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SEMINAR

V

Modelling of Structures

Modèles pour l'étude de structures

Modellbildung für Tragwerke

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Models in Engineering Science and Structural Engineering Design

Modélisation dans la science des ingénieurs et la conception des structures

Modellbildung in Ingenieurwissenschaften und Tragwerksentwurf

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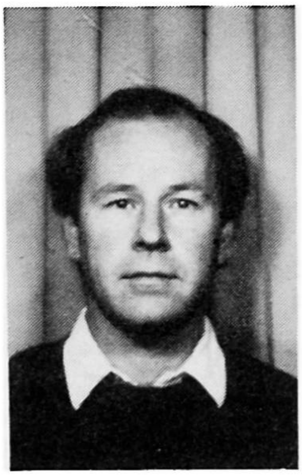

Bill Addis studied engineering and philosophy at Cambridge University. Following some years as a designer he has been researching into structural design and lecturing at Reading for 9 years.

SUMMARY
The paper considers the uses of mathematical and physical models by engineering scientists and engineering designers. The different uses are characterised by the different goals of these two professions. The criteria for choosing models appropriate to these goals are discussed, and suggestions are made for research into the effectiveness of different models in providing adequate justification of designs of structures.

RÉSUMÉ
Cet exposé traite de l'utilisation des modèles mathématiques et physiques par ceux qui s'engagent dans la science des ingénieurs et dans la conception et le calcul des structures. Les utilisations différentes s'expliquent par les buts différents des deux professions. On discute les critères pour le choix des modèles les plus appropriés. On propose quelques nouvelles voies de recherche pour évaluer l'efficacité des différents modèles dans l'étude de la conception des structures.

ZUSAMMENFASSUNG
Dieser Aufsatz behandelt die Anwendungen von mathematischen und physikalischen Darstellungen in der Arbeit der Ingenieurwissenschaftler und Tragwerks-Entwerfer. Die verschiedenen Anwendungen sind entsprechend den verschiedenartigen Zielen dieser zwei Berufe charakterisiert. Die Kriterien, nach denen Darstellungen gewählt werden, die den verschiedenen Zielen entsprechen, werden ebenfalls behandelt. Auch wird die Wirksamkeit der verschiedenen Darstellungen zur Rechtfertigung der Strukturentwürfe diskutiert und einige neue Forschungsthemen werden vorgeschlagen.

1. THE USE OF MODELS IN STRUCTURAL ENGINEERING

The use of models, both physical and mathematical, to represent the behaviour of a "real" structure has been well established in the field of structural engineering for at least 150 years.

During this period, however, relatively little has been written upon the aim or function of using models in engineering, in general, and in design, in particular. Their use has come to be rather taken for granted, not, of course, without justification, for they are indeed very useful.

In consequence, there has arisen a degree of confusion about the use and purpose of models. This confusion is typified by the developement, within a University research program, of sophisticated mathematical models relating to the behaviour of, say, a steel-framed building, which are not taken up by professional designers of such buildings. A gap is thus perceived between academics and practising structural engineers which is found to be difficult to bridge, despite many Government studies to seek more effective ways of putting the research work of Universities into practice in industry.

While there are no doubt many reasons for this state of affairs, perhaps the most significant is a fundamental misunderstanding about the activities of academics and designers. They are each engaged in different work with different aims - the one with engineering science, the other with engineering design; and this distinction seems to be more appropriate here than the more common distinction between theory and practice [1].

2. ENGINEERING SCIENCE

2.1 The Nature of Engineering Science

Engineering scientists are engaged in a branch of experimental or theoretical science with the same methodology and aims as other scientists, such as physicists. Their aim is to seek a deeper understanding of the world about them and to be able to explain the phenomena which they observe in it.

2.2 Models in Engineering Science

Engineering scientists use models in conjunction with theories or hypotheses as part of the process of explaining certain phenomena. They are also likely to be used in conjunction with an experiment designed in some way to test a hypothesis. This whole process constitutes the "scientific method" and has been thoroughly discussed by, for instance, Popper [2] and Kuhn [3]. Models in engineering science are almost invariably tested under laboratory conditions where the environment can be carefully controlled. Because of the complexity of the real world, both physical and mathematical models are usually much simplified and idealized to avoid the influence of many phenomena which might possibly interfere with the particular phenomena under scrutiny.

2.3 Criteria for Choosing between Models in Engineering Science

In a sense, the scientist's primary goal is to invent better and better models of the world in order to improve our understanding of it. The judgement as to just what constitutes a better model is influenced by a number of criteria of excellence, and by the relative importance ascribed to the different criteria [3], [4].

Examples of some criteria typically used by an engineering scientist might be:

- accuracy
- simplicity
- generality
- elegance
- ability to account for and explain past phenomena
- ability to predict the outcome of experiments

The result of these various circumstances is a rather protected environment in which certain highly specialized fields are investigated at a level which is simplified and idealized enough to be effective, using methods which are well understood. Nevertheless, despite being thus simplified, the work is often highly sophisticated, rigorous and complex - as reference to any of the many technical papers to which this type of work leads, will confirm.

3. ENGINEERING DESIGN

3.1 The Nature of Structural Design

The structural designer is faced with two distinct tasks:

- to conceive and describe a solution to a structural problem (the "specification" of a solution)
- to prove to the satisfaction of various persons, including the designer him/herself, that the solution is viable (the "justification" of a solution)

For both the specification and the justification, the designer may draw upon many types of knowledge and data, including intuitive knowledge of structural behaviour, precedent, empirical design rules and proof tests, as well as the use of mathematical and physical models of the structure or its parts. These different elements are taken into account and combined by means of a "design procedure", nowadays closely related to the Codes of Practice for the design of structures.

3.2 Models in Engineering Design

The role of models in structural engineering design is different from their role in engineering science by virtue of the different goals of the two activities. Designers do not have the luxury of being able to simplify and restrict their field of inquiry in quite the way that scientists can. They have to meet the challenge of a proposed building design by whatever means they can lay their hands on.

3.2.1 The use of mathematical models

Mathematical models form the basis of all the various techniques of structural analysis and need not be discussed here. The important point is that, according to the degree of simplification, and the way the designer conceives of the way in which the structure is behaving, so different models may be chosen. This possibility of choice will be discussed below.

3.2.2 The Use of Physical Models

Designers only resort to the use of physical models when they perceive a need to do so. This is likely to result from a direct knowledge or, sometimes, only a more subtle "gut feeling", that the use of design procedures based wholly upon mathematical models will be inadequate and not provide the required justification of the proposed design. This is typically the case for unusual structures, such as the Sydney Opera House, and for structures, such as suspension bridges, subject to complex and dynamic loading.

In these cases the model is designed with an intent very different to that of the engineering scientist. Rather than be as simple as possible to reduce the interferance of extraneous factors, it has to model the structure in the way most appropriate to providing the required justification of the proposed design. This has to be a compromise, somewhere between a totally accurate model, which would be a replica of the structure, and one that would be too simple to be convincing in its ability represent the relevant behaviour of the structure.

3.3 The Designer's Choice between Models

Within this wider context of a design procedure, the choices facing a designer about the use of mathematical or physical models are different from those which face the engineering scientist.

Designers are invariably faced with a structure which is much too complicated to model fully and in every detail. Even to attempt to do so would usually be inappropriately complex, and far too time consuming and expensive.

Designers consequently choose very simple models of the structure they are designing. For example, it is still common to design steel-framed buildings as a grid of columns to support vertical loads and a series of simply supported or partially restrained beams spanning between the columns; even in cases where the rigidity of joints is fully taken into account, it is common to ignore the extra strength and stiffness added by the system of enclosure used (e.g. cladding).

The use of such simple and "out of date" methods is wide open to the criticism of engineering scientists who are generally accustomed to using much more sophisticated mathematical models. And yet the structural design profession is notably reluctant to adopt the latest and most sophisticated techniques of structural analysis and design. One example is the almost total rejection of the results of the Steel Structures Research Committee recommending the adoption of more rational elastic design procedures [5]. Other examples are the continuing rejection by some designers of plastic and load factor design methods for steel structures [6] and of the recent more complex Codes of Practice for the design of concrete structures [7].

Such rejection of new methods is often with good reason, since the use of well-established methods has a strong attraction; but it belies an important underlying issue, namely the nature of the factors which influence a designer's choice of design procedure. A new model or new use of an old model will only be adopted by designers if some benefits of doing so are perceived.

3.4 Criteria for the Designer's Choice between Models

In the cases of both mathematical and physical models, designers are forced to make choices which engineering scientists do not have to make - choices about which model and which level of sophistication would be appropriate. Furthermore, these choices might be made according to criteria very different from the criteria used by engineering scientists when choosing between models in the pursuit of their goals. Or, while the criteria might be similar, the weight and importance they are given might be very different.

Thus, the criteria applied by designers to their choice of model might include:
- simplicity
- rationality
- speed
- cost-effectiveness
- accuracy
- reliability
- level of sophistication
- the power to justify a proposed design

The most significant difference between this list and the one given above, of the criteria applied by engineering scientists, is that it is more open to subjective opinion. At a particular time and place in history, the opinions, by and large, will lie within a narrow band of concensus which reflects the appropriate "engineering climatology" (as Pugsley calls it [8]).

4 THE CRITICAL APPRAISAL OF DESIGN PROCEDURES

The subjectivity of some of the opinions about the relative merits of the different models and their use in design procedures, ought somehow to lead to frequent and open debate on the subject: yet, in general, it does not. Three exceptions to this norm have interesting conclusions.

In the 1930s the Steel Structures Research Council was set up to review the current elastic design methods for steel framed buildings. It concluded that "the method of design of steel-framed buildings in common use had no firm rational basis" [4]: and yet the more rational methods proposed were strongly rejected by the structural design profession because they were more complex and did not yield significantly better, safer or cheaper structures.

Some 20 years later a different type of survey was contained in the Report of the Conference on the Correlation between Calculated and Observed Stresses and Displacements in Structures [9]. This conference aimed to present the results of tests on structures, some real, some part-structures and some models, in the hope of providing designers with better methods of designing structures. The results were, however, extremely confused, presenting neither the current state-of-the-art of engineering science, nor of engineering design procedures. This confusion was largely because the criteria for assessing the different types of results by the two different professions (scientists and designers) were neither stated nor discussed. The conference left the designers with no clear advice as to how to design better, safer or cheaper structures, and the engineering scientists appeared entirely happy that their duties had been adequately discharged in performing highly specialised research of little direct interest to the designers.

The final example comes from a paper in which a contributer noted that a reinforced concrete slab designed to the very latest procedure was not obviously safer or better and yet cost some 13% more than if it had been designed according to the out-of-date Code of Practice [10].

5. CONCLUSIONS

This paper has sought to draw attention to the need to look carefully at the different actities and goals of engineering scientists and designers. These differences are particularly important when considering the use of both mathematical and physical models and the criteria for choosing between different models which could be used.

The main conclusion of the paper is that there is an important area of research to be pursued - the critical appraisal of models and the criteria for choosing between them, particularly in the field of structural engineering design. For design procedures, the relative merits of, for instance, simplicity and accuracy, or cost-effectiveness and sophistication, need to be evaluated.

Perhaps most importantly, the role of models in engineering design needs to be more fully investigated and understood. Many structural designers are able to design entire structures on the sole basis of their experience and without any recourse to the use of models and structural analysis. They are then required to justify these designs by means of models. The processes of justification could be made more acceptable and reliable if there was a clearer understanding of the powers of justification of the various means outlined above - precedent, the use of mathematical models of different complexity and sophistication, and the testing of physical models.

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Structural Information via Modal Testing

Information structurale par des essais modaux

Tragwerksinformation durch Modalversuche

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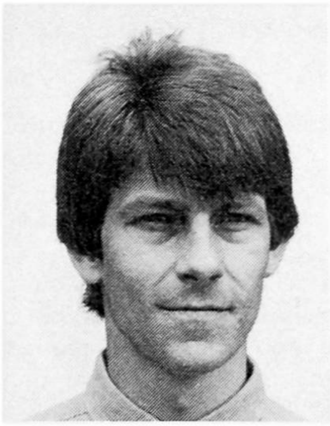

Sven Ohlsson, born 1952, received both his civil engineering degree and his doctoral degree at Chalmers University of Technology. His research work has been focused on structural problems for buildings and bridges and especially dynamic ones. He is presently leading a research group for dynamic structural problems at Chalmers.

SUMMARY
Modal testing of structures is a powerful experimental technique for dynamic problems. It is based on Fourier transformed time records and curve-fitting of experimentally obtained frequency response functions. This paper deals with different possibilities to use this technique as a tool for non-destructive testing. The tests aim at determining static structural properties including buckling loads. A pilot experimental study of columns is summarized as well.

RÉSUMÉ
L'essai modal de structures est une technique expérimentale efficace pour des problèmes dynamiques. La méthode utilise une transformation de Fourier d'enregistrements des fonctions de temps pour créer des fonctions de réponse de la fréquence. Cet article traite des possibilités d'utilisation de cette technique comme un outil pour des essais non destructifs. Le but des essais est la détermination d'attributs structuraux statiques et les charges critiques. Une étude expérimentale préliminaire de colonnes est aussi présentée.

ZUSAMMENFASSUNG
Die modale Versuchstechnik für Tragwerke ist ein wirkungsvolles experimentelles Verfahren für dynamische Probleme. Dieses Verfahren geht von fouriertransformierten Zeitaufnahmen und Kurvenanpassungen von versuchstechnisch ermittelten Frequenzantwortfunktionen aus. Dieser Artikel behandelt verschiedene Möglichkeiten der Verwendung dieses Verfahrens als ein Werkzeug für zerstörungsfreie Prüfung. Das Ziel der Versuche ist es, statische Tragwerkseigenschaften einschliesslich der Knicklasten zu bestimmen. Eine experimentelle Vorstudie anhand von Stützen wird kurz zusammengefasst.

1. INTRODUCTION

Fourier transform based experimental modal analysis has shown an inherent capability to enable reliable and accurate estimations of modal parameters. Repeated experiments have yielded results with exceptionally small scatter. This is especially the case for eigenfrequency estimates, where almost identical results can be obatined from tests with different kinds of dynamic loading, different location of measurement points and so on. The experience is gained within various research projects related to bridges |6|, floors |5| and other civil engineering structures. Modal testing based on impact loading is rather easily performed, at least compared to static tests, which typically need rather heavy equipment and arrangements.

Numerous relevant questions concerning structural properties and loading conditions are very difficult and costly to answer by means of traditional static testing. This is especially true for civil engineering structures in use, but it is also valid for a variety of questions dealing with properties of structural details.

One important class of problems includes the question: "What static stiffness will a specific structural member or joint show under a certain kind of load?" Rotational rigidity of column footings (bolted steel or wooden members) and deflection stiffness of beam members of unknown concrete quality or with unknown amount of cracks are common examples of desired structural information. The unknown degree of semi-rigidity inherent in many connections, which are assumed to act as simple supports, defines another class of problems.

A third type of problems is related to axially loaded slender structures and members, where instability is a major concern. The dynamic characteristics of such members are dependent on the compression force present, cf. |8| and |9|. This fact can be used in the following situations:

a) The structural properties are known and modal testing can be used to estimate the (unknown) existing axial load.

b) Provided that the present load and the structural configuration are known, some estimates of the elastic critical buckling load can be made.

The idea to use the modal testing concept to estimate static structural properties has grown out of the above-mentioned observations about the accuracy inherent in the technique. A research project is under development at Chalmers University and the main aim of this paper is to introduce the basic ideas.

2. MODAL TESTING

Modal testing is a concept, which here refers to a technique for dynamic structural testing aiming at estimates of one or more of the modal parameters: eigenfrequency f_n, modal relative damping $(c/c_{cr})_n$, mode shape function $\Phi_n(x,y)$ and modal mass m_n. Such estimates are usually established for a limited number of eigenmodes. Modal testing typically includes the following phases:

- Choice of representative measurement points and directions.
- Dynamic broadband excitation by means of impact or random force applied at one point of the structure.
- Measurement and parallel recording of force and response time histories. The response is typically measured as acceleration.
- Fourier transformation of force and acceleration records respectively yielding complex spectral representations.

- Dynamic acceleration flexibility functions - *accelerances* - are established by division of spectra. The accelerance (point or transfer) is a complex function of frequency.
- The process described above is repeated for other combinations of loading point and point of response, resulting in a set of accelerances.
- Using theoretical accelerance functions based on the theory of modal analysis of linear systems, a curve-fit procedure is carried out. When minimizing least squares of errors, estimations of the above-mentioned four modal parameters can be made. An example of mode shape estimates is shown in fig. 1. Flexibility type functions are illustrated by fig. 2 . Reference |1| is a rather complete presentation of experimental modal analysis.

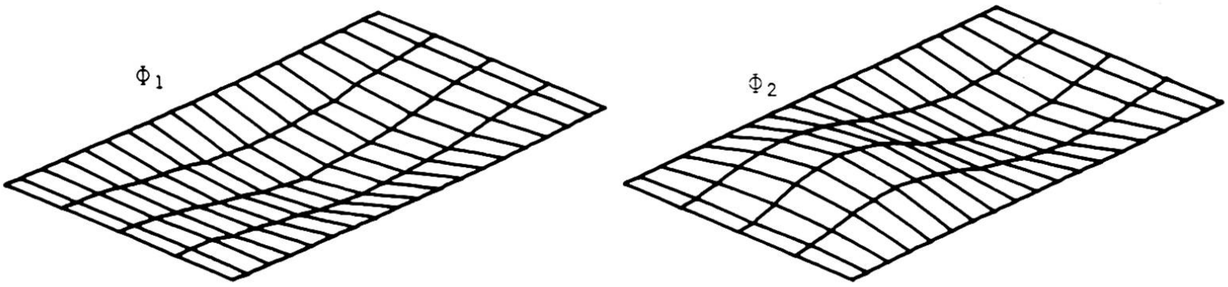

Fig. 1 Measured mode shape functions Φ_n for a corrugated web of a steel plate girder. Result from curve-fitting of 94 transfer accelerances and one point accelerance.

3. ESTIMATING AXIAL LOAD AND BUCKLING LOAD

3.1 Basic theory

Consider a simply supported beam with constant cross section. The eigenfrequencies f_n for corresponding bending modes $\Phi_n(x)$ are calculated from Eq.(1):

$$f_n = (n^2\pi/2)\sqrt{EI/mL^4} \quad \text{(Hz) or } (s^{-1}) \tag{1}$$

where n is the mode number, EI is bending rigidity, m is mass per unit length and L is span length. If a constant static axial force N is added, the corresponding eigenfrequencies $f_n(N)$ of the resulting beam-column can be written:

$$f_n(N) = f_n(N=0)\sqrt{1 - \frac{1}{n^2}\frac{N}{N_{el}}} \quad (s^{-1}) \tag{2}$$

where $f_n(N=0)$ is the eigenfrequency of the unloaded beam as calculated from Eq.(1) and N_{el} is the elastic buckling load:

$$N_{el} = \pi^2 EI/L^2 \quad \text{(N)} \tag{3}$$

Equation (2) can serve as a basis in a non-destructive test approach. Some of the eigenfrequencies $f_n(N)$ can be measured for a given, but unknown, axial force N. As apparent from Eq.(2), the influence from axial force on the eigenfrequencies is stronger for lower modes of vibration. The relation between two successive eigenfrequencies can be derived from Eq.(2) as:

$$\left(\frac{f_{n+1}(N)}{f_n(N)}\right)^2 = \frac{(n+1)^2}{n^2}\,\frac{(n+1)^2 - (N/N_{el})}{n^2 - (N/N_{el})} \tag{4}$$

This expression can be used to *estimate the present axial load* as a fraction of the Euler buckling load N/N_{el} based on measurements of two successive eigenfrequencies, preferably f_1 and f_2.

Another way to use Eq.(2) is by repeated measurements of one eigenfrequency f_n. The first measurement gives a value $f_n(N=N_0)$ for the existing unknown axial force N_0. A well-known additional force δN is then applied. The second measurement gives a value for $f_n(N_0+\delta N)$. If both sides of Eq.(2) are squared, the resulting relation is a straight line in a diagram like the one in fig. 3. The slope of this line is defined by:

$$df_n^2/dN = f_n^2(N_0+\delta N) - f_n^2(N_0))/\delta N \qquad (5)$$

The knowledge of this slope value enables us to estimate the axial load N_R, which is the difference between the present load N_0 and the buckling load N_{el}:

$$N_R = -f_n^2(N_0) \cdot dN/df_n^2 \qquad (N) \qquad (6)$$

The additional force δN could in a real situation be applied by means of an added mass on top of a column. The fundamental mode eigenfrequency is the most sensitive to a given small additional force.

3.2 Laboratory tests

This section is a presentation of a pilot study. The aim is to study how accurately the load estimates discussed in the previous section can be made. The test specimen is an aluminium column with rectangular hollow cross section, which is simply supported with span length of 1.20 m. One of the supports is equipped for application and measurement of axial static force, cf. fig.2. Accelerances are established by means of hammer impact modal testing for 13 different levels of

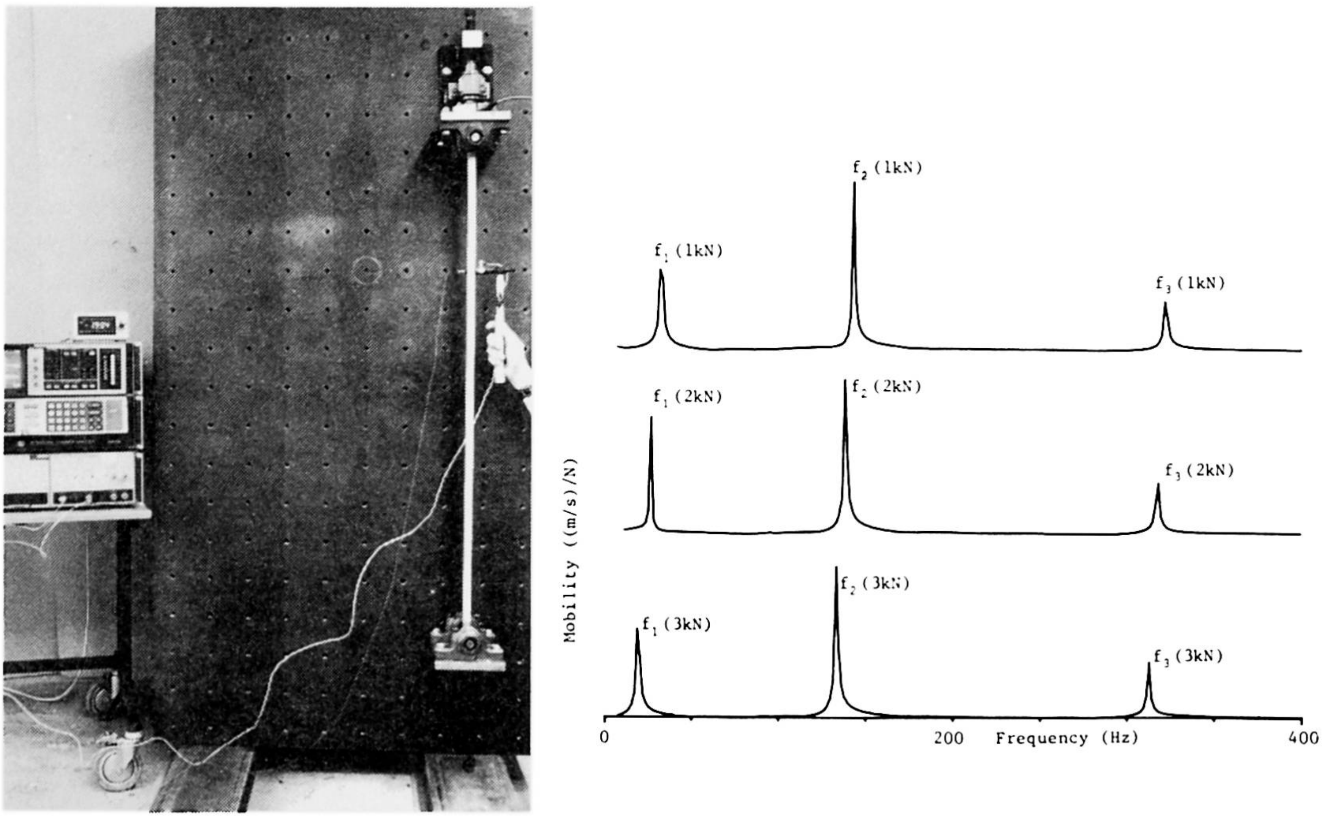

Fig. 2 Test set-up (left) and measured point mobilities related to three different levels of axial force N = 1, 2 and 3 kN respectively (right).

axial force N. The accelerances are transformed to mobilities (vibration *velocity*/force) and some are shown in fig.2. The three lowest eigenfrequencies are determined via curve-fitting for each load level. The fundamental frequencies are plotted in fig.3. The straight line approximation results in an Euler buckling force N_{el} = 4.16 kN. The two load levels N_0 = 1.50 and $N_0+\delta N$ = 1.60 kN can be used to test the approach related to Eq.(5) and (6). Eq.(5) gives:

$$df_1^2/dN = (28.76^2 - 29.34^2)/100 = -0.3370 \tag{7}$$

and Eq.(6) gives:

$$N_R = 29.34^2/0.3370 = 2554 \text{ N} \tag{8}$$

This estimate could then be compared with the difference $N_{el} - N_0$ = 4160 - 1500 = = 2660 N. The estimate is then appr. 4% lower than this value, which must be considered as promising.

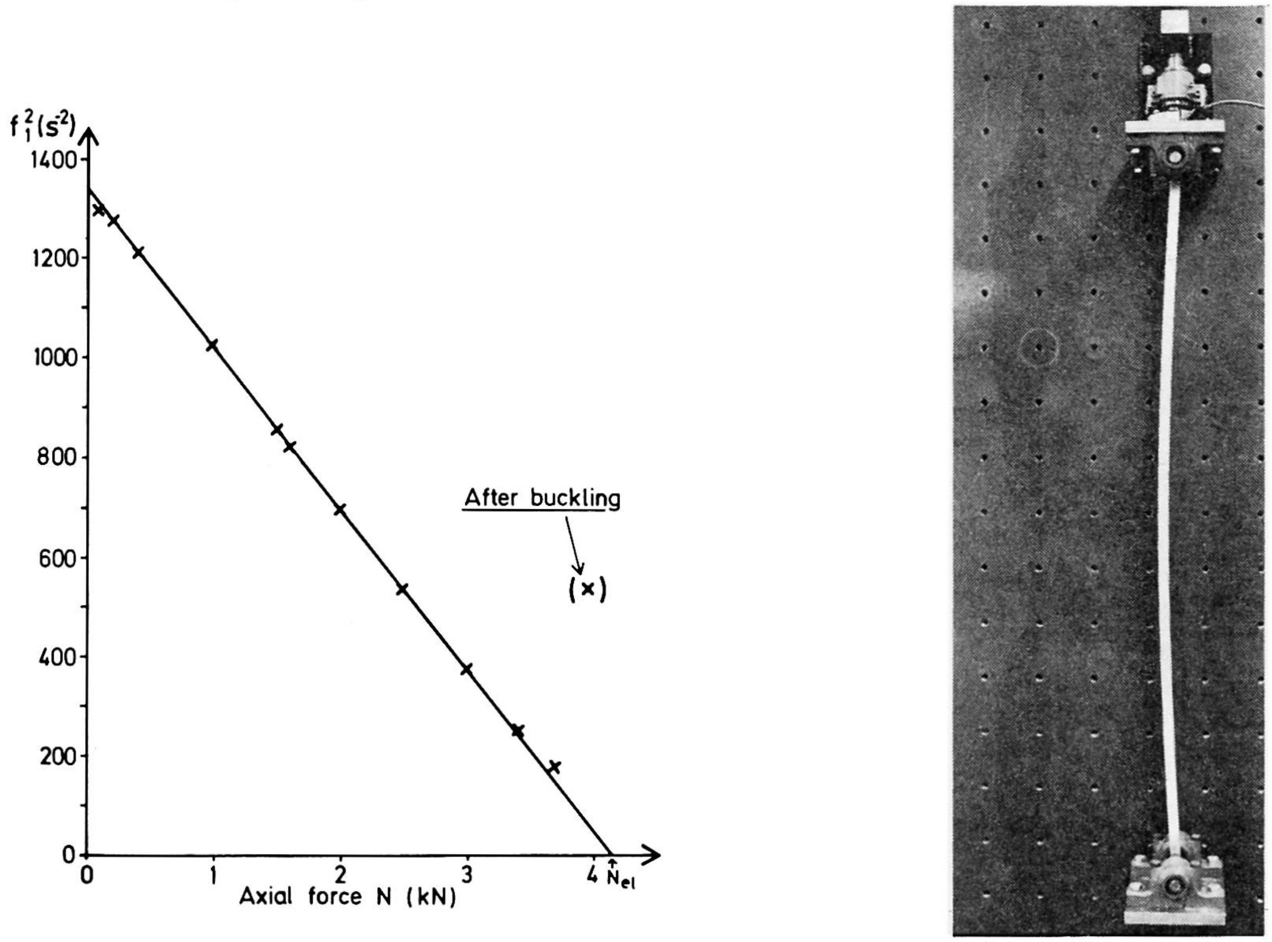

Fig.3 Plot of experimentally determined fundamental mode eigenfrequency (squared) versus axial force N (left) and illustration of buckled specimen (right). Experimental buckling load is approximately 4.0 kN.

4. CONCLUDING REMARKS

The approach to column force estimation problems illustrated in chapter 3 only included one of the four modal parameters - the eigenfrequency. If mode shapes and modal masses are taken into account as well, more complex structures may be handled and with still better accuracy. This holds also for the other kinds of problems, where bending rigidity or the degree of end rotational rigidity are sought. The references illustrate some practical problems that may be handled with non-destructive modal testing. One important possibility is to use repeated tests over time, aiming at detection of structural changes due to ageing, fatigue or material deterioration, cf. Ref. |3| and |10|.

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Model Techniques for Determining Loads and Responses of Hydraulic Structures

Modèle pour la détermination des charges et du comportement d'ouvrages hydrauliques

Modelle zur Bestimmung von Lasten und Verhalten bei hydraulischen Anlagen

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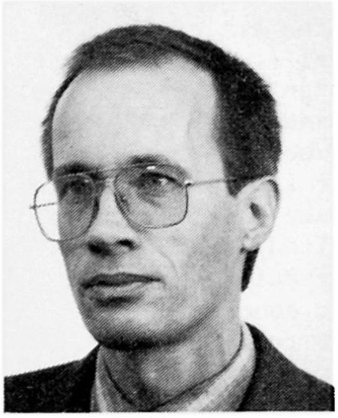
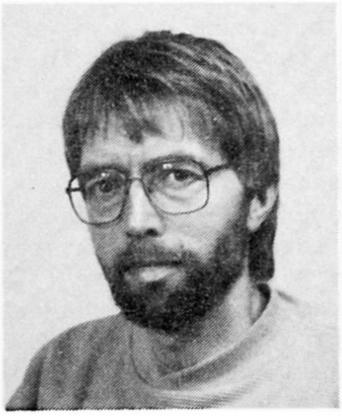
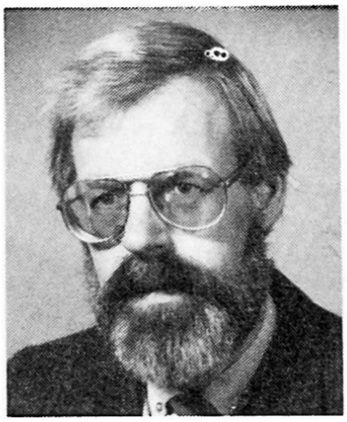


Rob de Jong, born 1946, received his civil engineering degree of the Technical University Delft. Since 1968 at Delft Hydraulics and involved in investigations on hydraulic structures.



Tom Jongeling, born 1947, received his civil engineering degree at the Technical University Delft. Worked till 1976 at the National Physical Laboratory, since 1977 at Delft Hydraulics and involved in research on gates with special emphasis on mathematical models.



Henk van der Weijde, born 1945, received his mechanical engineering degree at the Technical University Delft, since 1967 at the Bridge Department of the Ministry and project manager of the gates of the Oosterschelde Storm surge barrier.

SUMMARY
This paper outlines the use of scale models and numerical models to investigate the dynamic response and the resulting stresses in hydraulic structures caused by hydrodynamic loads e.g. flow and wave-induced forces. Numerical models provide additional analyzing techniques and it is stated that in the case of complex structures preferably both modelling techniques should be applied in a hybrid approach.

RÉSUMÉ
Cet article présente l'usage des modèles réduits et numériques pour l'étude du comportement dynamique et des tensions qui résultent des forces hydrauliques dues aux courants et vagues – sur les ouvrages. Les modèles numériques offrent des possibilités supplémentaires d'analyse. Dans le cas des ouvrages complexes, les deux techniques sont à utiliser d'une façon complémentaire.

ZUSAMMENFASSUNG
In diesem Artikel ist die Benutzung der Untersuchungen mit Modellen und numerischen Berechungen zu resultierenden Schwingungsformen und Spannungen hydraulischer Kräfte dargestellt. Numerische Berechnungen geben die Möglichkeit zusätzlicher Analysen. Es empfiehlt sich im Fall einer komplizierten hydraulischen Anlage die beiden Techniken zu kombinieren.

1. INTRODUCTION

The range of scale models that can be used in hydraulic engineering was greatly enlarged by the introduction of the elastic similarity models in about 1960. This development was stimulated by the Delta Project and the canalisation of the Rhine in the Netherlands, where large control gates were used. Economic considerations demanded a new approach to design procedures. The application of elastic similarity models in hydraulic engineering had been hampered prior to this by the lack of suitable modelling materials. These materials have to meet special elasticity requirements in addition to low internal damping and linear properties if they are to fulfil the scale laws, which are dictated by free surface flow. In addition the recent availability of computer simulation techniques has resulted in a wider application of dynamic models for hydraulic structures.

2. INVESTIGATION STRATEGY FOR THE DYNAMIC BEHAVIOUR OF THE GATES OF THE OOSTERSCHELDE STORM SURGE BARRIER

Hydraulic loads and the dynamic behaviour of hydraulic structures are often strongly related to the shape of the structure. In a scale model study this might necessitate reshaping of the model in order to improve the design. Vibrations due to feed back phenomena can be dangerous and are not always reproduced satisfactorily using simple dynamic models. Elastic similarity models are a better research tool. However, because modifications to elastic similarity models are cumbersome, these models are usually preceeded by more simple models such as single mass spring models. This investigation strategy was applied for the gates of the Oosterschelde storm surge barrier (Figure 1), but because of the complex fundamental modes and the interaction with the adjacent elastic concrete beams (see Figure 1) the initial, simple, models could not contribute to an understanding of the dynamic behaviour as they normally do. Early in the design process, therefore, the possibility of using elastic similarity models for two gates (with different heights) to finalize the design, rather than to check it had to be considered. The progress made on scale modelling techniques and on numerical modelling techniques made it possible to got for such an approach. The elastic models were therefore designed in such a way that the fundamental mode frequencies (in the vertical and horizontal bending and torsion directions) could be changed without drastic reconstruction of the models (e.g. diagonals were initially fixed by screws). The effects of schematizing details of the model gates could also be predicted by using FEM models of the steel gates and the model gates. In this approach mathematical models of the gates proved to be indispensible and the need for scale models to check the validity of the mathematical models was essential.

3. SCALING TECHNIQUES

3.1 Scaling laws for elastic similarity models of hydraulic structures

Scaling techniques for elastic models of hydraulic structures are described in literature [1,5]. The most important scaling laws are summarized briefly below. Elastic similarity models with free surface flow must reproduce both the Cauchy and Froude numbers and when the length scale of the model is n_L, this produces the following scale factors:

Fluid:

(1) $n_a = n_g = 1$ (a=acceleration; g= gravitational acceleration)

(2) $n_v = n_L^{0.5}$ (v=flow velocity)

(3) $n_t = n_L^{0.5}$ (t=time)

(4) $n_p = n_\rho \cdot n_v^2$ (p=pressure; ρ=specific density)

Elastic structure:

(5) $n_\omega = 1/n_t = n_L^{-0.5}$ (ω=angular frequency)

(6) $n_\varepsilon = n_\sigma / n_E = 1$ (ε=relative deformation, σ=stress) (E=modulus of elasticity)

(7) $n_m = n_\rho \cdot n_L^3$ (m=mass)
(8) $n_\gamma = 1$ (γ =ratio of damping to critical damping)

In general the vibration of a structure in water produces pressure variations on the structure which can be broken down into components which are in phase, out of phase or in opposite phase with the structure displacements. These pressures (or interaction forces) appear as rigidity, damping or mass components in the equation of motion and are referred to as "added terms". In general this effect is always present to some extent.
This means for instance that the scaling law for the mass (7) holds for the total mass, i.e. $m_{struct.} + m_{water}$. In the case of geometrical reproduction the added forces will be reproduced correctly by the water and it is only necessary to pay special attention to the reproduction of the structural mass, rigidity and damping.

The scaling laws given above are used when modelling structures. For instance if $n_E = n_\rho = 1$ is selected, it follows that $n_\varepsilon = n_v^2 = 1$, which means that a geometrical model, which is tested in the same fluid and with the flow velocity values which occur in reality will correctly reproduce deformations of the structure. This, however, is in general not possible in hydraulic models.
Combination of (4) and (6) with $n_\rho = 1$ gives an additional condition:

(9) $n_L / n_E = 1$

In practice there is no material which satisfies all conditions, but often a solution can be found by adapting the material thickness in the model. DELFT HYDRAULICS has gained a considerable amount of experience with Trovidur (R), a kind of PVC, which has low damping and a fairly linear elasticity.

3.2 Oosterschelde models [3,4]

A scale factor n_L = 40 was chosen for the Oosterschelde storm surge barrier scale models. The PVC material has an n_E value of 60, and therefore the plate thicknesses were increased by a factor of 1.5. The low weight of the plastic material was compensated for by attaching concentrated lead blocks to the plates in such a way that moments of inertia were correctly reproduced without significantly influencing the fluid flow.

Since the gate structure had a low torsional stiffness to prevent it from jamming in situ, the retaining plate structure, the vertical support system (trusses) and connections were important details of the design. FEM models of the two model gates were made to facilitate correct schematization of the scale models. Comparison of results obtained with the scale and numerical models led first to a refining of the numerical models and then to an adaptation of the scale models. It was possible to simulate the bending stiffness within a few percent and the torsional stiffness to within 10 percent of the values indicated by the FEM models.

4. USE OF MODELS IN THE DESIGN PROCESS

Important hydraulic phenomena in the design of the gates were vibrations, quasi-static wave loads and above all wave shocks. These wave shocks could easily occur on the plate girders, which were designed on the sea side of the vertical retaining plate, and were thus exposed to sea waves. The research on these wave shocks is discussed in detail below in connection with the use of hydraulic models. Reference should also be made to [2,3,4].
A hybrid investigation program was set up to determine design wave shock loads using:

A) stiff models equipped with pressure transducers to measure local pressures and to enable the type of wave shock to be identified, wich was important in view of the choice of the scaling rules, see Figure 2.

B) elastic similarity models to obtain information about the shock momentum in all fundamental response modes, see Photograph 1.
C) FEM models of the steel gates to investigate the overall response and the local steel stresses due to wave shocks.

The wave shock loads on the steel gates (in a time domain) were deduced from A and B. In order to use the numerical models of the steel gates added mass components had to be incorporated. Because the added mass phenomenon is related to modes of the structure, it could not be simulated as an invariable mass. A computer program, based on potential flow was used to establish the three-dimensional added mass matrix for the gate structure (see Figure 3). The method was already in existence but had not been used for thin walled structures with water on both sides of the gate. In this case the scale model results could be used to check the validity of the added mass computer program. The response of the gate to wave shocks could then be computed in various vibration modes, see Figure 4.

The study enabled two concepts to be compared for the main support system of the gates: one with easy to manufacture flat steel plates but inviting heavy shock loads, and therefore needing too heavy gates (see Figure 1), the second with truss girders but with greatly reduced shock loads (see Figure 5). Because of this aspect the girder design with a truss on the seaward side was chosen.

5. CONCLUDING REMARKS

- Elastic similarity models can be used as a final check on the dynamic behaviour of slender or thin-plated hydraulic structures. If stiff models are used to obtain information about local pressures caused by wave shocks, elastic similarity models can be used to measure the shock momentum on the main support system.
- FEM models can be used to analyse the local dynamic stresses which result from hydraulic shock loads and with these models the design of complicated hydraulic elastic similarity models can be refined. Results of scale models can also be understood more easily by analysing vibration modes with FEM calculations. Comparison with scale model tests may be necessary in order to refine FEM models of complicated structures.
- Fluid-structure interaction effects should be incorporated into numerical models. DELFT HYDRAULICS is collaborating with the National Physical Laboratory (TNO) in the implementation of a fluid module in the FEM DIANA package, which will enable a full dynamic analysis to be made of structures including interaction effects.

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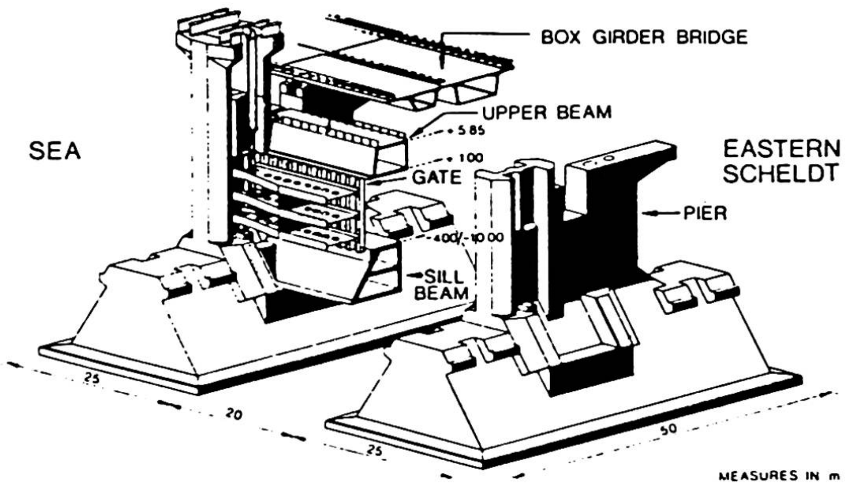


Figure 1

Elements of the Oosterschelde Storm surge barrier.
In three channels 66 piers, forming 63 apertures, have been placed on 45 m center to center line distances.

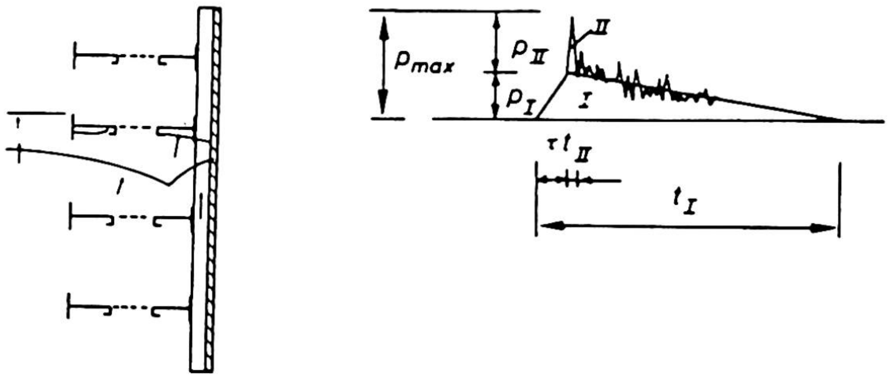


Figure 2

Wave impact and schematization as decided with the scale model studies.

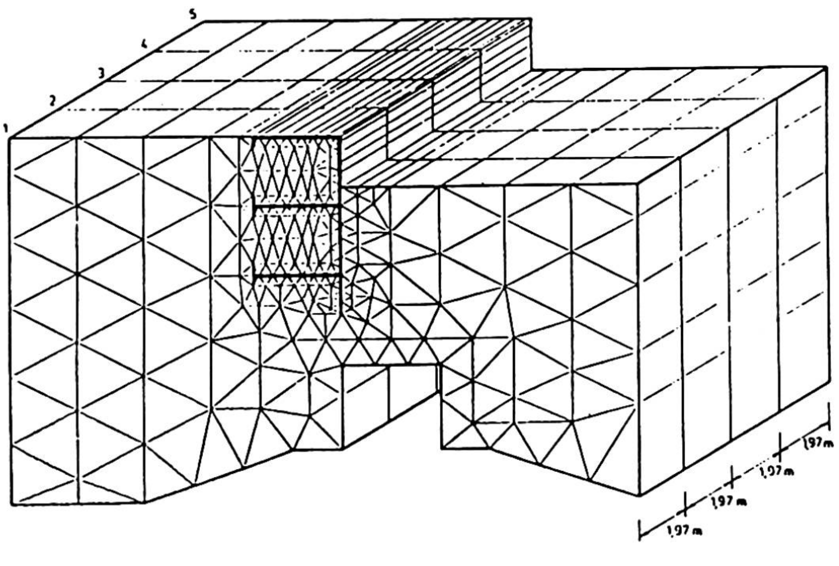


Figure 3

Four segment FEM model for computing the added mass mabrix of a gate with plate girders.

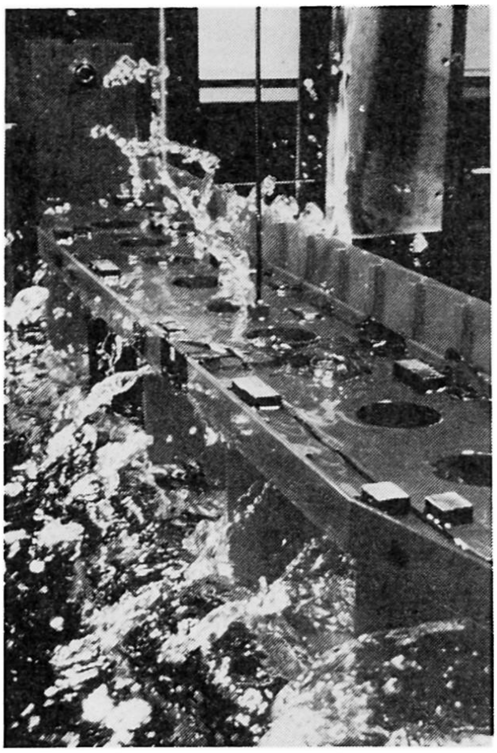


Photo 1

Wave impact of the elastic scale model

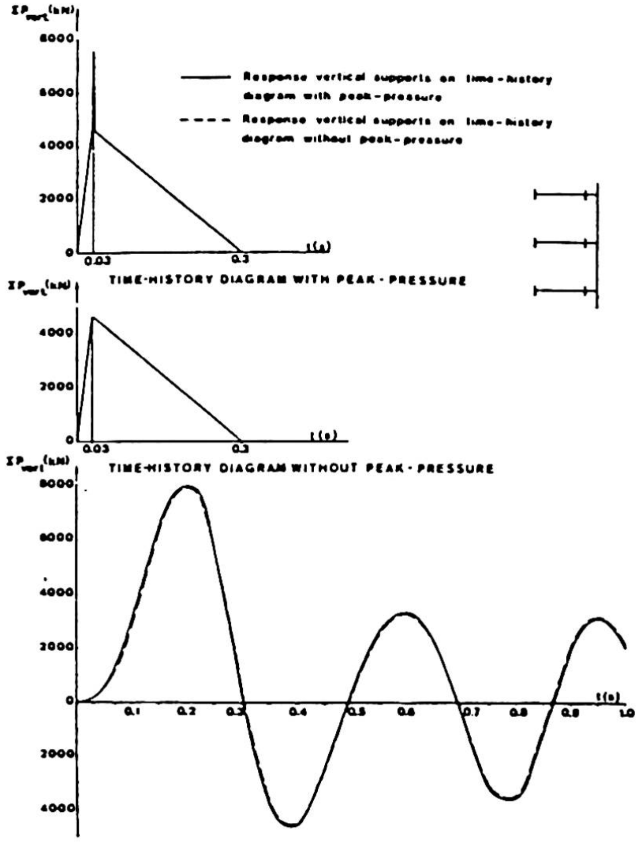
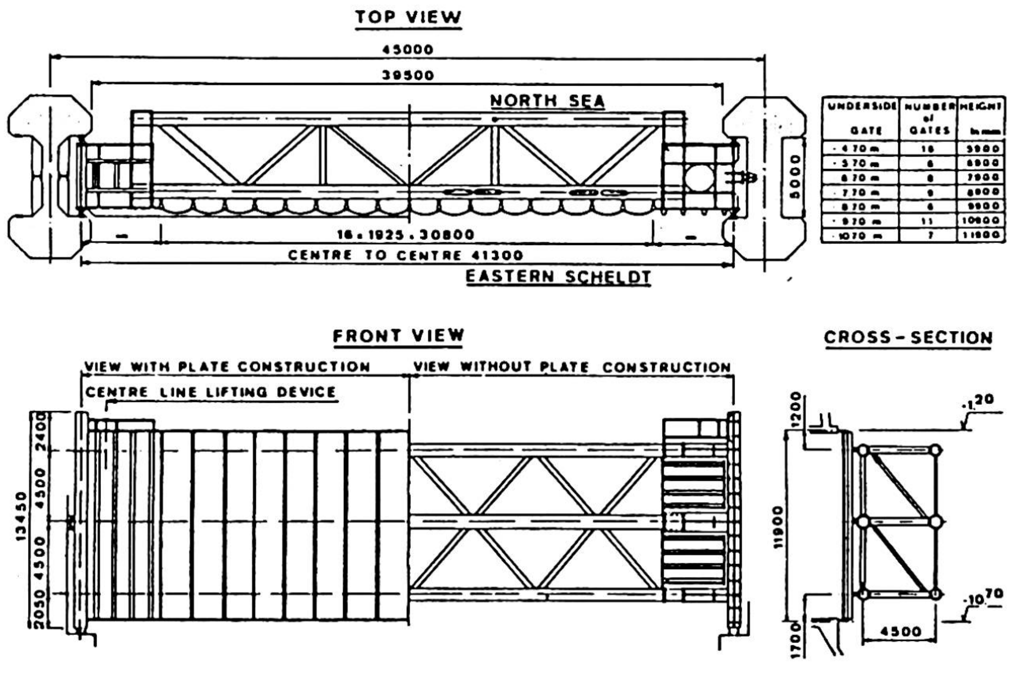


Figure 4

Response vertical supports (loading and response for a half FEM model).

Figure 5

Construction of gates



Earthquake Response Simulation by On-line Computer Test Control Method

Etude et contrôle du comportement sismique à l'aide de l'ordinateur

Simulation von Erdbebenreaktionen mit Hilfe einer On-line-Computer-Kontrolle

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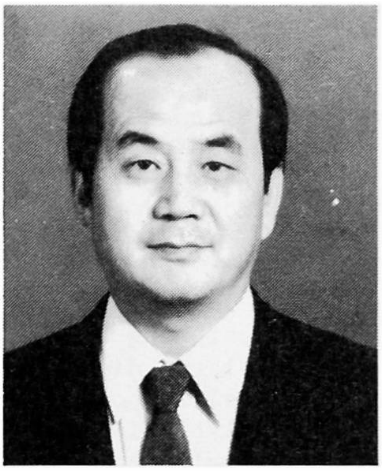

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SUMMARY
Analytical modelling of a structure is an important task in the seismic design procedure. Response calculation is so sensitive to the analytical model that formulation of the model must be carefully adjusted by simulation results. For this purpose the "on-line" computer test control method, which comprises computer analysis and experiment in a closed loop, is suitable.

RÉSUMÉ
Le modèle analytique d'une structure est un aspect important du processus de dimensionnement vis-à-vis des tremblements de terre. Les résultats des calculs de la réaction d'une structure sont très sensibles au modèle analytique, et la formulation du modèle doit être ajustée avec soin par des résultats de simulation. Pour cette raison, la méthode du contrôle des essais en ligne, incluant la simulation et l'expérience dans un cercle fermé, est appropriée.

ZUSAMMENFASSUNG
Die analytische Modellierung von Tragwerken ist eine wichtige Aufgabe für die Bemessung unter Berücksichtigung seismischer Beanspruchungen. Das Reaktionsverhalten dieser Modelle ist jedoch sehr empfindlich, so daß ihre Formulierung vorsichtig, anhand von Simulationsergebnissen, angepaßt werden muß. Zu diesem Zweck ist eine On-line-Computer Kontrolle angebracht, die in einem geschlossenen Kreis mittels eines Computers Daten von Experimenten erfaßt und gleichzeitig die Anpassung der analytischen Modelle durchführt.

1. INTRODUCTION

Research on the seismic performance of structural system against earthquake loading is being intensively conducted in the world, especially in the earthquake-prone countries. Much development of computers and their application techniques has exerted a great deal of influence on earthquake engineering. In designing structures, computer analysis to obtain the response behavior to earthquake ground motion consumes large amounts of time and effort since careful structure modeling must be carried out. Idealization of structures, equations to be solved, calculation methods suitable for the problems, analytical models such as the elastic-plastic characteristics of materials etc. are included. In addition, the exact values of physical constants are necessary before the execution of calculations. These values are usually obtained by experiments.

Highly developed computer techniques demand high quality of experiments. Fortunately testing techniques have also rapidly progressed. They are greatly owed to improved production techniques. In fact materials suitable for specific tests are now available, and can be manufactured to precise standards. Testing machine can be controlled in a precise manner, in most cases by computer.

Thus great advances have been made in both analytical and experimental fields. Both of them must be utilized in designing earthquake resistant structures. In behavior of structures, sometimes large discrepancies are found between idealized analytical solutions and actual response. Therefore, even in practical design procedure more realistic analytical models are required. These models depend on the accurate constitutive relations of materials, and more on correct formulations of elastic-plastic behavior of members and connections.

Loads or load effects acting on structures are determined by referring to statistical data on weights and meteorological phenomena. Seismic load, however, is exceptional. It cannot be determined independent of structural dynamic properties, because it is a load effect induced by the vibration of structures under earthquake excitation. The change of the dynamic properties due to yielding of members may alter the seismic load. It is difficult to determine analytical models by experimental results. In a strict sense the load applied to a test structure during test cannot be determined before execution of the experiment.

In order to overcome this dilemma, a new type of experimental technique was devised at Institute of Industrial Science, University of Tokyo[1],[2]. This technique is a combination of the numerical analysis and the experiment. It makes possible to trace the earthquake response of structural systems in the time domain without using a shaking table. It is called the on-line computer test control method, or the on-line test in abbreviation. In this presentation the basic concept of the on-line response test is described at first and then some test examples are highlighted in order to emphasize the importance of the on-line response test. Discussions are focussed on the accuracy of analytical models which are compared with the results of the on-line tests.

2. BRIEF DESCRIPTION OF ON-LINE COMPUTER TEST CONTROL METHOD

The basic concept of the on-line test is outlined in Fig.1. This technique is essentially a numerical analysis of spring-mass systems using the direct integration method. In the on-line test, however, the restoring force characteristics of the analyzed system are not postulated but are measured in a test carried out in parallel with the numerical integration. From this test, the earthquake response of the analyzed system can be obtained directly, and the tested structure itself undertakes the motion as if it were subjected to ground motion but at a slower speed. The on-line test is superior in several respects to the shaking table test, which is the most direct method for simulating the earthquake response of structural systems. The most significant advantage of the on-line test is that the loading of the on-line test can be quasistatic and can be halted at any time upon request, allowing close monitoring of detailed local behavior of the tested structure during earthquake response.

The on-line test has several advantages over other methods. Because the loading can be quasistatic(slow loading), less actuator capacity is required in the on-line test than in the shaking table test of the same specimen. In other words, large-scale or even full-scale test structures can be tested. Since the loading can be a repeated process of loading and pausing, conventional measuring devices used in quasistatic tests are sufficient in the on-line test, while shaking table tests must use simultaneous, continuous measuring. Finally, velocity adjustment loading in which the actuator ram velocity can be automatically controlled is possible when the restoring force properties of the test structures are affected by loading speed.

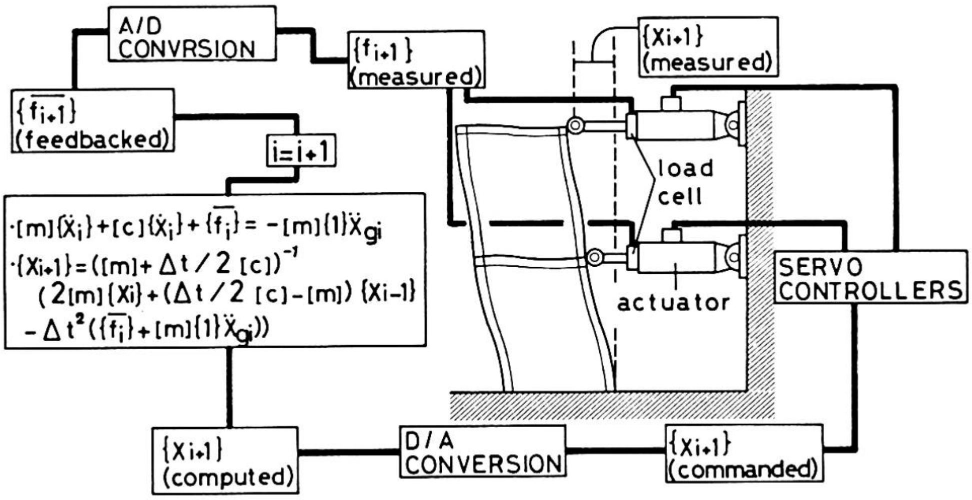


Fig.1 Block diagram of On-line test procedure

3.VERIFICATION OF ANALYTICAL MODELS

3.1 Importance of simulation

To emphasize the importance of verification of analytical models by simulation results, sensitiveness of calculation results to the model are examined in the case of the restoring force characteristics of a steel frame. A steel frame of three story and one bay bent is taken as an example. A 1/10 scaled model shown in Fig. 2 was tested by a shaking table to obtain the real characteristics of the restoring force of the first story [3]. The story shear force Q(the horizontal force induced at the level of columns' tops during excitation) versus the story displacement δ (the side-sway of the columns' top) relationship was obtained by the experimental model, as shown in Fig.3(a). An analytical model, namely, a mathematical expression describing the relation between the force and the displacement was formulated and then the hysteretic loops in Fig. 3(b) were produced with the mathematical expression according to the measured response displacement represented as the solid curve in Fig. 4. The two hysteretic loops is Figs. 3(a) and 3(b) are resembling each other from general appearances. The response displacement calculated by using the mathematical expression, however, is quite different as shown by the dushed curve in Fig. 4. This is a typical example to demonstrate how response calculation results are sensitive to the preciseness of analytical models used.

The adequateness of models used in response calculation must be examined by simulations.

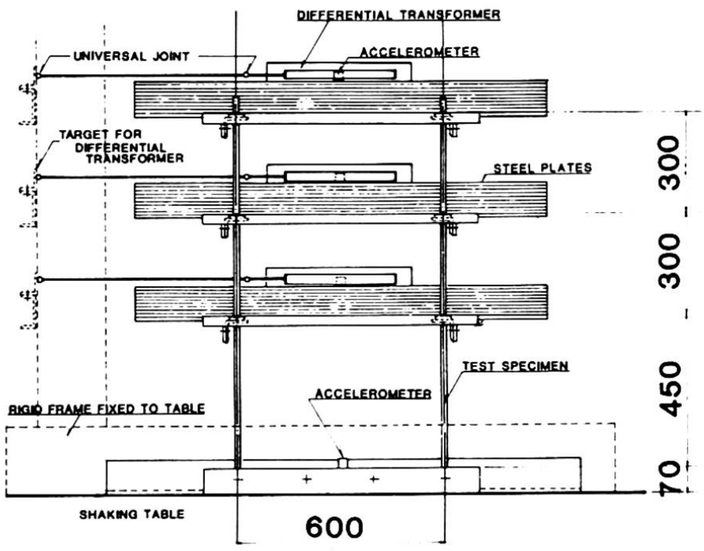
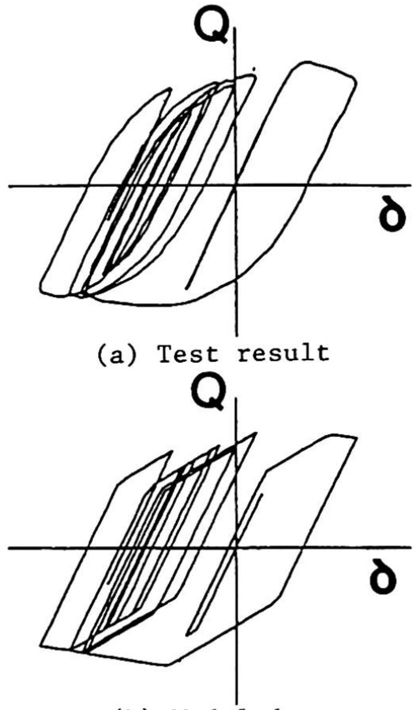


Fig.2 1/10 scaled model for shaking table tests



(a) Test result

(b) Modeled

Fig.3 Hysteretic loops

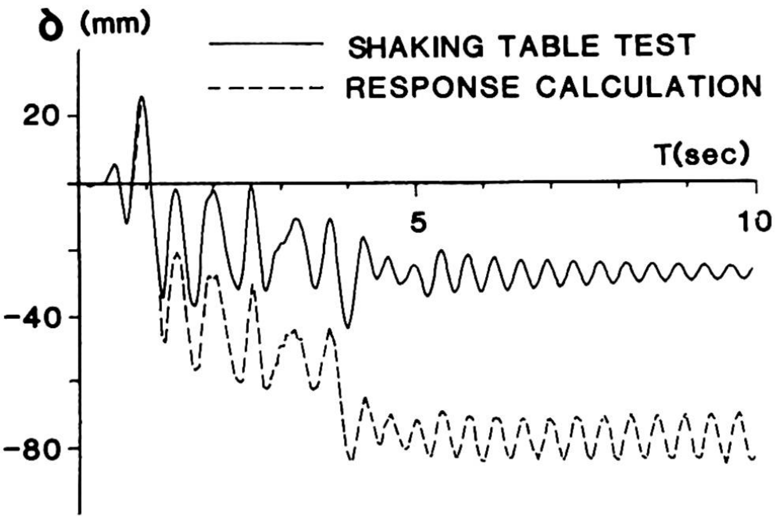


Fig.4 Response displacement measured and calculated

3.2 Earthquake response of a steel frame with high-strength bolt connections

High strength bolt connections are often used as in on-site erection work. The high-tension force provided by the high-strength bolts creates high-friction forces between connected plates, and therefore, the friction-type bolt connection is a very rigid one. The onset of yielding at the connection, however, causes a thickness reduction in the connected plates, which triggers slipping of the connected plates. After slipping the dynamic behavior becomes very complicated. A precise analytical model expressing such behavior is difficult to develop. Here, an analytical model was tried to be formulated upon cyclic test data that include the load at which the first slip occurs and the amount of slip rotation as well as the flexural rigidities. This model, called as Bolt Slip Model, postulates the hysteretic behavior of the beam with a bolt connection as shown in Fig. 5, though it is a rather simple expression being contrary to the real hysteretic loop. The adequateness of the analytical model was examined by comparing the calculation result with the simulation result obtained from the on-line test.

The on-line test was conducted on a frame with bolt connections [4]. The test frame is one-half of a two-story steel frame with high-strength bolt connections at the beam ends (Fig. 6) in lien of the frame as in Fig. 1 because of symmetry in deformation. From results obtained, the response displacement of the top of the frame is presented in a time history expression as the solid curve in Fig. 7. Meanwhile, the calculation result using Bolt Slip Model is shown as the dashed curve in the same figure. Agreement in both curves is satisfactory during dominant vibration. Hysteretic curves by the same on-line test is also shown in Fig. 8, in order to emphasize that the on-line test technique can simulate such a very complicate response behavior. It should be noted as one of the advantage of this test technique.

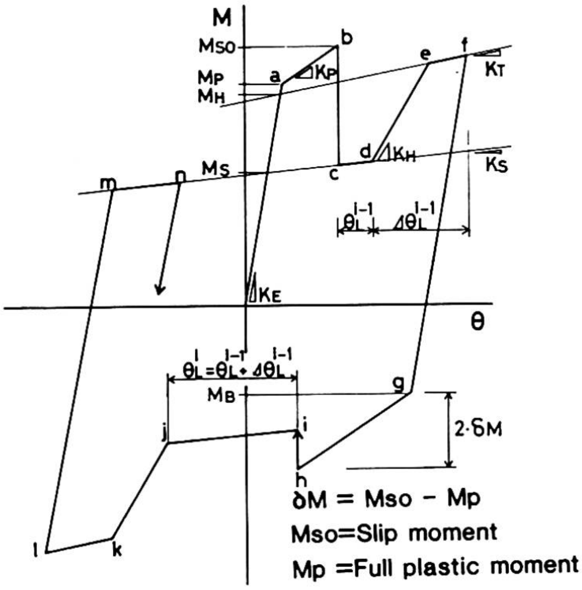


Fig.5 End moment M - end rotation θ of beam with bolt connection (BOLT SLIP MODEL)

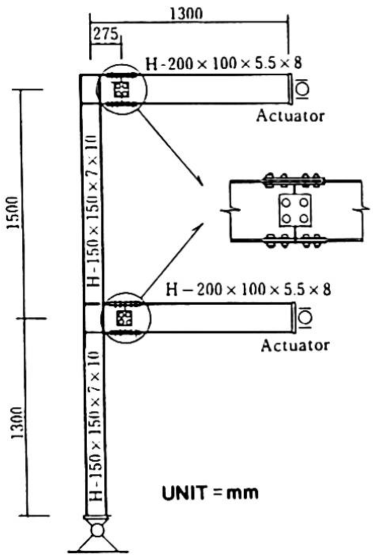


Fig.6 Test frame with bolt connections

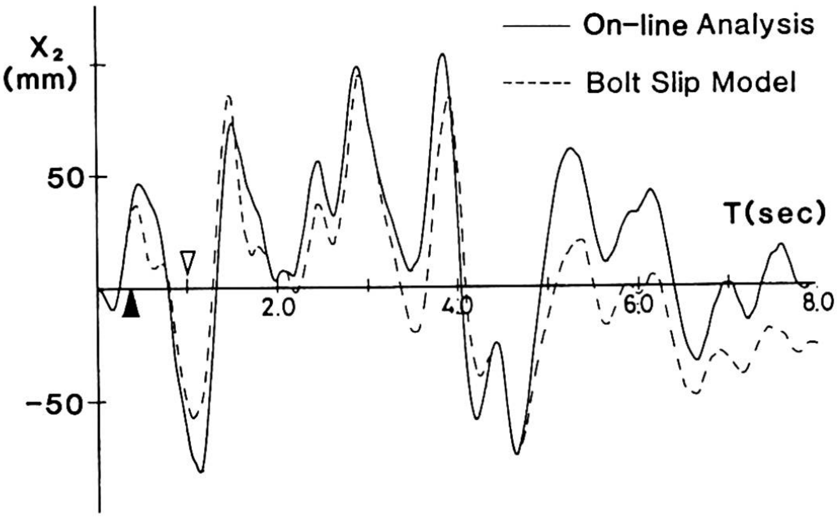


Fig.7 Response displacement simulated and calculated (Frame with bolt connections)

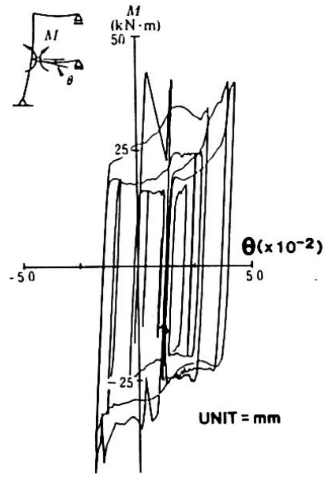


Fig.8 Moment-rotation relation by On-line test

In practical design procedure, a simple mathematical expression such as the bi-linear model as shown in Fig. 9 is often used. The preciseness of this model was also examined by the on-line test on a similar frame but without bolt connections. The response displacement of the top of the frame are compared with each other in Fig. 10. Agreement in these results is considered as being in the same degree as the previous case.

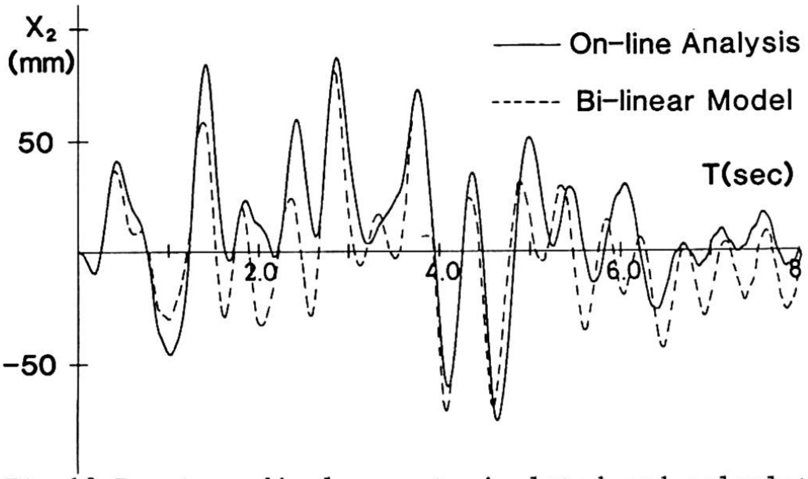


Fig.10 Response displacement simulated and calculated (Frame without bolt connection)

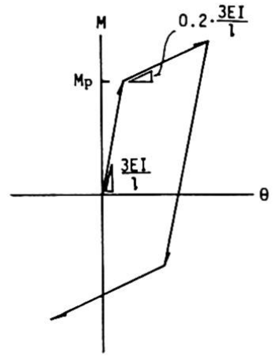


Fig.9 End moment M - end rotation θ relation of beam

4.CONCLUSION

For simulation of seismic behavior of structures, shaking table tests are often used. But these tests sometimes are not sufficient to predict real response behavior, because only small-scaled models of structures can be tested. The small-scale model tests are due to the limitation in capacity of a shaking table. It is not rare, however, that large-scale model tests, even full scale, are required to know precise response behavior of structures and structural elements. The on-line test described here is a suitable technique for such a purpose.

Modeling of structures is an important task in seismic design, but it is also difficult to achieve a satisfactory formulation which can describe real response in as easier way as possible. It needs a sort of compromise to be settled between preciseness and work. Analytical models must always be examined by the suitable simulation way. The raison is that damage sequence in disastrous stage of structures has never been recorded during earthquakes.

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Hysteresis Models for Earthquake Response Simulation

Modèles d'hystérésis pour la simulation de réponse sismique

Hysteresismodelle zur Simulation des Erdbebenverhaltens

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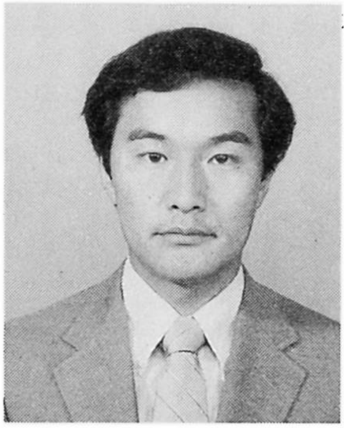


Hiroto Kato, B.S., has been actively engaged in research in prestressed concrete structures and earthquake response of buildings.

SUMMARY
This paper presents experimental investigations into the effect of loading rate on the hysteretic behavior of steel components and systems. Steel frames and columns were loaded both dynamically and quasi-statically, and the relationship between the loading rate and the restoring force was examined. A procedure to estimate the hysteresis for dynamic loading based on the hysteresis obtained under quasi-static loading is also proposed.

RÉSUMÉ
Ce document présente les études expérimentales de l'effet du niveau de charge sur le comportement d'hystérésis des éléments et ensembles en acier. Des poteaux et ossatures ont été soumis à des charges dynamiques et quasi-statiques, et la relation entre la charge et la force de restitution a été examinée. Une procédure pour évaluer l'hystérésis due aux charges dynamiques basée sur les résultats des charges quasi-statiques est aussi proposée.

ZUSAMMENFASSUNG
Diese Veroeffentlichung befasst sich mit experimentellen Untersuchungen des Einflusses der Belastungsgeschwindigkeit auf das hysteretische Verhalten von Stahlkomponenten und Systemen. Stahlrahmen und Stuetzen wurden sowohl dynamisch als auch quasi-statisch belastet und der Zusammenhang zwischen Belastungsgeschwindigkeit und Reaktionskraeften wurde ermittelt. Eine Methode zur Ableitung der dynamischen Hysteresis von der quasi-statisch ermittelten Hysteresis wird vorgeschlagen.

1. INTRODUCTION

To simulate the inelastic response of structural systems under earthquake loading, one should assign to each structural component a hysteresis model that represents the component's restoring force characteristics. Some of the popular models are, for example, the bilinear model and the Ramberg-Osgood model. When using those models, the analyst should assign a certain value to each of the model's individual parameters such as the yield force, the maximum strength, and the elastic stiffness. To do this, one often uses information obtained from the experimental investigations in which the tests were performed quasi-statically (slow loading). Here, a simple but serious question is addressed; whether or not the hysteresis models whose properties are determined based upon the statically loaded test results are valid for the purpose of earthquake response simulation, because the earthquake loading is naturally dynamic. It is well known that the stress-strain relationship of steel is affected by the strain rate. Under a high strain rate, both the yield and maximum stresses increase, while the change in Young's modulus is minimal. Then, the hysteresis of steel structural components and systems should also be affected by the loading rate since their constituent is rate dependent. There are studies that examined the loading rate effect on the hysteresis of steel components and systems. Some studies [1,2] suggested that this effect is not significant as long as the loading rate remains in a level achieved by the earthquake loading, while some other studies [3,4] warned that this effect cannot be neglected by any means. This inconclusiveness in their comments seems to stem from the difficulty to bridge the material's strain rate effect on the stress-strain relationship with the structure's loading rate effect on the hysteresis. Further, this difficulty is given rise to, because (1) as the earthquake loading is nonstationally, the loading rate continuously changes with respect to the time, and (2) as the structural components and systems extend into three dimensions, the strain as well as the strain rate vary from one fiber to another even at a given time.

The goal of the study is to propose simple but systematic procedures to estimate the loading rate effect on the hysteresis of steel components and systems by utilizing the knowledge of the strain rate effect on the steel materials. This paper introduces experimental studies that examined the effect of loading rate on the earthquake response of steel frames and on the hysteresis of steel components. A preliminary proposal to estimate the loading rate effect on the hysteresis of steel components is also included in this paper.

2. EFFECT OF LOADING RATE ON EARTHQUAKE RESPONSE

2.1 Pseudo Dynamic Test

The shake table test is known as the most direct experimental procedure to simulate the earthquake response of structural components and systems. Another experimental procedure for the earthquake response simulation, designated as the pseudo dynamic test, has also been devised [5]. This test is a combined numerical analysis and experiment and capable of reproducing the earthquake response of structures under quasi-static loading. The basic algorithm of this test follows; (1) to fabricate the test structure whose earthquake response one wishes to examine, and to set up this structure on the test bed; (2) to assume this structure as a discrete mass-spring system and establish the corresponding discrete equations of motion in a computer; (3) to connect a load applying actuator to the test structure at each assumed discrete mass position and in the direction of vibration; and (4) to solve the equations of motion by the direct integration method, but, rather than assigning a hysteresis model for each assumed spring, to measure the restoring force directly from the test in which the structure is loaded quasi-statically to the computed displacement positions. By repeating the computation, quasi-static loading, and measurement, the test structure traces a displacement time history as if it were subjected to the real earthquake loading, but the velocity attained in the test is much smaller than the velocity to be achieved in the real earthquake loading. Details in the concept of the pseudo dynamic test and a summary of the previous applications can be found elsewhere [6].

2.2 Comparison Between Shake Table Test and Pseudo Dynamic Test

As the first step to investigate the loading rate effect, a shake table and pseudo dynamic tests were carried out for a steel frame. Comparison of the responses obtained from those two tests makes it possible for us to learn directly the effect of loading rate on the earthquake response, because, from the basics of the pseudo dynamic test, the major difference between the two tests is the loading rate; i.e. dynamic in the shake table test and quasi-static in the pseudo dynamic test. The basic dimensions of the tested steel frame are shown in Fig. 1. Figure 2 shows the test setup for the pseudo dynamic test. Two identical test specimens were fabricated; one for the shake table test and the other for the pseudo dynamic test. Further, the same input ground motion was applied to the two specimens. The obtained displacement time histories and force-deflection relationships are compared in Figs. 3 and 4. Figure 3 shows that the waveforms of the two displacement responses are very similar but there is an offset in the datum line between the two responses. Figure 4 indicates

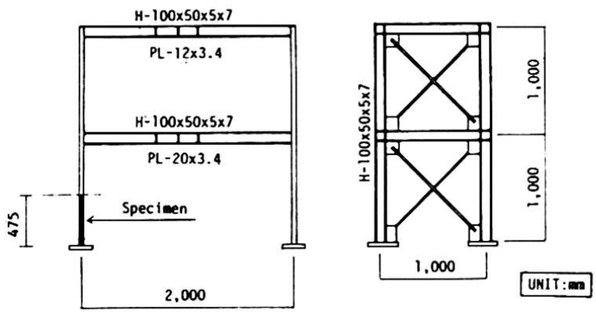


Fig. 1 Specimen for Steel Frame Test

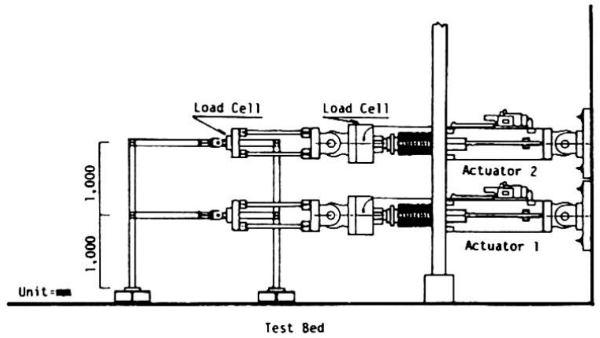


Fig. 2 Test Setup for Pseudo Dynamic Test

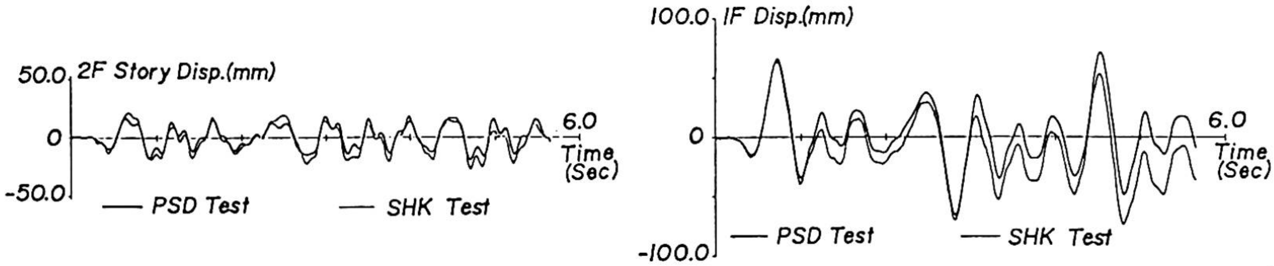


Fig. 3 Displacement Time Histories Obtained from Shake Table and Pseudo Dymanic Tests

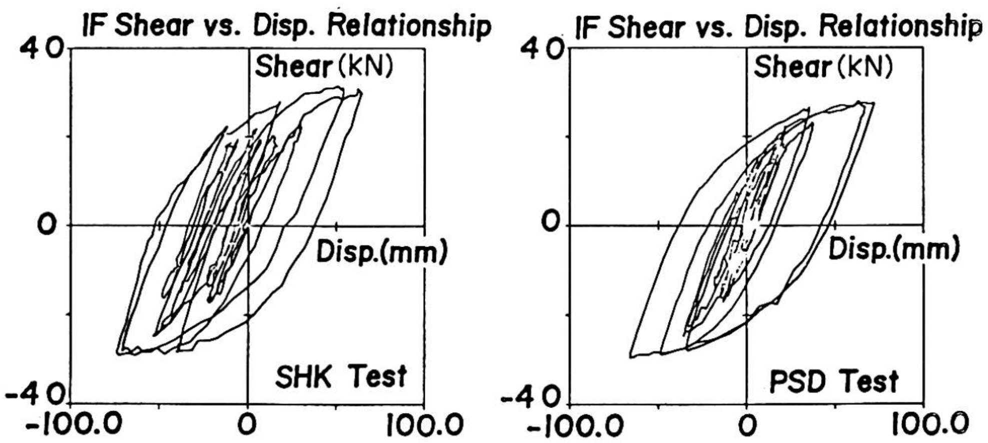


Fig. 4 Hysteresis Curves Obtained from Shake Table and Pseudo Dynamic Tests

that the restoring force in the inelastic range is greater by about 10 to 15 percent in the shake table test. It was found from the post numerical analysis that the difference in the restoring force (Fig. 4) was mainly responsible for the difference in the displacement response (Fig. 3). Details in the shake table and pseudo dynamic tests as well as the post numerical analysis are described in [7].

3. EFFECT OF LOADING RATE ON HYSTERESIS

From the results indicated above, it was found that the loading rate indeed affected the hysteresis and accordingly the earthquake response of the steel frame. To further examine the mechanism that changed the hysteretic characteristics, dynamic and quasi-static loading tests were carried out for a cantilever H-shaped steel column (Fig. 5). The test setup is shown in Fig. 6. The tested column had the same material and cross sectional properties as the columns of the steel frame (Fig. 1), and the column length was exactly half the first story's clear span length of the steel frame. Since, in the steel frame, the beam stiffness was significantly larger than the column stiffness, the point of contraflexure of the column remained in the middle of its clear span. Further, the acutuator, attached to the free edge of the column, imposed a displacement time history that was exactly half in magnitude the first story displacement response obtained from the pseudo dynamic test (Fig. 3). In those conditions imposed, the cantilever column's hysteresis was representative of the first story hysteresis of the steel frame. Two cantilever specimens were fabricated, and one (designated as the fast test) was tested by imposing the displacement time history dynamically with the velocity that was computed in the pseudo dynamic test, whereas, the other (designated as the slow test) under slow loading in which the velocity was reduced to 1/100 of the velocity in the fast test. Since the two tests sustained the exactly the same displacement time history, comparison in the restoring force between the tests provided us direct information on the loading rate effect.

The major findings obtained from those tests can be stated as follows. (1) The two obtained hysteresis curves are compared in Fig. 7, in which the restoring force obtained from the fast test is greater in the inelastic range by 10 to 15 percent. This difference agreed with the difference observed in the steel frame test. A closer look of those curves, however, indicated that, in the fast test, the restoring force was constantly higher in the inelastic range, but decreased rapidly as shown in Fig. 8 when the displacement approached its extreme value (the maximum displacement in Fig. 8). In fact, the force almost matched with the restoring force obtained in the slow test at its extreme position, where the velocity was reduced to zero. This observation supports our intuition that, at the extreme displacement where the velocity is reduced to zero, the restoring force should be the same in both the fast and slow tests. (2) The stiffness in the elastic range was almost identical between the two tests, indicating that the loading rate did not influence the elastic behavior. This observation also agrees with the known fact that Young's modulus of steel remains unchanged regardless of the strain rate. (3) In the fast test, the yielding region was more confined near the clamped edge (the most stressed section). Since the displacement at the free edge was taken to be the same, in turn, this confined yielding region resulted that the curvature at this region was greater in the fast test. (4) Figure 9 shows the relationship between the curvature rate and the restoring force ratio at the cross section 20 mm inside the clamped edge (designated as SEC1 in Fig. 5). Here, the curvature rate was estimated from the strain rates measured at SEC1 during the fast test, and the restoring force ratio was taken as the restoring force in the fast test relative to that in the slow test. Furthermore, the plots were made only for the region where SEC1 yielded significantly. The figure shows that the maximum curvature rate and strain rate was about 0.01 (1/mm/sec) and 0.2 (1/sec) respectively, and the restoring force ratio scattered between 10 to 15 percent.

4. ESTIMATION OF EFFECT OF LOADING RATE ON HYSTERESIS

From the above experimental observations, the relationship between dynamically and quasi-statically loaded hystereses can be stated quantitatively as follows. (1) The elastic stiffness remains unchanged regardless of the loading rate. (2) Under dynamic loading, the restoring force increases in the inelastic range, and this increase is relatively constant until the displacement approaches its extreme value (Fig. 8). (3) Under dynamic loading, the restoring force drops rapidly near the extreme displacement and merges with the restoring force obtained under quasi-static loading. Since, as shown in Fig. 8, this drop occurs only very near the extreme displacement, one may reasonably neglect this drop and assume that the restoring force is constantly higher up to the extreme displacement.

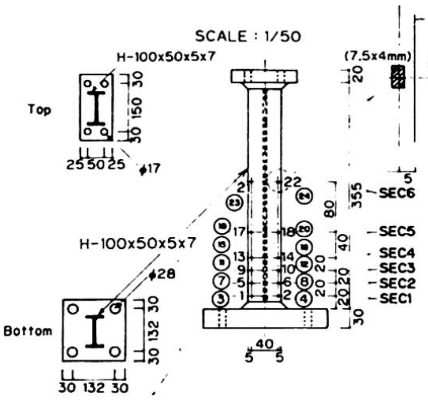


Fig. 5 Specimen for Steel Cantilever Column Test

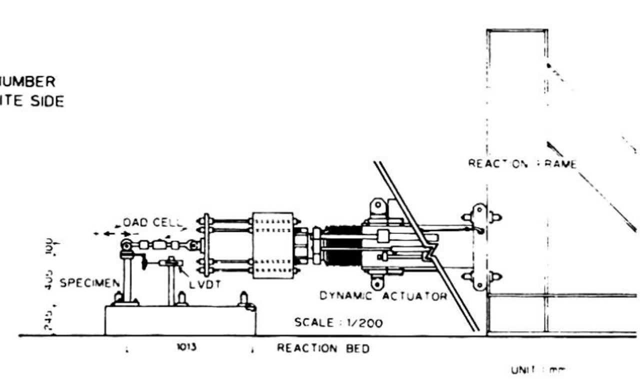
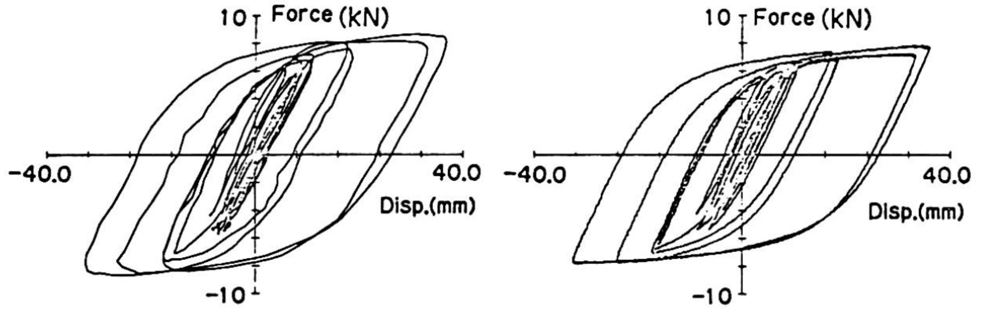


Fig. 6 Test Setup for Dynamic and Quasi-Static Loading Tests



(a) RATE1 (Real Time) (b) RATE100 (Real Time/100)

Fig. 7 Histeresis Curves Obtained from Dynamic and Quasi-Static Loading Tests

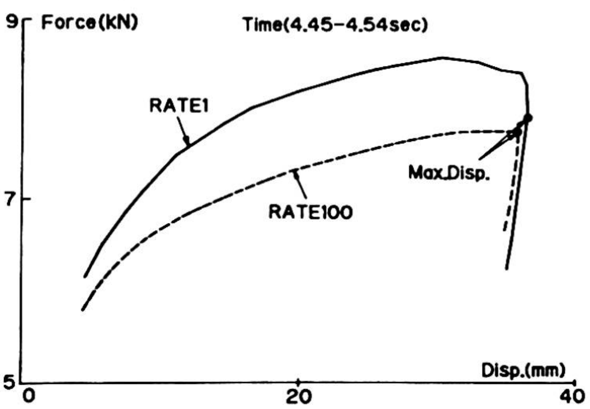


Fig. 8 Comparison of Dynamic and Quasi-Static Hysteresis Curves

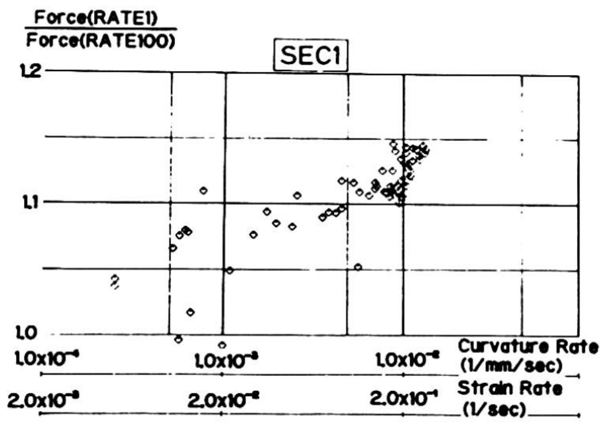


Fig. 9 Curvature Rate vs. Reactional Force Ratio Relationship

Considering the statements (1) to (3), the hysteresis under dynamic loading can be estimated reasonably but without much elaboration by simply adding, in the inelastic range, some restoring force to the restoring force obtained under quasi-static loading. Then, the question that remains is how to estimate this additional restoring force. A procedure to estimate the additional restoring force for cantilever steel beams (like the one in Fig. 5) is proposed below; (1) To estimate the curvature rate at the clamped edge from the velocity at the free edge. In fact, this estimate is made possible if the moment vs. curvature relationship is specified for the cross section. Further, it is presumed that the velocity is a quantity that can be obtained in the numerical computation; (2) to bridge the curvature rate with the strain rate by assuming that the plane section remain plane after deformation; (3) To estimate the moment increase based upon the strain rate at the section. Here, the stress increase in relation to the strain rate is taken to be known through our previous knowledge; (4) To estimate the restoring force increase by dividing the moment increase by the length of the beam. The validity of this procedure was calibrated for the cantilever steel column (Fig. 5). According to the test result, at 4.45 sec, the velocity at the free edge was 277.5 mm/sec, and the restoring force increase was by 945 N. On the other hand, the computation using the proposed procedure gave a value of 1,010 N for the restoring force increase. The estimated value is only 7 percent larger, demonstrating the appropriateness of this procedure. Details in the procedure proposed as well as the fast and slow test results are depicted in [8].

5. CONCLUSIONS

This paper presented experimental studies in which steel frames and columns were loaded both dynamically and quasi-statically, and demonstrated how the loading rate affected the hysteresis of those structures. This paper also described a procedure to estimate the hysteresis for dynamic loading based upon the corresponding hysteresis obtained under quasi-static loading.

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Modelling of Load Carrying Capacity of Plastic Structures

Modélisation de la capacité portante de structures en plastique

Modellierung der Tragfähigkeit von Tragwerken aus Kunststoffen

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Dr. Helen Doubravszky, born 1939, received her architect degree at the Technical University of Budapest. For 18 years she was involved in research concerning load-bearing capacity of plastics, and received her PhD. degree at the Hungarian Academy of Sciences in that field. Now, as senior research associate, she is responsible of the mechanical testing of non-traditional materials and structural connections.

SUMMARY
The verification of the safety of structures and structural elements, the load-bearing capacity of which depends upon mechanical properties of plastics, can not be performed with appropriate confidence on the basis of traditional methods. Their limit states are consequences of loading processes, and can be controlled but through limit strains. This paper shows a design system derived from this statement, and based on the assumptions of linear viscoelasticity.

RÉSUMÉ
La sécurité de structures et d'éléments structuraux, la capacité portante dont dépend les propriétés mécaniques des plastiques, ne peuvent pas être vérifiées avec certitude sur la base de méthodes traditionnelles. Les états de rupture sont la conséquence du processus de charge et ne peuvent être contrôlés que par des déformations limites. La contribution présente un système de calcul et de projet découlant de cette affirmation et basée sur les hypothèses de visco-élasticité linéaire.

ZUSAMMENFASSUNG
Die Sicherheit von Tragwerken und Tragelementen aus Kunststoffen kann mit den traditionellen Methoden nicht genügend zuverlässig ermittelt werden. Die Grenzzustände von Tragwerken aus Kunststoffen sind eine Folge der Belastungsgeschichte und können nur über die Grenzdehnungen erfasst werden. Dieser Beitrag stellt eine Bemessungsmethode vor, die auf dieser Basis beruht, und setzt die Annahmen der linearen Visko-Elastizität voraus.

INTRODUCTION - AN ALTERNATIVE FOR STRUCTURAL ANALYSIS

Structural analysis is based upon models of physical reality. These models provide with patterns for analysis:

- separate phenomena of the first importance from those of secondary importance,
- construct a closed system from phenomena of first importance,
- give approximative methods that take into account phenomena of secondary importance,
- ensure calculation method that optimally conforms to the closed system.

For load-bearing structures and structural elements we use elastic materials. We suppose that they deform under load and recover after the load has been released. If the deformation and the recovery time are short /as with steel and concrete/, we use ideally elastic material model for structural analysis. /Instantaneous elastic deformation is of first importance./ We take a constant design strength as granted, and examine rupture of the structure by comparing design strength to design loading effects /stresses from design loads/. Thus the closed system for verification of the safety of structure /by ISO 2394/ can be summarized by the following relation:

$$\sum_{i=1}^{n} S_{d,i} \leqq R_d \qquad /1/$$

where $S_{d,i}$ is the design loading effect from the i-th design load, and

R_d is the design strength.

Deviation from the ideally elastic behaviour /fatigue, creep, plastic deformation/ of these structural materials is comparatively small /of secondary importance/, and is taken into account by reducing the design strength.

Structural plastics - both thermoplastics and thermosets, homogenous and composites, solids and cellulars - deform and recover long after their state of stress has been changed. The ultimate time-dependable part of their strain is of the same order of magnitude as the instantaneous part. Furthermore, under stress, simultaneously with the increase of strain, their strength decreases. These phenomena are of first importance, and demand viscoelastic model for analysing load-carrying properties of structures as well as structural elements with such materials.

In case of viscoelastic material model, strength is not considered as constant any more, thus a design loading effect can not be related to design strength unless duration and circumstances of loading are known. When examining ultimate limit states, the structural response to the design process of loading effects has to be analysed and compared to a material constant. For this material constant - according to results of the research of last 20 years [1,2], we can take the limit strain. Thus the closed system for verification of the safety of plastic structures can be summarized by the following relation:

$$\sum_{i=1}^{n} \varepsilon_{d,i}(\sigma_i, t_i, T_i, e_i) \leqq \varepsilon_{lim} \qquad /2/$$

where $\varepsilon_{d,i}(\sigma_i, t_i, T_i, e_i)$ is the material response to the i-th design loading effect: strain caused by σ_i stress of t_i duration acting at T_i temperature and e_i environment,

ε_{lim} is the limit strain, beyond which viscoelastic properties of plastics change irreversibely*.

* Irreversible changes of viscoelastic properties occur when the integrity of the material ceases.

This paper shows how a closed structural analysis system and a calculation method can be constructed on the basis of the viscoelastic material model.

MAKING USE OF THE HYPOTHESIS OF LINEAR VISCOELASTICITY

Definition of linear viscoelasticity given by Ferry[3] sais: "if both strain and rate of strain are infinitesimal ... the ratio of stress to strain is a function of time /or frequences/ alone, and not of stress magnitude", in other words isochroneous strains are proportional to the respective constant stresses.

Plastics remember loading history. Boltzmann superposition principle helps us in analysing this phenomenon. Its simple definiton by Ferry[3] sais, that "the effects of mechanical history of linear viscoelastic body are linearly additive". It means that the sequence of different loads does not influence their summarized material response.

Let us see the degree of the error made when utilizing the assumption of the linear viscoelasticity:

- with composites there is no error, because in most cases their limit strain /o.oo2...o.oo5/ is far below their proportional limit;
- with plastic foams there is no error, because their limit of reversible changes coincide with their proportional limit /o.o1...o.o15/[4];
- with thermoplasts there can be a problem, but it can be solved by allowing a certain error /by ST SEV 5060 - 5 % deviation, by US Plastics Design Manual[5] and by Powell[2] - 15 % deviation from linearity/. In addition, ST SEV 5060 gives for these materials two limit strains: a proportional limit /o.oo5...o.oo8/ and a limit of reversible changes /$<$ o.o25/.

Applying the hypothesis of linear viscoelasticity, we get two important facilities for structural analysis:

- stress analysis based on traditional elastic formulae, and
- summing up strains /material response to different loading effects/ on the basis of the Boltzmann superposition principle.

All the engineers agree in the importance of the first assumption, though the importance of the second one does not fall behind it. It merely was not a problem with the instantaneously elastic materials, because loading effects could be summed up without any regard to their succession and duration

A CONSEQUENTLY USED TIME-DEPENDENT LINEAR STRESS-STRAIN RELATIONSHIP

In course of the development of structural analysis, one of the main questions was the overall resistance of the structure, i.e. proving that it had enaugh rigidity to endure the consequences of loading. Obviously, modulus was used for expressing stress-strain relationship: the resistance of the structure was proportional to the material characteristic /modulus/.

With plastic structures, according to eqn. /2/, we are interested both in local and overall deformation of the structure. Moreover, we have to analyse the whole deformation process. This means that first we calculate the state of stress of the structure, after that we calculate the time-dependent deformation caused by this state of stress. Using compliance we do not only describe a property of the structure by a material characteristic proportional to it - this is the only rheologically correct way of engineering calculation:

$$D(t) = \frac{\varepsilon(t)}{\sigma} \qquad /3/$$

where $D(t)$ is the compliance describing retarded deformation, i.e. time-

-dependent strain caused by a given stress*.

Engineers prefere models consisting of simple relationships. It is a convenient method to have an initial compliance /corresponding to some conventional time, e.g. 1 min./, and a time characteristic:

$$D(t) = D_o \times \delta_{ct} \qquad /4/$$

where D_o is the initial standard compliance, and
δ_{ct} is the time characteristic, indicating strain increment after t hours period of loading.

The time characteristic δ_{ct} can be easily determined by a creep experiment as a ratio of the deformation after t hours loading and that of 1 min. loading. The strain in the experiment must not exceed the proportional limit strain /or the limit of reversible changes, if this is the lowest of the two/.

CALCULATING RECOVERY

The amount of the recovered deformation depends upon both the creep time and recovery time. Experiments as well as model-calculations carried out according to basic laws of linear viscoelasticity [6] proved that for practical purposes the following is a good approximation:

$$t_{ro} \approx 10 \times t_{ct} \qquad /5/$$

where t_{ro} is the recovery time corresponding to t_{ct} creep time, and
t_{ct} is the creep time /load duration/.

Fig.1 shows its graphical presentation. In practice, the part of the compliance representing retarded deformation $D_o(\delta_{ct}-1)$ has to be multiplied by the recovery ratio δ_r. The last can be approximated - according to Fig.1 - by the following formula**:

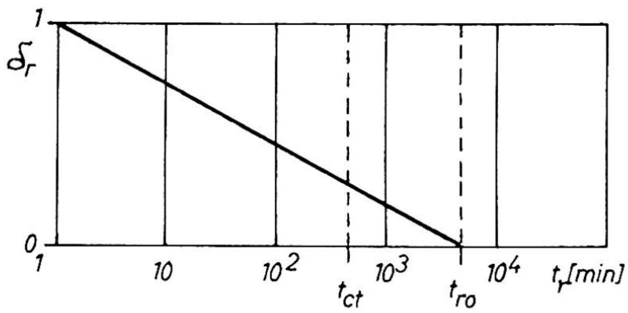

Fig.1

$$\delta_r = 1 - \frac{\lg t_r}{\lg(10 \times t_{ct})} \qquad /6/$$

where δ_r is the recovery ratio, indicating fraction of the retarded deformation not yet recovered during t_r time.

This simple rule was included in ST SEV 5060 and is permitted to apply in cases when max. strain does not exceed neider proportional limit, nor the limit of reversible changes.

CALCULATING STRUCTURAL RESPONSE TO TEMPERATURE AND OTHER ENVIRONMENTAL EFFECTS

Temperature change modifies creep rate of the plastic bodies. This phenomenon can be expressed by shifting "lg compliance - lg time" relationship along the horizontal /and sometimes along the vertical/ axis [3]. This means a simple multiplication:

$$D(T) = D_o \times \delta_T \qquad /7/$$

where δ_T is the temperature characteristic, the ratio of the initial stand-

* With viscoelastic materials, modulus describes relaxation: $E(t) = \sigma(t)/\varepsilon$, time-dependent stress caused by a given strain

** Formula /6/ is valid between t_o=1 min. and $t_{ro}=10 \times t_{ct}$, with t expressed in minutes ard compliance and that of an other temperature.

According to rules of the shifting, formula /7/ applies to compliance at any load-duration.

Other environmental effects /mainly humidity/, that modifie creep rate /but do not cause degradation/, can be taken into account by the same model[7]:

$$D(e) = D_o \times \delta_e \qquad /8/$$

where δ_e is the environment characteristic, the ratio of the initial standard compliance /corresponding to standard atmosphere/ and the compliance at another humidity.

CONCLUSION - THE STRAIN-BASED DESIGN METHOD IN FRACTICE

The strain-based system for the verification of the safety is advantageous in case of those structures of which at least one of the limit states depends upon the load-bearing capacity of plastics /including polymer concreet and steel reinforced polychloropren bearings/. It provides with method for differentiation between loads of short and long duration, cold and hot periods of service etc.

Fig.2 shows an example for how to sum up consequences of different loads with different loading time. With stress-based system for the verification of the safety, this could not be realized, so the reliability of such design would be much poorer than with the system presented in that paper.

a/ Deviding of the loading process to parts of constant load intensity

b/ Fitting time-function of the compliance to each load part

c/ Calculating time-function of the deformation from each load part

d/ Summing up deformation from each load part

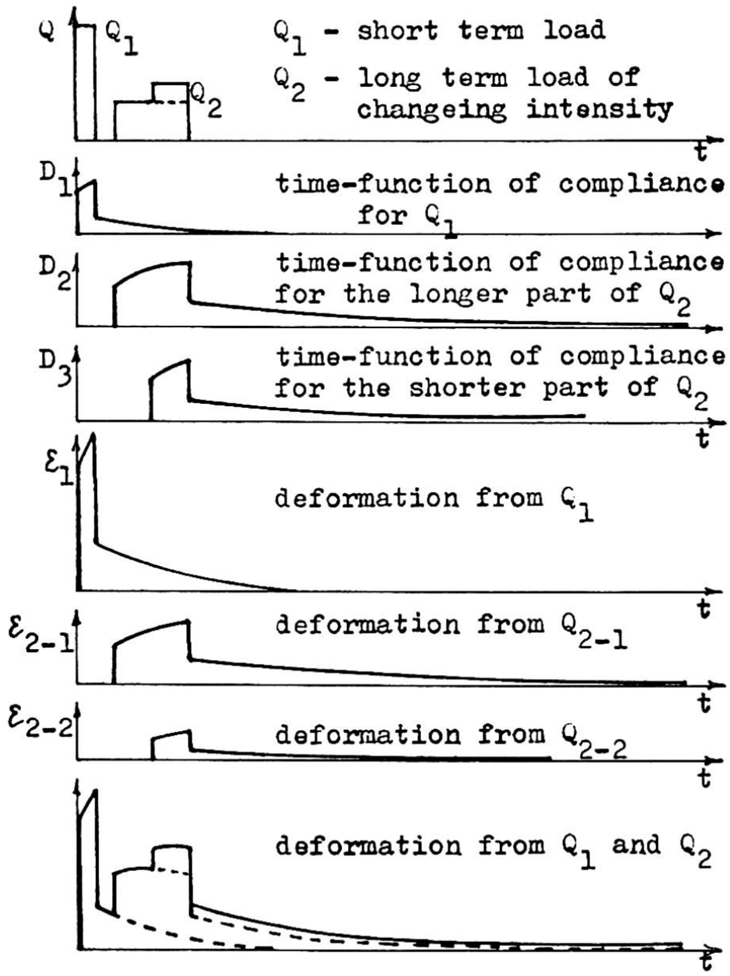

Fig.2

The basic differences between the principles of the strain-based and those of the stress-based design methods, as well as the great variety in the field of application of plastics in structures and structural elements make considerable obstacles in the way of putting this method in practice. It would be preferable to issue a series of documents beginning with the vocabulary and with the principles of structural design of plastic structures, followed by rules for constructing the design loading process, by rules for determination of design characteristics of structural plastics, by analysis methods for various kinds of plastic structures, and by rules of performing experiments with full-size elements for the verification of their safety.

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REFERRED INTERNATIONAL STANDARDS

ISO 2394-1973 General principles for the verification of the safety of structures

/ST SEV 5060/ СТ СЭВ 5060-85 Надежность строительных конструкций и оснований КОНСТРУКЦИИ ПЛАСТМАССОВЫЕ Основные положения по расчету

Design of Transmission Towers: Challenge for the Practicing Engineer

Calcul des pylônes de lignes électriques: défi à l'ingénieur

Bemessung von Hochspannungsmasten: Herausforderung für den praktischen Ingenieur

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SUMMARY
Transmission towers are among the most complex civil engineering structures. Though a feasible preliminary design can be based on rather simple computer calculations, the final check often requires more sophisticated computer programmes. The paper is aimed at stressing the fundamentals of both approaches and at presenting a comparison between the corresponding results.

RÉSUMÉ
Les pylônes de lignes électriques comptent parmi les structures les plus complexes du génie civil. Un dimensionnement préliminaire peut être conduit à partir de calculs relativement élémentaires. Une vérification finale à l'aide de programmes de calcul plus élaborés reste néanmoins nécessaire, ainsi que le montre l'analyse comparative des résultats selon les deux approches mentionnées.

ZUSAMMENFASSUNG
Hochspannungsmasten sind sehr komplexe Ingenieur-Tragwerke. Obwohl eine sinnvolle Vorbemessung mit einfachen Computer-Berechnungen möglich ist, erfordert die endgültige Bemessung oft wesentlich verfeinerte Rechenprogramme. Dieser Beitrag beschreibt die wesentlichen Punkte der beiden Methoden und vergleicht deren Resultate.

1. INTRODUCTION

Building transmission towers made with steel angles is a common and widely used practice. Such structures are nevertheless amongst the more complex ones in the field of civil engineering, for reasons which are commented briefly here below.

Though a simple section at first glance, the angle is normally subject to combined axial force, bending moment, shear and torsion, as a result on the one hand, of the non-coincident centroïd and shear center and, on the other hand, of the usual practice to make end connections on one angle leg only. Transmission towers are truss-type structures and are thus likely to produce kinematic mechanisms ; with a view to avoid these latter, an adequate geometry is necessary. Secondary bars, called redundants, are often used to reduce the buckling length of compressed chords and legs, with the result of zero axial force in these redundants, as far as it is referred to a first order analysis. A proper design must however account for unavoïdable geometrical imperfections (especially lack of bar straightness) and erection misalignments ; the redundants have thus to be designed to resist the associated destabilising forces. It is also of common practice, though unsafe, to assume pin-ended bars and to simplify the design by partitioning the tower and analysing it as an assemblage of non interactive plane trusses ; consequently load eccentricities and spatial behaviour are mostly neglected.

A lot of transmission towers collapsed in the past and lessons must be drawn from these accidents. The loading conditions should be revised with the result of a more accurate assessment of wind loads, differential bearing settlements, loads resulting from conductors failure,... Also it is presently attempted to improve the design procedures and allow the designer for more elaborated approaches. In this respect, it is while mentioning the publication, by ECCS, of Design Recommendations [1] and, more recently, of computer programmes and comments to which the junior author contributed much [2] ; these programmes enable the interactive preliminary design of several parts of a transmission tower.

Because the transmission tower is probably the sole huge reproducable structure in the field of civil engineering, a special care should be brought to its design with a view to optimization and to cost and material savings. Therefore the authors are of the opinion that refined calculations should be made finally, on base of the aforementioned preliminary design, by using non linear finite element computer programmes.

2. A STEP TOWARDS SPECIFIC DESIGN RULES

In 1985, ECCS produced recommendations for the use of angles in lattice transmission towers [1]. These recommendations were derived from a first draft issued in the Introductory Report to the Liège Colloquium of 1977 [4]. They are mainly aimed at giving rather simple calculation rules when designing the main structural components of transmission towers made with angles. The design buckling stresses are usually higher than those adopted for other steel structures, because they were drawn from a lot of test results of structures loaded up to failure, which make the transmission towers the best known steel structures.

The basic buckling curve adopted for angles in transmission towers is valid for equal and unequal leg rolled or cold-formed angles, which buckle in the plane of minimum stiffness without local failure due to plate instability. The European column buckling curve a_0 is adopted as this basic curve. The influence of local and torsional buckling on the overall buckling strength of rolled and cold-formed angles is accounted for in a simple and practical way by using a reduced yield stress; the latter is deduced from the yield stress with regard to the manufacturing process and the b/t ratio of the walls.

The buckling length and the radius of gyration to be used of legs or chords depend on the type of bracing, either symmetrical or staggered in two normal planes. Provided the slenderness ratios be computed accordingly, the engineer is allowed to disregard the secondary unfavourable effects due to the eccentricity of the loads from the web members.

The end connections of web-members in compression usually produce eccentricities and restraints (when 2 or more bolts), both of which affect the carrying capacity. It is assume that both effects cancel each other at a non-dimensional slenderness close to $\Lambda = \sqrt{2}$. For lower slenderness ratios, there is a resulting detrimental influence on the carrying capacity ; a modified increased slenderness ratio $\bar{\Lambda}$ (> Λ) is defined according to the type of eccentricity (at one or both ends) and the buckling axis (minimum axis or rectangular ones). For higher slenderness ratios, the beneficial effect of the end restraint exceeds the detrimental one due to connection eccentricity. A modified reduced slenderness ratio $\bar{\Lambda}$ (< Λ) is recommended, which depends on the type of member (continuous or not) and of connection (one or more bolts). When end restraints and member discontinuity cannot be clearly defined, simplified rules are suggested, where the slenderness ratio $\bar{\Lambda}$ depends on the buckling axis only.

Some general guidelines are also given for what regards the lengths to be used as buckling length of web member types and bracings.

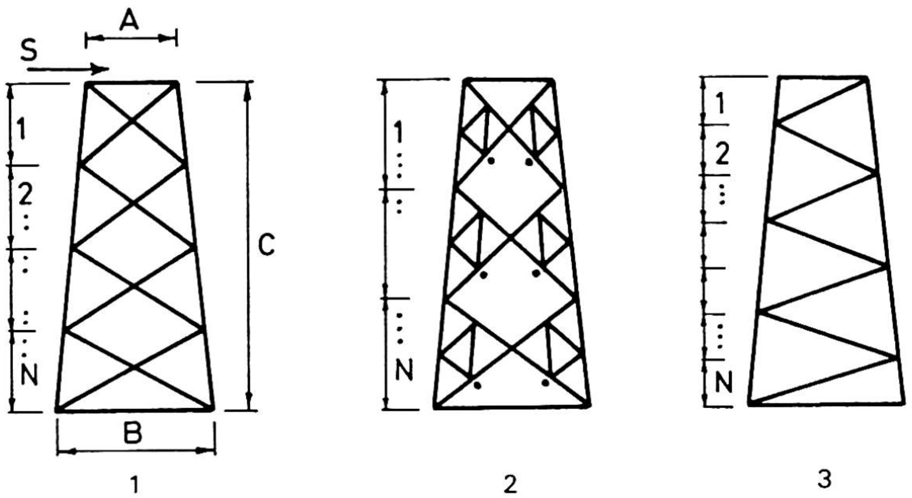
The ECCS Recommendations can thus be used only when the forces in the several bars are known. It can also be observed that each structural element is designed with nearly no account for the others. As the ECCS design rules are derived from failure test results, they could be expected to be safe. The systematic use of the ECCS Recommendations in the frame of a preliminary design and the assessment of their actual safety are two matters which are discussed in the following sections.

3. AN ATTEMPT TO USER'S GUIDANCE AND OPTIMIZATION

ECCS recently issued [2] a suite of programmes which were intended to assist the designer in arriving at the final stage of design by quickly looking at various alternatives and assessing the influence of changing some of the design parameters. The package consists of : i) a manual and a floppy disc containing four design programmes suitable for use on an IBM P.C. and ii) a file of angle member properties. The design rules are in accordance with the ECCS Recommendations [1] and the BSI Code of Practice [3].

As latticed transmission towers are generally statically indeterminate, advantage is taken of symmetry conditions to separate the four faces of the tower body and treat each face individually as a plane statically determinate problem. The loads applied to the tower generate a horizontal shear force S applicable to the panel under consideration, wherefrom the member forces of the bracing can be derived.

The programme REDCROSS is aimed to design these bracing members ; it is carried out interactively and allows for an optimum solution, that is characterized by the least weight at the most uniform buckling security within the individual bars. The input comprises four sections : i) the general geometry of the panel: widths at top and bottom and height ; ii) the applied residual shear force S at the top of the panel ; iii) the yield strength of the bracing members, and iv) the type of bracing. Four types of bracing are possible (fig. 2). The programme is not intended at all to replace a full analysis of the tower; it is just aimed at guiding the designer in coming to an acceptable preliminary design, that can be fully checked later. The user can takes advantage of the dialogue with the computer to reach a quasi-optimal solution for a specified type of bracing and eventually choose another type of bracing.



Note:Type 2 is with or without stayed across lower corners

Figure 1 - Types of bracing.

The second programme, called REDLOAD, facilitates the design of K-panels in latticed transmission steel towers, and more especially of redundants and secondary bars. Several patterns of redundants are possible. From the basic geometry information, the member of subpanels over the height of the frame, the leg load and a code number that denotes the minimum angle size it is desired to use, the programme gives automatically design loads and length of bars and then proceeds to design the sections, specify the bolts and gives reserve factors for both bars and bolts. It finally gives the weight of redundants in the panel.

Programmes REDBRACE and REDHIP are intended at designing automatically K-brace panels ; the components of the design are : i) the leg, ii) the main horizontal and iii) the main diagonal members.

4. A WAY TOWARDS THE ACTUAL BEHAVIOUR.

The actual behaviour of transmission towers cannot obviously be reflected by the previous design rules, but only either by a full scale test of the structure or by a numerical simulation on computer. The latter approach is quite suitable for parametric studies and is in addition not destructive. In this respect the junior author contributed much to the nonlinear finite element programme FINELG [5], that is able to compute the critical loads and the associated buckling modes as well as the whole behaviour up to and even beyond the ultimate load, account taken of geometrical and material non linearities. He developped more especially a very efficient spatial beam finite element [6] , the main pecularities of which are : account of warping, open or closed shape of cross-section made of thin walls, asymmetric cross section, eccentricities and semi-rigidity of end connections, residual stresses, elastoplastic material constitutive law, large displacements,... The reader who is interested in more details concerning this F.E. is begged to refer to [6].

With such a marvellous tool in hands, it was attractive to simulate the actual behaviour of a transmission tower that was tested in Great-Britain [7]. With a view to limit the duration of the computations, only a part of structure was studied; this part is composed of the experimentally collapse region and the adjacent space panels. The computations were carried out with account taken of initial bar out-of-straightness, rolling residual stresses, plasticity,dead load and eccentricities of connections. It is while mentioning that the F.E. used allows for a progression of yielding across the section and over the length of this element, so that a refined discretization is not at all necessary. An excellent agreement between the numerical and experimental results was found. On

the one hand, the collapse modes are identical and correspond to instability (fig. 2) with excess of plastic deformations in two same compression web members. On the other hand, the computed collapse load exceeds the experimental one by less than 6 %, what can be regarded as very successful account taken of the complexity of the problem. Another simulation was conducted by simply changing the orientation of the web members ; that means that the non-connected leg of the angles is oriented inwards instead of outwards previously, for compression members, and vice-versa, for tensile members. This change results in an increase of the computed collapse load, which now exceeds the experimental one by more than 13%. A non negligible gain of ultimate load can thus be obtained by connecting the compression members in such a way that their free leg be oriented inwards the tower. The bar out-of-straightness is another parameter which was also investigated, as well as the effect of the remaining part of the tower.
For the above computation, the initial geometrical imperfection was choosen similar to the first instability mode. The reference amplitude was equal to 1/1000 of the length of the bar, which was found the weakest by a bifurcation analysis; its sign was associated with the displacement of the same bar obtained by a first order computation. A change of that sign for all the bars yields a slight difference (< 1 %) in ultimate load and is thus not significative. Allowance was made for axial rigidity of the chords which connect the studied part to the foundation by introducing elastic spring supports (instead of fully restrained ones as considered in the previous computations); here too, only a slight drop (1,5 %) of the ultimate load is observed.

A step further [8] was to compare the results of some design rules (REDCROSS) for the chords, legs and bracing members with those drawn from the study, by means of FINELG, of the weakest plane panel extracted from the spatial tower part examined above. In other words, it was aimed to investigate whether the collapse locations coincide when either analysing successively the different bars of the structures or considering a plane panel. From this comparison, following conclusions are while being emphasised :

- The identification of the weakest panel, according to calculation of the axial forces in the bars based on the assumption of pin-ended bars, may be wrong. As a consequence, bracing members designed accordingly could be undersized.
- A more realistic value of the ultimate load would require that the spatial behaviour of the structure - i.e. the restraint provided by adjacent panels - be accounted for in the design. However it is still 20 % higher than the one found by a complete spatial analysis.
- It is compulsory to take account of connection eccentricities, otherwise the ultimate load can be overestimated by more than 80 %.

5. CONCLUSIONS

Though valuable design rules for the members and bracings of transmission towers are presently available, they must be reserved to preliminary design. It appears indeed that in spite of the fact that they were deduced from experimental evidence, they are not able to account for all the peculiar parameters which are likely to influence the ultimate load of such structures. A refined nonlinear analysis is the sole able to warrant for a specified safety. The old adage "You only get for what you pay for" has here an obvious relevance.

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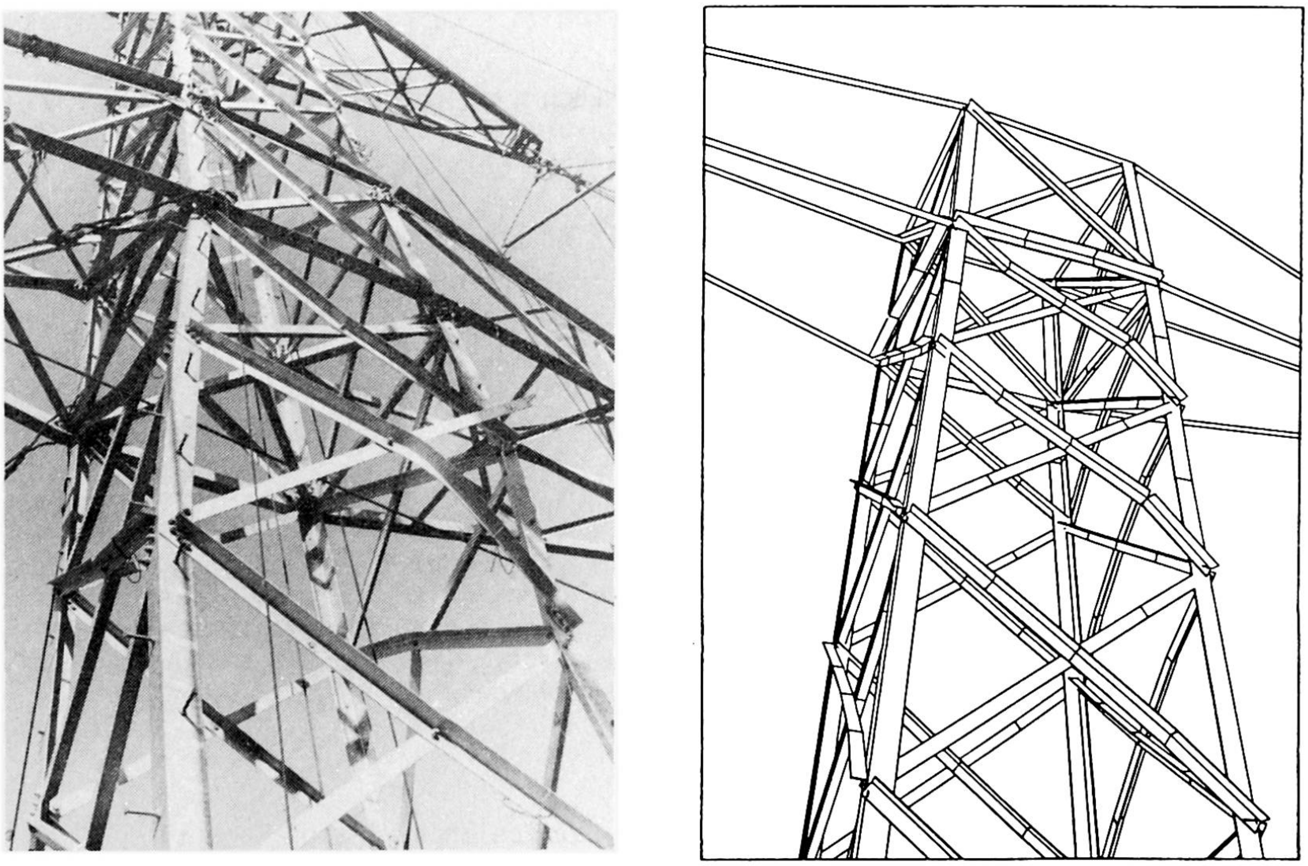

Figure 2

Challenge and Promise of Assessment of Structural System Reliability

Défi de l'évaluation de la fiabilité d'un système structural

Herausforderungen bei der Ermittlung der Zuverlässigkeit von Systemen

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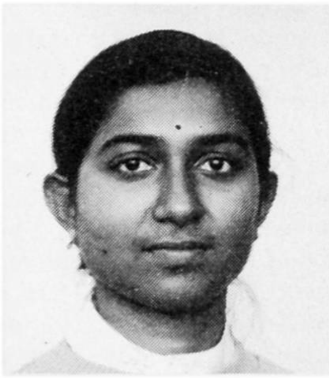

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SUMMARY
A brief description of some of the widely used methods for the evaluation of structural system reliability is given in the first part of the paper. An approach, in the form of software for a microcomputer, which uses simulation to compute the probability of failure of a structure is suggested. The method would yield reliable results and pinpoint the critical issues, enabling the engineer to make improvements in design.

RÉSUMÉ
Certaines méthodes largement utilisées pour l'évaluation de la fiabilité des systèmes structuraux sont présentées. Une approche en forme de logiciel pour micro-ordinateur, basée sur la méthode de simulation pour le calcul de la probabilité de ruine d'une structure est aussi présentée. La méthode proposée permet d'obtenir des résultats fiables, indique les paramètres critiques de la stucture et permet à l'ingénieur d'améliorer la conception.

ZUSAMMENFASSUNG
Der erste Teil dieser Arbeit behandelt häufig benützte Methoden zur Bewertung der Zuverlässigkeit von Bausystemen. Eine hier vorgeschlagene Methode macht Gebrauch von Software für Microcomputer, um die Wahrscheinlichkeit des Versagens einer Struktur zu bewerten. Diese Methode ist verlässlich und zeigt die kritischen Punkte, was dem Ingenieur erlaubt, den Entwurf zu verbessern.

1. INTRODUCTION

It has been established that most factors influencing the design and performance of structures are, in reality, uncertain. The structural design codes, consequently, are edging towards probability based methods with a view to rationalize and unify them. Engineers and researchers now recognize that the 'safety' or 'reliability' of a structure is of prime importance and have been formulating methods to quantify the reliability of a structural system. The objective of this paper is to review the methods currently used for the assessment and to suggest an approach which, in the authors' view, is likely to yield more reliable and realistic results by modelling the system with the aid of the rapidly increasing power and speed of microcomputers.

The existing methods used for the assessment of structural system reliability are classified and critically reviewed in the first part of the paper. The implementation of the more 'exact' methods, if not impossible, require the use of considerable statistical skill while the assumptions made in the simpler approaches leave the reliability of the prediction in some doubt; simulation techniques being used at times to assess the reliability and adequacy of the results obtained from these analytical approaches. The most commonly used approach to assessing the probability of failure of a structural system is to determine the upper and lower bounds of the probability. Researchers have suggested many methods for their computations but the reliability of the result depends on the assumptions made in the statistical theories of the idealized world and the narrowness of the margin between the bounds determined.

Monte Carlo simulation has often been used to model the failure mode of structural systems and to verify the results obtained by other methods but seldom as a tool for obtaining the probability of failure directly. The commonly stated reason for the reluctance to use simulation is the perceived computer time required to obtain a fairly reliable estimate. Great strides have been made in the development of computer technology and speed in recent years, presenting the possibility of overcoming this obstacle. The greatest advantage of using simulation techniques is that, unlike other methods, assumptions need not be made regarding the probability distribution of the variables and their correlations. Researchers have, additionally, been discouraged by the perceived difficulty in analysing the effects of even slight changes to the model. The development of specially suited software for use with a microcomputer would remove this difficulty and would not only give an estimate of the probability of failure, but would also be able to study the sensitivity of the objective to variation in the model and that of the model to changes in the variables within each model, enabling the most critical issues to be pinpointed [1].

2. REVIEW OF EXISTING METHODS

2.1 Elementary concepts of reliability

The values of relevant variables which would result in failure, or, in other words, the failure domain, may be separated from the safe region by a failure surface, the equation of which is termed the failure function. In the simplest case of a structural element with resistance R subjected to a load or load effect S (in similar units), the failure function G may be taken as the difference

$$G = R - S \quad ..... \quad (1)$$

Since R and S usually exhibit statistical dispersions, the probability of failure is found as follows if both variables are independent. The probability that the load lies in the range x, x+dx ($f_S(x)$) is found and multiplied by the cumulative probability that the resistance lies below x ($F_R(x)$). This product is summed over all possible values to give the probability of failure P_f as

$$P_f = \int_{-\infty}^{+\infty} f_S(x).F_R(x).dx \quad ..... \quad (2)$$

The reliability index β is defined as

$$\beta = \frac{\mu_G}{\sigma_G} \quad ..... \quad (3)$$

where μ_G - mean value of G
σ_G - standard deviation of G

The methods used for structural reliability assessment were classified into three categories, levels I, II and III, in early research and this grouping is still of some use [2,3]. A detailed description of the classification is given elsewhere [2].

2.2 Evaluation of reliability of structural elements

The first step towards the computation of the reliability of a structural system is the evaluation of the probability of failure of the elements in the system. The failure function g(X) in terms of the relevant variables $X_1, X_2, \ldots, X_n$ is formed and the joint probability density function

$f_{X_1, X_2, \ldots, X_n}(X_1, X_2, \ldots, X_n)$

of the variables is integrated over the region of failure to obtain the probability of failure P_f of the element.

$$g(X) = g(X_1, X_2, \ldots, X_n) \quad ..... \quad (4)$$

$$P_f = \int\int\ldots\int\int_{g(x)<0} f_{X_1, X_2, \ldots, X_n}(X_1, X_2, \ldots, X_n).dX_1.dX_2 \ldots dX_n \quad ..... \quad (5)$$

Due to the difficulties in forming the joint probability density function, simplifying assumptions are resorted to in determining P_f. Some of the methods used for the evaluation are described in the next few sections.

2.2.1 Numerical integration

The most reliable of the methods available, this involves the computation of P_f using equation (5) by numerical methods. In most circumstances, however, as the joint probability density function cannot be defined, the method would not be of much use.

2.2.2 Maximum entropy distribution method [4]

The first four statistical moments of the basic variables are used to determine the statistical moments of the failure function. A maximum entropy distribution is generated to fit the failure function and P_f is then computed using numerical integration. The method has been shown to yield reliable results under certain conditions.

2.2.3 Second moment methods

A simplification is made by expanding the failure function g(X) in a Taylor series about a point lying on the failure surface ($X_1^*, X_2^*, \ldots, X_n^*$). The series is then truncated at first order terms and approximate values found for the mean and variance of g(X). For uncorrelated variables,

$$\mu_g \simeq -\sum_{i=1}^{n} X_i'^* \left(\frac{\partial g}{\partial X_i'}\right)_* \quad ..... \quad (6)$$

$$\sigma_g^2 \simeq \sum_{i=1}^{n} \sigma_{X_i'}^2 \left(\frac{\partial g}{\partial X_i'}\right)_*^2 = \sum_{i=1}^{n} \left(\frac{\partial g}{\partial X_i'}\right)_*^2 \quad ..... \quad (7)$$

The first derivatives are evaluated at the chosen point on the failure surface and X_i' is the standard normal transformation of the variable X_i. The derivation of the equations is discussed at length elsewhere [5].

2.2.4 Monte Carlo simulation

Several trials are performed to model the failure function. Within each trial, a random value of each relevant variable is generated and the value of the failure function determined. The probability of failure can then be estimated by one of two approaches. The first involves dividing the number of trials where g(X) was found to be negative or zero by the total number of trials to give Pf. The second approach uses the values of g(X) generated to find the distribution of g(X) from which the area below zero is computed and taken as Pf. It must be pointed out that often researchers assume each basic variable to follow a stylised probability distribution, for example a normal distribution. Additionally, the basic variables are at times assumed to be independent. If a large number of variables are involved in a purely additive problem it may be acceptable to assume that the distribution of g(X) is normal, but this is seldom the case in reality. Monte Carlo simulation is advantageous in such situations as the probability distributions of the variables need not be assumed to follow a stylised distribution and correlations between variables, if they can be perceived and quantified, could easily be incorporated in the evaluation of the value of g(X) within each trial.

2.3 Evaluation of reliability of structural systems

It is not usually feasible to link the probability of failure of the elements to that of the structure directly as the elements can interact with one another. A simplification is made by classifying systems as either series, where the failure of any one element results in the failure of the system, or parallel, where each element must fail to cause the collapse of the system. In reality most structural systems are a combination of the two. If for example a framed structure exhibiting plastic mechanisms of collapse is considered, the occurrence of each mode may be taken as equivalent to a parallel system as all the plastic hinges necessary to cause the mechanism must occur. The failure of the structure, however, would be equivalent to a series system of different modes of failure i.e. different mechanisms, as the occurrence of just one mode results in system collapse. Since there are complications in deriving the probability of failure of the system, relatively simple methods are used to compute the upper and lower bounds of the probability of collapse Pf. The most commonly used bounds are the simple bounds and the Ditlevsen's bounds. In the former, the bounds are given by

$$\operatorname*{Max}_{i=1}^{m} (Pfi) \leqslant Pf \leqslant 1-\prod_{i=1}^{m}(1-Pfi) \quad \text{for a series system and}$$

$$\prod_{i=1}^{m} (Pfi) \leqslant Pf \leqslant \operatorname*{Min}_{i=1}^{m} (Pfi) \quad \text{for a parallel system}$$

where

m - number of modes of failure

Pfi - collapse probability of the ith mode

The Ditlevsen's bounds apply for a series system and are more reliable as the range between the bounds is narrower. The expressions for the bounds are

$$Pf1 + \sum_{i=2}^{m} \operatorname{Max}(Pfi - \sum_{j=1}^{i-1} Pfij;\ 0) \leqslant Pf \leqslant \sum_{i=1}^{m} Pfi - \sum_{i=2}^{m} \operatorname*{Max}_{j<i} Pfij$$

where Pfij is the probability that the failure functions of the ith and jth modes both indicate failure. These bounds require the consideration of all possible pairs of failure modes.

Many researchers have considered different types of structures and suggested different methods by which an estimate of the probability of failure of the system may be found. For example, in the case of framed structures, Stevenson and Moses [6], Kam, Corotis and Rossow [7], Moses [8], Murotsu et. al. [9], Ang and Ma [10], Bennett and Ang [11] and Ranganathan and Deshpande [12] are only a few of the many researchers who have suggested varying methods of estimating Pf. Most of the researchers have assumed the basic variables as following a normal distribution when using examples to illustrate their methods. Some researchers have considered the variables to be correlated while others have assumed them to be independent. A few researchers have used simulation to verify the results obtained by using their proposed methods. It becomes clear that many assumptions can once again be avoided if simulation was used to determine the value of Pf directly.

3. THE CHALLENGE AND THE PROMISE

3.1 The challenge

It can be seen from section 2 that estimating the probability of failure of a structure is a formidable task and has been tackled only by making simplifying assumptions which may not be applicable in real situations. There exists a need, therefore, for developing an approach which is simple to use and yields more reliable results.

3.2 The promise

It has been stated that Monte Carlo simulation is the only method by which a reliable estimate of system collapse probability may be found using the statistics of collapse of each mode [8]. Engineers are often discouraged from using probabilistic analysis to solve a problem but welcome the concept of modelling a structure using Monte Carlo simulation. It is the authors' view that developing computer software using Monte Carlo simulation to predict the probability of failure would therefore not only yield realistic results but would also be more widely accepted. A minimum of statistical knowledge would be necessary for such an approach. The next section gives a brief description of the method.

4. THE RECOMMENDED APPROACH

Figure 1 shows in the form of a flow chart the steps involved in using Monte Carlo simulation for the prediction of the probability of failure Pf. The program 'Venturer' [1] could be used with ease to determine the probability distribution of the basic variables if very little information is available. It is envisaged that the structure would be fed into the program in the form of a network or tree diagram, enabling the computer to determine the various modes of failure that would be possible. The correlation between variables can easily be accommodated in the simulations carried out to determine the probability of failure of each mode of collapse. These would then be combined to estimate the probability of collapse of the structure itself. The software would plot on the screen the possible modes of failure, indicating that which is most likely to occur. By ensuring that the software is user-friendly, it could be used to encourage the user to try a variation in the model or in the variables and observe the effect of these changes. Relatively fewer simulations could be carried out at this stage until the model and variables had been defined to the user's satisfaction. A large number of trials could then be performed to get an estimate for the value of Pf.

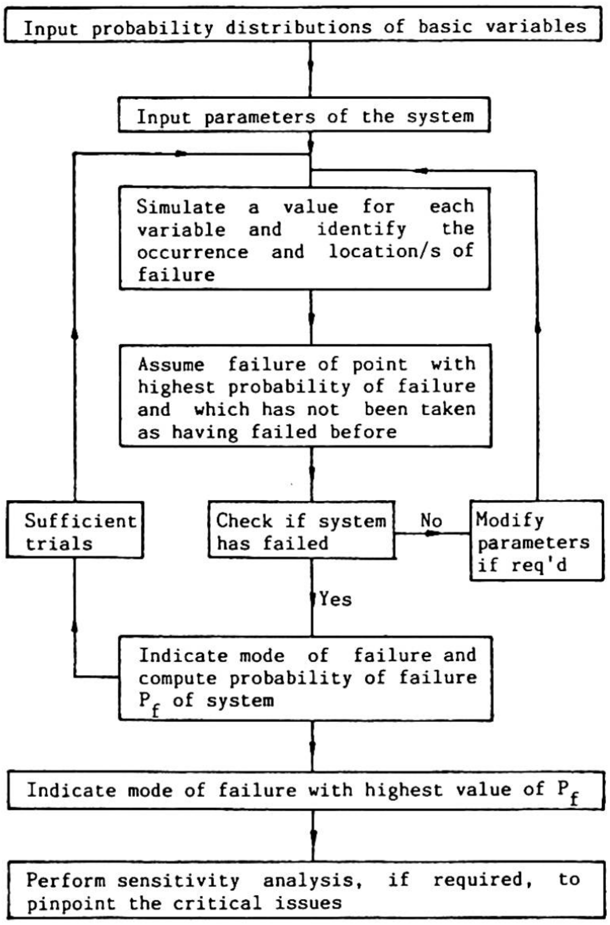

Fig. 1 Flow chart for method

Another advantage in the package would be the sensitivity analysis that could be carried out to determine the sensitivity of the model to changes in the variables. The variables that are most critical would be pinpointed at this stage giving the engineer an opportunity to alter the design. The changes in the value of P_f due to changes in the model could also be analysed and would give the user more insight into the problem. The authors have pursued the approach detailed above and found the method to yield promising results.

5. CONCLUSIONS

It has been pointed out that at present, most methods available for the evaluation of structural reliability or probability of failure involve making assumptions. The suggested approach using Monte Carlo simulation would result in a simple and acceptable software which can be used for a variety of problems and would result in reliable results. The advancement of computer technology has introduced the possibility of using simulation as a viable method for finding the system collapse probability in its own right instead of being used as a calibration tool.

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Zuverlässigkeitsuntersuchungen an Stahlkonstruktionen

Reliability Analyses of Steel Constructions

Analyses de la fiabilité de constructions en acier

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ZUSAMMENFASSUNG
Der Beitrag berichtet über Ergebnisse von Zuverlässigkeitsanalysen, die an eingeschossigen Zweigelenkrahmen aus Stahl durchgeführt wurden. Besonders herausgearbeitet wird die Tatsache, daß der Sicherheitsindex nur in Verbindung mit den Eingangsdaten (stochastische Modelle der Lasten und Materialeigenschaften) und den Analysemethoden interpretierbar ist.

SUMMARY
Results of reliability analyses are reported concerning singlestory double hinged steel frames. The fact is emphasised that the safety index is only interpretable in the context of both the basis date (stochastic models of loads and material properties) and the methods of analysis used.

RÉSUMÉ
L'article présente des résultats d'analyses de fiabilité faites sur des portiques en acier à deux articulations et à un étage. On insiste sur le fait que l'indice de sécurité ne peut être interprêté qu'en rapport avec les données d'entrée (modèles stochastiques des charges et des caractéristiques des matériaux) et les méthodes d'analyse.

1. EINLEITUNG

Die Zuverlässigkeitstheorie der Baukonstruktionen ist ein leistungsfähiges Instrument zur Einschätzung der Sicherheit von Tragwerken. In den letzten Jahren ist diese Theorie immer weiter vervollkommnet und verfeinert worden. Eine nur noch schwer überschaubare Anzahl von Veröffentlichungen ist dieser Theorie gewidmet. Von den guten zusammengefaßten Darstellungen sei nur auf [1, 2, 3] verwiesen.

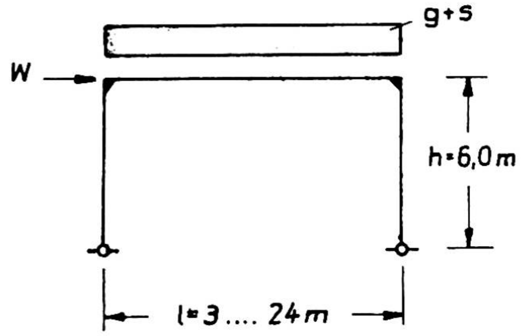
Als spärlich dagegen ist die Zahl der Veröffentlichungen zu werten, die Ergebnisse zuverlässigkeitstheoretischer Untersuchungen und dabei gewonnene praktische Erfahrungen vermitteln.

Der vorliegende Beitrag berichtet über Ergebnisse diesbezüglicher Untersuchungen am Lehrstuhl Metallbau der Technischen Hochschule Leipzig.

2. ERGEBNISSE VON ZUVERLÄSSIGKEITSANALYSEN AN ZWEIGELENKRAHMEN AUS STAHL

Es wurden u. a. auch Zweigelenkrahmen aus Stahl untersucht, und es soll hier über dabei gewonnene Ergebnisse berichtet werden.

Diese Zweigelenkrahmen könnten z. B. die Haupttragkonstruktion von Hallen repräsentieren. Abmessungen und Belastung zeigt Fig.1.



Die Rahmen wurden nach Elastizitätstheorie 2. Ordnung mit den in der DDR verbindlichen Lastannahmen gemäß TGL 32274 bemessen, d. h. folgende Normlasten, Lastfaktoren und Lastkombinationen liegen der Bemessung zugrunde.

Fig.1 Abmessungen und Belastung des untersuchten Stahlrahmens

Schnee: $s = 0{,}5\ \mathrm{kN/m^2}$ $\gamma_f = 1{,}1 \cdot 1{,}4 = 1{,}54$

Wind : Staudruck $0{,}55\ \mathrm{kN/m^2}$ $\gamma_f = 1{,}2$

Eigenlast: $g = 0{,}5\ \mathrm{kN/m^2}$ $\gamma_f = 1{,}1$ (d. h. es handelt sich um eine sehr leichte Dacheindeckung)

Lastkombination Eigenlast + Schnee : $1{,}1 \cdot g + 1{,}54 \cdot s$

Lastkombination Eigenlast + Wind : $1{,}1 \cdot g + 1{,}2 \cdot w$

Lastkombination Eigenlast + Schnee + Wind: $1{,}1 \cdot g + 0{,}9 \cdot 1{,}54 \cdot s + 0{,}9 \cdot 1{,}2 \cdot w$

Nach in der DDR gültigen Stahlbaunorm ist weiterhin ein Materialfaktor $\gamma_m = 1{,}15$ bei der Bemessung zu berücksichtigen.

Nachrechnungen nach Normen anderer Länder und Vergleiche mit Anbietern bei internationalen Ausschreibungen zeigen, daß sich nach DDR-Normen bemessene Stahlkonstruktionen nur unwesentlich von anderen unterscheiden (insbesondere hinsichtlich der erforderlichen Querschnitte).

Die so bemessenen Rahmen wurden anschließend mittels zuverlässigkeittheoretischer Methoden 1. Ordnung untersucht. Die Grenzzustandsbedingung wurde aus der Fließgelenktheorie 2. Ordnung hergeleitet. Das Ergebnis ist aus Fig.2 ersichtlich.

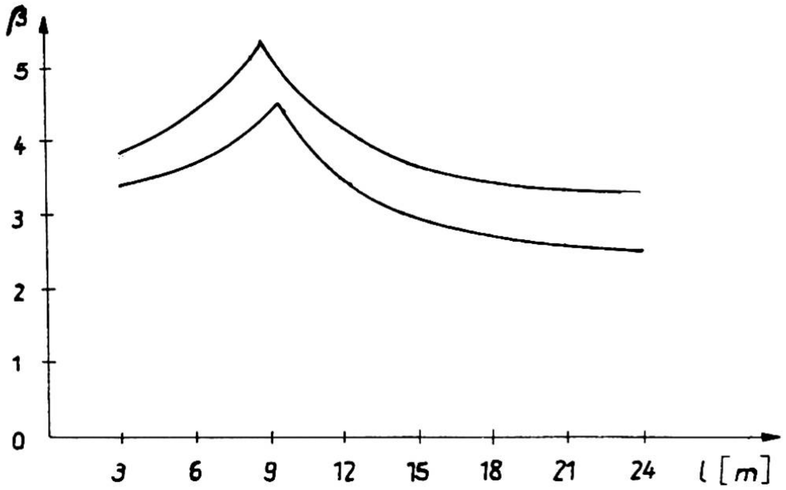


Fig.2 Sicherheitsindex ß (Einschrankung) für nach DDR-Norm bemessene 6 m hohe Zweigelenkrahmen, die durch niedrige Eigenlast, Schnee und horizontalen Wind beansprucht werden.

Bei der Beurteilung dieser Werte ist zu beachten:

- Mittelwert und Standardabweichung für die als extremwert-I-verteilten Schnee- und Windlasten wurden aus statistischen Daten meteorologischer Stationen hergeleitet (she. Abschnitt 3.1 und 3.2). Das führt zu einer schärferen Betrachtungsweise, als wenn man (günstigerweise) annimmt, daß die derzeitigen Lastannahmen einer 98 %- oder 99 %-Fraktile entsprechen.
- Es wurde ein Bezugszeitraum von 50 Jahren zugrunde gelegt.
- Die untere Kurve ergibt sich, wenn man für die Querschnittsfestigkeit von der unteren Fließgrenze ausgeht, die zudem nicht unbeträchtlich streut (s. Abschnitt 3.4), wenn man die möglichen Versagensmechanismen (Fließgelenkketten) als Seriensystem auffaßt und wenn man zwischen den Fließgelenken bezüglich deren Festigkeit vollständige Korrelation voraussetzt. Man kann also bezüglich der Tragfähigkeitsannahmen von einer unteren Schranke sprechen.
- Die obere Kurve ergibt sich, wenn man für die Querschnittsfestigkeit von der oberen Fließgrenze ausgeht (was eine durchaus übliche Annahme ist), wenn man den Sicherheitsindex ß nur bezüglich des kritischen Versagensmechanismus bestimmt und wenn man zwischen den Fließgelenken bezüglich deren Festigkeit keinerlei Korrelation voraussetzt. Man kann also bezüglich der Tragfähigkeitsannahmen von einer oberen Schranke sprechen.

Tab. 1...3 zeigen Forderungen an den Sicherheitsindex ß im Grenzzustand der Tragfähigkeit, wie sie von verschiedenen Richtlinien bzw. Empfehlungen erhoben werden.

Sicherheitsklasse	1	2	3
ß für Bezugszeitraum 1 Jahr	4.2	4.7	5.2

Tabelle 1 Na Bau: Grundlagen zur Festlegung von Sicherheitsanforderungen für bauliche Anlagen (BRD)

Versagensfolgen	weniger schwer	schwer	sehr schwer
ß für Bezugszeitraum 1 Jahr	3.1	3.7	4.2

Tabelle 2 Recommendation for Loading- and Safety Regulations for Structural Design NKB Report No.36 The Nordic Committee on Building Regulations

Zuverlässigkeitsklasse	V	IV	III	II	I
ß für Bezugszeitraum 1 Jahr	3.2	3.7	4.2	4.7	5.2
ß für Bezugszeitraum 50 Jahre	2.5	3.0	3.5	4.0	4.5

Tabelle 3 Zuverlässigkeitskonzeption für tragende Baukonstruktionen, Bauakademie der DDR, Mai 1987

Aus der unterschiedlichen Höhe dieser Forderungen kann man nicht auf unterschiedlich hohe Zuverlässigkeitsniveaus schließen, vielmehr ist es notwendig, die Rahmenbedingungen festzulegen, innerhalb derer geforderte ß-Werte gelten (Fig.2 verdeutlicht die Streuung, die allein aus unterschiedlichen Annahmen auf der Tragfähigkeitsseite bzw. aus der Berücksichtigung des Systemcharakters resultieren). Bemerkenswert ist auch, daß man bei der Erarbeitung der neuen Stahlbaunorm in den USA von einem erforderlichen ß = 2.7 ... 3.0 ausging [4] was u. E. auf eine realistische Betrachtungsweise schließen läßt.

3. DISKUSSION DER DEN BASISVARIABLEN ZUGRUNDE GELEGTEN STOCHASTISCHEN MODELLE

3.1 Windlasten

Die Norm-Windgeschwindigkeit v_{TGL} nach TGL 32274/07 ist das von der Höhe über Gelände abhängige 2 Minuten-Mittel, das durchschnittlich in 5 Jahren einmal erreicht oder überschritten wird. Für die Jahresmaxima der Windgeschwindigkeit in Europa gilt die Extremwertverteilung vom Typ I

$$F_I^{(1a)}(v) = \exp\{-\exp[-a_w(v - \hat{v})]\} \qquad (1)$$

Der Modalwert $\hat{v}$ und das Streuungsmaß a_w bestimmen sich mit

$a_w\hat{v} = 10$ für Mitteleuropa und

$$F_I^{(1a)}(v_{TGL}) = \exp\{-\exp[-a_w(v_{TGL} - \hat{v})]\} = 1 - \frac{1}{5} = 0,8$$

zu $a_w = 11,50/v_{TGL}$ und $\hat{v} = 0,8696\ v_{TGL}$.

In der DDR ist der Lastfaktor für Wind 1,2, für turmartige Bauwerke 1,3. Nimmt man an, daß er nur die Streuung der Windgeschwindigkeit repräsentiert, so ist der Rechenwert der Windgeschwindigkeit $\sqrt{1,2}\ v_{TGL} = 1,095\ v_{TGL}$. Einsetzen dieses Wertes in Gl.(1) ergibt eine Unterschreitungswahrscheinlichkeit von 0,9282, das entspricht einer Wiederholungsperiode von 14 Jahren.

Die Verteilungsfunktion der Windgeschwindigkeit für die betrachtete Lebensdauer von 50 Jahren berechnet sich zu

$$F_I^{(50a)}(v) = [F_I^{(1a)}(v)]^{50}$$

Für die Geschwindigkeit v_{10} [m/s] in 10m Höhe ist $v_{TGL} = 29,6$ m/s, und man erhält

$$F_I^{(50a)}(v_{10}) = \exp\{-\exp[-0,3885\ (v_{10} - 35,81)]\}$$

und für den v_{10}^2 proportionalen Staudruck $q_{10}[kN/m^2]$

$$F_I^{(50a)}(q_{10}) = \exp\{\exp[-8{,}33\,(q_{10} - 0{,}801)]\}$$

Die Höhenabhängigkeit des Staudrucks wurde berücksichtigt. Des weiteren wurden die Staudruckwerte analog [5] mit dem Faktor 0,75 reduziert, da der Wind nicht immer aus der für die Konstruktion ungünstigsten Richtung weht.

3.2 Schneelasten

Für die Jahresmaxima der Schneelast wird ebenfalls eine Ex I-Verteilung angesetzt.

$$F_I^{(1a)}(s) = \exp\{-\exp[-a_s(s - \hat{s})]\} \qquad (2)$$

Für die Schneelast auf dem Boden im Flachland der DDR (bis 250 m Seehöhe) gilt

$$\hat{s} = 0{,}13 \text{ bis } 0{,}24 \text{ kN/m}^2$$
$$a_s = 4{,}35 \text{ bis } 5{,}88 \text{ m}^2\text{/kN}$$

Im folgenden wird gerechnet mit $\hat{s} = 0{,}24\ (0{,}13)$ kN/m^2 und $a_s = 5$ m^2/kN. Auf dem Dach werden 70 % der Schneelast auf dem Boden angesetzt.

$$\hat{s}_D = 0{,}7 \cdot 24 = 0{,}168\ (0{,}091) \text{ kN/m}^2$$
$$a_{sD} = 5/0{,}7 = 7{,}142 \text{ m}^2\text{/kN}$$

Setzt man diese Parameter in Gl.(2) ein, so ergibt sich für die Norm-Schneelast im Flachland $s_{TGL} = 0{,}50$ kN/m^2 eine Unterschreitungswahrscheinlichkeit von 0,9109 (0,9475), entsprechend einer Wiederholungsperiode von 11(19) Jahren. Mit dem Lastfaktor 1,4 betragen die analogen Werte für die Rechen-Schneelast 0,9779 (0,9872) bzw. 45 (74) Jahre. Rechnet man auf 50 Jahre um, so sind in Gl.(2) die Parameter

$$\hat{s}_{D,50} = \hat{s}_D + \ln 50/a_{sD} = 0{,}716\ (0{,}639) \text{ kN/m}^2$$
$$a_{sD,50} = a_{sD} = 7{,}142 \text{ m}^2\text{/kN}$$

Für die kombinierte Belastung durch Schnee und Wind wurden entsprechend der Regel von Borges-Castanheta sowohl der 50-Jahres-Schnee mit dem 1-Jahres-Wind als auch der 1-Jahres-Schnee mit dem 50-Jahres-Wind gemeinsam angesetzt.

3.3 Eigenlast

Die Untersuchung bezieht sich auf einen niedrigen Eigenlastanteil. Der Normwert der Eigenlast verhält sich zum Normwert der Schneelast wie 1 : 1. Weiter wird angenommen, daß der Mittelwert der Eigenlast gleich dem Normwert und der Variationskoeffizient 0,05 beträgt.

3.4 Querschnittsfestigkeit

Auf der Grundlage einer umfangreichen Studie [6] von internationalen Veröffentlichungen zur Fließgrenze des Stahles und zu den Toleranzen des Walzstahles wurden für die Zuverlässigkeitsanalysen folgende Annahmen getroffen:

- Die Fließgrenze des Stahles wird als normalverteilt angenommen
  a) bei Zugrundelegung der von der Metallurgie "gewährleisteten" Fließgrenze: $\bar{x} = 1{,}2 \cdot R_{Norm}$, $v = 0{,}10$
  b) bei Zugrundelegung der unteren Fließgrenze: $\bar{x} = 1{,}1 \cdot R_{Norm}$, $v = 0{,}085$

- Die Querschnittswerte werden normalverteilt angenommen mit folgenden Parametern: $\bar{x}$ = 1,0 · Normwert, v = 0,03 bei Blechen und Profilen, v = 0,06 bei Rohren [7].

4. SCHLUSSFOLGERUNGEN

Für die Weiterentwicklung der Stahlbauweise, insbesondere für die weitere Normenarbeit, ist es wünschenswert, die Ergebnisse an unterschiedlichen Stellen durchgeführter Zuverlässigkeitsanalysen über Veröffentlichungen auszutauschen.

Für die Vergleichbarkeit ist es notwendig, nicht nur Sicherheitsindices oder Versagenswahrscheinlichkeiten anzugeben,sondern auch die Voraussetzungen, unter denen diese gewonnen werden, also gewählte stochastische Modelle für die Basisvariablen,Methoden, die der Analyse zugrunde liegen usw. Noch effektiver wäre es,für massenhaft durchzuführende Zuverlässigkeitsanalysen eine Rahmenrichtlinie zu erarbeiten und zu veröffentlichen, was nicht im Widerspruch zur Weiterentwicklung der Theorie an sich steht. Fig.2 deutet die Streubreite der Ergebnisse allein aus unterschiedlichen Annahmen auf der Tragfähigkeitsseite an. Unterschiedliche Annahmen auf der Lastseite haben noch größere Auswirkungen. Es ist ein realistisches Herangehen bei Zuverlässigkeitsanalysen gefragt, auch wenn dies dazu führt, Forderungen bezüglich der Größe von ß für die Normenarbeit neu zu überdenken. Versagenswahrscheinlichkeit bzw. Sicherheitsindex sind zu diesem Zweck operative Größen, die nur aus Rückrechnungen an über Jahrzehnte bewährten Konstruktionen bestimmt werden können. Als relative Bezugsgrößen sind sie aber durchaus geeignet, das Sicherheitsniveau der Baukonstruktionen zu homogenisieren. Z. B. erweist sich die Lastkombinationsregel der DDR-Norm nach Fig.2 als konservativ, da sie im mittleren Bereich, wo Wind und Schnee etwa gleichen Einfluß auf die Bemessung haben, zu deutlich höheren ß-Werten führt.

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