überzeugenden Versuchsergebnissen hatte die Dimensionierung der Kaminröhre unter Festlegung genügend hoher Sicherheit zu erfolgen.

7. Bestimmung der zulässigen Spannungen

Die Bemessung tragender Bauteile ist so vorzunehmen, daß ein Versagen durch Bruch, Fließen, erheblich bleibende Formänderungen des

Baumaterials oder durch instabiles Verhalten von Konstruktionsteilen mit Sicherheit vermieden wird. Hierzu werden die zulässigen Spannungen unter Berücksichtigung der Werkstoffeigenschaften, konstruktiven Durchbildung der Bauteile und Beanspruchungsart des ganzen Bauwerkes von den Versagensspannungen abgeleitet. Besondere Aufmerksamkeit hinsichtlich höherer Temperaturen, Feuerwiderstandsverhalten und allgemeiner Beständigkeit gilt den organischen Konstruktionsmaterialien. Die von Fall zu Fall zu bestimmenden Abminderungs- und Sicherheitsfaktoren 1/A, bzw. (A) oder S ergaben beim anstehenden Ansaugkamin aus GFK-Handlaminat folgendes Ergebnis:

Der Abminderungsfaktor A_1 dient der Berücksichtigung zeitabhängiger Verminderung von Beul- bzw. Bruchfestigkeiten gegenüber in Kurzzeitversuchen ermittelter Werte. Obwohl die Bruch- bzw. die krit.Beulspannungen eindeutig bekannt sind, wurde der Wert 0,5 (2,0) zugrunde gelegt. Diese hohe Sicherheit wurde auch seitens der genehmigenden Behörde als gut ausreichend angesehen, so daß die unter den allgemeinen Sicherheitsfaktoren S angeführten und hier zutreffenden Gesichtspunkte als mitberücksichtigt anzusehen sind.

Der Abminderungsfaktor A_2 beinhaltet die Alterungs- und Korrosionseinflüsse. Hinsichtlich der Alterung speziell mit den UP-Harzen VESTOPAL® liegen über gut 1 1/2 Jahrzehnte zuverlässige und gute Erfahrungen vor, so daß bei der in diesem Fall ohnehin geringen ständigen Belastung bei fachgerechter Fertigung eine Werkstoffalterung auf Jahrzehnte kaum zu befürchten ist. Bei erstmaliger Ausführung dieser Art wurde ein Minderungsfaktor von 0,9 (1,11) in die Bemessung eingeführt.

Der Abminderungsfaktor A_3 gilt dem Einfluß erhöhter Temperaturen, der beim GFK erkennbar ist. Bis zu Temperaturen um + 80°C ist je nach Glasfasergehalt eine Festigkeitsminderung von 10 - 15 % allgemein festzustellen. Da der Luftansaugkamin, wenn überhaupt, sich in den Breitengraden seines Aufstellungsortes auf 60° erhitzen kann, wurde eine Abminderung 0,7 (1,43) als ausreichend angesehen.

Der Abminderungsfaktor A_4 berücksichtigt nachteilige Fertigungseinflüsse, die sich vor allem bei kleineren Konstruktionsabmessungen stärker auswirken können. Der Schaftdurchmesser und die Mindestwandstärken konnten bei der hier vorgesehenen Fertigung exakt eingehalten werden. Letztlich unerforscht sind noch die interlaminaren Kohäsions- und Spannungsverhältnisse, die mit 0,8 (1,25) Abminderung beachtet werden sollten.

Die Sicherheitsfaktoren S_B gegen Bruch, S_P gegen Überschreiten der Schädigungsgrenze, S_I gegen Versagen durch Instabilität können gemäß den für Bauteile aus bekannten Werkstoffen abgeleiteten Werten angesetzt werden. Bei dem Ansaugkamin konnten die Sicherheitsfaktoren, soweit nicht schon unter Faktor A_1 berücksichtigt, vernachlässigt werden, da praktisch nur durch übermäßig hohe, kaum zu erwartende Windbelastung ein Tragwerksversagen eintreten und sich ohnehin der Werkstoffbruch nur mit Vorankündigung vollzieht. Die Gefahrenklasse, die einem derartigen Kamin zukommt, ist selbst bei einem vollständigen Einsturz gering.

Insgesamt stellte sich ein zusammengefaßter Sicherheits- und Abminderungsfaktor ein:

Abminderungsfaktor	1/A	A
A_1	0,5	2,0
A_2	0,9	1,11
A_3	0,7	1,43
A_4	0,8	1,25
A	0,252	3,97

Als zulässige Spannungen ergeben sich,
für Sicherung gegen Beulen

$$\sigma_{zul.} \leqq \sigma_{krit.\,B} \cdot 0{,}252$$

gegen reinen Stauchbruch ohne Berücksichtigung der Stabilität

$$\sigma^{D}_{zul.} \leqq 1050 \cdot 0{,}3 = \sim 300 \; kp/cm^2$$

und gegen reinen Reißbruch

$$\sigma^{Z}_{zul.} \leqq 1323 \cdot 0{,}3 = \sim 400 \; kp/cm^2$$

Mit diesen der Bemessung zugrunde gelegten zulässigen Spannungen ergeben sich z.B. die in Abb. 3 zusammengestellten Nennspannungen.

8. Bauaufsichtliche Zulassung

Baustoffe, Bauteile und Bauarten, die nicht allgemein gebräuchlich und bewährt sind, dürfen bekanntlich nur angewendet werden, wenn ihre Brauchbarkeit als Tragkonstruktion nachgewiesen ist. Dieses trifft für den hier zur Anwendung gelangten GFK zu, der zwar als Baustoff nicht erstmalig verwendet wird, für den aber noch keine offiziellen Vorschriften für Bemessung, konstruktive Behandlung und Prüfmöglichkeiten vorliegen. Nur eingehende Festigkeitsuntersuchungen können hinsichtlich der Bauverwendbarkeit Klärung schaffen und darauf aufbauend genügend hohe Sicherheit dem Bauwerk ausreichende Standfestigkeit verleihen.

Bei dem hier beschriebenen GFK-Kamin mit beachtlichen Abmessungen und verhältnismäßig hoher Beanspruchung wurde mit der wiedergegebenen Berechnungsart, der nach zuverlässigen Versuchen vorgenommenen Bemessung und den zugrunde gelegten Sicherheitsfaktoren nach Einschaltung des Prüfingenieurs offiziell das Baugenehmigungsverfahren eingeleitet, das sämtliche hierfür notwendigen Instanzen der BRD durchlaufen sollte. Am 18. Nov. 1971 wurde nach verhältnismäßig kurzer Laufzeit von der obersten Bauaufsichtsbehörde nach Konsultation des Institutes für Bautechnik, Berlin, und des Innenministeriums des Landes Nordrhein-Westfalen die Zustimmung zur Errichtung für den Einzelfall erteilt.

9. Werkstattfertigung und Montage

Die Kaminröhre mit 2,50 m Ø wurde in ganzer Länge von 30 m im Handauflegeverfahren stoßfrei hergestellt. Eine spannbare Stahlform von entsprechendem Durchmesser und 6 m Länge, die nach der Fertigstellung eines Röhrenabschnittes verzogen wurde, diente gewissermaßen als Gleitschalung. Stufenweise war zunächst mit einer Manteldicke von rund 5 mm die Röhre ganz gefertigt worden, um anschließend gemäß eines Laminatplanes die festgelegte Manteldicke mit vorgegebenem Laminataufbau zu bewerkstelligen (Abb. 10).

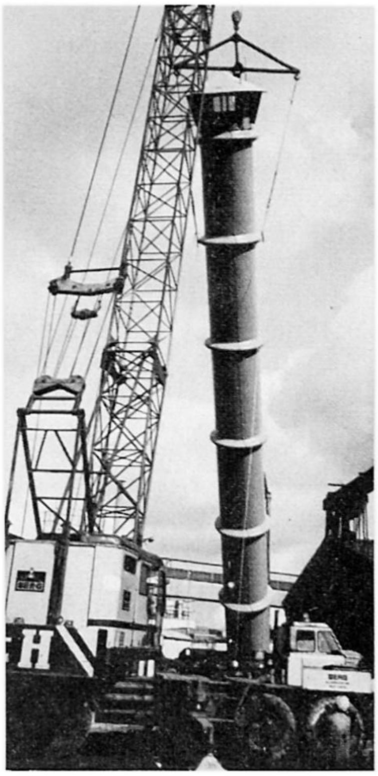
Werkstattfertigung der 30 m langen GFK-Röhre mit Ø 2,50 m, Abb. 10

Vor dem Auflegen der beiden letzten Matten wurden aus GFK vorgefertigte Viertelsegmente als verlorene Schalung, aber später mittragender Bestandteil für die hohlquerschnittigen Versteifungsringe aufgelegt. Die letzte Laminatschicht war über die Hohlringe zu ziehen, wodurch sich eine einwandfrei kraftschlüssige Verbindung zur GFK-Röhre ergibt. Die beiden äußeren Mattenlagen garantieren in Verbindung mit einem zweifachen Außenanstrich aus Vestopal 400® neben hoher Alterungsbeständigkeit einen wirkungsvollen Schutz gegen Witterungseinflüsse und aggressive Atmosphäre. - Um eine ausreichende Aushärtung des Reaktionsharzes zu erreichen, sollten nach der Laminatherstellung genügend hohe Temperaturen vorliegen, was in der hier zur Verfügung stehenden beheizten Werkstatthalle gewährleistet war.

Am unteren Abschluß des Kamines werden die Einspannkräfte über einen Stahlring in die GFK-Röhre eingeleitet, was, wie erwähnt, mit VK-Verbindungen vorgenommen wurde. Auf den inneren Stahlring ist der GFK-Mantel, der durch HV-Schrauben von außen angepaßt wird, zum genauen und lückenlosen Sitz anlaminiert.

Bei der Fertigung von GFK-Röhren derartiger Durchmesser im Wickelverfahren lassen sich mit den bisher bekannten Vorrichtungen nur Einzelschüsse von etwa 6 - 8 m herstellen. Das Verbinden der Einzelschüsse erfolgt in doppellaschigen Stumpfstößen mittels Matten- und Gewebelaminat, das zweckmäßig im Handauflegeverfahren ausgeführt wird. Die Ringversteifungen können auch bei dieser Fertigungsart entweder über Schaumstoffkerne oder dünne GFK-Schalen anlaminiert werden.

Der Transport zur Baustelle und die Montage des fertigen GFK-Kamines gestaltete sich in einem für den Bauingenieur bekannten Vorgang. Das Aufstellen des Kamines mit einem Gesamtgewicht von rund 6 Mp war durch einen Mobilkran in kürzester Zeit erledigt.



Montage des GFK-Ansaugkamines, Abb. 11

10. Schlußbetrachtung

Über eine Reihe von Jahren konnten Festigkeit und Beständigkeit in Kurz- oder Langzeit erforscht und umfangreiche Erfahrungen nach einer Vielzahl unterschiedlichster Anwendungen gesammelt werden, so daß grundsätzlich das GFK-Werkstoffverhalten weitgehend bekannt ist. Auch konstruktiv und verbindungstechnisch dürfen die auftretenden Probleme als gelöst angesehen werden. Speziell für turmartige Bauwerke liegen nunmehr die Stabilitätsergebnisse an GFK-Kreiszylinderschalen vor, so daß sich hier ein weites Anwendungsgebiet erschließen läßt, wenn die speziellen Werkstoffeigenschaften, wie z.B. leichtes Gewicht, hohe Alterungsbeständigkeit, insbesondere gegen Seewasser, und Isolierfähigkeit gegen elektrische Spannungen ausgenutzt werden. Funk- und Fernmeldemaste, Leucht- und Signaltürme für Luft- und Seefahrt, Kamine aller Art, Masten für Hochspannungsanlagen oder Windkraftwerke und dergl. können wahlweise eingefärbt, auf Jahrzehnte wartungsfrei hergestellt, leicht transportiert sowie schnell und einfach montiert werden.

Der weiteren Entwicklung wird es nunmehr überlassen bleiben, inwieweit bei der Verwirklichung derartiger Bauwerke der neue Baustoff GFK selbsttragend künftig eingesetzt werden wird.

Zusammenfassung

Neben den heute bekannten Baustoffen ist der glasfaserverstärkte Kunststoff (GFK) für tragende Konstruktionen einsatzbereit. Seine Werkstoffeigenschaften, Herstellungsverfahren und konstruktive Anwendbarkeit sind bekannt. Zur Berechnung und Bemessung eines 30 m (60 m) hohen Ansaugkamines waren Stabilitätsuntersuchungen notwendig, deren Ergebnisse allgemein verwendbar vorliegen. Speziell für selbsttragende Turmbauwerke gibt es bei den speziellen Eigenarten des GFK eine Vielzahl von Anwendungsmöglichkeiten.