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

Band: 7 (1964)

Rubrik: Ib: Function and use of model tests

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Ib1

The Aid of Models in the Analysis of the Behavior of Structures Within and Beyond the Elastic Range

La contribution des modèles à l'étude du comportement des ouvrages au-delà et en deçà de la limite élastique

Modellversuche für die Untersuchung des Verhaltens von Bauwerken inner- und außerhalb des elastischen Bereiches

GUIDO OBERTI
Prof., Politecnico di Torino

1. The possibility of investigating on scale models the behavior of structures both within and beyond the elastic range has been long proved by the writer analytically and experimentally on a great number of structures. These were, in particular, plain or reinforced concrete structures, for whose design the writer was directly responsible alone or in association with others.

A model is a highly valuable element, and its importance is now universally acknowledged for analyzing structural behavior under elastic conditions in order to obtain the magnitudes of the strains, and hence of the stresses, likely to originate in the full-scale structure under working load. These results are valuable for many reasons. First, they make it possible to compare the values determined experimentally with those obtained by using the conventional calculation methods which, as is known, are impaired by a number of limitations, assumptions and simplifications unnecessary in a model. Second, when the model is used as a clever calculating machine, it can provide the numerical solution of three-dimensional elasticity problems which cannot be solved analytically both because of their extreme complexity (only partly reduced by the use of electronic computers) and, even worse, because of the difficulty encountered in introducing in the theoretical considerations sufficiently accurate boundary conditions.

But, even if the investigation should basically remain within the elastic range, its extension beyond that range, though less rigorous than the former and, under certain aspects, complementary to it, is still always an invaluable source of precious information and training for the engineer who is charged with the high responsibility of designing and erecting the structure.

In fact, this investigation may lead to determine and to locate the weak and less efficient points of the structure. It may also furnish (or at least confirm) the order of magnitude of the factor of safety and thus provide the designer and the commissioning party with an assurance that is final and conclusive or complementary, depending on the individual point of view.

2. In the ambit of structural problems, the following classification for models may be made:

a) according to the type of problem:
   1. static models,
   2. dynamic models;

b) according to the extent of the expected results:
   1. elastic models (i. e., valid in the elastic range only),
   2. global models (indicating the structural performance also under unelastic conditions up to failure).

The present-day trends of modelling engineering, with respect to the research and experimentation carried out by the writer (especially at ISMES, the Experimental Institute for Models and Structures, Bergamo, Italy), may be summarized as follows:

— The interest in modelling is increasingly being shifted to global models, along with the general tendency to have all designed structures calculated to the breaking point.
— As to elastic models, their use is steadily growing as very rough research instruments for investigating problems wherein the elastic behavior supplies, by interpolation, adequate information regarding also the unelastic performance. Of the recent uses in this branch of research we shall mention in particular the investigation of slabs and bridge decks by the Moiré method.
— The materials for elastic and global models are undergoing constant improvement in relation to the problems being investigated, so as to have them adjusted in the best possible way to the aims pursued.
  For elastic models, ISMES has recently succeeded in using epoxilic resins mixed with various materials. This permits to obtain a wide range of elastic moduli, in accordance with the requirements of each individual case, and stress-strain relations that are proportional also when the stresses are high.
— There is an increasing interest on the part of the customers, especially for large hydroelectric schemes, to investigate the equilibrium of vast valley systems affected by the schemes and whose original conditions may somehow be modified by the presence of the hydro plants.
  At ISMES there have lately thus been studied on models equilibrium problems of dam and abutment mountain systems, whose geomechanical characteristics have conveniently been schematized on the basis of geognostic investigations.
— With respect to the foregoing paragraph it may be pointed out that, as an aid to studies on models, are increasingly used and recommended in-situ and laboratory investigations of the geomechanical and geotechnical features of the materials.

— Similarly, there is a growing interest in tests dealing with thermal stresses in structures, with particular regard to dams as a consequence of impounding and drawing down the reservoir water in hydroelectric plants. The same is true also with respect to problems relating to the statics of reinforced, prestressed or not, concrete containers for nuclear reactors.

Summary

The writer first recalls the importance of scale models in analyzing the static and dynamic behavior of structures stressed also beyond the elastic range. He then uses the investigations carried out under his supervision, especially at ISMES, Bergamo, to present a classification of the research possibilities now available.

Résumé

On rappelle tout d'abord le rôle important des maquettes dans l'étude du comportement statique et dynamique des constructions sollicitées aussi au-delà du domaine élastique. L'auteur se réfère ensuite aux recherches qu'il a conduites, notamment à l'ISMES, à Bergame, pour présenter une classification des possibilités qui s'offrent maintenant à la recherche.

Zusammenfassung

Es wird zuerst die große Bedeutung der Modellversuche zur Abklärung des statischen und dynamischen Verhaltens von Bauwerken inner- und außerhalb des elastischen Bereiches gezeigt. Anhand der Ergebnisse der Forschungen an der ISMES, Bergamo, welche unter seiner Leitung durchgeführt wurden, klassifiziert der Autor die heute zur Verfügung stehenden Forschungsmöglichkeiten.

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Ib2

Dynamic Structural Studies on Models

Essais sur modèles dynamiques

Untersuchungen an dynamisch beanspruchten Modellen

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1. Introduction

Important improvements in dynamic structural studies have only recently been possible due to better understanding of the fundamental character of most of the dynamic loads. It is particularly so for wind and earthquake actions.

Machine vibrations may, in general, be well represented by sinusoidal varying forces. The same is not true for wind and earthquakes loads. Convenient information on the random character of these actions has been obtained only very recently [1] and [2].

Two types of randomness must be distinguished. The first refers to the probability of occurrence of an action with a given intensity. The second to the random character of the vibration itself, in the sense that, although with different power levels, random vibrations simultaneously include a large range of frequencies. In the following this last type of randomness is the only one considered.

Civil engineering studies on wind actions usually assimilate these actions to static forces, the wind velocity being supposed constant. Even in the problems of aeroelasticity, particularly those concerning the aerodynamic stability of suspension bridges and stacks, the real turbulent character of wind is, generally, not considered.

Most of the modern earthquake engineering studies suppose the random character of seismic vibrations, defining this through mean velocity spectra. This representation, very convenient for the study of the behaviour of one-degree of freedom linear oscillators, can only be generalized to systems of several degrees of freedom through simplifying hypotheses and is not applicable for non-linear behaviour.

Recent studies showed that earthquakes can be well represented by a random vibration of constant spectral density of acceleration in the range 0 to 5 Hz, and zero density beyond this range. For reference, the spectral density may be taken equal to 675 $cm^2 s^{-4} Hz^{-1}$ which corresponds to the recorded N-S component El-Centro, 1940, earthquake. Duration is generally taken equal to 30 s.

A more accurate representation of an earthquake could be obtained by assuming that the spectral density of acceleration changes in function of the frequency according to a law analogous to that of a simple oscillator transfer function. The data so far available, based on recorded earthquakes, is not yet sufficient to justify this refinement.

2. Studies in the Linear Range

The matric formulation of dynamical problems allows the analytical study even of involved structures. Numerical solutions can be easily obtained with modern electronic computers.

For determining the vibration modes, the knowledge of the stiffness and mass matrices is sufficient. The analytical determination of the stiffness matrix is sometimes involved and can be substituted by an experimental determination based on model tests. Model tests may also be very useful for a check of the simplifying assumptions to be considered in the analytical methods.

If complete information on the dynamic behaviour is needed, damping can not be disregarded. In this case dynamical tests on models may give convenient information but it is necessary to use models built of the same materials as the prototype. Even so, as the influence of scale in the damping factors is not yet well known, it is necessary to judge the results taking in consideration the values of the damping factors determined in dynamic tests of real structures of the same type. The problem is particularly involved because damping may increase with the vibration amplitudes and, in general, it is only possible to study the behaviour of real structures for vibrations of very small amplitude.

Theoretically, if the structure behaves linearly, it is possible to derive the behaviour under the action of random vibrations by studying the behaviour for sinusoidal vibrations. The simplest way would be to experimentally determine the transfer functions of the magnitudes to be determined. By multiplying the spectral densities of acceleration by the corresponding values of the square of the transfer function, the spectral density of the response of the structure is computed. Integrating this response for the range of frequencies considered, the mean square value of the interesting quantities (displacements, strains, stresses) is then obtained. Finally, if maximum values (mean maximum values or extreme values with a given probability) are to be obtained, the root mean square values must be multiplied by suitable coefficients.

The method just described is, in practice, difficult to apply and, in general, the results obtained are not sufficiently accurate. This is why a different testing technique has been adopted at the Laboratório Nacional de Engenharia Civil in Lisbon [3].

Models are directly submitted to random vibrations and the magnitudes of interest, such as displacements and strains are directly recorded. In the case of earthquake studies, the duration of the earthquake is also reduced to scale, several tests being performed for a level of acceleration. So, it is easy to determine for each test the maximum values of the magnitudes of interest, and to compute from several tests the mean of these maximum values. It has been shown that these mean maximum values are in general the magnitudes of interest for design purposes [4].

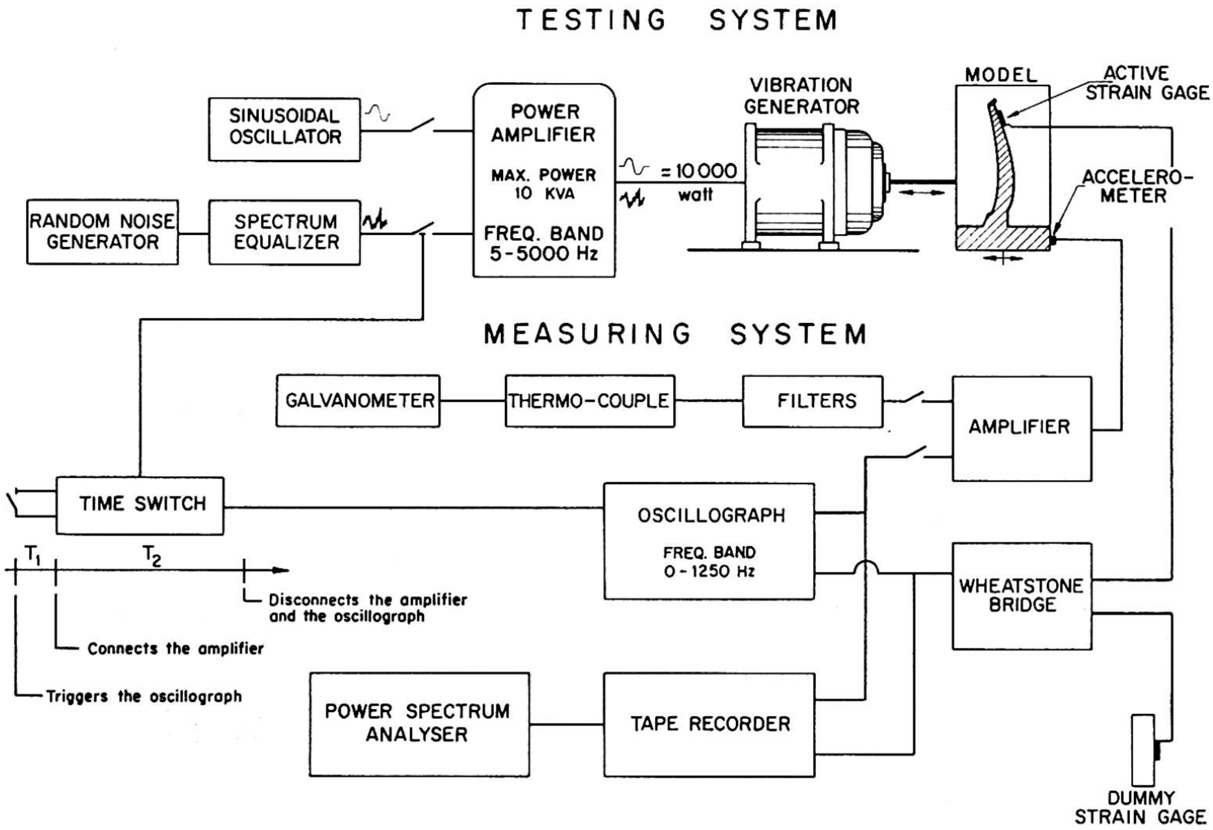
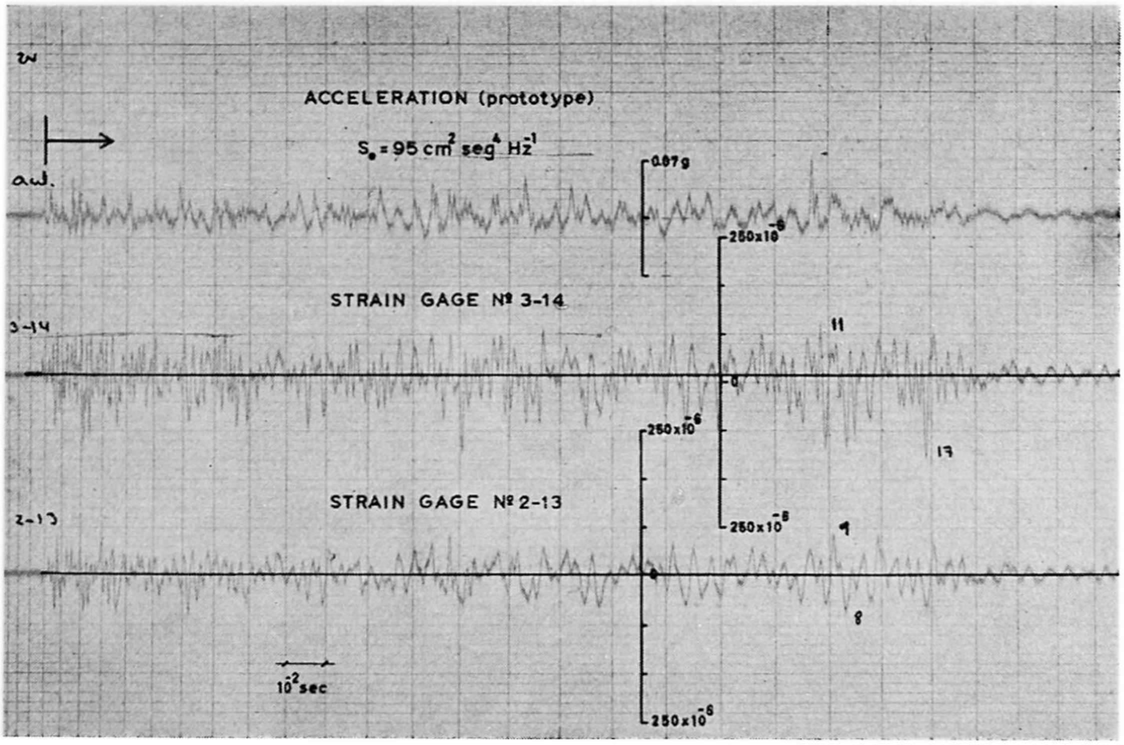


Fig. 1.

A diagram of the testing set-up used for this type of tests is presented in fig. 1. Both sinusoidal and random vibrations may be induced in the model by an electromagnetic vibrator and for the latter it is possible, by acting on a spectrum equalizer, to adjust the convenient values of the acceleration spectral density at the different frequency ranges. The quantities of interest such as acceleration, displacements and strains, can be directly recorded on paper (fig. 2) or in magnetic tape. Tape records are used in an electronic spectrum analyser to determine spectral density diagrams.

The testing of a buttress dam model is shown in fig. 3 and the testing of



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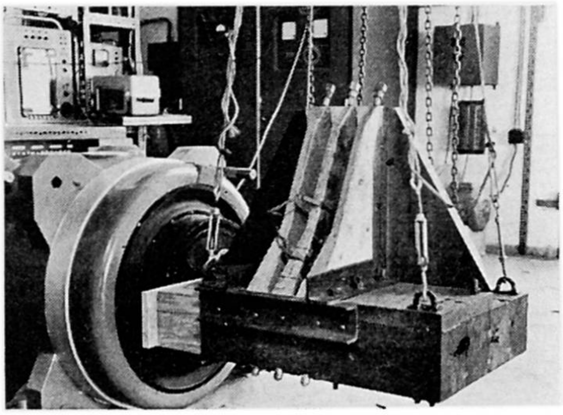

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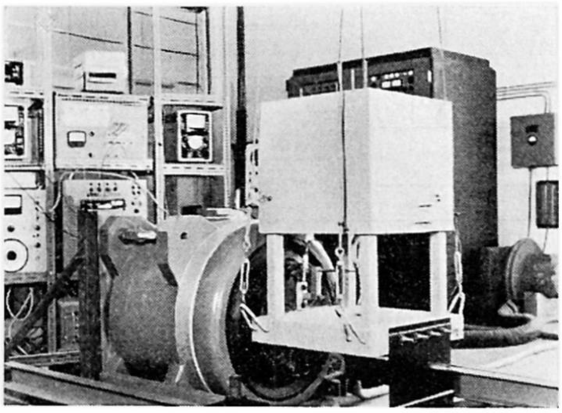

Fig. 4.

a reinforced concrete model that reproduces an usual type of building is presented in fig. 4.

3. Studies on the Non-Linear Range

The need to study structural behaviour in the non-linear range is well recognized. In fact if safety against rupture is to be judged, plastic deformations must be considered. This is particularly true for earthquake actions.

In fact, earthquake actions usually give rise to displacements exceeding those of elastic behaviour. By increasing the stiffness of the structure the situation is not modified because, if the stiffness increases, seismic forces also increase.

The analytical study of non-linear behaviour, both elastic or hysteretical, is much more involved than in the linear case. At present only very simple problems can be dealt with.

Numerical methods based on finite differences are easy to program for the digital computer and may give valuable information. Results of similar type may be obtained in analogical computers, even in a more economical way. But it must be considered that these methods may only be used by introducing many simplifying hypotheses whose validity it is very difficult to judge.

Structural models are particularly useful for studies on the non-linear range. If the testing technique described above is used, it is sufficient to conveniently increase the spectral level of acceleration to follow the behaviour in the non-linear range, till rupture is attained.

Also in this case the most convenient materials to build the models are those of the prototype, this being the easiest way to maintain their rheological properties, but, it can not be forgotten that dynamic similitude imposes a change of frequency range from the prototype to the model, and this change may affect the interesting mechanical properties. This is a problem that deserves further study.

Also it may occur that, after a certain level of vibration, the behaviour of the structure be modified by incipient ruptures or cracks, the structure being, even so, able to sustain important horizontal forces.

In this case it is necessary to further analyse the behaviour taking in consideration that the initial deterioration may affect the similitude conditions. Gravity forces may then be of paramount importance as compared with elastic forces. Concrete dams are a good example of cases where problems of this type occur. Also for this purpose model tests may give very valuable information, difficult to obtain by other means.

4. Conclusions

The main conclusions that can be derived concerning dynamic structural studies are the following:

a) Improvement in dynamic structural studies must be based in the convenient representation of dynamic actions. Particularly, the random character of wind and earthquake actions must not be forgotten.

b) The statical determination of the stiffness matrix by model tests may be of much help for the further analytical solution of vibration problems.

c) Dynamical model tests in the elastic range may give complete information on structural behaviour, the determination of transfer functions being particularly recommended.

d) For judging the safety against rupture it is necessary to explore the dynamic behaviour in the non-linear range. In the case of brittle structures it may be necessary to study the behaviour after cracks have developed.

e) The testing technique described seems to be particularly convenient, since it directly allows the measurement of the magnitudes of interest.

Bibliography

1. A. G. DAVENPORT: "The Application of Statistical Concepts to the Wind Loading of Structures". The Institution of Civil Engineers, Proceedings, vol. 19, August, 1961.
2. G. W. HOUSNER: "Characteristics of Strong-Motion Earthquakes". Bulletin of the Seismological Society of America, vol. 37, 1947.
3. J. FERRY BORGES, J. PEREIRA, A. RAVARA and J. PEDRO: "Seismic Studies on Concrete Dam Models». Symposium on Concrete Dam Models, Laboratório Nacional de Engenharia Civil, Lisbon, October, 1963.
4. J. FERRY BORGES: "Structural Behaviour and Safety Criteria". Preliminary Publication, Seventh Congress, International Association for Bridges and Structural Engineering, Rio de Janeiro, August, 1964.

Summary

The usefulness of model tests for dealing with dynamic structural problems is discussed. Attention is called to the possibility they afford to study the dynamic behaviour both for sinusoidal and random vibrations, in the linear and non-linear ranges.

Special reference is made to the testing techniques adopted at the Lisbon research institute, LNEC.

Résumé

L'auteur discute de l'utilité des essais sur modèles dans la résolution des problèmes dynamiques. Il rappelle que ces essais permettent d'étudier le comportement dynamique pour des vibrations aussi bien sinusoïdales qu'aléatoires, dans les domaines linéaire et non-linéaire.

On mentionne tout particulièrement les techniques d'essai employées au laboratoire de recherches de Lisbonne, LNEC.

Zusammenfassung

Es wird über die Nützlichkeit von Modellversuchen bei der Behandlung dynamisch beanspruchter Tragwerke gesprochen. Besonders hervorgehoben wird die sich dabei bietende Möglichkeit, das dynamische Verhalten sowohl unter sinusförmigen als auch unter beliebigen Schwingungen im linearen und nicht linearen Bereich zu untersuchen.

Die verschiedenen vom Laboratório Nacional de Engenharia Civil (LNEC) in Lissabon angewendeten Prüfmethoden werden besonders hervorgehoben.

Ib3

The Use of Model Tests in Bridge Analysis

L'emploi d'essais sur modèles dans l'étude des ponts

Modellversuche im Brückenbau

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1. Introduction

The lack of a rigorous mathematical treatment of certain types of bridge structures very often compels designers to use model tests to understand their behaviour. Such is the case, for instance, of bridge decks formed by a rectangular, tee-beam or hollow slab, non-rectangular in plan, or with marked curvature in horizontal and vertical planes (fig. 1) and of all bridge solutions with an important three-dimensional behaviour.

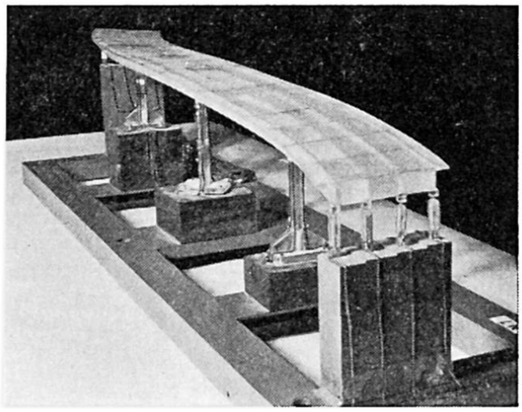


Fig. 1.

These difficulties are related in the papers of R. E. ROWE and BEST [1] and VOJTECH MICHÁLEK and VLADIMIR BKEZTNA [2] presented at the Congress under theme Ib.

In the present contribution the author summarizes his experience in model tests of bridges developed at the "Laboratório Nacional de Engenharia Civil" (Lisbon) illustrating it with some studies he had the opportunity to perform.

Some aspects connected with the above mentioned papers are discussed.

2. Construction of the Model

2.1. Scales

In order to minimize the construction of the model, structural studies for determination of the internal forces (i.e. M, N, T) in some cross sections of bridges are usually carried out on models as small as the dimensions of the measuring instruments available allow.

The scales of overall models range from $^1/_{50}$ to $^1/_{500}$, being as a rule approximately $^1/_{100}$ for bridges with a total apan of about 100 m. Models are sometimes distorted with regard to the similitude of their cross section: The members of the model are constructed with the same proportions as those of the prototype with regard to the moment of inertia but the shape of the cross sections is simplified, the similitude of normal and shear forces being therefore changed.

Structural studies of details in which the field of stresses in certain zones has to be analysed are usually carried out with larger models (scales $^1/_2$ to $^1/_{30}$). In these models only the member or group of members to be studied are reproduced as is emphasized by G. K. JEWGRAFOW and B. W. BOBRIKOW in their paper [3] presented at the Congress.

For these models boundary conditions are maintained as accurately as possible.

2.2. Materials

After attempts to use different materials, notably celluloid as employed by Prof. EDGAR CARDOSO in his elastic model studies [4], we have finally settled on acrylic resins for overall model studies. Other plastics, such as polyester resins and polyethylenes, were abandoned either because of difficulties with their mechanical characteristics or with the stability of measuring devices [6].

The acrylic plastics or methyl methacrylate resins are on sale under different trade-names such as "Perspex" (I.C.I.), "Plexiglas" (Röhm and Haas), "Lucite" (Dupont), etc. as sheets and profiles, 0.5 to 50 mm in thickness.

The average mechanical characteristics of acrylic plastics are as follows:

Modulus of elasticity	$30—40 \times 10^3$ kg/cm^2
Poisson's ratio	0.38
Ultimate tensile strength	300—500 kg/cm^2
Coefficient of thermal expansion	$7—10 \times 10^{-5}/°$C

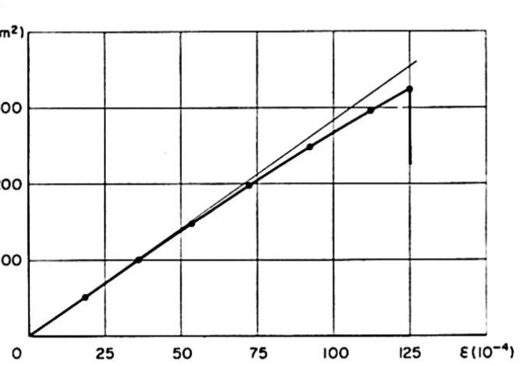
The stress-strain diagram is nearly a straight-line (fig. 2) and there are a time dependent moduli of elasticity. The loading-readings are carried out in a constant period of time ($\sim$5 seconds) for all measurements.

The pieces cut from acrylic resin sheets or profiles are glued together with a chloroform solution of the material itself. A satisfatory gluing requires perfectly smooth flat surfaces which can be obtained by machining with a milling-cutter or a shaper.

In models constructed to smaller scales for detail studies of concrete bridges micro-concrete is often used to reproduce the properties of prototype material as is also emphasized in paper [1].

2.3. *Loading System*

The structural problems posed by the action of permanent loads and live loads can be solved, as a rule, knowing the influence surfaces of the internal forces in some cross sections due to vertical loads on the deck. These influence surfaces are obtained in our models by applying concentrated loads to various points of a grid system drawn on the surface of the deck. The loads are applied by means of a hook fitted with a counterweight (fig. 3).



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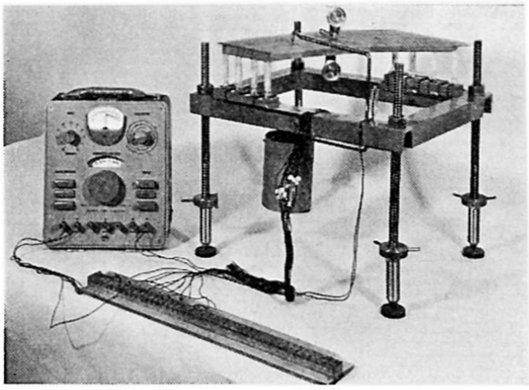

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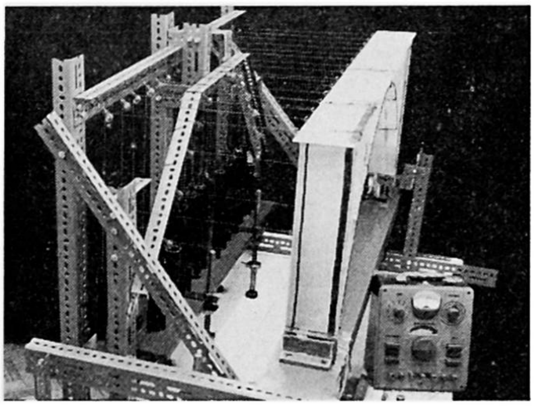

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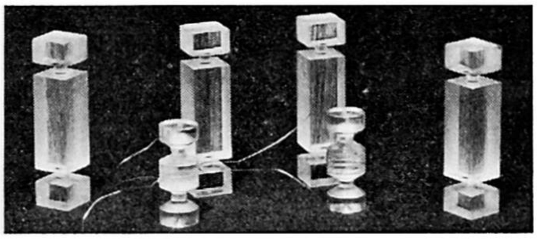


Fig. 5.

In order to minimize creep effects, strains and deflections are measured point by point 5 seconds after application and removal of the loads.

Other actions, such as wind loads, are studied by replacing them by the equivalent static forces (fig. 4).

3. Tests

3.1. *Measuring Instruments*

In our small-scale models we use the smallest and least stiff strain meters and dial gauges available. Strain-meter bases range from 5 mm to 20 mm, according to the dimensions of the model and the stress gradient around the observed point.

Paper-based electrical resistance strain gauges and rosettes glued on the models with "DUCO" cement are still the most widely used, in spite of attempts, due to their cost and inevitable loss, to replace them by small reclaimable strain meters.

The reactions of fixed or movable supports are measured in the models by replacing the supports by previously calibrated cells of acryllic material with a pendular behaviour (fig. 5).

Deflections in reference to one outside fixed system are measured with dial gauges.

3.2. Results

The internal forces in the cross sections observed are calculated from the experimental values by means of the similitude theory [5].

As a rule, the known quantities are the strains $\epsilon_1, \epsilon_2, \epsilon_3$ at the points observed on the surfaces on the model and the displacements u, v, w of some points of the model with respect to an external fixed system. The internal forces — M, N, T — and the displacements are required at the homologous points of the prototype.

The accuracy of the results obtained is always checked on basis of the deviations between the equilibrium of the applied forces and of the internal forces in the sections observed. Desviations up to 5% are deemed tolerable.

4. Examples

4.1. Increase of internal forces in the member of a ribbed deck due to the asymmetry of the live loads

M_m, N_m and T_m being the average values of the internal forces in a cross section of a deck rib (assuming that the action of the live load is uniformly distributed in all the members) and M_{max}, N_{max} and T_{max} being the maximum values of these same magnitudes in the same cross section due to asymmetric loads in the cross section under consideration, the designer usually requires the values of the coefficients:

$$\eta_M = \frac{M_{max}}{M_m}; \qquad \eta_N = \frac{N_{max}}{N_m}; \qquad \eta_T = \frac{T_{max}}{T_m} \qquad (1)$$

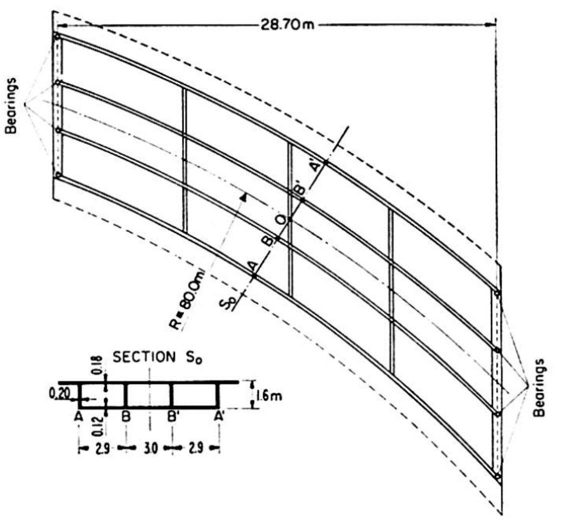
for each rib and cross section.

Although for some typical decks analytic methods are available for determining η_M, model tests have to be resorted to in current cases for obtaining more accurate values.

In fig. 6 is shown the plan and the middle cross section S_0 of the three-cells box girder, curved in plan, presented in fig. 3.

The influence line along S_0 of the moments M_A in web A for vertical loads on the deck is indicated by the full line in fig. 7, and the values obtained by an approximate analytical computation by means of a mesh analogy are shown by the dotted line.

It was found that for our standard truck with wheel axles 2.0 m apart, the experimental value η_M is 1.5 and the expected analytical value 2.5.



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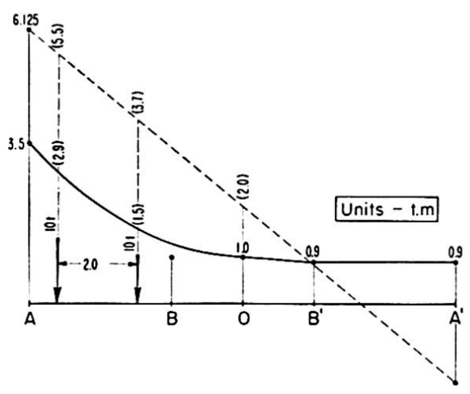

Fig. 7.

4.2. *Influence Surfaces in Slab-Decks*

This is a study, more general than 4.1, which can be included in it as a particular step of preceding analysis.

ϵ_x and ϵ_y being the strains due to a normal concentrated load measured along two orthogonal directions in one face of the slab-deck model, the following moments act along the same directions in the prototype*):

$$M_x = \chi\lambda\frac{E_m}{1-\nu^2}(\epsilon_x+\nu\,\epsilon_y)\frac{e^2}{6},$$
$$M_y = \chi\lambda\frac{E_m}{1-\nu^2}(\epsilon_y+\nu\,\epsilon_x)\frac{e^2}{6}. \qquad (2)$$

$1/\lambda$ being the scale of the model, $\chi=F_p/F_m$ the ratio of the forces acting on the prototype to those acting on the model, E_m and ν the modulus of elasticity and Poisson's ratio of the material of the model and e the thickness of the slab in the model. F_p are usually assumed equal to the unit force. If we had instead a ribbed or hollow slab the experimental problem would not be difficult as elongation ϵ_1 would be measured along the beam and ϵ_2 along a direction normal to it.

The moments in the beam in the prototype are now given by

$$M_1 = \chi\lambda\frac{E_m}{1-\nu^2}(\epsilon_1+\nu\,\epsilon_2)\,W_1. \qquad (3)$$

---

*) Internal normal forces in the slab are not considered.

W_1 being the modulus of the section of the beam in the model with respect to the face in which ϵ_1 was measured.

Fig. 8 shows the influence surface of the moments in cross section between a a' of rib 3′ of a skew ribbed slab.

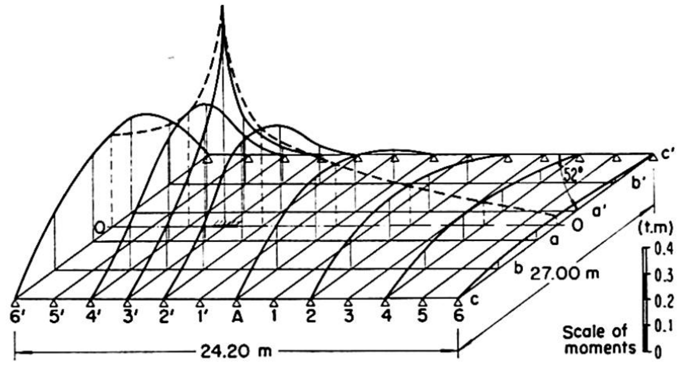

Fig. 8. Bending moments influence surface in beam 3′ section 0—0′.

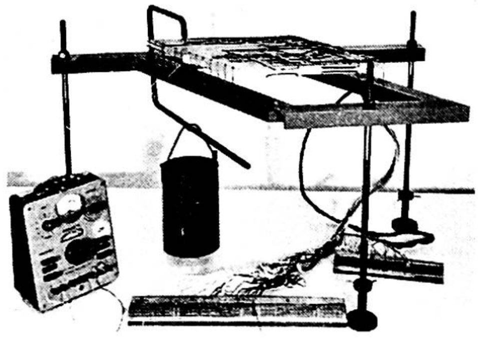

Fig. 9.

The model, made of "Plexiglass" to scale 1:75, was loaded with a concentrated force of 7.5 kg (fig. 9) and consequently $\chi\lambda = 10{,}000$.

The moments in the longitudinal and cross beams that meet at an angle of 52° were computed by (3).

4.3. *Three Dimensional Behaviour*

As a rule the analytical study of a bridge with an important three-dimensional behaviour is very difficult and inaccurate, whereas a model study is comparatively easier and more reliable. An instance of this is presented in the paper [2] about the model study of an important box-girder bridge.

These studies are also of particular interest in decks carried by very deformable substructures such as slender piers, deformable arches, ties, and so on.

As a rule the aim of a model study of this type is to obtain the influence surfaces of the internal forces M, N and T in a given cross section of a member of the bridge, under a vertical load acting at different points of the deck.

Assuming the common case of uniaxial stress let W_{I} and W_{II} be the moduli of the section with respect to the principal axes, A the section area and ϵ_{I_1}, ϵ_{I_2}, ϵ_{II_1}, ϵ_{II_2} the strains at surfaces along the planes containing the axes of the member and the principal axes I and II of the section, we have.

$$\begin{aligned} N &= \chi A\, E_m \frac{\epsilon_{\mathrm{I}_1} + \epsilon_{\mathrm{I}_2} + \epsilon_{\mathrm{II}_1} + \epsilon_{\mathrm{II}_2}}{4}, \\ M_x &= \chi\lambda\, W_{\mathrm{I}}\, E_m \frac{\epsilon_{\mathrm{I}_1} - \epsilon_{\mathrm{I}_2}}{2}, \\ M_y &= \chi\lambda\, W_{\mathrm{II}}\, E_m \frac{\epsilon_{\mathrm{II}_1} - \epsilon_{\mathrm{II}_2}}{2}. \end{aligned} \tag{4}$$

Shear forces in a particular cross section of a member can be determined from the strain measurements in models with dimensions enabling the installation and reading of strain rectangular rosettes applied at an angle of 45° with the axis of the member.

Let ϵ_{a_1}, ϵ_{a_2} and ϵ_{b_1}, ϵ_{b_2} be the strains measured along two directions at 45° to the axis of the member in opposite directions in respect to a longitudinal plane containing one of the principal axis of the section. We can write for a rectangular profile of the model:

$$T_m = \frac{G_m A}{3} (\epsilon_{a_1} + \epsilon_{a_2} + \epsilon_{b_1} + \epsilon_{b_2}), \tag{5}$$

in which G_m represents the modulus of elasticity in shear of the material of the model.

In a prismatic piece subjected to linear bending the measurement of bending strains ϵ_1 and ϵ_2 at two cross sections, of the same face at a distance d, enable to calculate the constant shear force:

$$T_m = \frac{E}{d} (\epsilon_1 - \epsilon_2) W. \tag{6}$$

In fig. 10 and 11 is presented, as an example of a study of this type, the influence surface of the bending moments M_x at the base of one pier (in the direction of its maximum moment of inertia) of a skew bridge under the action of a concentrated vertical load applied in the deck. It is noteworthy that the points where bending moments change signs are difficult to determine by analytic means.

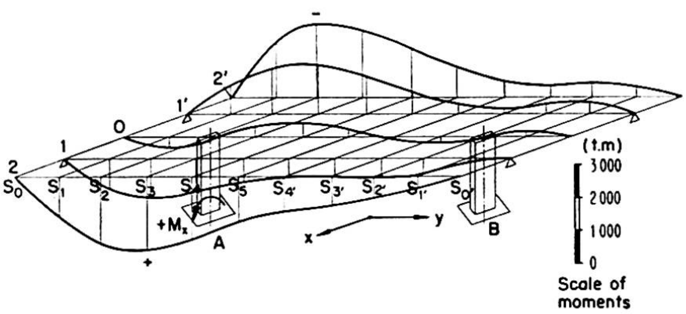

Fig. 10. Moments M_x in the base of pier A.

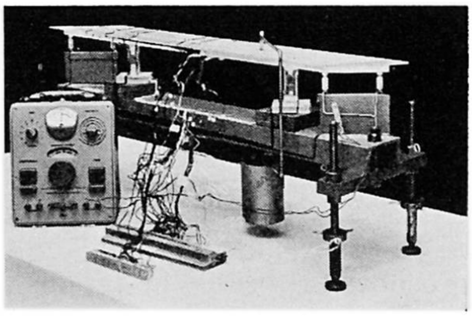

Fig. 11.

Another exemple of a three-dimensional study is presented in figure 12 in which are summarized some experimental results of the three dimensional behaviour of a deck of an arch bridge 211 m in span.

In figures 12a and 12b are represented the influence lines of the bending moments in two cross section of the longitudinal deck beams for different positions of a vertical load moving along the three vertical plans of the arch ribs.

Considerably differences was found between the experimental values and those that could be obtained from the current methods of analysis.

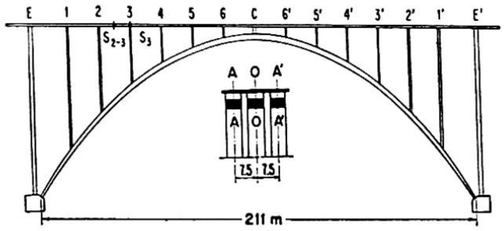

Fig. 12.

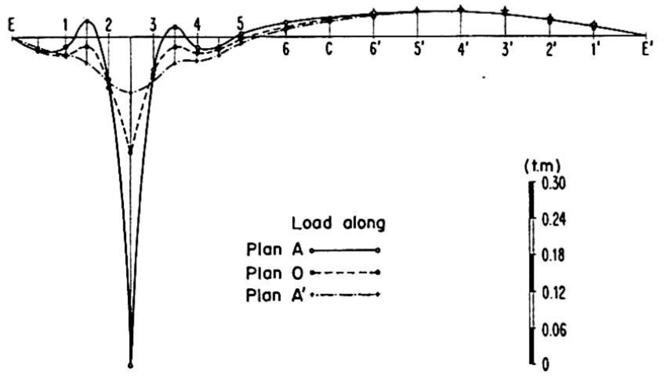

Fig. 12a. Moments in section S_{2-3} of deck beam A.

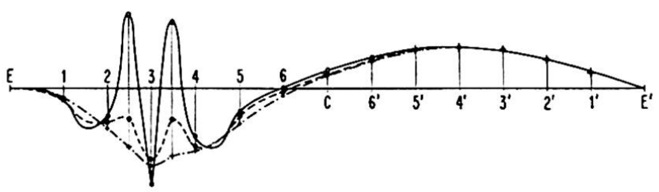

Fig. 12b. Moments in section S_3 of deck beam A.

5. Conclusions

Although not new, but now merely more exploited, model tests open new prospects in bridge design as the examples and the studies presented at Congress clearly show.

A. In bridge structures for which mathematical treatment is either insufficiently developed or deemed less accurate, model tests are an important basis for a research study program and a source of important design data.

B. In bridge structures for which mathematical methods of analysis exist, model tests sometimes suggest anomalies in design assumptions enabling the adoption of others, closer to their real behaviour.

C. Model tests also enable the development of new structural shapes, suggested by designer's experience and intuiton and seldom applied due to lack of an easy checking basis.

References

1. R. Rowe and B. Best: "The Use of Model Analysis and Testing in Bridge Design." Paper presented at 7th Congress of IABSE.
2. Vojtěch Michálek and Vladimír Březina: "Model Tests on a Box Girder Bridge." Paper presented at 7th Congress of IABSE.
3. G. K. Jewgrafow and B. W. Bobrikow: «Modelluntersuchungen an einem Brückenfachwerk aus Spannbeton.» Paper presented at 7th Congress of IABSE.
4. E. M. Cardoso: «Alguns métodos de cálculo experimental e sua aplicação ao estudo de pontes.» Lisbon, 1950.
5. M. C. Rocha: «Dimensionamento experimental das estruturas.» Técnica N. 215, 216 and 217, Lisbon 1951.
6. J. F. L. Fialho: "The use of plastics for making structural models." Bulletin RILEM No. 8, 65—74, Sept. 1960.

Summary

The author sums up the model testing methods in use at the LNEC (Lisbon) in the study of bridge structures, presenting the results of major engineering interest of some completed studies.

Résumé

L'auteur résume les techniques employées au LNEC (Lisbonne) pour les essais sur modèles d'ossatures de ponts; il présente les résultats techniquement les plus intéressants de quelques-uns des travaux réalisés.

Zusammenfassung

In der vorliegenden Arbeit faßt der Autor die im LNEC (Lissabon) verwendeten Untersuchungsmethoden für Brückenmodelle zusammen. Er stellt einige Versuchsergebnisse von hoher technischer Bedeutung dar.

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Diskussion - Discussion - Discussion

Theoretische und experimentelle Untersuchung eines unsymmetrischen Kastenträgers mit zwei Flanschen (R. Dziewolski)[1]

Theoretical and Experimental Researches of an Asymmetrical Box Girder with two Flanges

Etude théorique et expérimentale d'une poutre en caisson asymétrique avec deux appendices

F. STÜSSI
Prof. Dr., Präsident der IVBH, ETH, Zürich

Im Beitrag von R. DZIEWOLSKI wird ein Kastenträger mit freien Flanschen unter Torsionsbelastung rechnerisch untersucht und die Ergebnisse der Berechnung werden durch Spannungsmessungen an einem Modell überprüft. Die Berechnung ist auf die Methodik der Elastizitätstheorie orientiert, und sie arbeitet mit einigen Begriffen wie $f(z)$ = Verwölbungsfunktion («fonction de gauchissement»), ϕ = sektorielle Fläche (Verwölbungsfläche) des geschlossenen Querschnitts («surface sectorielle [surface de gauchissement] des profils fermés»), $S_{\phi c}$ = sektorielles statisches Moment des geschlossenen Querschnitts («moment sectoriel statique des profils fermés») usw., die in der normalen Festigkeitslehre nicht vorkommen und die darum wenig anschaulich sind. Es dürfte deshalb vom Konstrukteur aus gesehen von Interesse sein, dieser Berechnung eine Methode gegenüberzustellen, die auf die Betrachtungsweise der normalen Baustatik orientiert ist. Es handelt sich grundsätzlich ja darum, daß der Konstrukteur selber das Kräftespiel in seinen Tragwerken beurteilen können muß, weil die konstruktive Durchbildung des Tragwerkes durch dieses Kräftespiel bedingt ist. Es dürfte deshalb wohl nicht ernsthaft bestritten werden können, daß immer dort, wo dies möglich ist, diejenige Berechnungsmethode den Vorzug verdient, die mit den gebräuchlichen Begriffen der Baustatik arbeitet und deshalb, von der Baustatik aus gesehen, anschaulich und direkt überprüfbar ist.

Diese baustatische Darstellung teilt das Torsionsmoment T auf in einen ersten und einen zweiten Torsionsanteil:

$$T = T_1 + T_2. \tag{1}$$

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[1]) Siehe «Vorbericht» — see "Preliminary Publication" — voir «Publication Préliminaire», Ib 3, p. 131.

Der erste Torsionsanteil T_1 wird durch den Schubfluß s_1 aufgenommen,

$$T_1 = 2\,F_m s_1, \tag{2}$$

wobei also in den freien Flanschen keine Schubspannungen wirken. Nun muß aber unter der Wirkung dieses Schubflusses s_1 die Querschnittsform erhalten bleiben; dies ist jedoch in der Regel nur möglich, wenn neben dem Schubfluß s_1 noch Normalspannungen σ auftreten. Die zugehörige Elastizitätsbedingung lautet somit

$$\frac{s_1'}{a_i d_i G} + \frac{\sigma_{i-1} - \sigma_i}{a_i b_1 E} = \varphi_1'' = \text{konst.} \tag{3}$$

Dabei bedeutet a_i den Abstand der Scheibe b_i mit Stärke d_i vom ersten Schubmittelpunkt 0_1, der mit dem von R. Dziewolski verwendeten Schubmittelpunkt übereinstimmt. Schreiben wir Gleichung (3) in der Form

$$\sigma_{i-1} - \sigma_i + \frac{E}{G}\,\frac{b_i}{d_i}\,s_1' = E\,a_i\,b_i\,\varphi_1'',$$

so heben sich bei der Summation über alle zum geschlossenen Querschnittsteil gehörenden Scheiben die Spannungen σ heraus und mit

$$a_{11} = \sum \frac{b_i}{d_i}, \qquad 2\,F_m = \sum a_i\,b_i$$

folgt

$$\varphi_1'' = \frac{a_{11}}{G\,2\,F_m}\,s_1'. \tag{3a}$$

Damit kann Gleichung (3) in der Form

$$\sigma_{i-1} - \sigma_i = \frac{E}{G}\left(\frac{a_{11}}{2\,F_m}\,a_i\,b_i - \frac{b_i}{d_i}\right) s_1' \tag{3b}$$

geschrieben werden und liefert nun die gesuchten Spannungswerte σ in Verbindung mit einer Gleichgewichtsbedingung

$$\int \sigma\, dF = 0$$

in Funktion von s_1' bzw. die Spannungsänderungen σ' in Funktion von s_1'' (Fig. 1).

Durch Aufsummieren der Spannungsänderungen σ' entstehen Schubspannungen τ_0,

$$\tau_0\, d = -\int \sigma'\, dF + C, \tag{4}$$

wobei die Integrationskonstante C passend so gewählt wird, daß Momentengleichgewicht besteht:

$$\int \tau_0\, a'\, dF = 0 \tag{4a}$$

oder durch die aus den Schubspannungen τ_0 entstehenden Scheibenquerkräfte $\mathfrak{Q}_0$ ausgedrückt

$$\sum \mathfrak{Q}_{0i} a_i' = 0. \tag{4b}$$

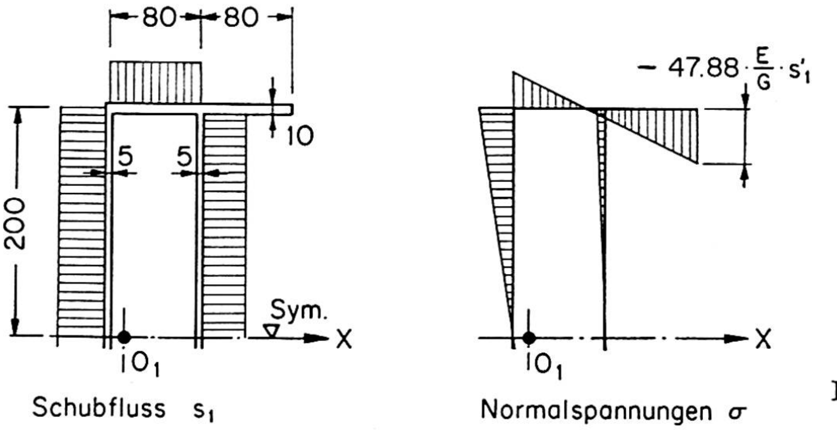

Fig. 1.

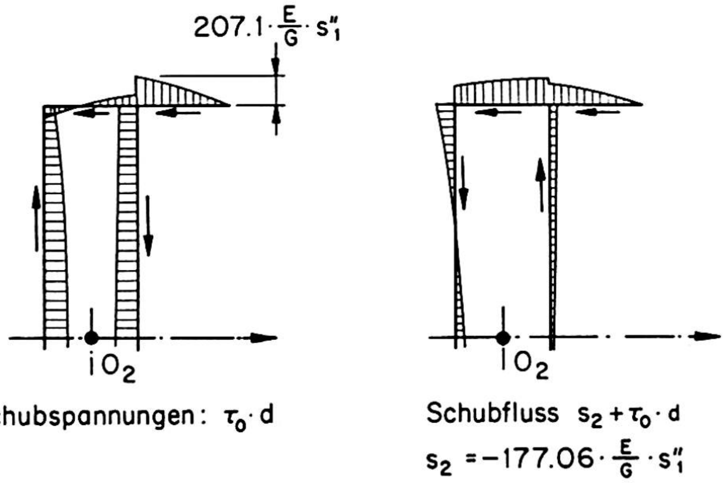

Fig. 2.

Dabei beziehen sich die Abstände a_i' auf den zweiten, durch die Schubverformungen allein bestimmten Schubmittelpunkt 0_2. Für den zweiten Torsionsanteil T_2 muß die Querschnittsform ebenfalls erhalten bleiben, und die entsprechende Elastizitätsbedingung lautet

$$\left(\frac{\mathfrak{Q}_{0i}}{b_i d_i} + \frac{s_2}{d_i}\right)\frac{1}{a_i'} = G \varphi_2' = \text{konst.} \tag{5}$$

Schreiben wir diese Gleichung in der Form

$$\mathfrak{Q}_{0i} a_i' + s_2 a_i' b_i = a_i'^2 b_i d_i G \varphi_2',$$

so heben sich bei der Summation über alle Scheiben des Querschnitts, einschließlich der freien Flanschen, die Momente $\sum \mathfrak{Q}_{0i} a_i'$ wegen Gleichung (4b) heraus und es bleibt

$$s_2 \sum a_i' b_i = s_2 \, 2 F_m = G \varphi_2' \sum a_i'^2 b_i d_i. \tag{5a}$$

Damit kann Gleichung (5) in der Form

$$\mathfrak{Q}_{0i} = s_2 \left[\frac{2 F_m}{\sum a_i'^2 b_i d_i} a_i' b_i d_i - b_i \right] \tag{5b}$$

geschrieben werden und wir erhalten aus der Summation über alle Scheiben des Querschnitts den Wert des zweiten Schubflusses s_2 in Funktion von s_1'' (Fig. 2),

$$s_2 = -C s_1''$$

und damit liefert die Gleichgewichtsbedingung mit

$$T_2 = 2 F_m s_2 = -2 F_m C s_1'' = -A s_1''$$

die gesuchte Differentialgleichung des Torsionsproblems zu

$$\underline{T = 2 F_m s_1 - A s_1''}, \tag{6}$$

$$T = 640 s_1 - 113318 \frac{E}{G} s_1''$$

oder auch mit $m_d = -T'$

$$\underline{s_1''' - \frac{2 F_m}{A} s_1' = \frac{m_d}{A}}. \tag{6a}$$

Damit ist die Aufgabe grundsätzlich gelöst.

Auf zwei Punkte sei noch besonders hingewiesen: Daß der Schubmittelpunkt 0_2, um den sich der Stabquerschnitt unter dem zweiten Torsionsanteil dreht, nicht mit dem Schubmittelpunkt 0_1 des ersten Torsionsanteiles zusammenfällt, ist aus dem von R. Dziewolski untersuchten Beispiel recht deutlich ersichtlich: wegen der gleichen Scheibenquerkräfte und der gleichen Stärke der beiden Stege muß der Drehpunkt 0_2 in der Mitte zwischen den beiden Stegen liegen. Ferner ist die Elastizitätsbedingung für den zweiten Torsionsanteil, Gleichung (5), häufig nicht für alle Scheiben (z. B. bei freien Flanschen) genau erfüllt, sondern es sind dann kleine Widersprüche gegenüber dieser «Verträglichkeitsbedingung» vorhanden, die einer weiteren Berechnungsstufe mit weiteren zusätzlichen Normalspannungen rufen könnten. Da aber der zweite Torsionsanteil T_2 ausgesprochen örtliche Bedeutung besitzt, kann im Rahmen einer technischen Biegungslehre ohne ungebührliche Einbuße an Zuverlässigkeit auf eine solche weitere Verfeinerung verzichtet werden.

Die von R. Dziewolski verwendeten Begriffe f, ϕ, $S_{\phi c}$ erhalten durch die skizzierte baustatische Theorie des Torsionsproblems nun ihre baustatische Deutung; so stimmt die Verwölbungsfunktion f überein mit dem Verdrehungswinkel φ_1 des ersten Torsionsanteils und die Querschnittswerte ϕ und $S_{\phi c}$ stellen (abgesehen vom Maßstab) die Verteilung der Normalspannungen σ und der Schubspannungen $\tau_0 d$ über den Querschnitt dar.

Zusammenfassung

Der elastizitätstheoretisch orientierten Untersuchung der Torsion von Stäben mit Kastenquerschnitt wird eine baustatische Berechnungsmethode gegenübergestellt, die auf der Zerlegung des Torsionsmomentes in zwei um die beiden Grenzlagen 0_1 und 0_2 des Schubmittelpunktes drehende Torsionsanteile beruht.

Summary

In connection with the investigation of the problem of the torsion of bars with a box-shaped section, a comparison is made between the study orientated towards the theory of elasticity and a method of calculation, based on applied statics, in which the torsional moment is resolved into two parts relative to the two extreme positions 0_1 and 0_2 of the centre of shear.

Résumé

Pour le problème de la torsion des barres à profil fermé, on compare l'étude orientée sur la théorie de l'élasticité à une méthode de calcul fondée sur la statique appliquée; dans ce procédé, on décompose le moment de torsion en deux parts relatives aux deux positions extrêmes 0_1 et 0_2 du centre de cisaillement.

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Research on Shells by Means of Model Tests

Recherches sur les voiles au moyen d'essais sur modèles

Modellversuche an Schalen

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During the last two years, an extensive programme of research on shells by means of models has been carried out at the "Instituto de Mecánica Aplicada y Estructuras". The main purpose of this paper is to present a survey of the programme showing the different kinds of models employed, and the most important conclusions reached.

Five shells were tested; four of them were hyperbolic paraboloidal shells of various shapes, the last model was a prismatic shell.

The first model was a hypar shell of considerable curvature, rhomboidal in plan, supported on two opposite corners. The diagonals of the prototype were 27 m. and 19 m. in plan. The supports consisted of two hypar shells

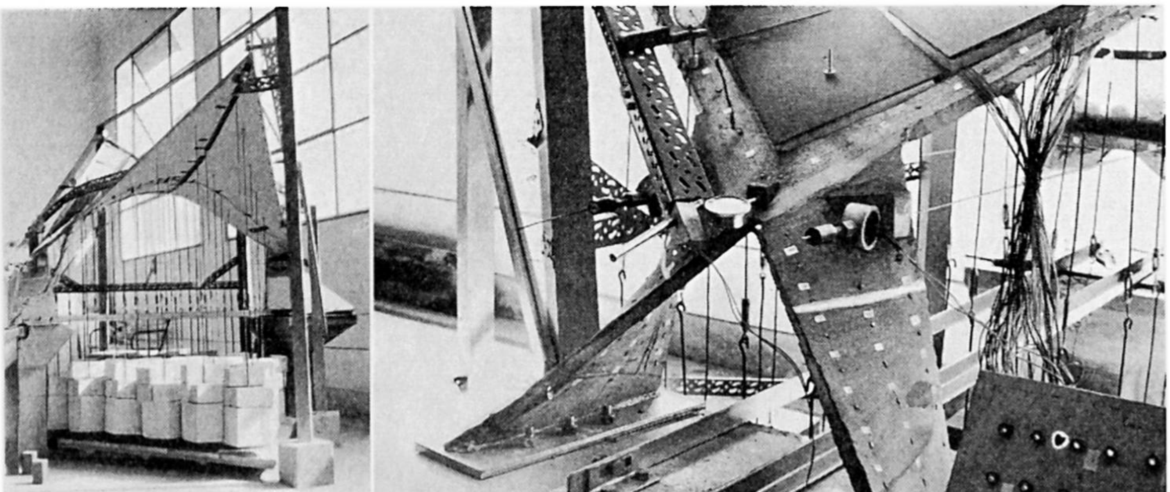

Fig. 1.

specially designed to absorb both the "arch" and the "rolling" effects. Tests were carried out on a $^1/_{10}$ model, made with reinforced mortar. In Fig. 1, a general view of the model prepared for testing is presented. The membrane stresses, bending moments and deflections under various load conditions were recorded. Typical bending moment curves along the diagonals of the free corners, due to uniform load, are presented in Fig. 2. Stresses due to bending moments in this structure did not exceed twice the stresses evaluated by means of the linear membrane theory. In Fig. 3, the distribution of edge forces under uniform load is presented. Actual edge forces are approximately one-half of those estimated by means of the simplified membrane theory.

The second shell tested was part of a basic research programme for the investigation of the stress distribution, edge perturbations and deflections of a hypar shallow shell. The model, as represented in Fig. 4 (left), was made

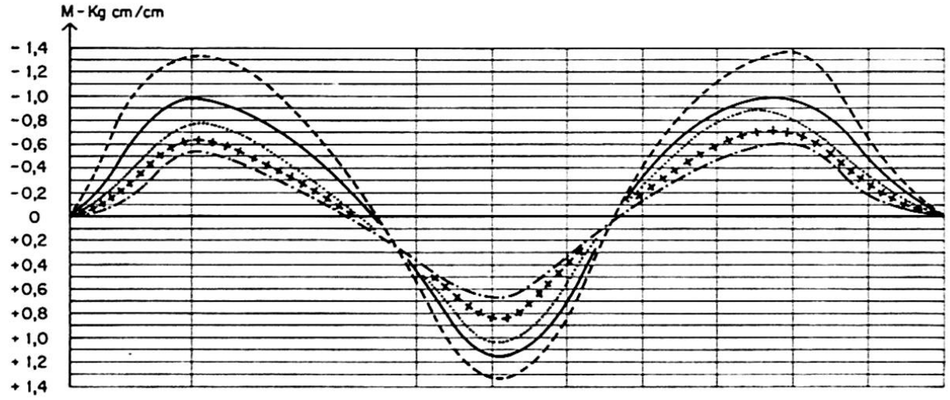

Fig. 2.

—·—·—·—· load $g = 320$ kg m^{-2} —————— load $g = 545$ kg m^{-2}
++++++++++ load $g = 395$ kg m^{-2} ---------- load $g = 620$ kg m^{-2}
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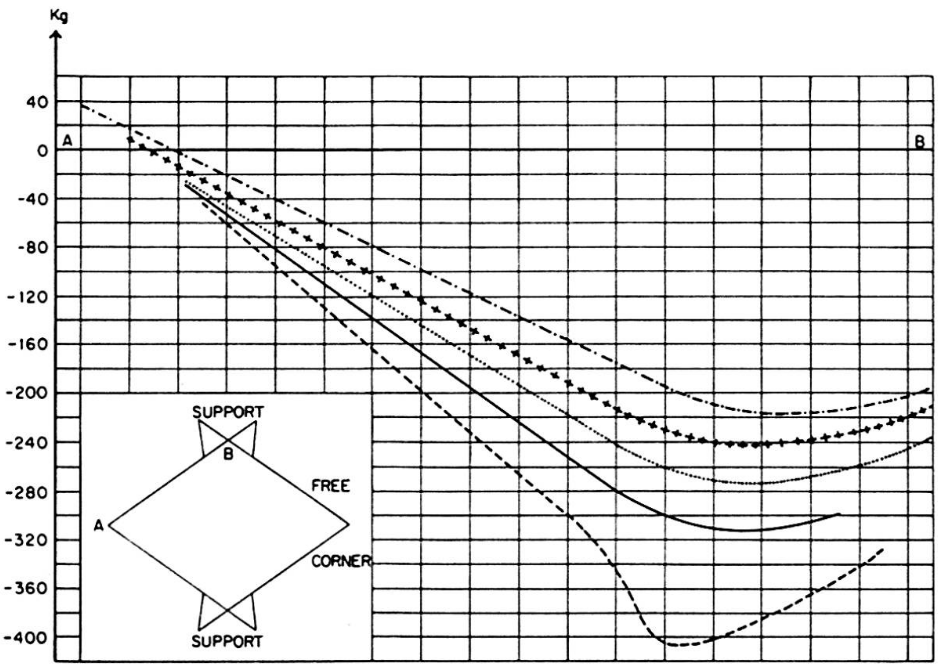

Fig. 3.

—·—·—·—· load $g = 320$ kg m^{-2} —————— load $g = 545$ kg m^{-2}
++++++++++ load $g = 395$ kg m^{-2} ---------- load $g = 620$ kg m^{-2}
·················· load $g = 470$ kg m^{-2}

of reinforced plastic material. Three different boundary conditions were considered in order to compare the influence of edge conditions on the stress distribution and moment perturbations. In Fig. 4 (right) the model arrangement is represented, when free edges were considered. A pneumatic device was used for loading and unloading.

Typical curves for membrane stresses, bending moments and shear forces

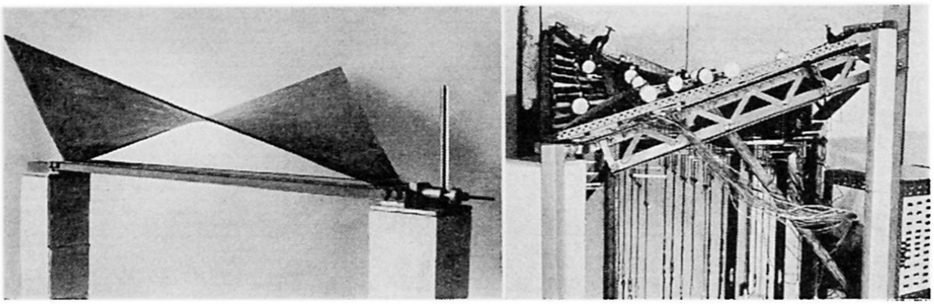

Fig. 4.

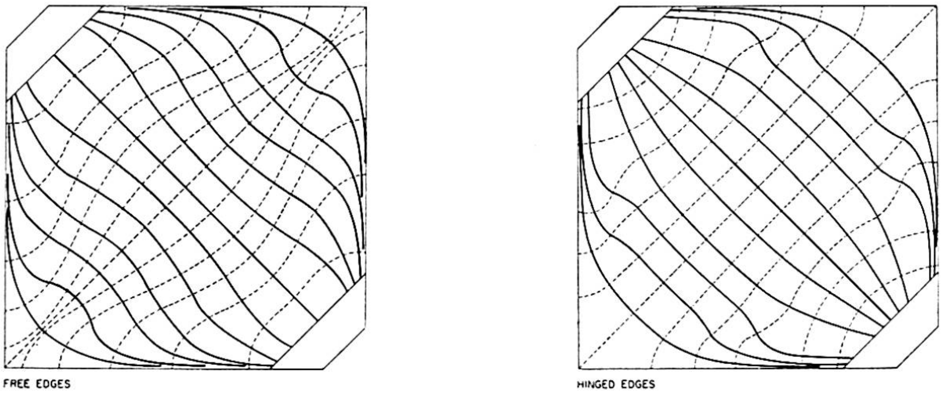

Fig. 5. Isostatics of Membrane Stresses.

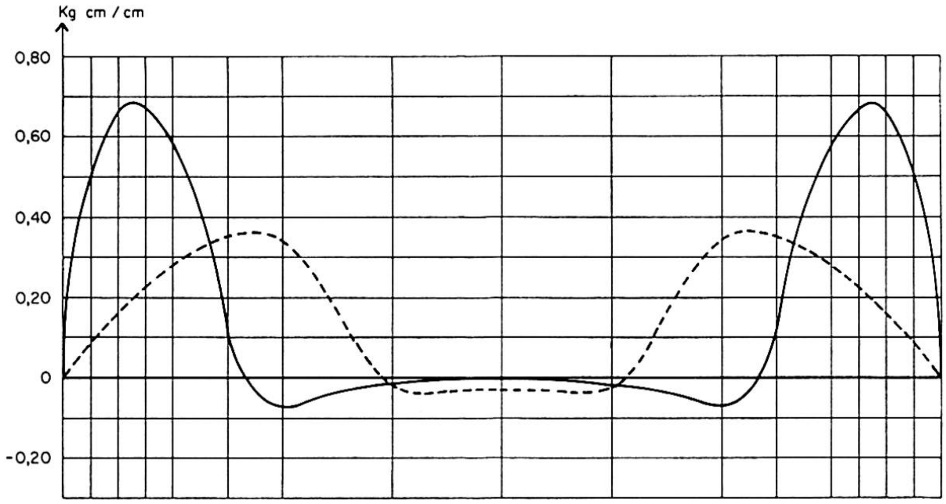

Fig. 6. Bending Moments along the Middle. Straight Generator.

----------- Experimental ——————— Theoretical were plotted along 5 straight generators for every edge condition. In Fig. 5, typical isostatic patterns are shown. Various perturbation theories were used in order to compare the experimental results. Fig. 6 shows a comparison between perturbation theorie, and the experimental results. A critical discussion of this research work has been published elsewhere [2]. This programme is now being completed with fixture designs which permit the application of edge forces and moments.

The third shell was a $^1/_{20}$ model of an umbrella shell. In Fig. 7 a view of the load model is shown. The height of the column was increased in order to make possible the inspection of the lower face of the shell. The main pur-

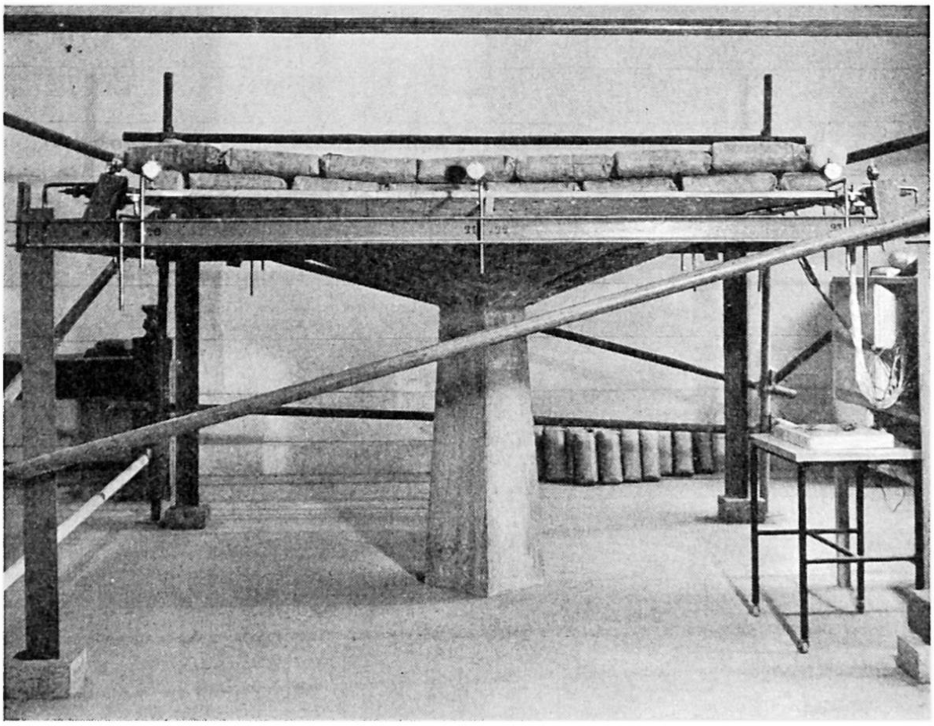

Fig. 7.

pose of this test was to check the general behaviour of the structural design of the umbrella in view of its rather large dimensions. In fact, the umbrella measured — in plan — 50 m. by 50 m. with a thickness of 10 cm. Membrane stresses, bending moments and deflections under design load, were recorded. A complete series of these typical stress distributions has been published elsewhere [3].

Although the stress distribution contributed to an understanding of the behaviour of the structure, a very interesting point was observed at failure. When the load was twice the design load, the umbrella collapsed abruptly, reversing its original shape. The collapse is clearly represented in Fig. 8. This break-through was unexpected, although the collapse occured 75 minutes after the load was applied. Practically no information on this problem was available to those responsible for this programme. Consequently a supplementary investigation was understaken in order to determine the causes of the failure. For this purpose, two models were tested. The first model was a single hypar shell which reproduced exactly one of the four elements constituting the umbrella. This model, made of cement mortar, was mounted on movable supports which were connected by an elastic tie. The purpose of this test was to examine in a single element, the buckling process. An asymmetrical buckling was obtained when the load reached the value at which the umbrella collapsed, Fig. 9 gives a general view of the shell after failure. The buckling process was succesfully interpreted theoretically, with the help of the experimental data obtained. Recently, the stability behaviour of a new umbrella model has been investigated. A full report on the subject will be published shortly.

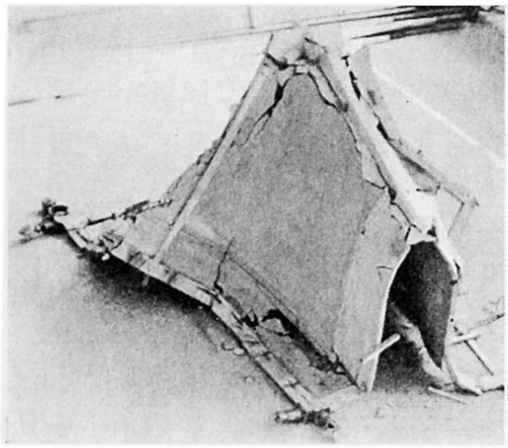

Fig. 8.

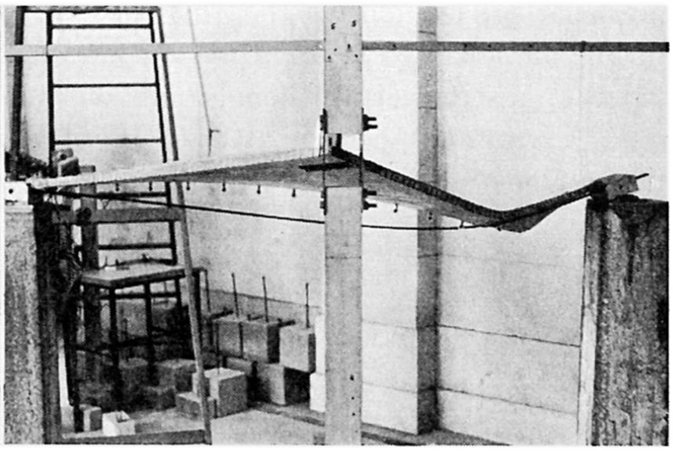

Fig. 9.

The last investigation dealt with an element of a prismatic shell structure. The model was made with Perpex and only information within the elastic range was required. Different loads were applied to the model in order to check experimentally the suitability of various technical theories. It was shown that great accuracy can be obtained by means of the theories for the evaluation of longitudinal and transverse stresses, except for the last plate for which the theories do not generally consider torsional effects.

Finally, some conclusions can be drawn from our experience in this field. It is shown that great advantages can be derived from model tests, not only for predicting the behaviour of actual structures, but also for making improvements on the original design and for obtaining experimental confirmation of the behaviour of structures whose theoretical analysis cannot be carried out rigorously.

References

1. E. Zamboni, J. N. Distéfano y C. Torregiani: «Diseño de apoyos laminares en cáscaras». IMAE, Unpublished Report, 1964.
2. J. N. Distéfano y C. Brebbia: «Influencia de las condiciones de borde en el comportamiento elástico de un paraboloide hiperbólico». IX Jornadas de Ingeniería Estructural, Montevideo, 1964. Publicación IMAE nº 10.
3. J. N. Distéfano y C. Torregiani: «Comportamiento elástoplastico de un modelo de cubierta tipo sombrilla». II Simposio Panamericano de Estructuras, Lima, 1964. Publicación IMAE nº 9.
4. M. H. Gradowczyck y M. Rubinstein: «Estructuras plegadas. Análisis teórico y experimental». IMAE, Unpublished Report, 1964.

Summary

The aim of this paper is to present a survey of the research programme on shells carried out at the Instituto de Mecánica Aplicada y Estructuras, during the last two years. Tests on models of five shells are briefly described and the most important conclusions obtained are presented. It is shown that great advantages can be derived from model test on shells which enable improvements to be made on the original design.

Résumé

Cette contribution a pour objet de présenter le programme de recherches que l'Instituto de Mecánica Aplicada y Estructuras a réalisé en matière de voiles au cours des deux dernières années. On décrit brièvement les essais qui ont été exécutés sur les modèles de cinq voiles et l'on en expose les principales conclusions. Il est montré que les essais sur modèles de voiles présentent un grand intérêt pour modifier avantageusement le projet initial.

Zusammenfassung

Es wird ein Überblick gegeben über die Modellversuche an Schalen, die in den letzten zwei Jahren am Instituto de Mecánica Aplicada y Estructuras zur Durchführung gelangten. Versuche an fünf Modellen von Schalen sind kurz beschrieben, die wichtigsten Ergebnisse werden angegeben. Es wird gezeigt, daß Modellversuche große Verbesserungen der Ausbildung und Bemessung von Schalen ermöglichen.

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Aerodynamische Modellversuche beim Donauturm Wien

Wind Tunnel Tests on Models of the "Danube Tower" Vienna

Essais en soufflerie sur maquettes de la «Tour du Danube»

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Der Wiener Donauturm wurde im April 1964 vollendet; die statische Bearbeitung erfolgte durch den Berichterstatter, wobei auch Professor LEONHARDT als Berater fungierte. Es waren bei der Erstellung dieser Konstruktion die aerodynamischen Probleme auf besonders windigem Standort zu lösen.

Die Höhe des Turmes beträgt 252,00 m; er besteht aus einem Rohrschaft aus Stahlbeton von 182 m Höhe über Erdniveau mit Aussichtsterrasse für 400 Personen, einer dreietageigen sogenannten Gondel, wovon 2 Etagen drehbar sind, und nicht zuletzt aus dem auf den Schaft aufgesetzten Stahlrohrmast von 71 m Höhe, von dem hier berichtet werden soll. Der Außendurchmesser des Stahlbetonschaftes, der auf einem kegelstumpfförmigen Schalenkörper aufgesetzt ist, verjüngt sich von 12 m auf Kote 0 bis auf 6,2 m auf Kote 150 m, mit Entasis.

Für weitere bautechnische und statische Detailbeschreibungen darf ich auf meine Publikation in der Zeitschrift des österreichischen Ingenieur- und Architektenvereines, Heft 5/64, hinweisen.

Statisch gesehen handelt es sich bei jedem Turmbauwerk um einen im Baugrund nachgiebig eingespannten Kragträger; die hauptsächlichen Belastungen stellten das Eigengewicht in Achsrichtung und der Winddruck senkrecht zur Bauwerksachse, dessen Verhalten viel zu wenig bekannt ist, dar. Die Verkehrslasten spielen verständlicherweise keine sehr große Rolle, wohl noch die Erdbebenkräfte.

Die Grundlage der Winddruckannahmen bildete der Normenwert des Staudruckes von $q = 130\,\text{kg/m}^2$; bei Spitzenböen muß im Aufstellungsgebiet des Turmes jedoch nach Messungen der Zentralanstalt für Meteorologie und Geodynamik Wien mit Geschwindigkeiten von 180 km/h gerechnet werden. Dieser Geschwindigkeit entspricht ein Staudruck von $q = 156\,\text{kg/m}^2$.

Die Kenntnis der Geschwindigkeit einer größtmöglichen Böe ist zu wenig; man muß auch auf die Windstruktur eingehen. Die dynamische Wirkung der zeitlichen Veränderlichkeit des Winddruckes war ebenfalls zu erfassen. Man erhielt die Ersatzlast $\max q = q + \beta \Delta q$, dem vergleichbar der Staudruck $q' = 195\,\text{kg/m}^2$ entsprach.

Zu den erforderlichen Berechnungszahlen gelangte man durch das Quellenstudium und die Heranziehung vergleichbarer Hochbauwerke [1]).

Andererseits wurde zwecks Feststellung des Formbeiwertes c das Gesamtmodell des Turmes einem Windkanalversuch unterzogen, durchgeführt im Institut für Strömungslehre der Technischen Hochschule Wien. Es wurde hiezu ein Turmmodell im Verhältnis 1 : 20 hergestellt, wobei der Beton-Teil des Turmes in Holz, der Stahlmast in Draht rekonstruiert waren. Die geforderte Aufgabe war die Ermittlung von c_w, um die Gleichung für den Windwiderstand

$$W = c_w q F$$

erfüllen zu können. Der Staudruck q war bekannt, ebenso F, die Projektionsfläche.

Um das Verhalten des c-Wertes bei geänderter Reynoldscher Zahl verfolgen zu können, wurden zusätzlich Teile des Turmes im vergrößerten Maßstab angefertigt: der drehbare Turmkopf im Verhältnis 1 : 50, ein Teil des Schaftes im Verhältnis 1 : 30 und ein Teil des Mastes (der ursprünglich als Gittermast geplant war) im Verhältnis 1 : 20.

Die Aufhängung des Modelles im Windkanal geschah derart, daß der ganze Turm zunächst einseitig umströmt wurde; durch eine besondere Aufhängevorrichtung war aber auch eine Drehung des Turmmodelles und Anströmung von den verschiedenen Seiten möglich. Dadurch konnten Größe und Richtung der Seitenkraft bestimmt werden.

Die erhaltenen Meßergebnisse umfassen die Widerstandsbeiwerte für den Turm, für den drehbaren Teil, für den Schaft-Teil und für den Mast.

Es wurden für die Windgeschwindigkeiten von 10 m/s und von 50 m/s Reynoldsche Zahlen ermittelt. Analog zu dem Verhalten des c_n beim Zylinder wurde nun ein ähnliches Verhalten des c_w bei den Teilen des Turmes angenommen. Demgemäß ändert sich das Wirbelgebiet nach der Ablösung der Grenzschicht. Daher nimmt man im Bereich von $Re = 1 \cdot 10^7 - 6 \cdot 10^7$ noch ein Ansteigen des c_w-Wertes an. Der endgültige c_w-Wert wurde mit $c = 0{,}55$ ermittelt und mit 0,6 berücksichtigt.

Es stand zunächst die Version des bekannten Stahlfachwerksmastes zur Diskussion, die auch bei den Windkanalversuchen im Modell vorgebildet war. Man kam aber von dieser schon mehrmals dagewesenen Ausbildung ab zugunsten einer Rohrkonstruktion, die hinsichtlich der günstigen Dimensionen sowie des Allgemeindruckes nun als eine architektonisch und technisch durchaus gelungene Lösung bezeichnet werden kann. Bemerkt muß hiezu noch

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[1]) A. Föppel: «Vorlesungen über techn. Mechanik». Bd. IV, Dynamik, 7. Aufl., Leipzig 1923. E. Rausch: «Maschinenfundamente und andere dynamisch beanspruchte Bauwerke». VDI-Verlag, 3. 4. 1959, Düsseldorf. F. Leonhardt: «Der Stuttgarter Fernsehturm». Beton- und Stahlbetonbau, 51/1956, H. 4.

werden, daß diese von VOEST-Linz gelieferte Rohrkonstruktion noch zwei Reklamezeichen mit je 9 m Durchmesser zu tragen hatte.

Es war bei der meines Wissens erstmaligen derartigen nicht abgespannten und auf einen elastischen Stab in 182 m Höhe aufgesetzten Rohrkonstruktion wohl ein gutes Stück Sonderforschung hinsichtlich ihres aerodynamischen Verhaltens nötig.

Es wurde getrachtet, dem Windangriff möglichst durch konstruktive Mittel zu begegnen.

Durch Fühlungnahme mit dem für aerodynamische Probleme bei zylindrischen Hochbauten spezialisierten Physiker C. SCRUTON im National Physical Laboratory in Teddington, Middlesex, England, wurden wichtige Fragen der Winderregung bei derartigen Türmen aufgeworfen. Es ist jedem Fachmann bekannt, daß eine windausgesetzte, rohrförmige Konstruktion Kármansche Wirbelstraßen erregt. Diese lösen sich mit einer durch die Umstände bestimmten Geschwindigkeit vom Rohr ab; mit der gleichen Frequenz pulsiert aber die quer zur Windrichtung wirkende aerodynamische Erregerkraft. Auf Grund dieser Vorgänge kommt es zu selbsterregten Schwingungen, deren Ausdehnung (Schwingungsweite) nur von der vorhandenen Gebäudedämpfung begrenzt wird. Es müssen daher diese Schwingungen unschädlich gemacht werden. Dies konnte geschehen durch die Anbringung von Störern, die die Grenzschicht turbulent machen, oder durch Verstärkung der Gebäudedämpfung. Die letztgenannte Möglichkeit wäre nur mit beträchtlichem konstruktiven Aufwand durchführbar gewesen.

So gab Mr. SCRUTON gute Ratschläge. Versuche hätten jedoch 1 Jahr gedauert; eine Anfrage beim Mechanischen Institut der Technischen Hochschule Wien wegen exakt wissenschaftlicher Erforschung des Dämpfungsdekrementes ergab, daß im besonderen Falle dies $^1/_2$ Jahr benötigt hätte.

Es wurde somit nach bekannten Autoren vom Berichter gefolgert, daß für Konstruktionen von kreisförmigem Querschnitt ausreichende aerodynamische Stabilität erreicht werden kann, indem man Gänge in Schraubenform, die äußere Oberfläche umlaufend, anbringt. Es ist eine entsprechende Anzahl von Gängen von bestimmter Höhe erforderlich.

Im Verlaufe verschiedener Versuchsmessungen im Windkanal durch SCRUTON, WOODGATE u. a. war gefunden worden, daß die erforderliche Höhe der Gänge von dem Umfang der vorhandenen Gebäudedämpfung abhinge, daß aber ca. $^1/_8$ des Durchmessers eines zylindrischen Mastes als Ganghöhe selbst für die leichtest gedämpften Konstruktionen genüge.

Die Stahlrohrspitze des Donauturmes Wien wurde somit endgültig festgelegt mit Durchmesser 2,50 m, Wandstärke ab 18 mm, 50 m hoch, glatte Oberfläche, darauf abgesetzter Teil mit Durchmesser 1,0 m, Wandstärke 8 mm, 20 m hoch mit angeschraubten verzinkten Blechstreifen.

Zur Materialbeanspruchung am Rohrfuß war auch die Dauerfestigkeitsbeanspruchung zu beachten. Die Größtspannung im Einspannquerschnitt beträgt 1600 kg/cm² mit Spiralen. Der Verfasser weist auch darauf hin, daß das gesamte Rohr geschweißt und im unteren Bereich aus St 52 T besteht. Das Gewicht beträgt 58 t. Vorteilhaft war für die Beurteilung auch die große Differenz der Eigenschwingungszahlen zwischen oberem und unterem Bereich. Mit Überschlagsformeln wurde die Eigenfrequenz wie folgt beurteilt:

im oberen Stahlrohrstummel (ca. 1,0 m ∅)	4,9 Hz
im unteren Stahlrohrmast (ca. 2,50 m ∅)	1,5 Hz

Für den abgesetzten Stahlrohrmast mit aufgesetzten Spiralengängen ist also die Resonanz sehr erschwert.

Die gewählte Form, Konstruktion und aerodynamische Beurteilung haben sich bisher bewährt und keinen Anlaß zur Klage gegeben, so daß auf die gute Wirkung von derartigen Spiralen geschlossen werden kann.

Zusammenfassung

Die Vornahme von Modellversuchen für den «Donauturm Wien» war angesichts der Turmhöhe (252,00 m), der 71,0 m hohen oberen nicht abgespannten Stahlspitze und der auftretenden Spitzenböen (180 km/h) erforderlich. Es wurde sowohl das Gesamtmodell des Turmes wie auch einzelne Turmteile im Institut für Strömungslehre der Technischen Hochschule Wien getestet. Als Formbeiwert wurde $c = 0{,}55$ ermittelt und danach die Baugestalt festgelegt.

Summary

Wind tunnel tests were necessary for the "Donauturm Wien" on account of its height (252,00 m), the steel spire 71,00 m high, and the occurence of gusts of up to 180 km/h. A complete model of the tower as well as models of separate parts, were tested at the Institut für Strömungslehre of the Technical University of Vienna. A shape coefficient $c = 0{,}55$ was determined, and the design of the building was based accordingly.

Résumé

La hauteur de la «Tour du Danube» (252,00 m), la présence d'une flèche métallique haute de 71 m et les bourrasques à considérer (180 km/h) ont nécessité des essais en soufflerie. Réalisés à l'Institut d'aérodynamique de l'Ecole Polytechnique de Vienne, les essais ont porté tant sur une maquette d'ensemble que sur divers éléments. On a mesuré un coefficient de forme $c = 0{,}55$ et la conception de l'ouvrage a été fixée en conséquence.