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RAPPORT INTRODUCTIF / EINFÜHRUNGSBERICHT / INTRODUCTORY REPORT

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Mr Chairman, ladies and gentlemen, when I was asked to prepare my paper for this symposium I was somewhat hesitant about accepting the invitation because I had never designed a concrete structure nor had I been directly concerned with the measurement of movements and strains in bridges or buildings. My only qualification is my experience of observing the behaviour of many types of concrete. Now that I have seen other papers printed in the Preliminary Publication and have read summaries of the reports submitted for the free discussion on this Theme, I am even more conscious that my contribution is quite different from what most members would seem to expect, and am therefore the more appreciative of the privilege of being invited to give this address. I can only trust that my simple, philosophical and perhaps unconventional remarks will help to improve the art of designing concrete structures to take account of the effects of creep, shrinkage and temperature changes.

You may have also noticed that my paper includes references drawn only from British sources. I hope this will not be regarded as any discourtesy to those in other countries who have made notable contributions to the subject of this symposium - it is only a measure of my lack of familiarity with work in this field, other than recent developments which have been taking place in my own country.

I have suggested in the conclusion to my paper that I may have a morbid view of the present ability of engineers to design concrete structures to accommodate the effects of creep and of changes in temperature and humidity. I feel that I owe you an explanation of this attitude.

My early experience of concrete was derived from a study of its properties under laboratory conditions. This meant that we attempted to keep the ambient temperature and humidity constant throughout the life history of the material under examination because variations in either were known to affect our results measurably. Any loading tests were short-term, and creep was ignored. After some years, the equip-

ment used for controlling the laboratory atmosphere gradually became less reliable, and experimentation of the conventional type therefore became more and more difficult. About the same time, we began to receive more requests for advice about unsatisfactory concrete in practice; the causes could only be discovered by visiting the sites to observe the circumstances under which faults had occurred. So the pattern of the work changed gradually from the experimental techniques of the laboratory to the experiential research of observation.

It soon became apparent that many of these unsatisfactory features - and especially the various types of cracking - were the result of the engineers' lack of appreciation of the importance of the changes in temperature and humidity which occur in practice. We then realised that research conducted in a laboratory, to study the relationship between the properties of concrete and its constituent materials and composition, was only of limited value in practice unless the results were interpreted in the light of experience of full-scale behaviour in normal environments. Hence, what appeared to be the misfortune of having to work in an unserviceable laboratory turned out to be a blessing because it forced us to a more realistic approach to research on the difficulties that occur in practice.

Although my own laboratory experience lies in the study of unreinforced concrete, my colleagues working on structural design problems have also found that it is convenient to work in laboratories where the variations in temperature and humidity are minimised. In recent years, too, the introduction of data loggers has enabled tests to be conducted so quickly that the time-dependent factors have been eliminated. These techniques have enabled the effects of particular man-made changes imposed on the test specimen to be determined more precisely than would have been possible if they had been obscured by the scatter in results arising from the effects of changes of temperature and humidity and from the effects of the time taken to complete the tests.

It is largely on the results of such laboratory investigations that criteria and recommendations for the design of structural concrete have been based. Doubtless, if all structures were built inside laboratories having controlled atmospheres and were used for only a short time, they would be perfectly satisfactory. But they are not; and it is quite unrealistic that the designer should depend so heavily on these results when he is going to build concrete structures under the varying conditions which exist in the natural world around us. It may be convenient for the research worker to do his tests under conditions controlled by man, because otherwise God would spoil his experiments and he would not have any well-ordered results on which to base specific recommendations. Designers should remember that the Scriptures say: "Vengeance is mine; I will repay saith the Lord": in the end, nature will have its own way and produce undesirable features in our structures if they ignore the predictable consequences of their actions.

I believe that research engineers have paid too much attention

to the study of structural design problems by theoretical analysis and experimental techniques in laboratories at the expense of the more practical observation of structures in real life. We delight in the pursuit of more and more sophisticated theories to explain complicated phenomena, rather than in examining whether these phenomena are of any significance in relation to the simple practical factors what actually affect the behaviour of structures. Unfortunately, this attitude is also found so often in designers who are perplexed by faults that have occurred in their structures because of their lack of appreciation of the simple nature of concrete and its response to changes in its environment.

Prof. Wästlund has already referred to three RILEM symposia which have a bearing on this symposium; in each case, attempts were made to bridge the gap between laboratory and site experience. Two of these were arranged by Prof. Rüschi and his staff in Munich. The 1958 symposium dealt with "The influence of time upon strength and deformation of concrete" and that held in 1968 with "The physical and chemical causes of creep and shrinkage". Both were primarily for research workers, about 35 being invited on each occasion, and much of the information had still to be digested into a form suitable for designers. However, even at the earlier symposium, there was little doubt about the complexity of the problem of predicting the creep of concrete in a given situation. It was known to be affected by the type of cement, the degree of hydration of the cement, the amount of carbonation and the intensity of loading or unloading as well as characteristics of the concrete; and it was on this occasion that I first realised the significant effect that fluctuations in load, temperature or humidity had on the creep compared with the values to be expected under the average conditions if held constant. Although more data had been collected by the time of the later symposium, especially in the field of design of pressure vessels for nuclear reactors, there still seemed little prospect of providing a simple basis for reasonably accurate prediction of creep.

Another symposium in 1968 was organised jointly by RILEM and Cembureau in Madrid just before the second Munich symposium; it was attended by 125 people having various interests. The subject was limited to "The shrinkage of hydraulic concretes", but even without having the complication of applied loading to contend with, it seemed that there were so many factors to allow for that the prediction of shrinkage was far from precise. In any case, shrinkage itself was of little direct interest: engineers were only concerned with its effects on deflection or cracking of members or on loss of prestress. It was generally agreed that one could not devise a realistic test for cement which could be specified by engineers to relieve them of the responsibility of worrying about the occurrence of cracking or excessive deflection of their structures. It was suggested that designers must still consider the proper questions relating to their designs - and to do this they needed to observe the structures they had built.

In my paper I have referred to "failure" rather than "faults" implying any aspect of design which has not been adequately ful-

filled; and I have tried to show how the limit state concept can be used to encourage more logical thinking about design to take account of the effects of creep and of changes in temperature and humidity so that the risk of failure is acceptable.

I would now like to take these thoughts a little further and suggest that failure do not happen by accident - as though they were "an act of God" to use the language of insurance companies. I believe that all failures are designed to occur just like any other attributes of the structure, and that the designer is the only person who can really be held responsible for them. Quite simply, if the building had not been designed by someone it would not have been built and there would have been no failure. Most failures occur because the designer has misjudged either the causes of stress or the partial factors of safety required to provide a reasonable probability against failure.

The failures to which I am referring are generally of limited significance in terms of social or economic values; structural engineers have studied how to design so that there will be little risk of any calamitous failure by collapse of the structure. But they have not paid enough attention to the sort of failures which involve maintenance problems - because of deterioration of the materials - or which cause concern and annoyance to the client - because of inconvenience and anxiety over the quality of the work. Admittedly, designers have concentrated on the most serious problems; but they have rather ignored the causes of minor failures and have hoped that they could depend on the experience and expertise of material producers and contractors alike to compensate for their lack of interest in these details. However, this is no safeguard if they blatantly violate the principles of good design and detailing.

It is my experience that designers find it extremely hard to come to terms with the ideas that changes in the temperature and humidity of the environment in which concrete is placed can have a significant effect on the whole of their design concept. They are well aware of the effects of the force of gravity, both in terms of the dead load of the structural members and the live loads that are placed upon them, but they find it hard to conceive that the effect of the sun shining on a part of the structure, or of the drying of the surface of some concrete compared with the middle of that same concrete, can cause greater stresses within the material than those which they associate with conventional loadings.

I have gained the impression that most, although not all, of the reported measurement of existing structures has been undertaken with a view to determining only the deflexions that have occurred. It seems to me that there is still considerable scope for the investigation of the effects of environmental changes and of creep on the stresses developed within structural members. It may well be that experience has shown that, with our traditional factors of safety and methods of design, creep and temperature and moisture changes do not have a very significant effect on ultimate failure of the structure. Whether this will always be so as further refine-

ments are introduced may well be a matter for investigation. But there is ample evidence from the observation of the structures that have been built in recent years that these factors play a very important part in design for the limit state of local damage as well as for the limit state of deflexion.

Local damage usually implies loss of integrity of the concrete by some form of cracking, while concrete, still in a high state of stress without cracking visibly, is regarded as being satisfactory. The distinction between the two states is not clearly marked as there is a gradual transition between them; a point of unacceptable behaviour has to be defined in some way as, for example, by specifying the maximum permissible width of crack at the surface of the concrete. Until such time as we have undertaken enough research to bridge the gap between the findings of the research worker in the laboratory and the results of observation of structures in practice, so that our design procedures for the limitation of local damage can be more formally codified, we must design intuitively.

It seems, therefore, that there is a considerable need for a reappraisal of the major influences affecting the design of structures. Fortunately this is being done gradually and our Codes of Practice are steadily being improved to give a better overall approach to the total design of a structure for all its functions whereas, hitherto, they tended to be limited to a rather specialised approach to one or two aspects which were of particular interest to structural engineers trained in the traditional way.

Our terminology may also have to be improved: for example, British codes have recommended minimum amounts of what is called "distribution" reinforcement for different types of structure. This word "distribution" gives no indication whatever that this reinforcement is required to control the tensile stresses which develop near the surface of a concrete member so that cracking will not occur there. The amount of this reinforcement is usually required to be a stated percentage of the cross-sectional area of the member, to satisfy typical situations. If it had to be calculated according to some formula which involved the drying shrinkage of concrete and the thermal contraction associated with the removal of formwork I am sure that engineers would have had a better appreciation of the effects of environmental changes even though the design may still have been somewhat arbitrary. As it is, designers tend to follow the recommendations blindly without appreciating the significance of the statements and they do not seem to realise that the minimum amount recommended for general purposes may be quite inadequate in some particular circumstances when the thermal and moisture gradients are excessive.

While the amount of reinforcement may be governed by the amount of tensile stress that is likely to be developed, the distribution of the reinforcement is governed by the need to control the width of cracks at the surface of the concrete. Cracking cannot be eliminated entirely. The designer's objective must be to distribute the tensile stresses arising within the concrete from all causes that at no point is there greater disruption of the concrete than that described

as microcracking, which is neither serious in its consequences nor objectionable visually. The designer must therefore try to induce as many cracks as possible in the concrete so that they will all be little ones - and not, as many believe, try to reduce the number of cracks! Thus, the detailing of the design, which is often left to the draughtsman, can also be a vital factor in the occurrence of such failures as I have in mind. Again more attention is being paid to improving this stage of design and to disseminating the information by the publication of relevant advisory literature and by training.

In my paper, I have referred in some detail to the visual observation of plain concrete: I make no apology for doing so even though the symposium is concerned with structural concrete. Structural concrete is surrounded by plain concrete; and plain concrete is the flesh which protects the skeleton of reinforcement or pre-stressing tendons. It is the plain concrete at the surface that suffers the first and most violent consequences of all variations in temperature and relative humidity. It is affected by carbonation and by attack from any adverse environment, whether it be wetting or drying, frost, chemical action, abrasion, impact or fire. And it is the first part of the concrete to suffer cracking. As I have explained in my paper, I believe that the flesh may even have a skin on it and that this too plays a vital role in the proper behaviour of the structural concrete. And whether or not it is true to say that "beauty is only skin deep", it is certainly true that the appearance of structural concrete is very much affected by the initial and weathered qualities of the concrete skin.

I would like to show two slides to illustrate my remarks about the skin on the surface of some types of concrete; these slides also illustrate the value of observing the behaviour of concrete in practice as a technique for understanding its nature.

The first may be aptly entitled "footprints on the sands of time" (to quote from a poem by Longfellow). It shows, I believe, how the same concrete can have a skin which is either weak and ineffectual in preventing attack on the underlying concrete, or one which is strong enough to protect the concrete against external hazards. I imagine that a man walked across the concrete within an hour or two after it has been laid, thus recompacting the surface concrete sufficiently to give the affected areas increased resistance to temperature and moisture changes.

The second slide shows an area of an experimental concrete road slab which was used for trials of concrete joint sawing equipment. At the time of casting, during one summer, little care was taken to protect the surface of the concrete and, as a result, so-called "plastic cracking" occurred within a few hours of laying the slab. The sawing trials had been completed some time before the following winter. The photograph, which was taken after frost damage had occurred, shows cracking parallel to the saw-cuts but none parallel to the plastic crack. It would appear, therefore, that in sawing the hardened concrete the skin had been broken and the underlying concrete was

damaged by the expansion of water as the temperature fell. On the other hand, as the plastic crack had formed in fresh concrete, the surface of the sides of the crack had become durable probably as a result of compaction under the influence of the surface tension of the moisture on the surface of the concrete.

This brings me conveniently to my analogy between concrete and human beings. Concrete is a creature whose behaviour is governed partly by heredity and partly by environment. Its hereditary characteristics are determined by the nature of the constituent materials of which it is composed. As these can be many and varied, concrete is a genus which includes very many species, races and nationalities, all with their distinctive individual characteristics. Conception of the new child takes place when the ingredients are brought together as a result of a marriage - by arrangement, in love or of convenience. After a short gestation period in the mixer concrete is born, as soft and pliable as a baby. The shape of its future can be easily moulded at a very early age; but nevertheless it rapidly gains strength and independence provided it is kept in a proper environment. In its youth it can accept a certain amount of ill-treatment without undue suffering but it is more likely to live a useful and active life if it is reared with care and sympathy. As it gets older it becomes harder and more brittle and less able to accept shocks without cracking under the strain and tension. Not only does it have a skin to protect it from disease, but it breathes or transpires, and as it does so it varies in weight and size. It has a wealth of experience behind it and its greatest attribute is that it will talk to you if you will only listen. I believe that concrete can grow old gracefully: but it will only do so if designers study its personality by observing its behaviour as it grows up in the world it seeks to serve.

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RAPPORT GÉNÉRAL / ARBEITSBERICHT / GENERAL REPORT

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INTRODUCTION

Measurements and observations of effect due to shrinkage, creep and temperature changes on concrete structures are carried out for various purpose. The 14 contributions from 11 countries published prior the symposium are a proof of this. They can be divided into three main groups.

The aim of the first group is a comparison of creep and shrinkage effects assumed in the design of a structure with actual values of those phenomena in the existing structure. The second group appears to include the investigations the purpose of which is a comparison of the experimental results obtained in controlled conditions of laboratories with the results of the measurements and observations obtained on the actual structures. The third group includes the contributions reporting on measurements and observations provoked by some defect or damage on the finished structure which can be imputed to unfavourable effects of creep, shrinkage, or temperature changes, respectively a suspicion has arisen that such a defect or damage could be done owing to those phenomena. In any case, the main aim of measurements and observations is to verify whether an actual structure can serve the purpose which it is designated to. Naturally, there are many other questions which are to be clarified by measurements and observations on actual structures and in laboratories.

A comparatively great number of measurements and observations to verify the values considered in the design of a structure have been made on a comparatively large scale. It is caused by the fact that it is difficult to transfer the results of measurements from one part of the world to another. The main reason for it lies in the different climatic conditions. Kokubu, Goto, Ozaka, Okamura, and Momoshima report in their contribution results of measurements on 7 highway and 14 railway bridges the main purpose of which was the evaluation of creep coefficients and shrinkage strains of bridge girders with a box cross section and an I-shaped one. The numerous measurements during a period of 2 - 5 years make possible a comparison of time-dependent behaviour of several bridges of different types, cross sections, spans, age of prestressing, climatic conditions, concrete compositions, construction methods, etc. The authors investigated also the influence of creep and shrinkage separately. They give the results of such measurements on three bridges. Generally, much attention is devoted to the long term behaviour of bridges. It is quite comprehensible. Bridges are mostly built as structures of prestressed concrete and the prestressing force is considerably influenced by creep and shrinkage effects. By new construction methods, especially cantilever method, the concrete elements have different ages and get their final loads in many steps. It may considerably influence the development of creep and shrinkage strains. The situation of a bridge with regard to the cardinal points and local climatic conditions may have a great influence on those magnitudes. Use of prestressed concrete makes possible bridges with large spans where the sustained load forms a great part of the total one. Keijer presents the results of measurements on four bridges in Sweden. Great attention is paid to the vertical deflections of bridges and to the problem of their stabilization during their lifetime. Some measurements lasted more than 12 years. Oberti presents in this contribution the results of measurements carried out during a period of 4 years. Aichhorn continued the measurements of three bridges in Austria and informs of measurements which lasted altogether 14 years. The aim of two reinforced concrete structure measurements the results of which are given in the contribution

of Ohno and Obata was the time-dependent change of expansion joint width.

The research works in which observations in laboratory conditions are combined with the measurements on actual structures are of great importance. We can find such a comparison in the contribution of Jungwirth where the results of laboratory research as well as results of measurements on bridges and other structures are presented. Swamy and Bennet report in their paper on a research programme the purpose of which is to predict the possible movements of actual structural members on the base of laboratory model tests. They compare the results of measurements on columns of a skeleton structure with the results obtained on laboratory specimens. Dilger, Ghali, and Kountouris observed creep and creep recovery on cylinder specimens to modify the principle of superposition and changes of the reactive forces on continuous prestressed concrete beams. Okada, Koyanagi, Yoshida carried out the measurements to find the effects of differential shrinkage on statical magnitudes of composite reinforced concrete beams. The object of the study presented by Gamble is the modification of the principle of superposition in order to clarify the non-linear creep and the verification of theoretical analysis on 22 specimens. Balázs and Horváth carried out the measurements on two bridges in order to compare the behaviour of structures with a normally treated and steam-cured concrete. Terteş, Onet, and Beuran investigated the long term behaviour of lightweight reinforced concrete beams. Results of an extensive experimental research of structures exposed to high temperatures are presented by Rickenstorff. The contribution of Cigánek and Meloun can serve an example when measurements and observations were provoked by the defects which appeared as a consequence of unfavourable effect of shrinkage on an existing structure. Similar reasons led to the measurements on a bridge and precast prestressed girders presented by Oberti.

The review of contributions to the present symposium shows that there is a variety of reasons which lead to organizing measurements and observations on actual structures or in

laboratories. Though the main aim of the measurements on actual structures is to make sure of the correct function and to compare the assumptions of design, they often require supplementary experimental research in controlled conditions of laboratories. On the other hand, an experimental laboratory research may be carried out to clarify some other problems. The detailed investigation of the influence of concrete composition, material properties, mode of reinforcing, mode of loading and its intensity, different temperature and humidity conditions, size and shape of specimens, etc., is possible only in a well equipped research institute. There are some more problems the solution of which can be expected only from a well prepared and programmed laboratory research. One of these problems is, for example, the determination of the limit of stable increase of displacements. It may be supposed that there is a certain ratio of the sustained load to the ultimate load at which the increment of displacement, e.g. of deflections, begin to be constant or increasing for the constant time interval. Another question connected with the safety of structures against undesirable displacements is the time-dependent change of statistical parameters. From the point of view of design philosophy it would be desirable to know the change of the distribution curve of strains or displacements at certain load and constant climatic conditions. A similar question is the investigation of creep and shrinkage effects on ultimate load where the change of statistical parameters would influence the safety of a structure against collapse. An experimental investigation of time-dependent change of standard deviation for deflection or cracking limit, respectively crack width, could answer the question of influence of creep and shrinkage on increasing or decreasing of dissipation of those values. Prestressed concrete structures are often reinforced with a supplementary reinforcement of mild steel. A redistribution of stresses between concrete and reinforcement due to creep may reduce the cracking limit of those structures. This is also a problem which can be solved only by a well prepared experimental research. As we can see from the presented papers, most of the research work concern with structural members subjected to flexural and compressive stresses.

An attention should be devoted to the other kinds of stresses, e.g. to the torsional one. Certainly, the problem of the influence of creep and shrinkage on structures stressed with repeated loading is of a great importance. In some cases it would be useful to examine by an experimental research the difference between the long term behaviour of structures under uniform and concentrated loads and investigate how the total deflections are influenced by a local concentrated stress.

TECHNIQUE OF MEASUREMENT AND INTERPRETATION OF RESULTS

An experimental program includes time-dependent measurement of several physical magnitudes. The measurement of displacements and strains should be considered as the most important together with the measurement of temperature and humidity of environment. As it is indicated by several authors /Oberti, Jungwirth, Rickenstorff/ the following quantities should be measured: vertical and horizontal displacements as a rule in the spots where these quantities are expected to obtain the greatest values, rotations of cross sections, local strains of concrete and those of steel, crack spacing and crack width, the value and situation of loads, temperature and humidity of environment and of the structure itself. The measurement of those quantities on the actual structure or in laboratory does not represent any substantial difficulties. The geodetical methods may be used for measurement of vertical and horizontal deflections on large constructions. Otherwise the simple dial gauges are used for the measurement of deflections /Terteau, Onet, Beuran/. As far as the measurement of strains is concerned, Jungwirth emphasizes that the greatest measuring length is necessary to use. A long steel bar is advisable for this purpose /Kokubu, Goto, Ozaka, Okamura, Momoshima/. Very often, however, contact gauges /Kokubu, Goto, Ozaka, Okamura, Momoshima; Oberti/, mechanical strain gauges /Swamy, Bennet; Cigánek, Meloun; Terteau, Onet, Beuran; Balázs, Horváth/, acoustic strain gauges /Swamy, Bennet/, resistance or inductance strain gauges /Oberti/ are used.

Certain difficulties arise at the evaluation of measured long term values when it is necessary to separate the creep values from those of shrinkage. Those difficulties are caused by

the fact that besides these effects the time-dependent strains are influenced by the initial state of stress and strain which must be estimated but the accuracy of such an estimation is unknown. The initial stress and strain distribution depends also on loads and effects occurring during the construction. The total strain due to creep and shrinkage is related with the value of sustained loads and thus the separation of those phenomena would be correct. Shrinkage strain may be observed on unloaded specimens situated in the same environment as the loaded structure /Kokubu, Goto, Ozaka Okamura, Momoshima; Okada, Koyanagi, Yoshioka; Cigánek, Meloun; Balázs, Horváth/. Though the results of such measurements are not fully satisfying, we have not another better way at present. Naturally, we are interested in the stress distribution in a structure at some age of it. The direct measurement of this distribution is very difficult. Kokubu, Goto, Ozaka, Okamura, and Momoshima used for that purpose the specimens cut out from the flanges of bridge girders. The flanges were made with appendages. In the case of prestressed concrete structures it is also necessary to separate the effects of steel relaxation which can be observed on specimens taken from the steel delivery and at an initial stress corresponding to the value of initial prestressing.

We can say that further attention to the improvement of measurement methods of time-dependent values should be devoted. The measurements are rather expensive, require much working effort and the results often do not fulfil expectations. All this is connected with the question of economy of measurements. In any case, it is necessary to carry out a great number of measurements when the evaluation of results is to be improved. Perhaps it would be possible to organize a research program based on the telemetrical transmission of measuring data from several structures to one point, where all changes of deflections, strains, temperature and humidity of structure and its environment would be registered. The obstacle to such a program form the above-mentioned difficulties in separation of several influences.

RESULTS OF MEASUREMENTS AND OBSERVATIONS

In order to compare the results of measurements and observations on actual structures it would be necessary to take into account all main factors influencing the value and curve of creep and shrinkage. As the variation of those factors is very extensive, it is clear that the comparison of values given in the reports presented to the symposium is not possible. The measurements were made on different types of structures, under different conditions and duration of observations.

Measurements of creep and shrinkage strains, reported by Kokubu, Goto, Ozaka, Okamura, and Momoshima, were taken from a comparatively great number of bridges. From the measurements a value of shrinkage of $200 \cdot 10^{-6}$ to $400 \cdot 10^{-6}$ was obtained for the period from 3 to 5 years. The authors emphasize the importance of situation of a bridge from the point of view of climatic conditions. A difference of $150 \cdot 10^{-6}$ in shrinkage strains between each side of the cantilever prestressed concrete girders was found as a result of the effects of sunshine, wind, rain, and snow. The measurements on two prestressed concrete bridges were carried out with the separation of shrinkage effects. In one of these bridges creep coefficients from 2 to 3 were determined for five years and in the other bridge a value of 1.2 was found from three years' observations. Creep coefficients derived from the measurements on twenty bridges in Japan varied from 1.5 to 2.5, which can be considered as a very good result of measurements. Oberti gives the results of measurements on pavement structure of Mont Blanc tunnel. The average horizontal strain reached a value of $550 \cdot 10^{-6}$ at the age of four years. In this structure the values of shrinkage strain at the end of the tunnel were found greater than in its middle. In the case of Bradano bridge, which had to be unloaded because of large long term deflections, 2.5 to 3 times higher values than those considered by the design were found. Oberti gives another example of prestressed precast beams in which about double creep and shrinkage strains were observed than those calculated by the design. Cigánek and Meloun measured shrinkage strain on vertical bearing walls of a multistorey

building and obtained values varying from 400.10^{-6} to 600.10^{-6} . A damage of a building is described due to differential shrinkage of cast-in-place walls and precast ceiling slabs. Swamy and Bennet report on the results of a programme of investigation of creep and shrinkage strains on a reinforced concrete structure during its construction and after its completion. Shrinkage strains were measured immediately after casting columns. Comparison of shrinkage and creep strains measured on columns and laboratory models shows that the laboratory specimens shrunk several times more at the initial stage than the exposed specimens. At the age of one year this difference was reduced so that the shrinkage strain reached a value of more than half of the shrinkage of specimens situated under controlled conditions. The authors investigated also the influence of a limestone and a granite aggregate. It showed that the limestone concretes had a greater movement than those of granite. The authors give also the results of variation of internal temperature and internal steel strains. Balázs and Horváth measured creep and shrinkage strains on two bridges, one of which was constructed of precast girders and the other was cast in place. In the first bridge values of 250.10^{-6} and 550.10^{-6} were found at the age of 100 and 1500 days respectively. The authors claim that the shrinkage was not finished. In the second bridge concrete specimens were steam-cured and showed the shrinkage 15 to 20 per cent lower than the normally treated ones. The authors compared the shrinkage of specimens situated inside of box girders with that of specimens situated on their surface. It was proved that the inside situated specimens showed lower shrinkage than the specimens situated on the surface of girders. In the case of steam-cured specimens the creep coefficients were 0.7 and 1.5 at the age of 100 and 1500 days respectively. In the case of normally treated concrete the coefficient of creep showed a value of 0.6 at the age of 100 days and 2.15 at 1500 days.

Jungwirth summarized various measurements and came to the conclusion that the end values of creep and shrinkage agreed at an accuracy of ± 15 per cent with the CEB Recommendations. The agreement of the time-dependent relations is less favourable. The long term strain curves show that the strains are re-

lated to the so called effective thickness of the body. The results namely show that the creep and shrinkage of thicker bodies is slower than those of thinner. The increase or decrease of temperature produces acceleration or retardation of creep at about 10 per cent. In summer the creep is 24 per cent quicker than in winter. Further Jungwirth remarks that not all evaluations give unambiguous results. It may be clarified by the fact that there is a considerable uncertainty as to temperature corrections. The effects of temperature are often merely assessed. Rotations of supports may cause considerable faults as well. Suppleness of shutterings may lead to a greater weight of structure than supposed in the evaluation. Authors presents a system of curves for creep and shrinkage strains suggested to be included into the improved Recommendations CEB.

Keijer in his contribution presents the time-deflection curves of bridges constructed by the cantilever method. The investigation showed that the creep coefficient of concrete has the prevailing influence on the magnitude and development of deflections. He remarks that the creep function containing logarithmic time-function shows a good agreement with the observed values. The conclusion that even after more than ten years stabilization of total deflection was not observed, is remarkable. Aichhorn presents results of measurements of creep and shrinkage at the age of 14 years. He remarks that the strain increment during the last 7 years was about 10 per cent, so that a conclusion results from it that the development of creep and shrinkage ceases approximately after 8 years and the four years' period considered in existing design is not sufficient. A different influence of seasons can be seen on the creep-time curves. That difference varies at a value of about 15 per cent. The similar tendency was observed for concretes of different strength.

Dilger, Ghali, and Kountouris report on very interesting experiments the aim of which was to observe the reaction changes in continuous prestressed concrete beams produced by differential settlement of supports. Those reactions are influenced by creep, change of the modulus of elasticity, and by the settlement-time curve. Experiments were carried out on five pairs of beams in such a way that a pair of beams were given

a constant deflection which was maintained and the time-dependent changes of force were measured. For one of the pairs the constant deflection was maintained during the whole period of observation while in the rest the deflection was changed at certain intervals. Authors compare the obtained force-time relations with those derived analytically for a modified principle of superposition which led to a good agreement between calculated and measured values.

The contribution of Terteá, Onet, and Beuran is devoted to the influence of creep and shrinkage on cracking and deflection development of lightweight reinforced concrete. Tests were carried out on two series of beams with simple respectively double reinforcement. Beams were subjected to sustained concentrated loads at the third-span points. Tests were carried out during a year period. The obtained results show that the total crack width grows and the number of cracks increases. In comparison with normal concrete the beams of lightweight concrete showed about 35 per cent greater crack width. For the production of testing beams two kinds of aggregates were used. The authors recommend that the depth of lightweight may be $1/15$ of their span. The experiments showed that the maximum crack width lies in admissible limits and thus reliability of such beams in construction is assured. The tested beams had also different steel percentage and the sustained load formed different portions of ultimate load. The long term deflections proved to have similar character of development for different values of sustained load.

Creep and shrinkage of lightweight prestressed concrete girder were investigated by Kokubu, Goto, Ozaka, Okamura and Momoshima. They found that creep of lightweight concrete is similar to that of normal concrete while the shrinkage was considerably lower.

Okada, Koyanagi, and Yoshioka were concerned with the influence of differential shrinkage on composite beams created in such a way that on the precast prestressed beam a slab was cast presenting the upper flange of the beam. Results of the experiments showed that the differential shrinkage reduce the cracking moment while no influence of that shrinkage on ultimate moment

was observed. The authors give a theoretical analysis of cracking moment with regard to the influence of differential shrinkage. The results show a good agreement with experimentally obtained values. The authors recommend to create the cast-in-place flange slab of lightweight concrete which may reduce the stresses produced by differential shrinkage because of low modulus of elasticity of lightweight concrete.

The validity of the principle of superposition is discussed by several authors /Jungwirth; Swamy, Bennet/. The study of Gamble is concerned with the modification of that principle in order to take into account non-linear creep. The validity of derived relations depends on determination of time function. The determination of that function is shown on example of 22 experiments data.

Ohne and Obata present the results of expansion joint movements during a year. The observations were carried out on two buildings. It was proved that the expansion joints show daily movements having a retardation of three hours in comparison with the extremal temperature. The changes of expansion joints diminish towards the column feet. The annual movements of expansions joints correspond with the atmospheric temperature and that of the roof slab. In the second building similar daily and annual movements were observed. The differences between the different parts of building exposed on differential cardinal points were noted. An approximately linear relation was found through a year between the daily minimum length of building and the daily minimum temperature.

The contribution of Rickenstorff is devoted to the study of long term effects of higher temperature /up to 300 centigrades/. The measurements on 81 reinforced and 4 prestressed concrete slabs and on 4 experimental chimneys present an extraordinary large programme and lead to many interesting conclusions. The speed of creep and shrinkage is accelerated by higher temperatures. The tendency to the higher permanent strains is shown even by heating old concretes. The thermal stress proved to be the main influence while the other factors and composition of concrete had a second-rate importance. The author presents also a mode of calculation of time-dependent stresses in thermal stressed concrete structures.

CONCLUSION

The review of the contributions shows that measurements and observations of structures enable broadening of our knowledge on behaviour of structures under sustained load and on shrinkage of concrete. The comparison of the results given in these contributions is, however, very difficult because of different types of structures, climatic conditions, duration of measurements, etc.

Some authors emphasize that it is very difficult to determine the exact values of shrinkage and creep on the base of measurements on actual structures. The difficulties also arise when the laboratory results are to be transferred on actual structures. However, damages on actual structures due to creep, shrinkage, and temperature changes lead to the conclusion that it is necessary to devote a concentrated attention to these questions. Oberti proposes that research and investigation among the laboratories and a cooperation among them should be organized.

I should like to add that besides the physical point of the problem it would be necessary to concern with criteria determininig the admissible limits according to which a designer can judge the danger due to long term effects on structures.

DISCUSSION PRÉPARÉE / VORBEREITETE DISKUSSION / PREPARED DISCUSSION**Experimental Observations of Prestressed Structures with Reference to Long-Term Deformations**

Observations expérimentales des déformations de longue durée sur des structures de béton précontraint

Beobachtungen an Spannbetonkonstruktionen in bezug auf Langzeitverformungen

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I. FOREWORD

In calculating prestressed concrete structures in Italy consideration is generally given to constant creep and shrinkage values, which the standards indicate as a minimum. In any case, these values are not related to the intrinsic characteristics of the concrete and structure nor to the environmental conditions under which the structure will be operating. When the slow deformations of the concrete considerably surpass the predicted standard values, the damage to the structure's serviceability might be of importance.

Here below are described two prestressed concrete structures whose abnormal deflections produced by the permanent and service loadings has made it necessary to undertake some experimental tests. This study has revealed the predominant importance of the slow deformation phenomena of the concrete in determining directly and indirectly (through large losses of prestress) the abnormal deflections under consideration.

It should be observed that this type of investigation, begun some time after the completion of the structure, usually encounters two kinds of difficulties, namely:

a) indirect measurements (deflections and rotations, pre-existent stresses, cracking effects) or direct but overall ones (strains due to different causes) must be performed, and they greatly reduce the accuracy of the results obtained;

b) it is generally not allowed to rigorously separate the phenomena investigated (e.g., the viscous phenomena from those of shrinkage) nor to determine a "weight" to be assigned to the various influencing parameters (nature of the material, curing age at prestressing time, environmental curing conditions, constructional

defects, etc.).

In view of these objective difficulties, independent experimental procedures have been developed in order to obtain, if not accurate values of the shrinkage and creep coefficients, at least possible value ranges.

2. BRADANO VIADUCT (I)

The viaduct is a post tensioned concrete structure. The cross-section is of the box type, with three webs containing the cables, and the cantilevered girders are articulated at mid-span (fig. 1).

The viaduct was built in 1959 and four years later particularly large deflections (8-10 cm) of the span center lines were observed. This made it necessary to undertake an especially accurate check program:

a) leveling measurements. These were made from time to time in 1959-1964 and went on systematically in 1964-1968, permitting to follow the lowering with time of the span center lines with respect to the fixed sections. It should be noted that, because of the post-tensioning and permanent loadings, a rise of the crown sections had to occur at first, due to creep. This indeed happened the initial period of the structure's life, thereby confirming the correct degree of initial prestress. But thereafter, because of the slow tension losses, the bending moments of the permanent loadings prevailed and the crowns of the spans began to sink (fig. 2). Besides the considerable size of the deflections, unusual is also the speed of the sinking, which is not yet completed at 10 years after the completion of the structure (fig. 3).

b) Static and dynamic loading tests. These permitted to establish the perfectly elastic behavior of the structure under the working loads and the natural vibration frequency, which proved to be very close to the calculated one. Indications were thus obtained regarding the E_b value, shown to be about $350,000 \text{ Kgcm}^2$, at 9 years.

Direct tests on five specimens extracted thereafter have yielded E_b values ranging from 290,000 to $350,000 \text{ Kgcm}^2$ (against the predicted value of $400,000 \text{ Kgcm}^2$). Ultimate load tests on the same date furnished $\sigma'_{br} = 344 \text{ Kgcm}^2$ against the predicted 375 Kgcm^2 at 28 days.

c) Evaluation of the residual prestress (σ_{ap}), in a direct way by testing the cable wires, and indirectly by tests on the stress release in the concrete. The former tests, made by cutting a steel wire, measuring its shortening and correlating it to Young's modulus as determined in laboratory, yielded the following average results (8 tests):

- at 4 years after completion of the structure	$\sigma_{ap} = 82,5 \text{ Kgcm}^2$
- " 5 " " " " " "	$\sigma_{ap} = 72 \text{ Kgcm}^2$

against the design σ_{ap} of 90 Kgcm^2 .

The indirect tests consisted in setting up 4 rosette-like strain gage bases read by a removable deformeter (fig. 4) at the intrados and extrados of the various fixed sections and then

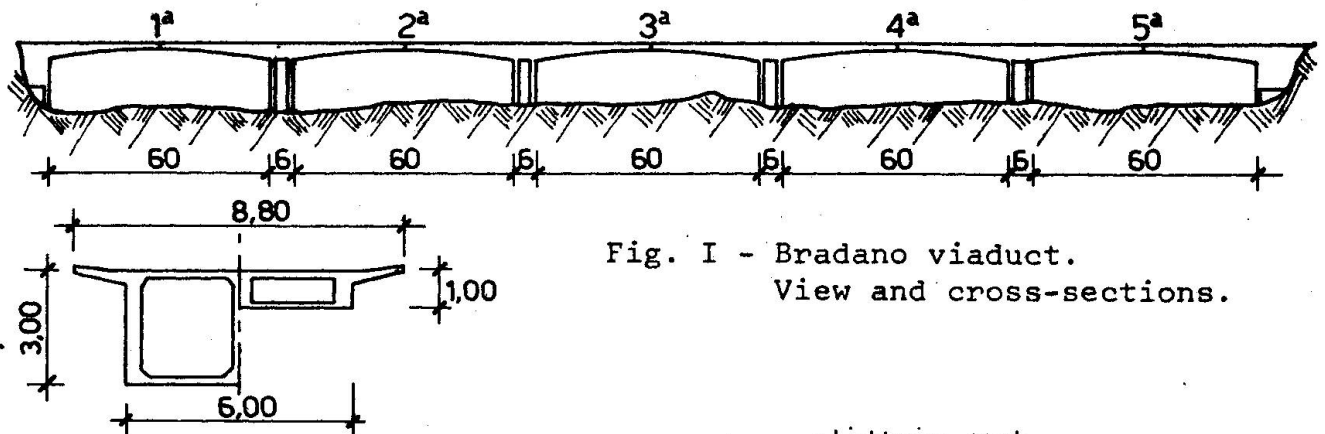


Fig. I - Bradano viaduct.
View and cross-sections.

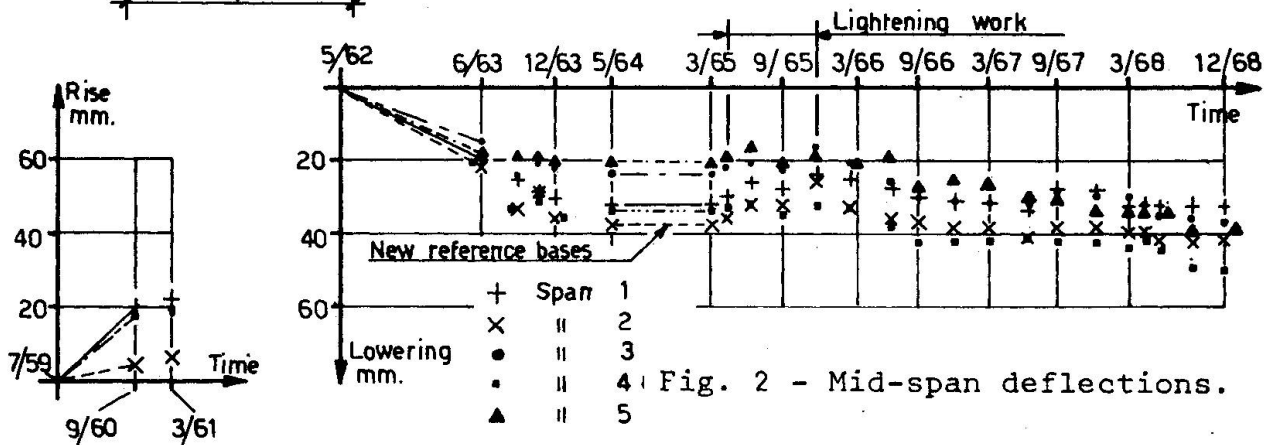


Fig. 2 - Mid-span deflections.

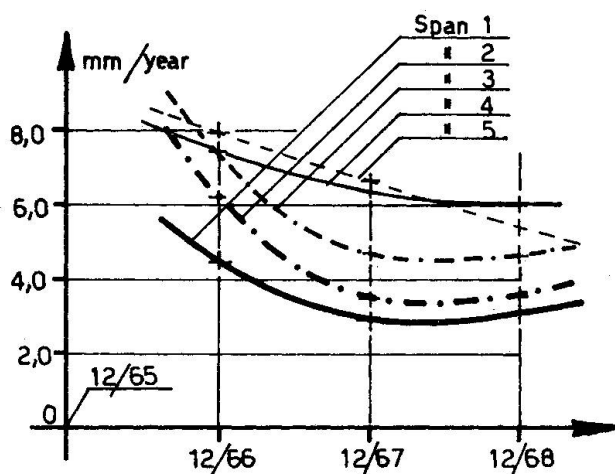
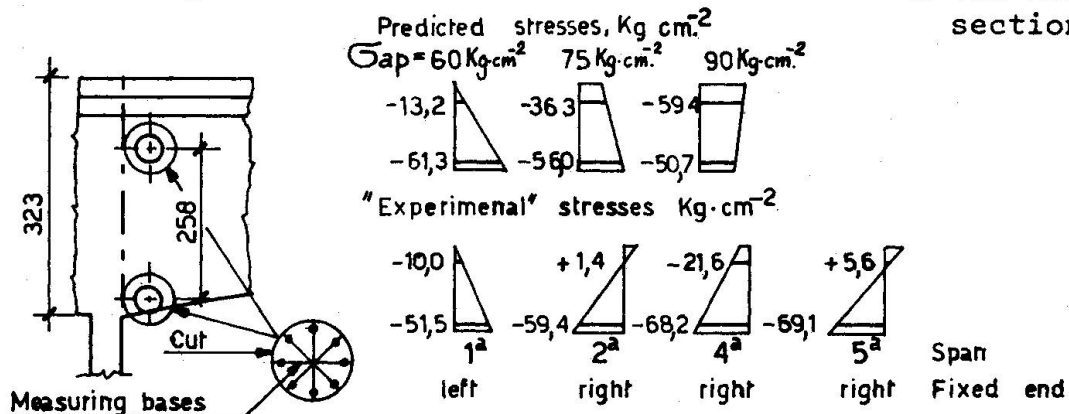


Fig. 3 - Mid-span lowering speed.

Fig. 4 - Predicted and actual stresses in the fixed sections.



cutting into by core boring. The strains thus released have permitted to pass by means of Young's modulus E_b and Poisson's ratio measured on the extracted specimens to the pre-existent stress values. Therefore, assuming a linear distribution of the stress, the "experimental" stress diagrams were obtained, comparable with those calculated for different values of the residual prestress (fig. 4). It proved to be $\sigma_{ap} = 60 \pm 70 \text{ Kgmm}^{-2}$ at 9 years of the structure's life. The total prestress loss has thus been $38-48 \text{ Kgmm}^{-2}$, against the 18 Kgmm^{-2} predicted for an infinite period of time.

d) Shrinkage tests. These tests were conducted in a room with constant temperature (20°C) and humidity (55%) on specimens prepared of the same aggregates and at the same proportions adopted for the prototype concrete.

Therefore, using the C.E.B. expression (2) for the shrinkage evaluation:

$$\epsilon_r = \epsilon_c \cdot K_b \cdot K_d \cdot K_t$$

the value K_b has been derived, and its application to the actual structure yielded:

$$\epsilon_r = 25 \pm 30 \cdot 10^{-5}$$

The results of the tests clearly show the substantial agreement between the high deflections values and the reduced residual prestress.

A comparison of the prestress tension losses, performed to bear out quantitatively these facts, leads to the following considerations:

a) assuming as valid the design value of steel relaxation (equal to 7% of the initial prestress) and taking into account the initial Young's modulus E_b (obtained from the experimental one) the creep factor, when supposing $\epsilon_r = 50 \cdot 10^{-5} (^\circ)$, is

$$\phi_N > 3,8$$

b) assuming as correct the design value of shrinkage and steel relaxation ($\epsilon_r = 25 \cdot 10^{-5}$), the creep factor is:

$$\phi_N > 4,8$$

These undoubtedly unusual values of the slow concrete deformations are believed to be caused by a particularly unfavorable concomitance of intrinsic and ecological factors, such as:

intrinsic factors: excess of fine material in the aggregates and, therefore, a high percentage of cement paste; high value of W/C; mineralogical and surface nature of the calcareous aggregates, quarried in a zone of predominantly tufaceous rock ($^\circ^\circ$).

ecological factors: particularly high annual mean temperature; frequent insolation throughout the year; on an average, low relative

($^\circ$) Due to exceptional climatic conditions.

($^\circ^\circ$) Not many studies exist on the influence of the aggregates on the phenomena under examination (known are those of Davis, U.S.A., and Kordina, Germany). However, it is believed that in some cases this influence may not be negligible.

humidity and low rainfall; high windiness.

3. PRESTRESSED CONCRETE FLOOR BEAM WITH POST-TENSIONED CABLES (I)

The beams, designed for an office floor, have a T like cross-section and a 14.3 m span (fig. 5).

The deflections, measured during the acceptance tests 3 months after the tensioning, were found to be about twice the calculated ones. This excessive deformability was clearly caused by a too early cracking.

Assuming as cracking moment the expression

$$M_f = W_i (\sigma'_{bi} + \sigma_{br}) \quad (I)$$

where

W_i = lower section modulus

σ'_{bi} = stress at the intrados edge

σ_{br} = tensile strength of the concrete

it was necessary, to explain the phenomenon, to assume a high loss of prestress, even when supposing that parasitic microcracks (due to the prevented shrinkage) had already at the beginning exhausted the tensile strength of the material.

Having ascertained that these losses were in no way to be attributed to steel relaxation, tests were carried out to find the actual state of stress in the cables and, hence, the slow losses that occurred.

The following was, therefore, determined experimentally:

- a) actual cracking moment, obtained by a distributed load test;
- b) state of stress in two sections of the concrete near the supports (fig. 6) determined by means of the strains "released" by core boring and Young's modulus E_b measured on the extracted specimen (fig. 7).

The largest possible cable prestress was derived from the experimental cracking moment, putting into equation (I) $\sigma_{br} = 0$ (pre-existent microcracking) whereas the lowest possible prestress value was deduced from the stress measured in the tested sections (in fact, a part of the prestress loss had to be attributed to the permanent deformations produced by the previous loading test).

The residual prestress, determined by the above outlined criteria, was 0,88-0,80 the design one for an infinite period of time. This accounts for the excessive deformability of the beam and, hence, its reduced serviceability.

A comparison between these prestress losses and those obtained analytically (considering the experimental E_b value of 230,000 $\text{Kgc}\bar{\text{m}}^2$ against the predicted one of 391,000 $\text{Kgc}\bar{\text{m}}^2$) yields, with

$\epsilon_r = 25 \cdot 10^{-5}$ like the design one, a maximum $\phi_N = 5.1$. Instead, assuming hypothetically $\epsilon_r = 50 \cdot 10^{-5}$, we get $\phi_N = 3.1$ against the predicted 2.5. In this case it is believed that the phenomenon should essentially be attributed to the intrinsic properties of the material which had a particularly high strength ($\sigma'_{br} = 420 \text{ Kgc}\bar{\text{m}}^2$, as an average of ten specimens at 210 days), associated with a low

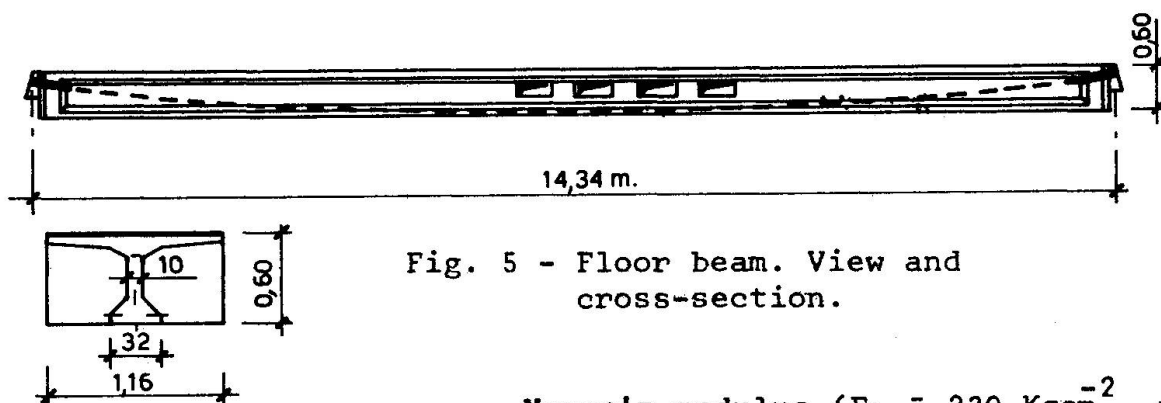
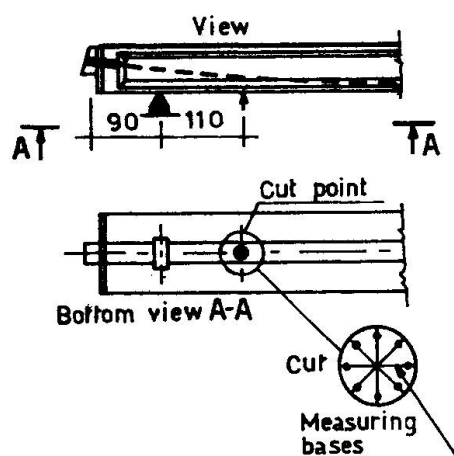


Fig. 5 - Floor beam. View and cross-section.

Young's modulus ($E_b = 230 \text{ Kgc} \cdot \text{cm}^{-2}$, an average of four specimens extracted at 210 days).



- Fig. 6 -
Test for evaluating
the pre-existent
concrete stress.

4. CONCLUSIONS

1) The experimental analysis of the phenomena under examination is particularly difficult when the study is undertaken after the start of the construction, which is very frequently the case in the structures behaving abnormally only after a certain period of time. In these cases it might be possible to indicate value ranges believed to comprise the creep and shrinkage deformations, whereas it is practically impossible to determine quantitatively, but only to suppose the "weight" to be attributed to the various influencing factors.

2) Not infrequently the slow deformation values are higher than the predicted standard ones, causing severe consequences to the deformability of prestressed concrete structures. It is, therefore, believed that in designing prestressed concrete structure it is necessary to check the slow

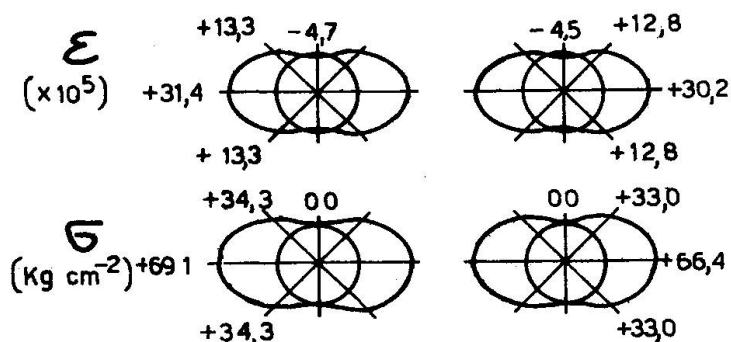


Fig. 7 - Polar diagrams of the strains and stresses released by core boring.

deformability characteristics of the material, possibly by conveniently extrapolated short-term tests or by evaluating parameters which empirically may be correlated to those characteristics. Moreover, particular care should be taken in evaluating the influence of the environmental factors, like humidity, temperature insolation, windiness, etc.

Finally it is suggested

that, inversely to the profundity of the quantitative analysis of the phenomena examined, it is advisable in the design stage to use adequate factors of safety.

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SUMMARY

The paper emphasizes the particular sensitivity of prestressed concrete structures as regards greater-than-average rheological phenomena of concrete.

In this connection, the investigation carried out on two structures is described, indicating its limits and results. The usefulness of an advance careful evaluation by the designers of the slow deformation phenomena of concrete is thus brought out.

RESUME

On met en relief la sensibilité particulière des structures en béton précontraint au point de vue des phénomènes rhéologiques du béton de grandeur supérieure à la moyenne.

On décrit à ce propos les recherches développées sur deux structures dont on présente les limites et les résultats, justifiant ainsi l'utilité pour l'ingénieur d'une étude soignée des phénomènes de déformations lentes.

ZUSAMMENFASSUNG

Es wird die besondere Empfindlichkeit der Spannbetontragwerke gegenüber ungewöhnlichen rheologischen Erscheinungen des Betons hervorgehoben.

Zu diesem Zwecke werden die an zwei Bauten vorgenommenen Untersuchungen beschrieben, wobei Schwierigkeiten und Ergebnisse angegeben werden und somit die Nützlichkeit gezeigt wird, während des Entwurfes eingehende Voruntersuchungen über die langsamen Betonverformungen anzustellen.

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Measurements of Shrinkage and Creep Strains on Post tensioned Waffle Slab

Mesures du retrait et fluage dans une dalle en caissons précontraintes

Messungen der Schwind- und Kriechdehnungen einer nachgespannten Rippendecke

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Introduction

In 1966 a post tensioned waffle slab was constructed for the Council of the City of Sydney. During construction and since the slab has been in service as a parking station, a detailed investigation has been carried out into the behaviour of the structure, and in particular a large series of measurements were made of shrinkage and creep strains.

Description of Structure

The maximum overall dimensions of the suspended concrete deck were 259'-0" x 191'-6" and as it was essential that large vehicles have easy access underneath, it was decided to locate the columns on a grid 58'-0" x 57'-4" with one span 62'-0" and with cantilevers 10'-0", 12'-0" and 13'-0".

The maximum live load used in the elastic analysis of the suspended slab was taken as 60 lbs. per square foot, which was equivalent to the whole area of the slab being completely covered with the heaviest vehicles which were likely to use it. However, under the most likely parking layout for this slab the uniformly distributed live load will be about 25 lbs. per square foot and it was decided that a loading of 30 lbs. per square foot would be taken as the live load when making calculations for load balancing.

For the ultimate analysis of the slab, the maximum possible load was taken to be a live load of 100 lbs. per square foot which was equivalent to 2000 people in each bay.

The suspended slab consists of a waffle slab of total depth 15" which includes a top flange 3" deep. The ribs of the waffle slab are 6" wide and 12" deep at 2'-6" centres in both directions. Over the inverted cone-type precast concrete capital there is a bottom flange to the waffle slab which is also 3" thick. A plan of the slab is shown in Fig.1.

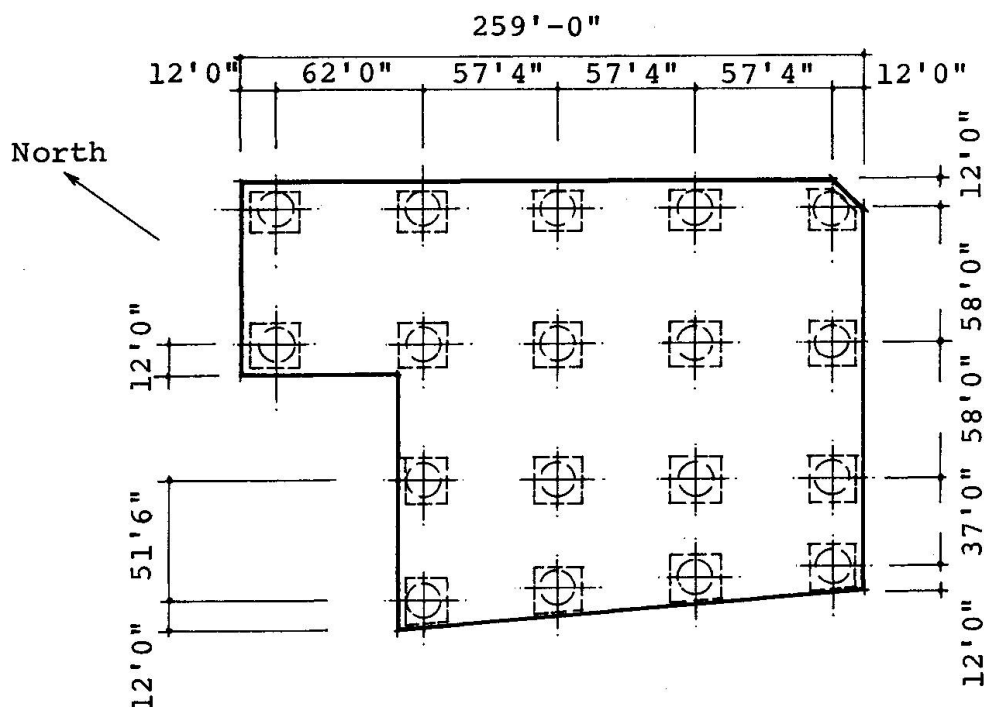


FIG.1 PLAN OF DECK AND COLUMN GRID LAYOUT

The slab is post tensioned with four $\frac{1}{2}$ " diameter stress relieved strands in each rib and thus the tendons are equally spaced in both directions. The four $\frac{1}{2}$ " diameter strands pass through oval shaped galvanised steel ducts $2\frac{1}{2}$ " x $\frac{3}{4}$ " which were filled with grout after the post tensioning. The amount of prestress in each tendon is identical, but the distance of the centre of the tendon from the neutral axis of the concrete section was varied from rib to rib and throughout the length of the tendon in order that the average prestressing force in the tendon would approximately balance the dead load of the slab plus a live load of 30 lbs. per square foot on the slab. In general, the strands were stressed from both ends and were continuous throughout the full length and breadth of

the slab.

Welded wire fabric was used to reinforce the upper flange of the waffle slab and the lower flange where it occurs over the columns, and in addition, over each column in the top of the slab there are $\frac{1}{2}$ " diameter cold worked deformed bars at 3" centres in both directions. The only other reinforcement in the slab is steel reinforcement stirrups in the ribs in areas surrounding the column supports.

The column Capitals consist of precast cones which are separated from the slab by teflon and neoprene bearing pads. These cones are 16'-0" in diameter with a total depth of 4'-0" and the cones together with the waffle slab are shown in Fig. 2. The columns consist of 24" diameter reinforced concrete poured in precast concrete pipes 1-5/8" thick, and the columns are supported on pile caps which rest on groups of piles to bedrock.

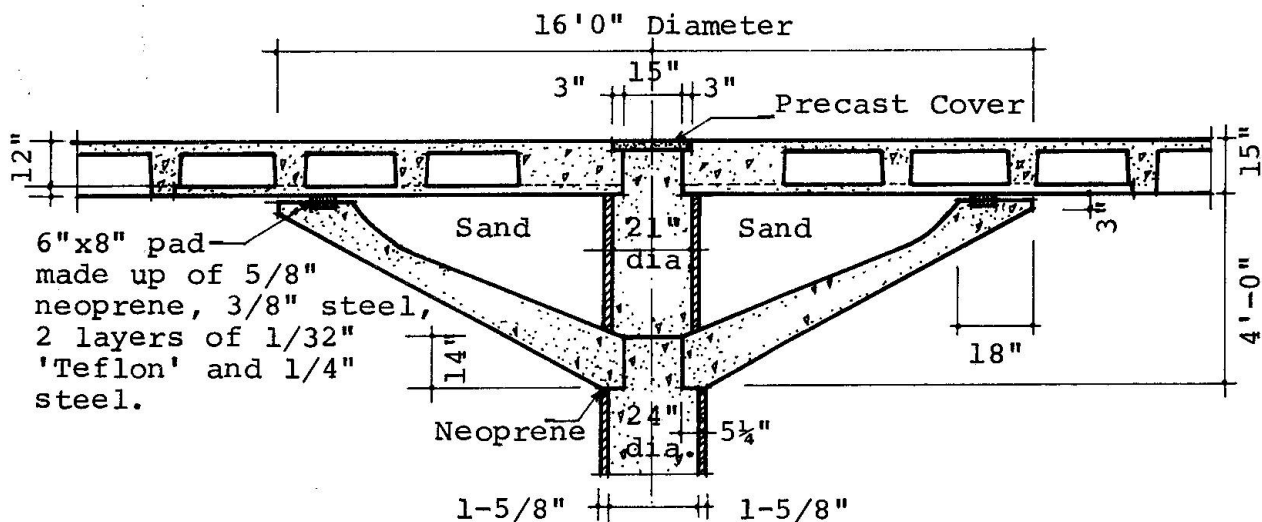


FIG.2 SECTION THROUGH WAFFLE SLAB AND COLUMN CAPITAL

Under dead load and a live load of 30 lbs. per square foot, the average concrete compression stress was 765 lbs. per square inch except over the column heads where it was 530 lbs. per square inch, in both cases after the assumed losses have taken place. The elastic stresses under dead load with live load of 60 lbs. per square foot comply with the provisions of the relevant Code, and there is a load factor of two against flexural failure under dead load plus a live load of 100 lbs. per square foot. The principal tensile stresses were checked under the combined action of prestress, flexure and shear. With a load factor of 1.8 and a live load of

60 lbs. per square foot, the maximum tensile stress at the critical section for shear was 188 lbs. per square inch, acting at an angle of 62 degrees to the neutral axis.

As no control joints were provided in the slab, construction joints were made so as to divide the slab into 5 pours, each of approximately 200 cu.yds. and a sixth pour was made of the access bridge to the main slab. The same aggregate, brand of cement, water cement ratio and mix proportion were used for each of the six pours, but as shown later, it is believed that different batches of cement which were ground to a varying degree of fineness resulted in a variation of concrete strength and shrinkage between different pours. The concrete was specified to have a compressive strength of 4000 lbs. p.s.i. in 28-day cylinders and it was specified that there should be a minimum increase in cylinder strength after one year of 25%. The total shrinkage was specified as not more than 500×10^{-6} in laboratory shrinkage tests on standard (A.S.T.M.) prisms. Curing consisted of an initial sprayed curing compound followed by a continuously water sprayed hessian covering. The curing continued until stressing commenced 25 days after placing of concrete.

Testing

The following tests were carried out -

- (1) Laboratory tests on cylinders at 7 days, 14 days, 28 days, 100 days and 1 year.
- (2) Laboratory shrinkage tests on 3"x3"x11" prisms.
- (3) Field shrinkage tests on 3"x3"x11" prisms. placed at top of the slab.
- (4) Field measurements of strain in the prestressed slab, using vibrating wire strain gauges.
- (5) Measurement of actual contraction of the slab at its edges from brass plugs embedded in the slab.
- (6) Measurement of deflection of the slab from about 50 brass plugs embedded in surface of the slab and measured by optical methods.
- (7) Measurement of bending moments over column heads by strain gauges attached to reinforcement in slab.

- (8) Measurement of strain, using a vibrating strain gauge in an unstressed concrete waffle slab 5 ft. long x 3 ft. wide, which has been placed on the main slab.
- (9) Measurement of temperature within the body of the slab.
- (10) Laboratory tests on the elastic modulus of concrete at various ages.

Results of Compressive and Shrinkage Tests

Table 1. Compressive strengths in 12"x6" cylinders in p.s.i.

POUR NO.	A g e a t T e s t				
	7 Days	14 Days	28 Days	100 Days	12 Months
1	2886	3796	3755	4280	4140
2	3548	3836	4383	4166	5026
3	3482	4028	4572	5010	5770
4	3344	3902	4343	4910	5400
5	3240	3946	4128	4470	5470
6	3402	4044	4419	4804	5290

Table 1 shows the average compressive strengths from cylinders at various ages. In the case of the cylinders taken at 28 days there were 24 cylinders taken in each of the six pours. For pour No.1 the average was 3,755 p.s.i. with a standard deviation of 435 p.s.i. for the 24 cylinders. For the average of the other five pours the mean was 4,368 p.s.i. for 120 cylinders. It can be shown that the difference of 613 p.s.i. between these two means is highly significant. Similarly, the average for pour No.3 at 28 days was 4,572 p.s.i. with a standard deviation of 241 p.s.i. for 24 cylinders. The difference between this mean and the mean for the other five pours, 4,204 p.s.i., is also statistically highly significant. In both cases the significance level is less than 0.2%. For this reason separate curves for development of compressive strength were plotted for pours 1, 3 and the remainder of the pours and these are shown in Fig.3.

Table 2 gives the results of 36 laboratory shrinkage specimens. These were cured for 18 hours in a humidity room at a relative humidity of 90% or above, and then immersed for 27 days in limewater. The specimens were then stored in a drying room in accordance with an A.S.T.M. Standardisation Procedure C157. From Table 2 can be seen that the shrinkage of the six specimens taken from pour No.1 appears to be rather less than the shrinkage of the specimens taken from the other five pours. Using the Wilcoxon's Sum of Ranks Test, it was shown

Table 2 Laboratory Shrinkage Specimens Strain $\times 10^{-6}$

SPEC- IMEN NO.	POUR NO.	28 Days	32 Days	35 Days	42 Days	56 Days	84 Days	91 Days	104 Days	117 Days	140 Days	252 Days	476 Days
97	1	17	187	257	313	440	524			439	452	482	462
99	1	-184	-40	36	91	241	336			335	328	373	315
101	1	-107	96	164	187	290	425			420	392	424	325
103	1	-101	68	133	234	329	484			436	425	485	445
105	1	-67	79	167	240	363	488			465	449	508	469
107	1	-100	70	162	244	392	492			470	450	495	471
109	2	-108	103	123	248	327	384		467		440	451	472
111	2	-71	142	162	271	387	463		517		522	559	525
113	2	-82	174	187	295	413	485		528		537	618	567
115	2	-88	128	147	261	366	449		509		515	547	499
117	2	-115	97	126	237	326	422		457		458	517	486
119	2	-73	132	144	221	346	421		444		439	493	438
121	3	-68	135	173	275	382	439		481		472	524	504
123	3	-133	54	106	221	354	409		470		451	498	479
125	3	-80	104	148	227	327	377		432		424	441	411
127	3	-90	112	177	262	382	438		506		499	560	537
129	3	-104	109	149	284	366	416		487		474	527	508
131	3	-89	116	171	271	388	439		501		492	544	521
133	4	-96	130	169	241	376	428		481		482	509	549
135	4	-141	111	150	232	370	420		480		500	546	540
137	4	-107	112	147	226	353	403		452		472	507	495
139	4	-86	138	182	254	386	430		481		495	551	538
141	4	-122	90	140	201	333	371		398		423	451	408
143	4	-79	140	192	262	418	455		479		485	538	512
145	5	-91	142	184	273	370	412		429		451	454	447
147	5	-85	111	159	250	377	446		479		479	489	469
149	5	-83	139	179	295	419	503		539		556	581	587
151	5	-60	158	200	300	414	489		528		518	556	560
153	5	-104	118	149	271	415	483		498		488	551	529
155	5	-149	69	102	208	321	391		425		426	450	436
157	6	-136	27	107	220	344	409	424			484	563	512
159	6	-67	121	192	315	438	492	517			584	648	589
161	6	-75	92	162	279	429	475	500			559	628	570
163	6	-106	47	115	232	353	408	433			476	572	491
165	6	-214	-17	63	190	325	364	386			429	505	475
167	6	-135	48	114	233	378	428	443			474	535	464

that the difference was highly significant. Therefore the results of the laboratory shrinkage for pour No.1 are shown separate from pours 2 to 6 in Fig.4.

It can be seen from a comparison of figures 3 and 4 that the concrete with the lower compressive strength had lower shrinkage. The average compressive strength of pour 1 at 12 months was 16% less than the other pours while at the same age the shrinkage from pour 1 specimens was about 14% less than the others.

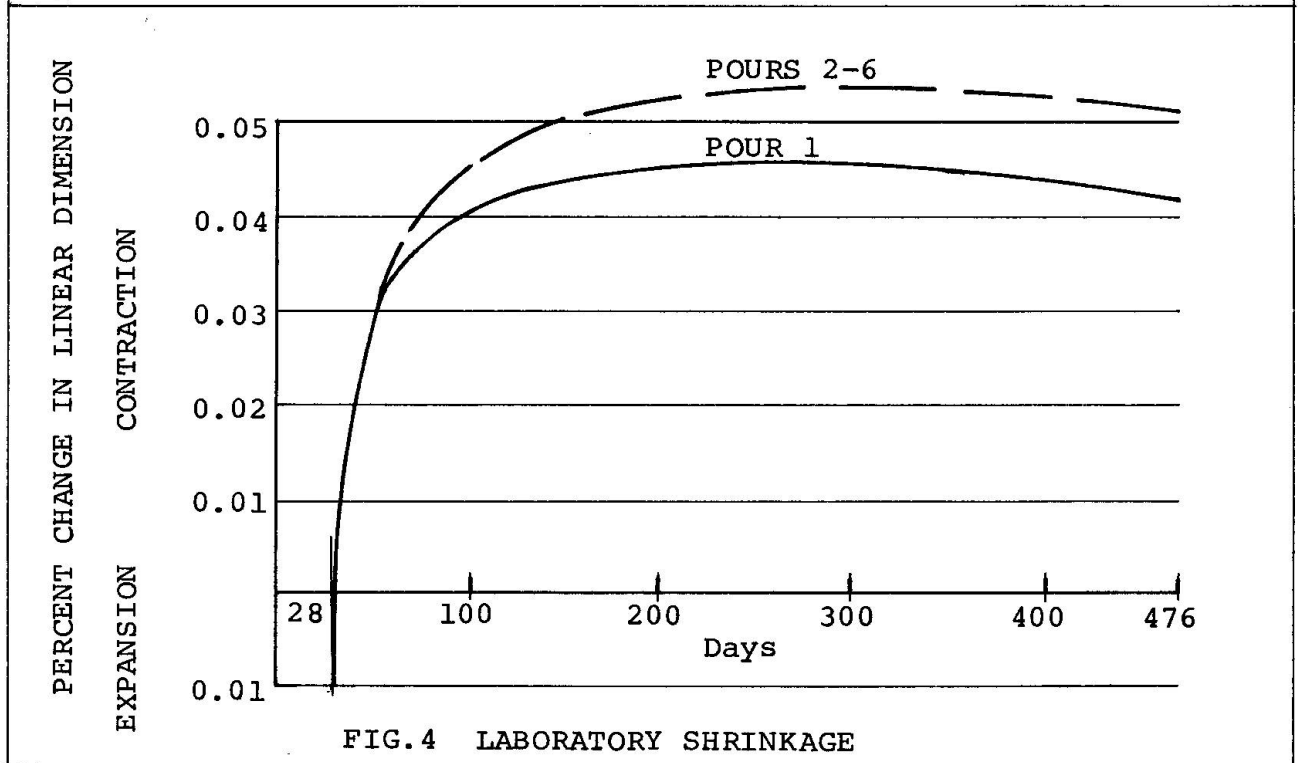
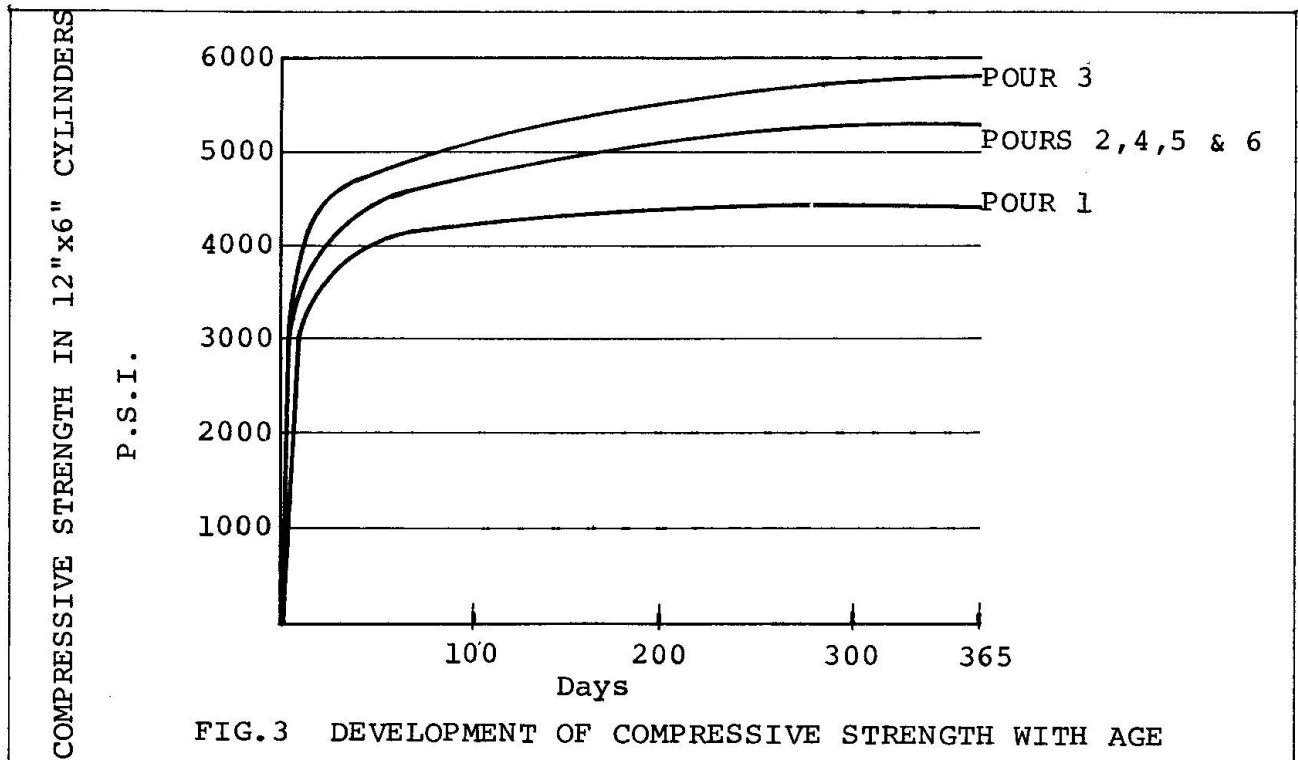
Table 3 shows the results of the 36 sets of field shrinkage prisms. These were cured for 18 hours in a humidity room and then taken onto the site and given the same curing and treatment as the

concrete slab. It is noticable that the strain for the field shrinkage specimens at 42 days was about one half the strain of the laboratory shrinkage specimens but at 252 days the field shrinkage is less than one third of the laboratory shrinkage. Also there is variation in field shrinkage with the position of the specimen on the parking deck. Further, the field shrinkage was affected by daily and seasonal variations in temperature and humidity.

Table 3 Field Shrinkage Specimens Strain $\times 10^{-6}$

SPEC- IMEN NO.	POUR NO.	28 Days	35 Days	42 Days	56 Days	70 Days	84 Days	105 Days	140 Days	175 Days	252 Days	336 Days	420 Days	504 Days
98	1	42	195	-34	63	140	194	188	110	105	138	247	130	136
100	1	89	190	-6	82	68	115	101	60	63	81	179	55	48
102	1	89	148	-9	61	118	151	141	93	89	102	213	108	103
104	1	74	153	-23	42	125	174	190	158	135	145	230	127	86
106	1	6	126	-80	-5	41	98	125	93	68	102	195	57	65
108	1	92	170	-34	69	126	189	197	139	141	145	229	131	94
110	2	140	-18	88	219	71	166	285	30	200	105	120	262	188
112	2	101	-52	40	188	27	120	224	-10	130	66	88	206	149
114	2	111	-40	71	199	40	146	262	6	199	89	107	246	202
116	2	169	-16	109	237	86	158	274	8	181	106	103	218	179
118	2	179	12	126	240	93	200	325	63	261	177	183	322	291
120	2	99	-48	73	182	102	141	223	4	184	114	114	223	192
122	3	110	-15	124	201	79	152	265	20	207	165	163	265	179
124	3	164	30	172	206	87	236	262	29	178	153	152	287	174
126	3	85	-32	98	153	43	108	226	-7	170	124	97	229	114
128	3	64	-21	131	195	69	179	284	38	219	162	146	287	180
130	3	72	-8	134	193	70	171	255	21	184	163	139	285	148
132	3	60	-10	130	187	31	117	251	-30	116	79	66	219	101
134	4	1	191	97	115	198	146	238	116	186	178	216	151	309
136	4	27	92	108	124	176	124	207	119	179	171	177	112	301
138	4	21	90	108	119	186	141	231	121	182	139	156	100	281
140	4	38	121	140	173	221	164	203	158	220	205	213	141	324
142	4	-30	75	87	117	175	125	201	95	161	113	115	65	230
144	4	-10	71	85	119	199	137	209	106	186	169	190	136	301
146	5	-22	148	258	115	205	140	100	127	224	156	172	135	296
148	5	-28	121	225	91	184	124	84	107	158	141	152	121	256
150	5	-24	131	228	116	187	135	115	128	148	127	148	98	265
152	5	-3	150	214	116	202	141	110	129	190	178	188	154	305
154	5	7	161	247	149	223	176	136	141	186	150	170	119	260
156	5	-14	108	183	76	152	68	32	118	150	120	120	50	219
158	6	93	190	156	62	165	383	89	156	63	184	202	182	73
160	6	161	214	230	101	229	391	133	159	91	279	148	227	130
162	6	101	183	168	69	174	372	118	119	68	189	189	168	70
164	6	101	175	181	54	164	334	80	119	62	202	182	181	95
166	6	91	202	179	67	135	369	79	79	6	129	113	100	-6
168	6	131	218	176	81	189	380	111	133	81	201	191	177	108

Negative strains denote expansion of the specimens.



Results from Vibrating Strain Gauges

The vibrating strain gauges were set into blocks of concrete approximately 3"x3"x5" on the day before the slab was poured and the block was then supported within the legs of large bar chairs which also supported the tendon ducts. Of the 44 gauges used, all

except 3 gave results which could be compared with the strains measured from the shortening of the slab. Naturally, there was considerable unexplained variation between one gauge and another and it was apparent that to obtain reasonable results, a considerable number of gauges must be used.

In general, the concrete showed an initial expansion which occurred for at least one week after the pouring of the concrete. The expansion varied between 50×10^{-6} and 170×10^{-6} and at the time of stressing the concrete was still in an expanded condition. The post tensioning caused an average contraction of 140×10^{-6} and a further contraction due to creep and shrinkage of 190×10^{-6} occurred in the 3 months after prestressing. In the following 21 months an additional contraction of 150×10^{-6} took place. On a gauged line of 259 ft. the total measured contraction 2 years after the initial post tensioning of the concrete was $1\frac{1}{2}$ " which corresponds to a strain of 480×10^{-6} . Fig. 5 shows the average strain using results both from vibrating strain gauges and measurements from brass plugs embedded in the slab.

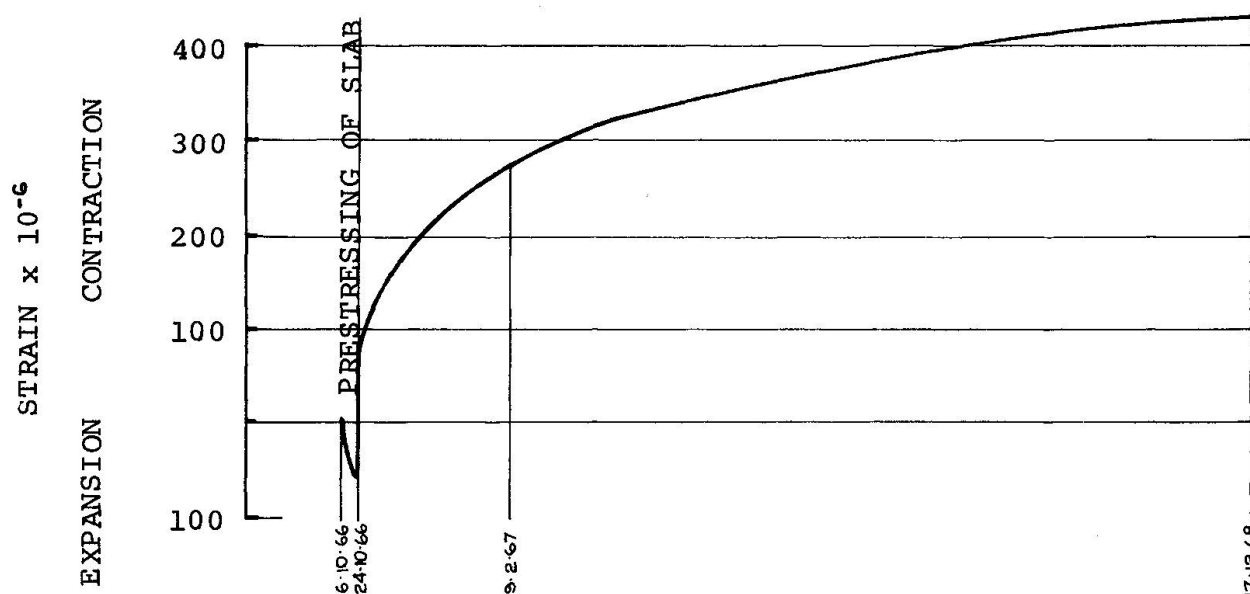


FIG.5 MEASURED STRAINS ALONG GRID LINE B

The gauge in the block which was not stressed, showed an expansion strain of 40×10^{-6} 21 days after pouring of the concrete. Contraction due to shrinkage then occurred so that 23 months later the measured strain was 170×10^{-6} . The result obtained by the vibrating wire strain gauge on the unstressed block was in agreement with the results on the field shrinkage prisms.

Acknowledgement

The Division of Building Research, C.S.I.R.O. and the Engineering Department of the Council of the City of Sydney carried out the detailed measurements of the structure during construction and in service. The Cement and Concrete Association of Australia made funds available to assist the C.S.I.R.O. in carrying out its share of the work.

SUMMARY

Results are presented of laboratory and field measurements of strains due to prestressing, creep and shrinkage in a post tensioned waffle slab. Prior to prestressing the concrete showed an initial expansion of about 100×10^{-6} . In the unstressed specimens the expansion turned to shrinkage after about 21 days with a contraction after 23 months of 170×10^{-6} . Measurements of the stressed slab showed a total shrinkage plus creep contraction of 340×10^{-6} . Measurements of shrinkage in the laboratory gave results more than twice those in the field.

RESUME

On présente des résultats de mesures effectuées en laboratoire et au chantier des dilatations provoquées par la précontrainte, le fluage et le retrait d'une dalle en caissons. Avant la précontrainte, le béton indiquait un allongement d'environ 100×10^{-6} . Pour le spécimen non-précontraint, l'allongement est devenu une contraction après environ 21 jours avec une contraction après 23 mois de 170×10^{-6} . Les mesures de la dalle précontrainte ont indiqué un retrait plus fluage total de 340×10^{-6} . Les mesures du retrait en laboratoire ont donné des résultats deux fois plus grand que celles sur l'ouvrage en place.

ZUSAMMENFASSUNG

Es werden die Ergebnisse von Dehnungsmessungen in Labor und Feld mitgeteilt, die sich aus Vorspannung, Kriechen und Schwinden in einer nachgespannten Rippendecke ergaben. Vor der Nachspannung wies der Beton eine Quellung von ungefähr 100×10^{-6} auf. In den unbelasteten Probestücken verwandelte sich das Quellen nach etwa 21 Tagen in Schwinden mit einem Schwund von rund 170×10^{-6} nach 23 Monaten. Messungen an der belasteten Decke zeigten einen totalen Schwund samt Kriechen von 340×10^{-6} . Schwindmessungen im Labor ergaben doppelt so hohe Werte wie jene im Feld.

Ergebnis von Dehnungs- und Temperaturmessungen in der Stauwand des Krafthauses eines Pumpspeicherwerks

Strain and Temperature Measurements on the Dam Wall of a Powerhouse
of a Pump Storage Station

Résultats des mesures de dilatation et de température dans le mur de
retenue d'une usine hydro-électrique par pompage

HANS-GERD MEYER

Dr. Ing.
Deutschland

1. Allgemeines

Das Krafthaus am unteren Becken des Pumpspeicherwerkes ist im Querschnitt als geschlossener Stahlbetonrahmen ausgebildet. Eine vertikale Seite des Rahmens ist die etwa 116 m lange, etwa 20 m hohe und etwa 2,50 m dicke Stahlbetonstauwand. (s. Bild 1)

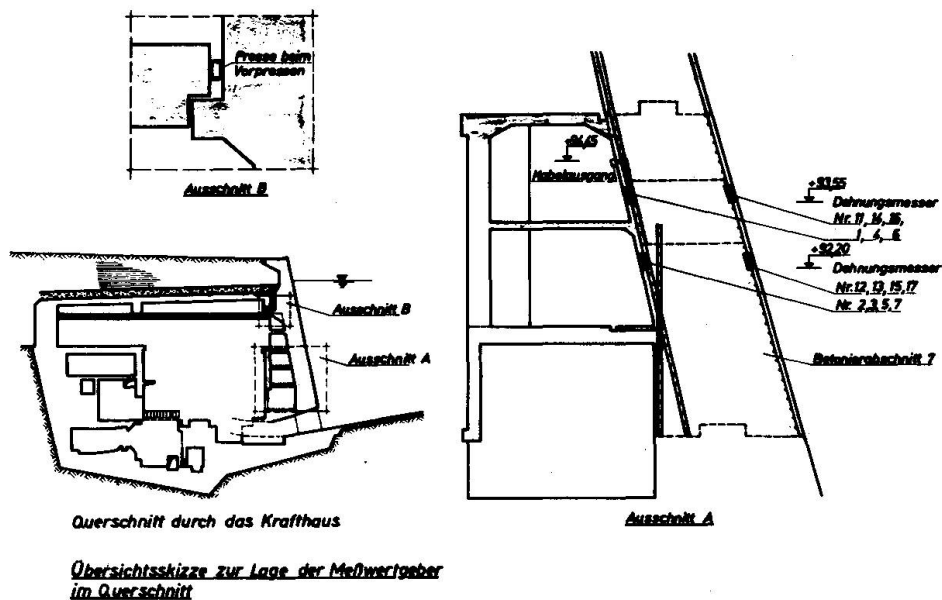


Bild 1:

In Längsrichtung war das Krafthaus in 4 Blocks - entsprechend den 4 Generatoren bzw. Pumpensätzen - durch Dehnungsfugen unterteilt. Alle Baumaßnahmen erfolgten blockweise.

Um auf der Unterwasserseite der Stauwand zur Sicherung gegen Risse auf alle Fälle einen Druckspannungszustand zu erreichen, wurde die Stauwand in Höhe der Krafthausdecke – nach Fertigstellung des Krafthauses bevor das Unterbecken mit Wasser gefüllt wurde – in Richtung Unterwasser vorgepreßt. Das Vorpresen erfolgte mit ölhydraulischen Pressen, die in einem dafür vorgesehenen Pressenspalt zwischen Stauwand und der auf einer Auskragung der Stauwand aufliegenden Krafthausdecke angesetzt wurden.

Zur Kontrolle der beim Vorpresen in der Bewehrung der Stauwand auftretenden Spannungen, aber auch, um einen Gesamtüberblick über die Spannungen der Stauwand zu erhalten, sollten die Dehnungen in der Hauptbewehrung der Stauwand gemessen werden. Die Messungen wurden vom Institut für Materialprüfung und Forschung des Bauwesens der Technischen Universität Hannover vom Verfasser durchgeführt.

2. Durchführung der Messungen

Für die Dehnungsmessungen wurde vom Prüfinstitut ein auf Dehnungsmeßstreifenbasis arbeitender temperaturkompensierter Dehnungsgeber entwickelt, der spannschloßartig in die Bewehrung ($\varnothing$ 22 mm, St III a) eingeschraubt werden konnte. Die Temperaturmessung erfolgte mit gesondert hergestellten elektrischen Widerstandsthermometern. Die Meßinstrumente wurden während der normalen Betonierarbeiten an der Stauwand mit einbetoniert. Die Meßstellen wurden sowohl auf der Unterwasserseite, als auch auf der Oberwasserseite jeweils in 2 verschiedenen Höhenlagen angeordnet. Insgesamt waren – verteilt auf die 4 Blocks des Krafthauses – 56 Meßstellen vorhanden. Eine Übersicht über die Lage der Meßstellen geben Bild 1 und Bild 2.

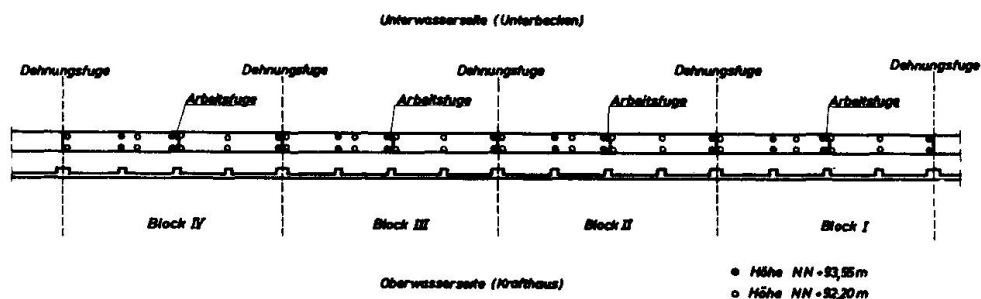


Bild 2:

Übersichtsskizze zur Lage der Meßwertgeber im Grundriß

Alle Meßstellen wurden in unregelmäßigen Abständen, je nach Baufortschritt, einzeln durchgemessen. Als Null-Messung wurde die Messung unmittelbar vor dem Betonieren genommen. Die Messungen wurden dann über einen Zeitraum von etwa 3 Jahren weitergeführt. Die Meßgeräte sind z. Zt. noch voll einsatzfähig, so daß jederzeit weitere Messungen vorgenommen werden könnten, die zweifellos interessant wären.

3. Auswertung der Messungen

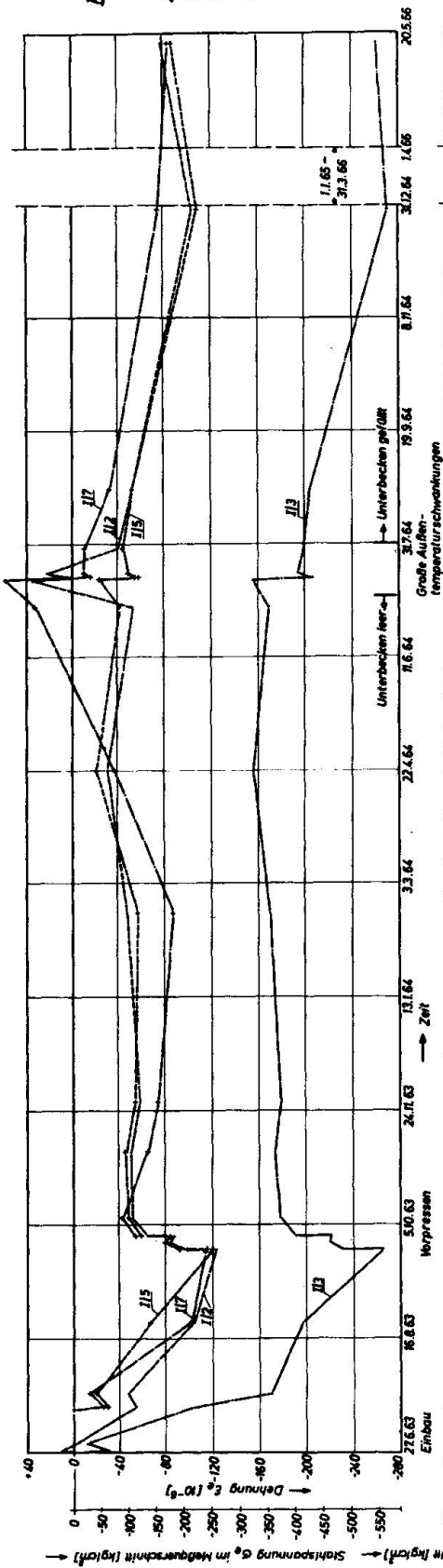
3.1. Dehnungsverlauf

Wie nicht anders zu erwarten, ergaben die Einzelmessungen auch an den an und für sich korrespondierenden Meßstellen teil-

Bild 3 a

Zeitlicher Verlauf der
Stahldehnungen und
Stahlspannungen
Block I (Oberwasserseite)

a) H=92,20 m ü. NN



b) H=93,55 m ü. NN

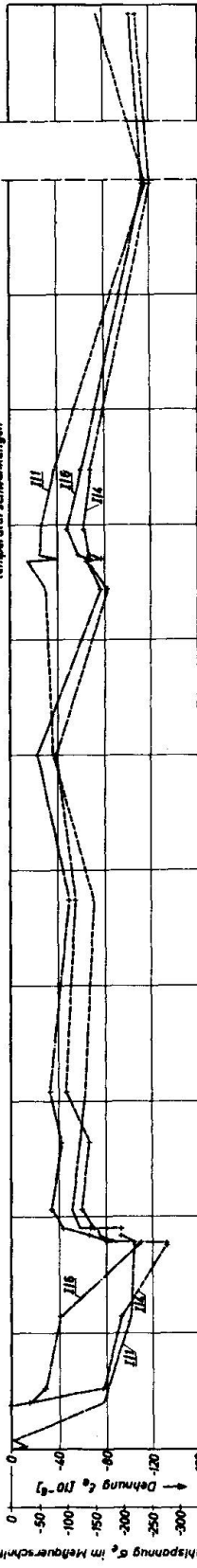
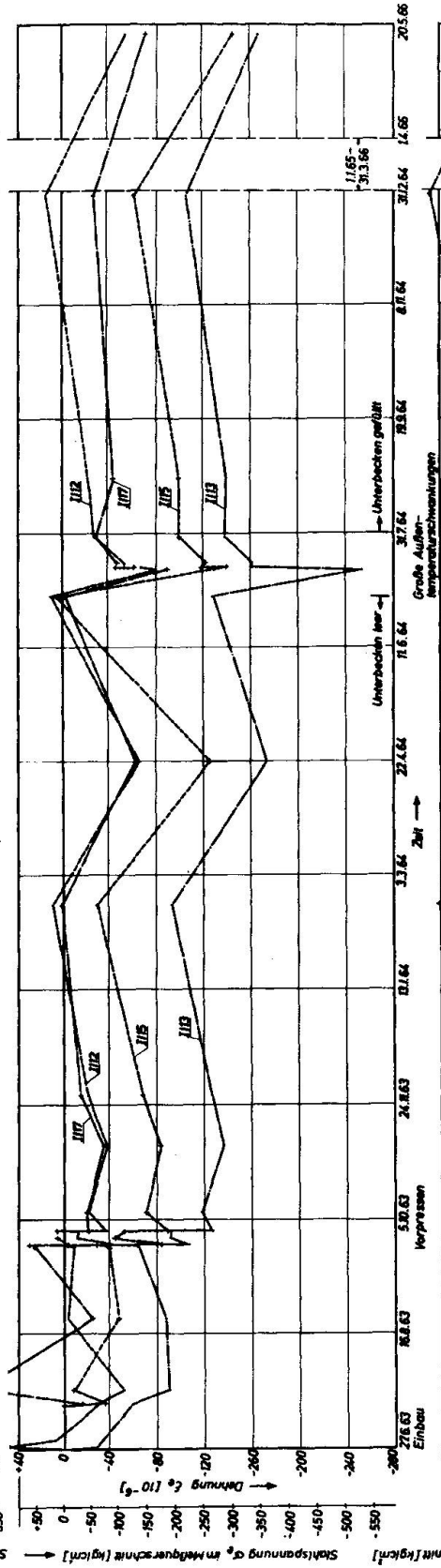


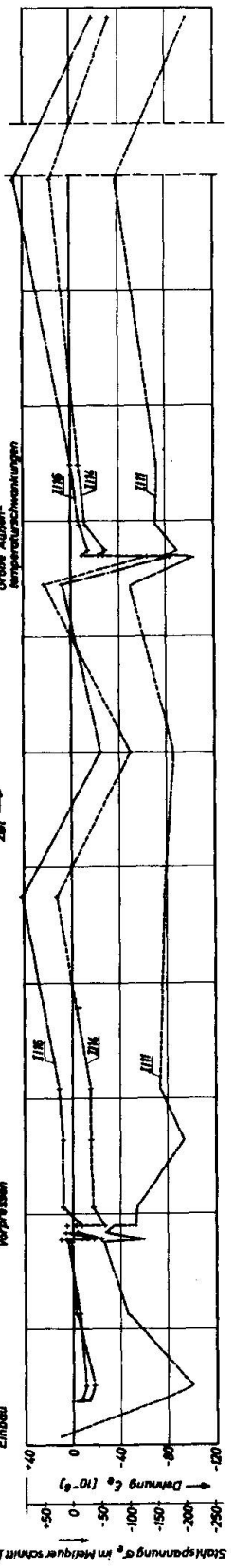
Bild 3 b

Zeitlicher Verlauf der
Stahldehnungen und
Stahlspannungen
Block I (Unterwasserseite)

a) H=92,20 m ü. NN



b) H=93,55 m ü. NN

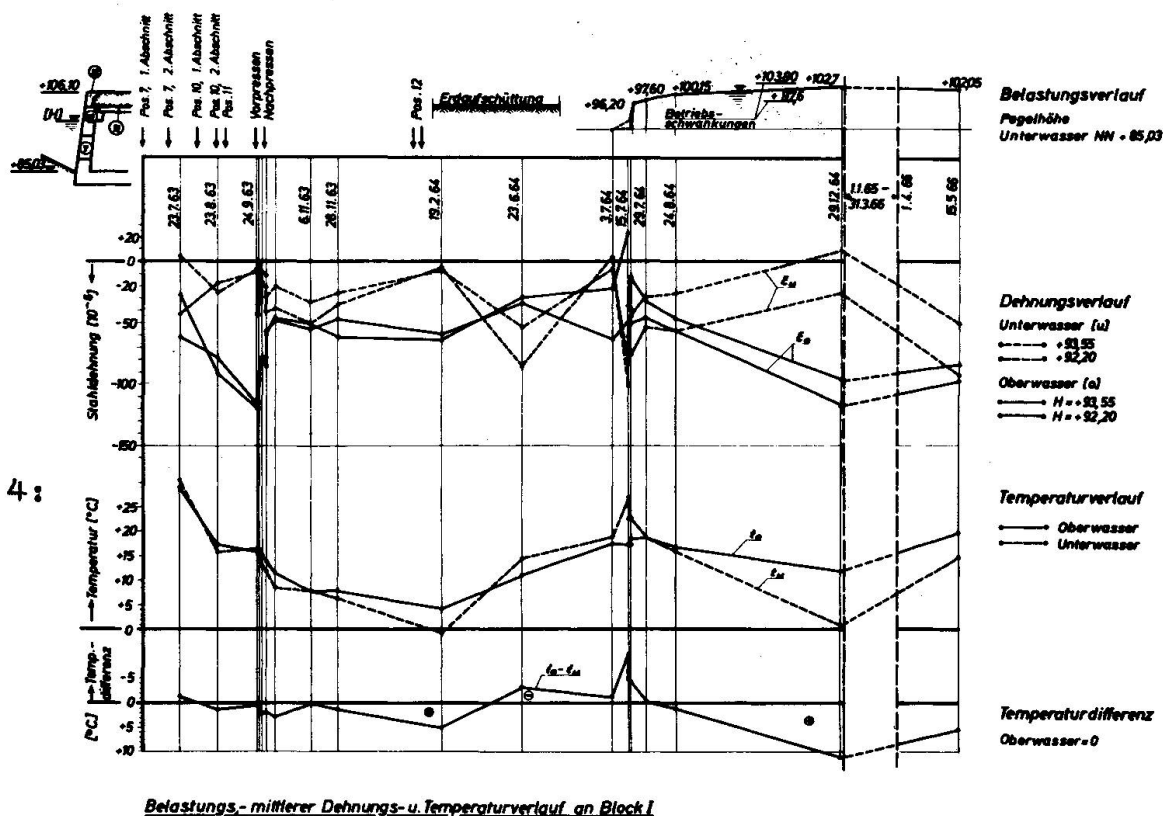


weise erhebliche Unterschiede, die eine Auswertung der Meßergebnisse im einzelnen kaum zuließen. (s. Bild 3 a und 3 b)

Um die hauptsächlichen Tendenzen des Spannungszustandes aber trotzdem erkennen zu können, wurden die Dehnungen und Temperaturen der auf gleicher Höhe liegenden Meßgeber, getrennt nach Ober- und Unterwasserseite, jeweils für einen Bauwerksblock gemittelt. Ein solcher gemittelter Verlauf der Dehnungen und Temperaturen wurde in Abhängigkeit von der Zeit für einen Bauwerksblock in Bild 4 dargestellt. Es sollte versucht werden, aus diesem Dehnungsverlauf den Einfluß bestimmter Bau- oder Belastungszustände auf den Gesamtdehnungsverlauf zu eliminieren. Diese Auswertung wurde erschwert, weil die einflußnehmenden Faktoren wie Eigengewicht, Vorpresse, Schwinden, Kriechen und Temperaturänderungen und auch der Wasserdruck größtenteils gleichzeitig auftraten. Die daraus herrührenden Dehnungen sind somit überlagert. Die quantitative Abschätzung des Einflusses der einzelnen Beanspruchungsarten wurde aber durch zeitlich begrenzte Messungen bei definierten Beanspruchungszuständen, so z.B. bei leerem Becken und größeren Temperaturunterschieden oder bei konstanter Temperatur und Betriebsschwankungen des Wasserspiegels trotzdem ermöglicht.

Die Beanspruchung der Stauwand wurde als Dehnung der Bewehrung gemessen. Da nach den Meßergebnissen die Stauwand in ungerissenem Zustand (Zustand I) verblieb, kann die Stahldehnung auch gleich der Betondehnung gesetzt werden. Für überschlägliche Spannungsermittlungen wurde der E-Modul des Stauwandbetons an gesondert hergestellten Probekörpern im Mittel zu $E_b = 2,1 \times 10^5 \text{ kp/cm}^2$ ermittelt, der E-Modul des Betonstahls kann zu $E_s = 2,1 \times 10^6 \text{ kp/cm}^2$ angenommen werden.

Bild 4:



Der geringe für diesen Beitrag zur Verfügung stehende Platz reicht nicht aus, um auf alle interessanten Einzelheiten der Meßergebnisse einzugehen. Im Rahmen des Themas dieses Symposiums soll daher hier nur der Größenanteil der Beanspruchung der Stauwand infolge Temperaturänderung den anderen maßgebenden Beanspruchungen wie Eigengewicht, Vorpresen, Wasserdruckbelastung insgesamt und Einfluß der Spiegelschwankungen während des Betriebes gegenüber gestellt werden. Vorweg sei noch gesagt, daß der Einfluß des Schwindens und Kriechens nicht exakt eliminiert werden konnte.

3.2. Dehnungen infolge Eigengewicht und Vorpresen der Stauwand.

Als erster definierter Belastungszustand kann der Zustand der Stauwand vor dem Vorpresen angesehen werden. Obwohl der Zustand bei den einzelnen Blocks zu verschiedenen Zeitpunkten und verschiedenen Außentemperaturen (aber Innentemperatur = Außentemperatur) auftrat, so sind doch die Dehnungen zu diesem Belastungszeitpunkt erstaunlich gleich. (s. Tabelle 1) Der Belastungszustand sei hier kurz "Eigengewicht" genannt, obwohl Schwindeinflüsse usw. auch darin enthalten sein können.

Tabelle 1: Zusammenstellung der Dehnungen vor und nach dem Vorpresen

Belastungszustand	Stahl-Dehnungen (10^{-6})			
	NN + 93,55 m		NN + 92,20 m	
	Oberwasser	Unterwass.	Oberwass.	Unterwass.
Vor dem Vorpresen				
Block I	- 115	0	- 119	0
" II	- 108	- 7	- 138	+ 6
" III	- 77	+ 8	- 107	+ 16
" IV	- 85	+15	- 145	+ 37
i. Mittel	- 92	+ 4	- 127	+ 15
Nach dem Vorpresen				
im Mittel	- 50	-30	- 77	- 41

Vor dem Vorpresen waren demnach auf der Unterwasserseite geringe Zugspannungen in der Bewehrung, während auf der Oberwasserseite Druckspannungen herrschten. Die beim Vorpresen der Wand gemessenen mittleren Dehnungen sind in Abhängigkeit von der Vorpreß-

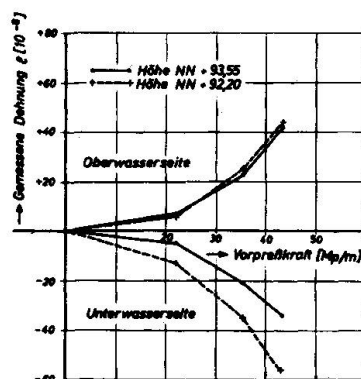


Bild 5:

Dehnungen in Abhängigkeit von der Vorpreßkraft (bezogen auf den Beginn des Vorpresens)

kraft in Bild 5 aufgetragen. Die Dehnungen wurden dabei auf den Zeitpunkt vor dem Vorpresen (Vorpreskraft = 0) bezogen. Der nichtlineare Zusammenhang zwischen Dehnung und Vorpreskraft muß auf Reibungsverluste der Vorpreskraft zurückgeführt werden. Die Vorpresung ergab auf der Unterwasserseite eine Stauchung (Druckspannung), auf der Oberwasserseite eine Dehnung. Damit wurde – wie geplant – auf der Unterwasserseite die Zugspannung aus Eigengewicht usw. überdrückt (s. Tabelle 1)

Durch das Vorpresen der Wand wird ein Biegespannungszustand erzeugt mit Dehnungen von etwa $+40 \times 10^{-6}$ an der Meßstelle NN + 93,55 m bzw. etwa $+60 \times 10^{-6}$ an der Meßstelle NN + 92,20 m; d.h. der Gesamtdehnungsunterschied allein in Folge des Vorpresens ist $\Delta \epsilon = 76 \times 10^{-6}$ bei Meßstelle 93,55 m und 138×10^{-6} bei Meßstelle 92,20 m. Das Vorpresen ergab demnach Dehnungsunterschiede in gleicher Größe wie die sonstigen Dehnungen aus "Eigengewicht usw."

3.3. Dehnungen infolge von Temperaturunterschieden.

Bei Betrachtung der Mittelwertkurven für die Dehnungen fallen erhebliche Veränderungen der Dehnungen auf, obwohl sich in bestimmten Zeiträumen die äußere Belastung nicht geändert hat und ein Einfluß von Schwinden und Kriechen wegen der Kürze der Zeitintervalle (z.B. 15. und 16.7.1964) nicht in Betracht kommt. Bei der weiteren Auswertung konnten diese Dehnungsänderungen auf Temperatureinwirkung zurückgeführt werden.

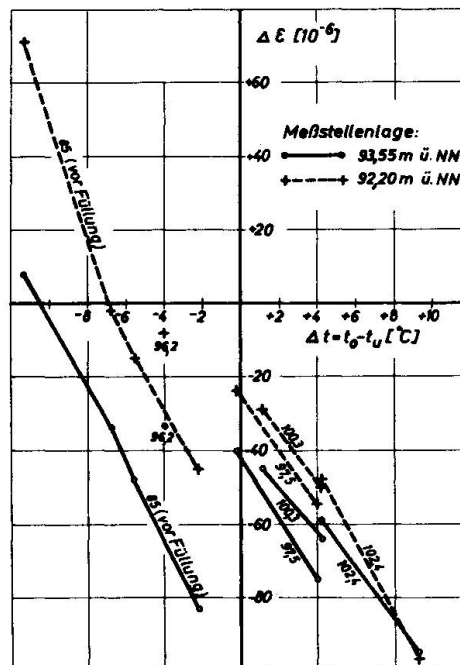


Bild 6:

Dehnungsdifferenz zwischen Ober- und Unterwasserseite in Abhängigkeit von der Temperaturdifferenz

(Parameter: Füllhöhe des Unterbeckens)

Die absoluten Temperaturen zeigen dabei keinen signifikanten Einfluß auf die Dehnungsänderung, wohl aber der Betrag des Temperaturunterschiedes $\Delta t = t_o - t_u$ zwischen Ober- und Unterwasserseite. In Bild 6 sind die Dehnungsdifferenzen zwischen Ober- und Unterwasserseite ($\Delta \varepsilon = \varepsilon_o - \varepsilon_u$) für verschiedene Meßzeitpunkte zwischen 1964 und 1966 in Abhängigkeit vom Temperaturunterschied aufgetragen worden. Als Parameter geht in die Darstellung die Füllhöhe des Unterbeckens ein, da die Messungen sowohl bei leerem als auch bei unterschiedlich gefülltem Unterbecken durchgeführt wurden. Die Verbindung der Punkte mit gleichem Parameter (Füllhöhe) ergibt für jede Meßhöhe eine Schar von Geraden mit gleicher Steigung; die Geraden für die zwei Meßhöhen unterscheiden sich lediglich durch einen konstanten additiven Wert, der den Ausgangszustand der Wand ohne "Temperaturbeanspruchung" darstellt.

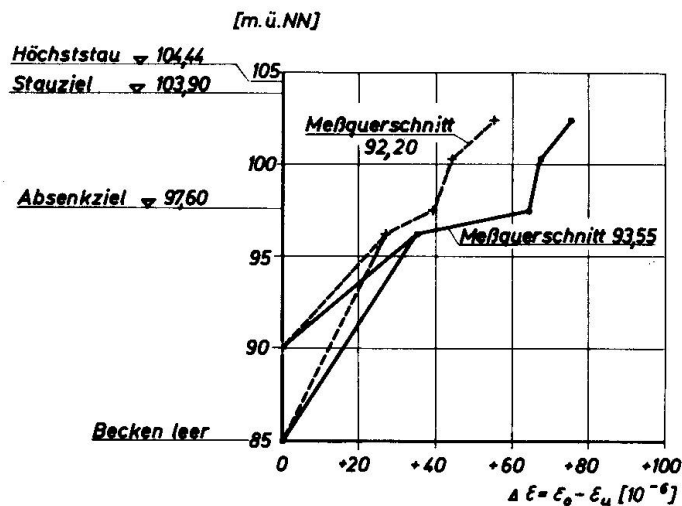
Bemerkenswert ist die Größenordnung der Dehnungsunterschiede. Zwischen einer Messung im Sommer, bei der die Temperatur unter Voraussetzung eines gefüllten Beckens an Ober- und Unterwasserseite etwa gleich ist ($\Delta t = 0$) und einer Messung im Winter ($\Delta t = 10^\circ$) kann ein Dehnungsunterschied von $\Delta \varepsilon = 80 \times 10^{-6}$ auftreten. Dieser Dehnungsunterschied ist genauso groß wie der durch das Vorpressen der Wand erreichte.

Die mittlere Steigung der Geraden in Bild 6 beträgt etwa $\alpha = 10 \times 10^{-6} / \text{grd } ^\circ\text{C}$ Temperaturunterschied, sie ist von gleicher Größe wie die Wärmeausdehnungszahl von Beton ($\alpha_t = 10 \times 10^{-6} / \text{grd}$) der aus Temperaturunterschieden zu erwartende Beton- bzw. Stahlspannungsunterschied der Stauwand kann demnach auf Grund der Meßergebnisse genügend genau nach dem Ansatz $\Delta \sigma = \alpha_t \times E \times \Delta t$ vorausberechnet werden. (Es sei hier ausdrücklich hervorgehoben, daß es sich hier um spannungserzeugende Dehnungsunterschiede handelt, da die spannungslose Temperaturverformung durch die Konstruktion der Meßgeber eliminiert wurden).

3.4. Dehnung infolge der Stauhöhe des Unterbeckens.

Als eine wichtige Einflußgröße auf die Beanspruchung der Stauwand muß die Stauhöhe des Wassers im Unterbecken angesehen werden. Der Einfluß der gesamten Stauhöhe, d.h. von der Füllung des Vorbodens des Beckens an, läßt sich aus den Meßwerten Bild 4 nicht direkt auswerten, da in ihnen gleichzeitig der Parameter "Temperaturunterschied" (vgl. Abschn. 3.3) enthalten ist. Aus Bild 6 kann jedoch der Einfluß der Stauhöhe ermittelt werden, wenn die Geradenscharen auf dem Wert $\Delta t = 0$ extrapoliert werden. Die auf der Ordinate für $\Delta t = 0$ abgelesenen Werte stellen die von der Temperaturdifferenz unabhängigen Dehnungsunterschiede (einschl. etwaiger Schwind- und Kriechdehnungen) dar; sie sind in Bild 7 dargestellt. Als Bezugszeitpunkt wurde der Zeitpunkt der Füllung des Vorbodens gewählt. *)

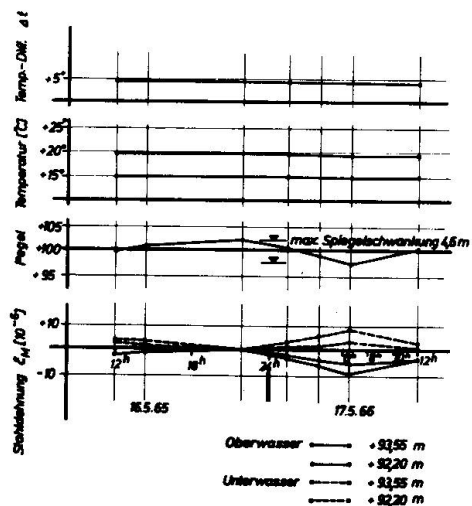
*) Infolge eines ungenauen Protokolls konnte die genaue Füllhöhe des Vorbodens nicht ermittelt werden, sie dürfte aber zwischen NN + 85 m (Vorboden leer) und NN + 90 m gelegen haben.



Differenz der Dehnungen auf Ober- und Unterwasserseite in Abhängigkeit von der Stauhöhe bezogen auf die Messungen am 15./16.7.1964 (Beginn des Füllens)

(auf eine Temperaturdifferenz $\Delta t = 0$ zwischen Ober- und Unterwasser extrapoliert)

Bild 7:



Dehnungsänderung in Abhängigkeit von der Wasserspiegelschwankung innerhalb von 24 Std.

Bild 8:

Der maximale Dehnungsunterschied von 60 bzw. 80×10^{-6} infolge der Füllung des Staubeckens ist nach dieser Auswertung gleich oder kleiner als der o.a. Dehnungsunterschied infolge Temperaturunterschied von 10°C .

Die in Bild 7 dargestellten Dehnungsunterschiede treten nur bei dem erstmaligen und einmaligen Füllen des Unterbeckens von NN + 85 m auf etwa NN + 104 m auf. Die täglichen Stauhöhen-schwankungen infolge des Betriebs des Pumpspeicherwerks betragen dagegen nur maximal etwa 6 m. Der Einfluß dieser Stauhöhenänderung (Betriebsschwankung) wurde bei einer durchlaufenden Messung über 24 Stunden festgestellt, deren Ergebnis in Bild 8 dargestellt ist.

Da der Temperaturunterschied während der Messung konstant blieb, können die Messungen direkt der Wasserspiegelschwankung zugeordnet werden. Bei einer vorhandenen Spiegelschwankung von 4,6 m ergaben sich maximale Änderungen des Dehnungsunterschiedes zwischen Ober- und Unterwasser von $\Delta\varepsilon = 18 \times 10^{-6}$ bzw. 9×10^{-6} , je nach Meßhöhe. Bei maximal geplanter Spiegelschwankung von 6,3 m sind dann Dehnungsunterschiede von $\Delta\varepsilon = 25 \times 10^{-6}$ bzw. 12×10^{-6} zu erwarten. Gegenüber den infolge des Temperaturunterschiedes entstandenen Dehnungsunterschieden sind die infolge des Betriebs des Pumpspeicherwerks entstehenden Dehnungsunterschiede demnach um mehr als $2/3$ kleiner.

ZUSAMMENFASSUNG

Im vorstehenden wurde versucht, aus an einer Stauwand über einen längeren Zeitraum durchgeführten Dehnungsmessungen die auf den Dehnungszustand einflussnehmenden einzelnen Beanspruchungszustände ihrer Grösse nach zu ermitteln. In Bezug auf die Themenstellung dieses Symposiums ergab sich, dass der Temperaturunterschied zwischen Ober- und Unterwasserseite der Stauwand gleiche oder grössere Dehnungsunterschiede (Spannungsunterschiede) hervorrief als die Beanspruchungszustände Eigengewicht, Vorpressen der Stauwand und Wasserdruck.

SUMMARY

Strain and temperature measurements were carried out over a considerable period of time on a dam wall. The results were used to determine the relative importance of the various factors affecting the state of strain.

Temperature differences between the upstream and downstream sides of the wall were found to produce differences in strain and stress which were as great as, and sometimes greater than those produced by hydrostatic pressure and the thrust forces.

RESUME

Dans ce qui précède, on a cherché l'ordre de grandeur de chaque cas de sollicitation, influant sur l'état de dilatation, à partir de mesures de dilatation faites sur un mur de retenue pendant une assez longue période.

En ce qui concerne le thème de ce symposium, on a trouvé que la différence de température entre les côtés amont et aval du mur suscitait des différences de dilatation (de contraintes) égales ou supérieures à celles dues au poids propre, à la précontrainte du mur et à la pression de l'eau.

I

Measurements of Creep, Shrinkage and Temperature Changes in Prestressed Concrete Bridges

Mesures du fluage, du retrait et de la température dans des ponts en béton précontraint

Kriech-, Schwind- und Temperaturmessungen an Spannbetonbrücken

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Observations made over a period of several years at the Research Institute of Civil Engineering in Bratislava /Czechoslovakia/ by akustical resonance string strain gauges, and geodetical methods, in 10 prestressed concrete bridges. The strain gauges were concreted directly into the structures. The following types of highway and railway bridges were measured:

- bridges with simple and continuous precast girders,
- with monolytical (cast in place) concreted framework structures,
- monolytical concreted cantilever bridges,
- cantilever bridges or precast elements and
- footbridge of prestressed lightweight concrete girders.

The process of change of deformations stabilized in the bridges constructed with good quality of concrete (Youngs Modulus $E = 350\,000 \text{ kp/cm}^2$) after three years. The total strain (creep and shrinkage) measured after 10 years was $- 200 \times 10^{-6}$ in a monolytical concreted framework prestressed concrete structure (Highway bridge "K" - Kotešová, span 63.40 m , Fig.No.1). The maximum deflection after 10 years was 5.57 cm at the center of the span. 4.87 cm of the above was found after the first three months the bridge was in use. After 3 years the deformations oscilated only due to temperature changes. The strains at the centre of the span are shown in Fig.No.2.

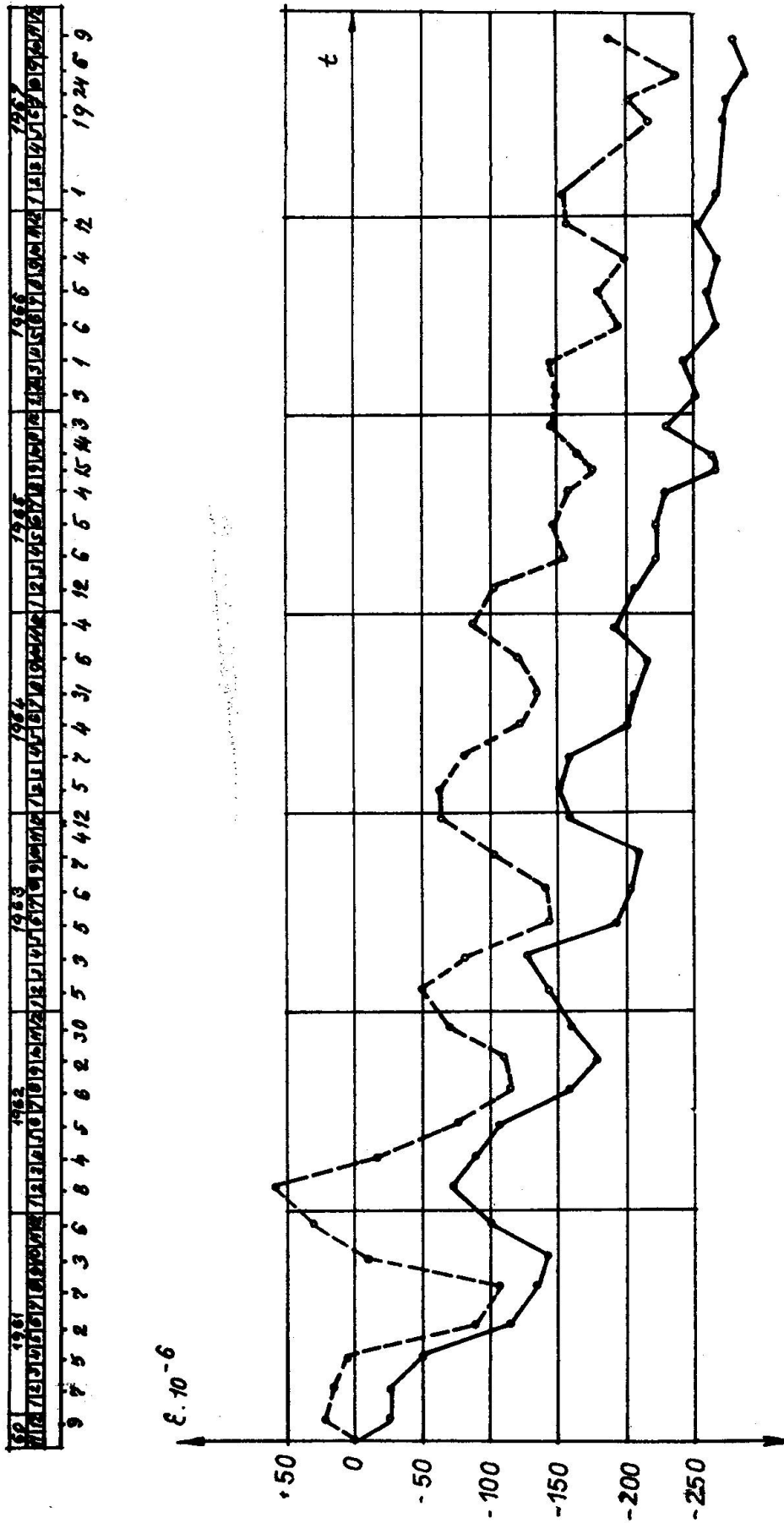


Fig. No. 2.: The strains in a monolithic concreted framework prestressed concrete highway bridge "K".



Fig.No.1.:The monolytical concreted framework prestressed concrete highway bridge " K " - Kotešová.

In the second framework structure (Highway bridge " B " - Bytča with the same span of 63.40 m) but of worse quality of concrete (Youngs Modulus $E = 250\,000 \text{ kp/cm}^2$) the maximum deflection was 11.77 cm after three years and is not stabilized. 8.5 cm of the 11.77 cm was found after the first 6 months of traffic on the bridge. The influence of temperature change between summer and winter on the deflection oscilations was $\pm 1 \text{ cm}$. However it is possible that these large deformations were partially caused by foundation movements and therefore recomandations were made to connect the bottom of the foundations with prestressed concrete bars in this type of structure.

The cantilever bridges with prestressed precast concrete elements had already larger deformations during prestressing in comparison with the monolytical concreted cantilever bridges. However one has to note, that the desired prestress in the highway bridge " S " (Sirník, span 60.00 m, Fig.No.3.) was -147.3 kp/cm^2 (the measured prestress -130.7 kp/cm^2) and in the monolytical bridge " N " (Nové Mesto, span 70.00 m) was -83.0 kp/cm^2 (the measured -84.9 kp/cm^2), which is smaller as in the bridge with precast elements " S " . From the measured results of the monolytical bridge " N " constructed with the inferior quality of concrete it

was observed that maximum deflection at the center of the span was 31.18 cm,

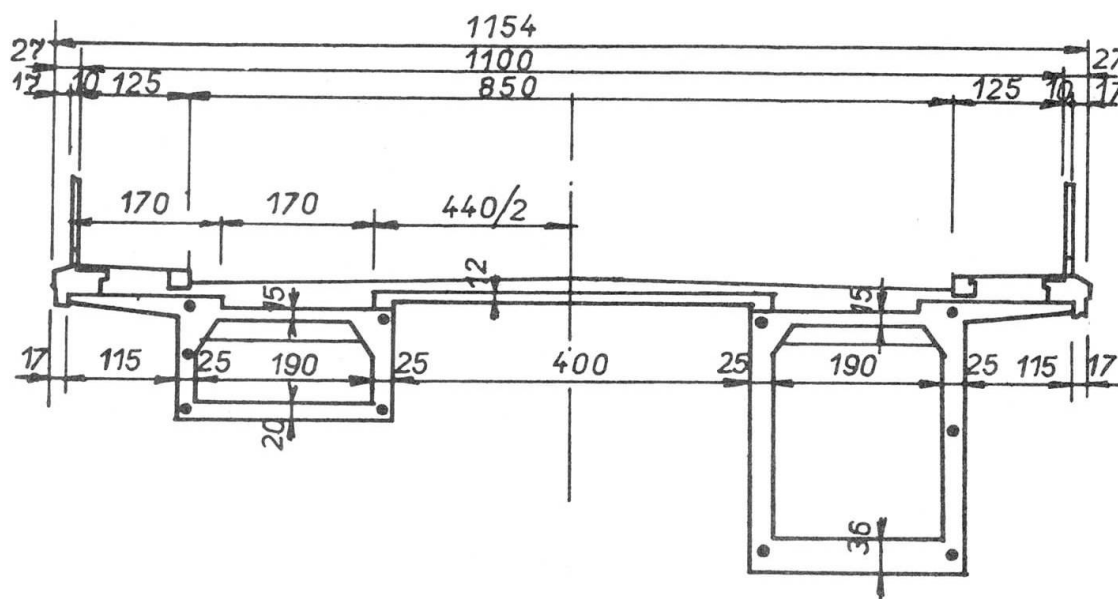


Fig.No.3.:The cross section of the cantilever highway bridge with prestressed precast concrete elements.

but the increase of the deflection in the last 3.5 years was only 6.95cm. In comparison, the monolytical concreted bridge "KO" (Kolárovo, span 61.61 m, Fig.No.4.) made with the better quality of concrete, the maximum deflection



Fig.No.4.:The monolytical concreted cantilever prestressed bridge "KO".

in 4 years was only 4.5 cm, and the strains, measured using akustical strain gauges, was 25 to 30 % smaller as in bridge "N". In the bridge "KO" the advantage of the metal joint was observed comparing it with the reinforced concrete joint in the center of the span in bridge "N". The measured stresses during prestressing and after the first year after prestressing is shown in Fig.No.5.

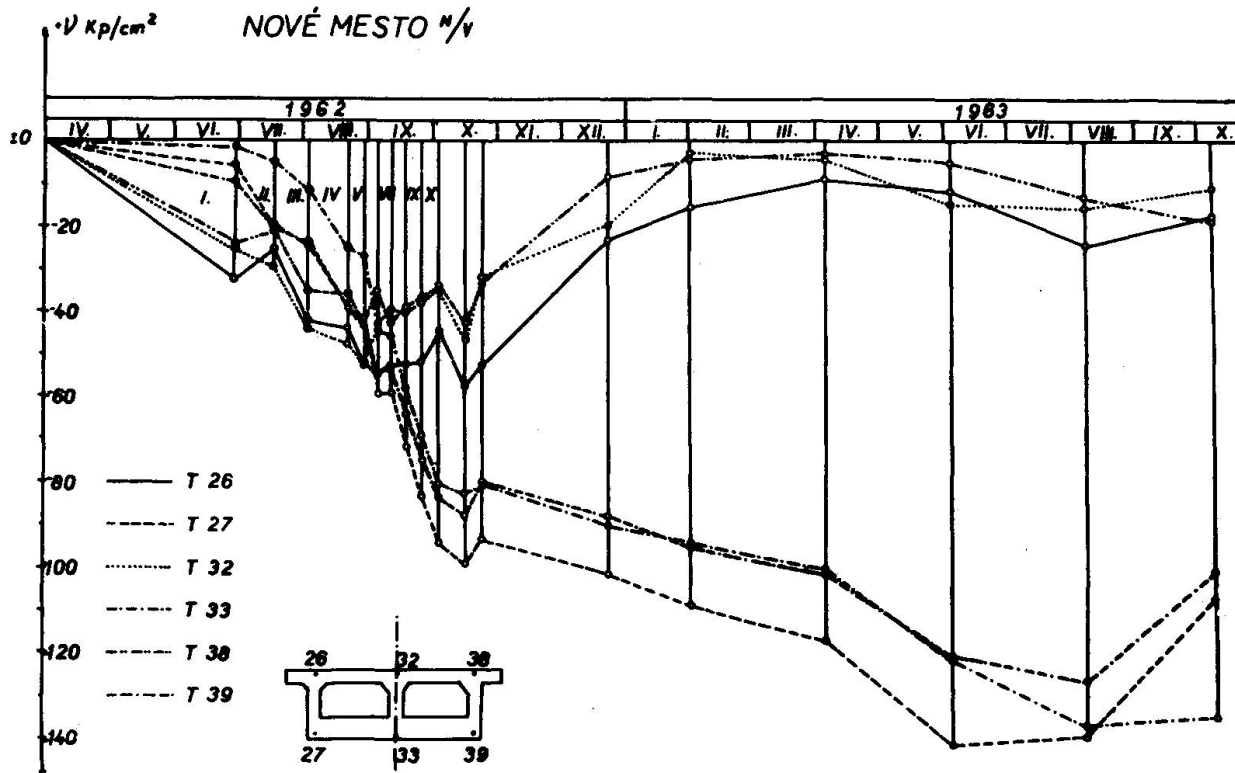


Fig.No.5.: The stresses in the cross section at the fixed end of the monolytical concreted cantilever prestressed concrete highway bridge " N " during prestressing and during the first year after prestressing.

An other interesting observation was that in the cantilever highway bridge " S " with precast elements, the strains after three years were 100 % higher (Fig.No.6.) as in the cantilever monolytical concreted bridge " KO ". The deformations in bridge " S " also stabilized in 3 years, since the quality of the concrete is not bad. The deflection increase was 45 mm in 1966, 13 mm in 1967 and 8 mm in 1968. In the cantilever bridge " S " the maximum strain was $- 850 \times 10^{-6}$ after 3 years, in the cantilever railway bridge " M " (Margecany, same span, Fig.No.7.) the maximum strain was $- 800 \times 10^{-6}$ after 2.5 years (50 % of the later is shrinkage).

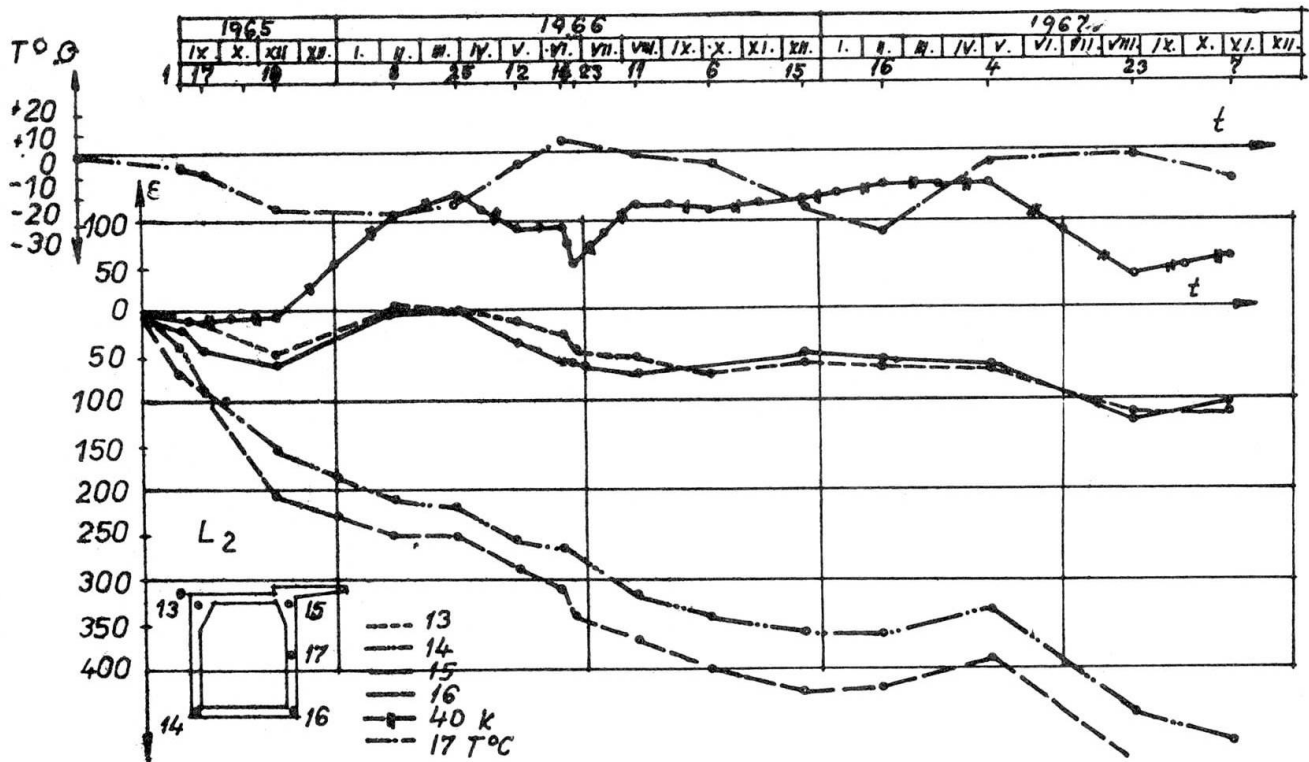


Fig.No.6.: The strains $\epsilon \times 10^{-6}$ in the cross section at the fixed end (first precast element) of the cantilever highway bridge " S " with prestressed concrete precast elements during the first 2 years.

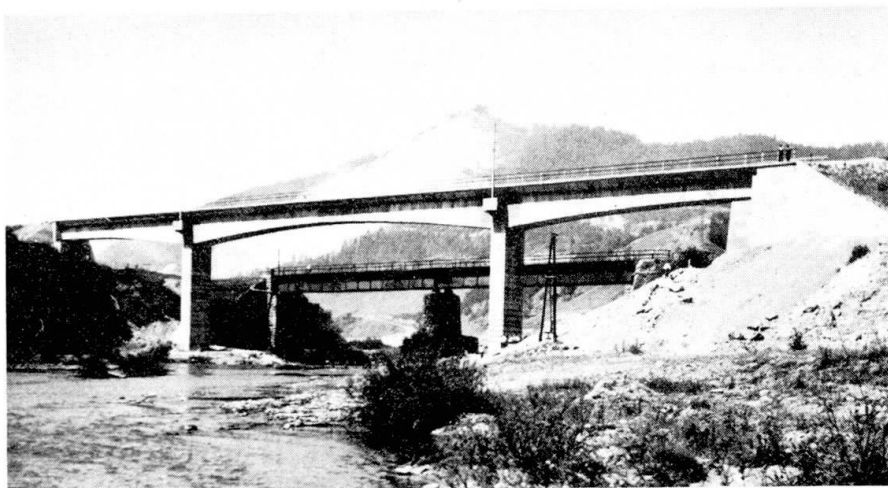


Fig.No.7.:The cantilever prestressed concrete railway bridge " M " with precast elements.

The process of deformation change in prestressed concrete bridges with simple and continuous girders had also stabilized after 3 years. In the railway bridge "KR" (Kramáre, span 24.00 m) the maximum strain was -180×10^{-6} after 4 years. In the footbridge " V " (Velim, span 11.20 m) after 3 years the deformation oscillation due to the deformation changes were $\pm 50 \times 10^{-6}$. In the highway bridge " NZ " (Nové Zámky , span 22.00 + 28.00 + 22.00 m), Fig.No.8., with continuous girders after 10 years the deformation oscillation due to the deformation changes were $\pm 10 \times 10^{-6}$.



Fig.No.8.:The highway bridge " NZ " with continuous precast prestressed concrete girders during construction.

According to our measurements we may conclude that between our results and the design values from the chechoslovakian code No.732 004 , p.108 (the code formulas based on the classical Dischinger theory of creep and shrinkage) the difference is small in the cantilever concrete bridges even where the deformations are relatively large. The creep coefficient for the highway bridge " S " is 2.07 from the code (from the measurements 1.78) and for railway bridge " M " is 2.06 (from measurements 2.07). The safety factor for the other bridges is much larger.

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SUMMARY

The paper gives an analysis of the measurements of creep, shrinkage and temperature changes in various 10 prestressed concrete highway and railway bridges. Observations were made over a period of 3 to 10 years at the Research Institute of Civil Engineering in Bratislava by acoustical resonance string gauges and geodetical methods. It is concluded that between the results of the observations and the design values by the classical Dischinger theory of creep and shrinkage the difference is small in the cantilever prestressed concrete bridges and the safety factor for the other bridges is much larger.

RESUME

Cet article décrit les mesures de l'influence du fluage, du retrait et de la température sur 10 ponts-routes ou ponts-rails. Ces observations ont été exécutées au moyen d'extensomètres acoustiques et par des méthodes géodésiques pendant des durées de 3 à 10 ans par l'Institut de Recherches du Génie Civil à Bratislava. On peut en déduire que la différence, entre les résultats des mesures et la théorie de Dischinger pour le fluage et le retrait, est petite dans le cas des ponts en béton précontraint construits en encorbellement et que le facteur de sécurité des autres ponts est encore plus élevé.

ZUSAMMENFASSUNG

In diesem Aufsatz werden Kriech-, Schwind- und Temperaturmessungen an 10 verschiedenen Strassen- und Eisenbahnbrücken aus Spannbeton gegeben. Die Beobachtungen wurden während einer Dauer von 3 bis 10 Jahren am Forschungsinstitut für Ingenieurbauten in Bratislava (Pressburg) mittels Saitendehnungsmesser und geodätischer Methoden durchgeführt. Es zeigte sich, dass die Differenz aus Messergebnissen und den klassischen Dischinger-Werten der Kriech- und Schwindtheorie für Freivorbaubrücken sehr gering und dass der Sicherheitswert der anderen Brücken noch grösser ist.

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**Mesures de dilatation et de poussée en clé sur un grand ouvrage en béton armé
(Pont de la RN 7 sur le canal de fuite de Donzère-Mondragon)**

Dilatations- und Scheiteldruckmessungen an einem großen Stahlbeton-Bauwerk
(Brücke der RN 7 über den Kanal von Donzère-Mondragon)

Dilatation and Force at the Top Joint on a Big Prestressed Concrete Structure
(Bridge of the RN 7 over the Channel of Donzère-Mondragon)

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I - INTRODUCTION

1-1 - Description de l'ouvrage

Le pont par lequel la route nationale 7 franchit le canal de fuite de l'aménagement de Donzère-Mondragon a été construit en 1951. C'est un grand pont en béton armé de 260,50 m. de portée comportant trois travées de 80,25m, 100m et 80,25m. Il a été réalisé pour le compte de la Compagnie Nationale du Rhône. Le projet a été conçu par M. le Président CAQUOT.

Ce pont est formé de deux poutres tubulaires parallèles de hauteur variable, supportant la dalle de chaussée. Les appuis de chaque poutre sont constitués par des rouleaux en béton frettés, formant au droit des piles, béquilles virtuelles. L'ouvrage présente un biais important (45°). Les deux poutres sont cependant droites, le biais est réalisé par un décalage longitudinal



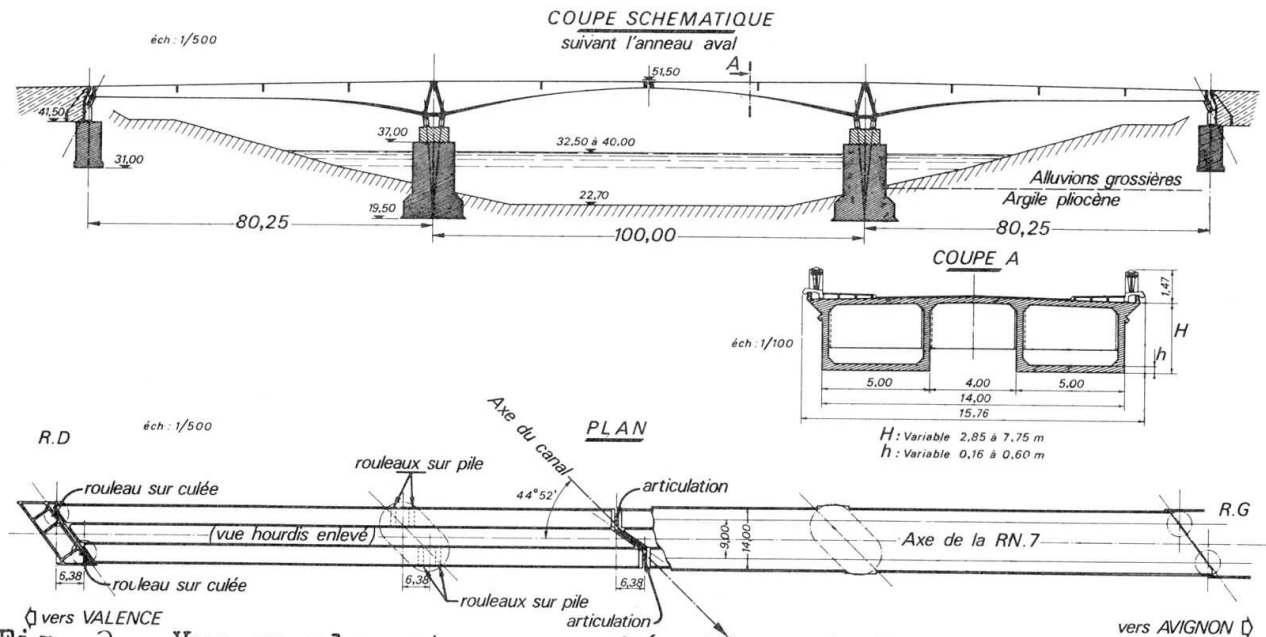


Fig. 2 - Vue en plan et coupe schématique du Pont.

des appuis de 6,38 m. La liaison entre les poutres est réduite au minimum par des entretoises de faible hauteur, à l'exception des entretoises sur appuis qui sont très rigides. Chaque poutre fonctionne sensiblement comme un pont droit.

Elles sont toutes deux coupées, en clé, par une articulation en béton fretté mise en charge au décintrement de façon à rester toujours en compression.

Les fondations de l'ouvrage sont des caissons foncés à l'air comprimé dans l'argile pliocène en place.

1-2- Campagne de mesures effectuées sur l'ouvrage

En 1955, après une période de froid intense, l'examen des articulations centrales permit de constater qu'à chacune des deux articulations l'un des plots était décollé de l'entretoise d'about de 2 à 3 mm. On concluait alors à un déclavage résultant de la disparition au moins momentanée de la poussée de clavage initiale.

Les études faites à la suite de ces premières constatations aboutirent à un projet de reclavage provisoire destiné d'une part, à rendre au pont ses conditions de fonctionnement normal en introduisant en clé une poussée égale à la poussée initiale et d'autre part, à permettre l'exécution d'une série de mesures à la suite desquelles un projet de reclavage définitif pourrait être établi.

Après mise en place des articulations provisoires formées de deux vérins, la Compagnie Nationale du Rhône a procédé à des mesures périodiques portant sur la poussée, la température et les déplacements.

Ces observations faites sur un peu plus de deux années, (de Novembre 1964 à Septembre 1966) c'est-à-dire sur deux cycles de saisons, ont comporté des enregistrements continus (enregistrements des températures,

variations de l'ouverture des joints de culée) et des mesures ponctuelles (mesures de poussée par exemple, dont la fréquence était en moyenne d'une par semaine). Elles ont fourni une masse assez importante d'informations sur le comportement de cet ouvrage.

De telles mesures, sur de telles durées, sont assez inhabituelles. C'est pourquoi il nous a paru intéressant de les relier ici.

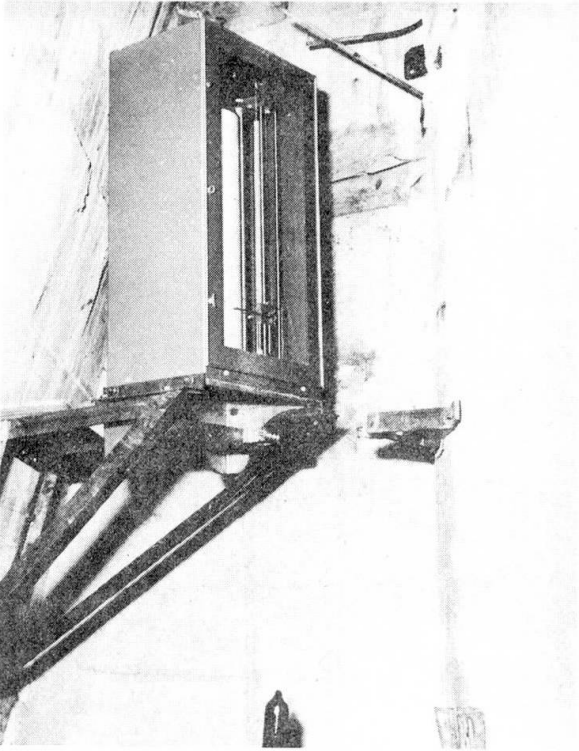


Fig. 3 - Appareils d'enregistrement des dilata-tions

2 - DILATATION DU PONT

Les observations relatives à la dilatation du pont ont comporté des mesures de température et d'allongement.

2-1- Mesure des températures

Des enregistrements continus de la température ont été effectués en trois points :

Le premier point de lecture donnait la température dite "extérieure" (te). L'enregistreur correspondant étant placé à l'intérieur d'un caniveau de trottoir du pont, au droit du joint de culée rive droite, donc à l'abri du soleil mais bien ventilé puisque le caniveau est coupé à cet endroit par le joint de culée.

Les deux autres donnaient la température dite "intérieure" (ti) des poutres à l'aide de deux enregistreurs disposés l'un dans la poutre amont, l'autre dans la poutre aval au droit de la pile rive droite, donc assez loin de la culée (80m). A noter que la seule ouverture établissant une communication entre l'intérieur des poutres et l'extérieur est une trappe de 0,50 m de côté environ, située à proximité immédiate de la culée. On peut, semble-t-il, admettre que cette ouverture ne joue qu'un rôle négligeable dans la transmission de la chaleur entre l'extérieur et l'intérieur des poutres.

2-2- Mesure des allongements

Les allongements du pont ont été enregistrés de façon continue à l'aide de quatre appareils disposés au droit des joints de culée rive droite et rive gauche, et des poutres amont et aval. L'appareillage comportait un cylindre vertical portant les feuilles d'enregistrement hebdomadaire, l'équipage mobile

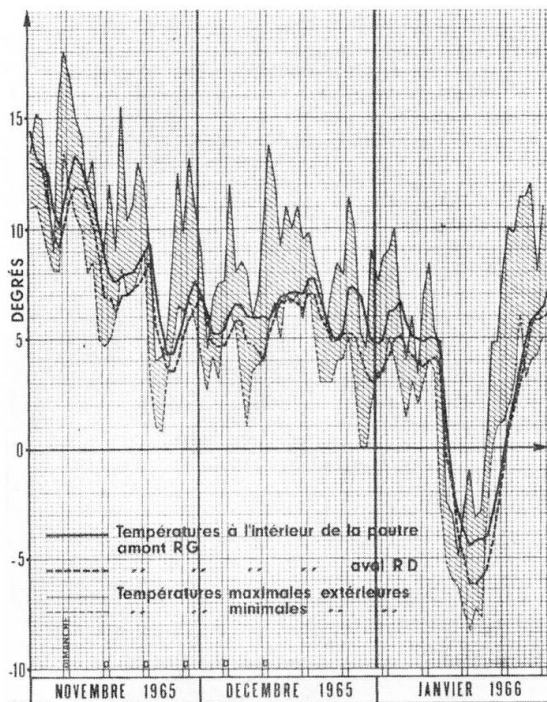


Fig. 4 - Variation des températures sur une longue durée

portant la plume était solidaire d'un fil d'acier reliant un des deux parements à un contrepoids ; l'appareil enregistreur était solidaire du parement opposé. Afin d'augmenter la sensibilité de l'ensemble, le fil d'acier était moufflé, ce qui nous a permis d'amplifier les mouvements dans le rapport de 1 à 5. Le pont étant biais, l'ensemble était disposé de manière à mesurer les déplacements réels parallèlement à l'axe de l'ouvrage.

Afin de contrôler le fonctionnement des enregistreurs, des mesures étaient effectuées au moins une fois par semaine à l'aide d'un pied à coulisses entre des repères fixes.

2-3- Résultats des mesures de température

Nous donnons ci-contre à titre d'exemple, un extrait de graphique semestriel des températures. Ce graphique comporte pour chaque jour quatre températures ; les températures à l'intérieur de l'anneau amont et de l'anneau aval et les maxima et minima extérieurs. On remarque que les températures intérieures du pont sont voisines l'une de l'autre et généralement assez

bien centrées sur l'intervalle minimum-maximum, avec toutefois, en cas de variation importante des extrêmes, un léger retard de quelques jours sur ceux-ci. D'une manière générale, il semble que la température à l'intérieur des caissons soit de l'ordre de la moyenne des extrêmes extérieurs.

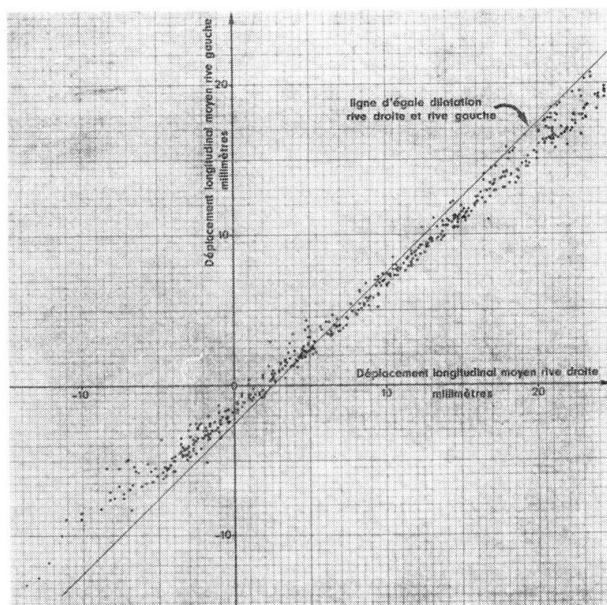


Fig. 5 - Comparaison des dilatactions rive droite et rive gauche.

2-4- Résultats des mesures d'allongement

Les variations des ouvertures des joints de culée ont été converties en variation de longueurs en faisant le total des variations de la rive gauche et de la rive droite, et cela pour chacun des anneaux amont et aval considérés séparément. Compte tenu de la précision des mesures, nous pouvons dire que les deux

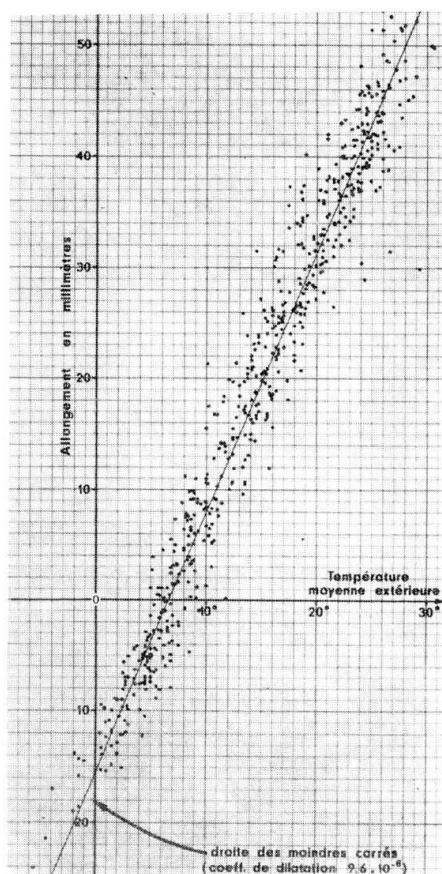


Fig.6 - Corrélation Allongements température

thermomètre placé dans le caniveau (voir paragraphe 2-1).

Pour les allongements, nous avons considéré la moyenne arithmétique des minima et maxima de chaque journée. On obtient ainsi un nuage de points donnant l'allongement du pont en fonction de la température. La droite des moindres carrés montre que la longueur augmente d'environ 2,5 mm par degré pour une longueur totale du pont de 260,50 m, cette variation donne un coefficient de dilatation de $9,6 \times 10^{-6}$, un peu faible en comparaison de la valeur admise couramment. Il faut noter à ce sujet que la dilatation du pont n'est pas entièrement libre, elle est freinée par les bielles de culée qui sont inclinées à 2 pour 1, ce qui entraîne une flexion des travées de rive et réduit le coefficient de dilatation apparent du pont. Cependant cette réduction est faible et n'intervient pas sur les deux premiers chiffres significatifs du coefficient de dilatation.

On peut noter sur ce graphique une assez forte dispersion. La température du pont ne peut se déduire de la longueur qu'à $\pm 3^\circ$ près.

Cette dispersion traduit la complexité de la distribution des températures à l'intérieur du pont sous l'influence des variations plus ou moins rapides et plus ou moins grandes de la température extérieure.

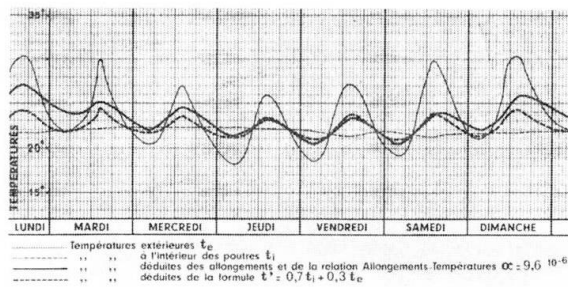
poutres amont et aval donnent lieu aux mêmes dilatations. Par contre, en comparant les variations d'ouverture des joints de culée rive droite et rive gauche, on constate que le joint de culée rive gauche joue moins que celui de rive droite. A une variation de longueur du pont de 10 mm correspond une variation de longueur de 4,6 mm en rive gauche et 5,4 mm en rive droite.

La symétrie du pont qui est géométriquement rigoureuse, n'est donc pas absolument conservée dans la dilatation. Il y a entre chaque rive, un écart de l'ordre de 10 % de la dilatation globale. Ce résultat peut être intéressant à connaître pour l'étude technologique des joints de chaussées.

2-5- Corrélation allongements-température, coefficient de dilatation

Un graphique de corrélation allongement-température a été établi à partir des données suivantes :

Pour les températures, nous avons porté en abscisse la température moyenne extérieure en faisant la moyenne arithmétique des maxima et minima de chaque journée d'après les enregistrements du



ig.7 - Enregistrement et calcul continu des températures du 4 au 11 Juillet 1966

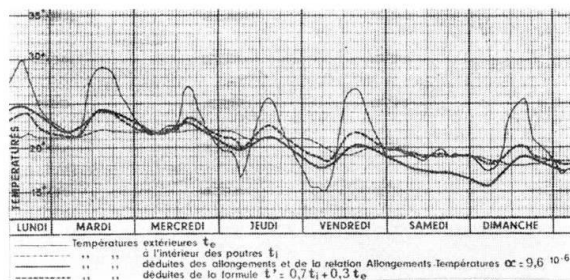


Fig.8 - Enregistrement et calcul continu des températures du 22 au 29 Août 1966

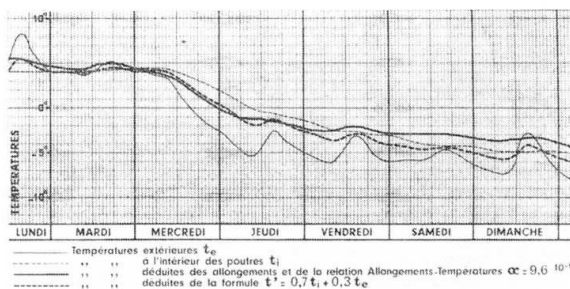


Fig.9 - Enregistrement et calcul continu des températures du 10 au 17 Janvier 1966

2-6- Température réelle du pont

C'est pourquoi nous avons cherché si l'on pouvait, à chaque instant, attribuer une température moyenne au pont et comment cette température pouvait se déduire de la température de l'air extérieur et intérieur. Les voiles de béton qui constituent le pont sont, sur une face, exposés à la température intérieure (t_i) et sur l'autre face, exposés dans les parties à l'ombre, à une température voisine de celle qui règne dans le caniveau (t_e). Dans les parties au soleil, la température extérieure peut être beaucoup plus élevée mais il a paru difficile d'en tenir compte. La solution la plus simple est d'admettre que le pont présente une température propre t qui est une fonction linéaire des températures de l'air ambiant.

La loi qui semble le mieux convenir est :

$$t = (0,70 t_e + 0,30 t_i) - 3h$$

Il y a en effet un déphasage assez net de trois heures entre la température du pont et la température de l'air.

La validité de cette loi apparaît sur les figures 7, 8 et 9, dont deux ont été établies en saison chaude et une en saison froide.

Nous avons reporté sur ces graphiques :

- les températures de l'air t_e et t_i (résultant d'un enregistrement continu) ;
- la température propre du pont t , déduite des précédentes selon la loi ci-dessus ;
- la température propre du pont (t') déduite de sa longueur à l'aide d'un coefficient de dilatation supposé de $9,6 \times 10^{-6}$.

On voit sur ces graphiques que les deux températures t et t' sont voisines, donc que la loi indiquée ci-dessus est assez bien vérifiée. Il subsiste cependant des écarts qui peuvent atteindre au maximum $2,5^\circ$. D'une manière plus précise le coefficient de corrélation entre t' et t est de l'ordre de 0,993 avec un écart type de $1,15^\circ$.

Si l'on se posait le problème des variations de longueur d'un pont projeté, on ne connaîtrait pas la température intérieure des poutres t_i mais seulement la température de l'air à l'ombre t_e à l'emplacement où l'on veut édifier le pont. Le plus simple est alors de remplacer t_i par la moyenne des températures extrêmes extérieures observées dans la journée précédente. La température propre du pont s'écrit alors :

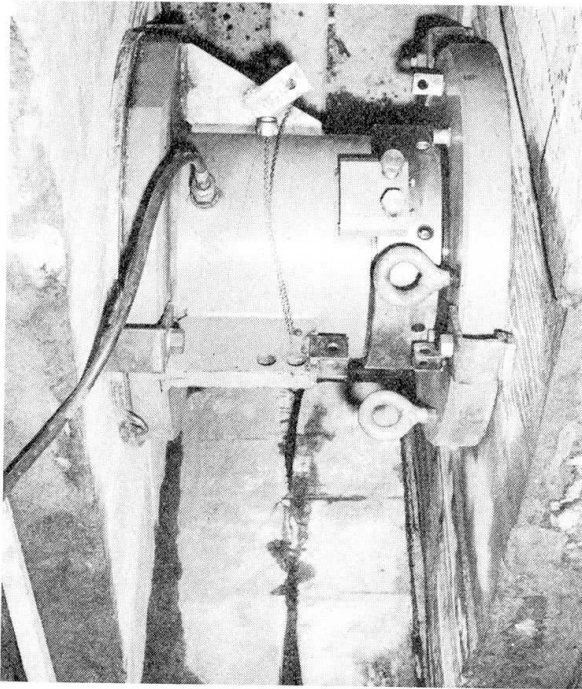


Fig.10 - Vérin de reclavage provisoire en place.

$$t = 0,70 t_e + 0,15 (t_{em} + t_{em}) - 3h - 1j$$

Cette loi n'est évidemment valable que pour un pont formé de poutres tubulaires analogues au pont de Donzère. Dans le cas d'un pont dalle il faudrait vraisemblablement réduire très fortement le coefficient qui est devant t_e .

3 - POUSSEE EN CLE

3-1- Articulation de clé provisoire

Au moment où le pont a été construit, une poussée a été introduite dans l'articulation de clé. Pour cela, des vérins ont été installés entre les entretoises de clé et la poussée exercée a été de 180 tonnes par poutre ; on a alors bétonné les articulations centrales qui ont été mises en charge par suppression des vérins. La poussée en clé devait ensuite s'élever à 214 tonnes par demi-pont par l'effet de charge correspondant à l'exécution des revêtements des trottoirs et des garde-corps.

Les opérations de reclavage qui ont été rendues nécessaires par la fissuration des rotules ont permis de faire d'assez nombreuses mesures de la poussée réelles qui régnait dans l'articulation.

Pour cela, les rotules ayant été démolies ont été remplacées par quatre vérins horizontaux à

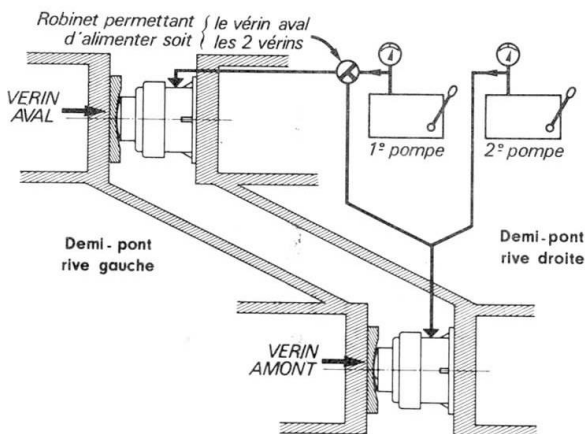


Fig.11 - Schéma des articulations provisoires.

raison de deux vérins par anneaux. Ces vérins étaient appliqués sur les entretoises d'about des deux demi-travées centrales de manière que leurs axes se situent dans le même plan que celui des rotules. Les pistons prenaient appui sur des crapaudines permettant la rotation des deux demi-ponts dans les mêmes conditions que les rotules primitives.

Les pistons pouvaient être bloqués par des écrous permettant de conserver la poussée sans qu'il soit nécessaire de maintenir en permanence la pression d'huile dans les vérins. Le circuit d'huile était conçu de telle sorte que la pression était la même dans chacun des vérins relatifs à un même anneau. Par contre, on pouvait pour chacun des couples amont et aval, brancher les vérins en parallèle sur une même pompe ou séparément sur deux pompes différentes.

3-2- Mesure des ouvertures du joint de clé

L'ouverture du joint de clé était mesurée à l'aide de deux comparateurs sensibles au $1/100$ de mm (1), situés l'un à l'amont, l'autre à l'aval dans le plan des axes des vérins. Cette ouverture était, bien entendu, constante en dehors de toute action sur les vérins. Ces deux comparateurs avaient essentiellement pour but de nous renseigner sur les variations d'ouverture du joint de clé lors des mesures de poussée qui se traduisaient par des variations d'ouverture pouvant dépasser 1 mm.

3-3- Inégalité des poussées sur les deux anneaux

Nous avons vu qu'il existait deux possibilités pour actionner les vérins :

- soit opérer avec deux pompes. Une pompe pour les deux vérins amont et une autre pompe pour les deux vérins aval. Dans cette hypothèse, on est maître à la fois des poussées et des écartements. La logique conduit à manoeuvrer les vérins de telle sorte que l'on enregistre les mêmes écartements à l'amont et à l'aval, ce qui a donné le plus souvent des valeurs de la poussée différentes pour l'anneau amont et pour l'anneau aval, l'écart type par rapport à la poussée moyenne étant de l'ordre de 14,5 %
- soit opérer avec une seule pompe pour les quatre vérins branchés en parallèle (méthode que nous avons le plus souvent employée). Dans cette dernière hypothèse, ils exercent tous le même effort mais le joint de clé ne s'ouvre pas parallèlement à lui-même, il se produit une légère angulation, assez faible d'ailleurs ; l'écart entre les comparateurs amont et aval est de quelques dixièmes de mm. Quelquefois cet écart est nul,

(1) Cette sensibilité peut paraître démesurée pour un ouvrage d'une pareille dimension. En fait, l'expérience a montré qu'une précision du dixième de mm était absolument nécessaire.

quelquefois il est dans un sens, quelquefois dans l'autre sens. Il est vraisemblable que cette inégalité résulte de l'ensoleillement qui chauffe différemment la face amont et la face aval du pont.

3-4- Détermination de la poussée

La détermination de la poussée est assez délicate. En effet,

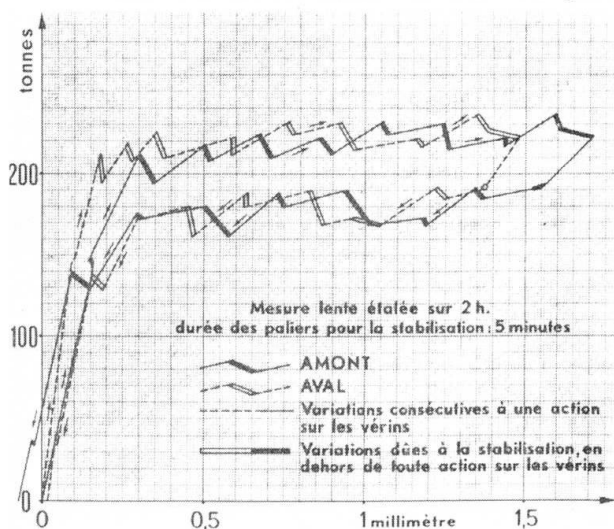


Fig. 12 - Variation de l'écartement du joint de clé en fonction de la poussée

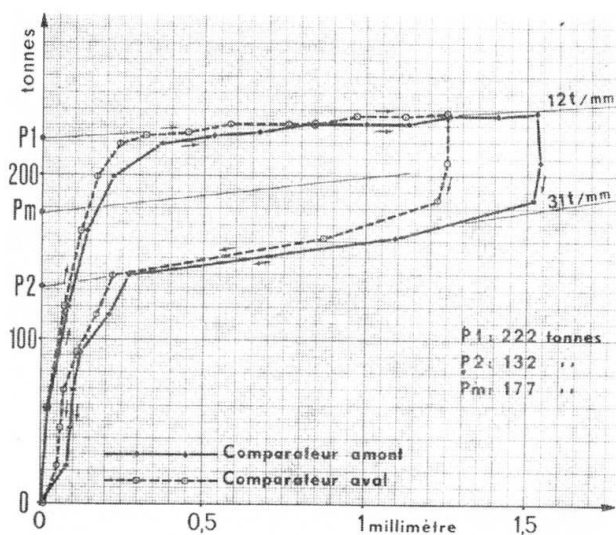


Fig. 13 - Mesure lente des variations de poussée en clé avec l'ouverture du joint

les valeurs que l'on mesure sur les manomètres donnent bien une poussée mais celle-ci ne correspond pas forcément à celle qui existait dans le pont avant que l'on agisse sur les vérins. Pour préciser cette définition de la poussée, nous avons établi lors de chaque mesure des graphiques donnant les variations d'ouverture du joint de clé en fonction de la poussée. Plus de 60 mesures de ce type ont été effectuées.

Les résultats d'une mesure courante sont représentés sur la figure 12. La mise en pression des vérins était opérée par paliers, en ne s'arrêtant sur chaque palier que le temps nécessaire pour lire les appareils.

La poussée croissait d'abord très vite, puis de moins en moins, c'était la phase de transfert de la poussée entre écrou et pression d'huile. Ensuite, la poussée croissait linéairement avec une pente beaucoup plus faible et très peu variable suivant les essais (12 tonnes par mm environ). Il y avait dans cette phase transfert complet de la poussée sur la pression d'huile et la réaction du pont exprimait uniquement le résultat des déformations provoquées dans les poutres par l'ouverture du joint de clé.

Si l'on cessait d'envoyer de l'huile dans le circuit, l'écartement ne variait plus. On constatait alors que la poussée

faiblissait. La perte de poussée ainsi obtenue pouvait atteindre 100 tonnes. Cette perte de pression ne peut s'expliquer que par une certaine raideur du pont qui ne se déforme pas instantanément sur toute sa longueur sous l'action des vérins.

Si l'on diminuait ensuite le volume d'huile, la poussée décroissait d'abord lentement à raison de 10 à 20 tonnes par mm selon une loi sensiblement linéaire mais dont la pente variait beaucoup plus d'un essai à l'autre que lors de la phase de montée en pression. Enfin, si le volume d'huile continuait de décroître, la pression tombait brusquement à partir d'un certain moment. Il y avait alors transfert entre pression d'huile et écrou.

Au total, il se produisait donc une sorte de phénomène d'hystérésis, les efforts appliqués ne se traduisant pas immédiatement par des mouvements de l'ensemble des poutres.

Dès lors, on peut se demander quelle était la poussée réelle dans le pont avant cette expérience. On peut par exemple extrapoler la montée de poussée linéaire jusqu'à l'axe des poussées, c'est-à-dire jusqu'au point d'ouverture nulle du joint on obtient ainsi une poussée P_1 . On peut de même extrapoler la descente de poussée linéaire, on obtient ainsi P_2 inférieur à P_1 . On peut aussi prendre la moyenne, soit P_m .

Pour mieux examiner ce phénomène, nous avons procédé à une autre expérience dont les résultats sont donnés figure 13.

La mise en pression a été faite par paliers successifs en laissant entre chaque lecture un temps de stabilisation ; l'opération a duré environ deux heures. Les appareils étaient lus immédiatement après les manoeuvres sur les vérins et immédiatement avant la manoeuvre suivante, soit au début et à la fin de chacun des paliers de stabilisation.

On constate que durant ces paliers, la poussée diminue dans la phase écartement du joint, elle augmente dans la phase resserrement. Si l'on considère les courbes enveloppes correspondant les unes au début des paliers, les autres à la fin, on constate une réduction du phénomène d'hystérésis. Il est probable qu'en faisant l'opération encore plus lentement ces phénomènes disparaîtraient complètement, mais la prévision ainsi obtenue serait illusoire car la température extérieure varie et cette variation trouble la mesure si celle-ci est trop longue.

L'analyse de ces graphiques nous a conduit à retenir la

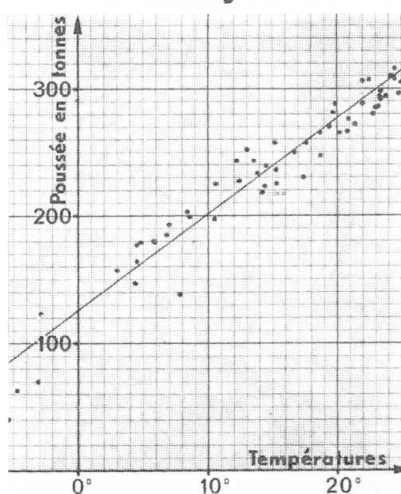


Fig.14 - Relation poussée-température

valeur moyenne P_m comme étant la plus représentative de la poussée qui existait en clé avant la manoeuvre des vérins.

3-5- Relation poussée-température

Le pont étant hyperstatique, la poussée en clé varie avec la température. Nous avons cherché à dégager la corrélation expérimentale entre ces deux éléments en prenant en compte, pour la poussée la valeur de P_m et pour les températures la valeur t définie au paragraphe 2-6 ci-dessus et déduite de la dilatation du pont à l'instant de la mesure de poussée. En adoptant ces paramètres, on pouvait espérer réduire la dispersion des points traduisant la relation poussée-température. La droite

des moindres carrés a l'équation suivante :

$$P(\text{tonnes}) = 7,53 \text{ t}' + 125$$

On voit que P peut s'annuler pour des températures voisines de -17° . On trouve là l'explication des désordres qui ont été à l'origine des opérations entreprises sur ce pont.

Cette corrélation est donnée dans la figure 14. On remarque qu'il subsiste une dispersion assez forte. Pour une température donnée la poussée est égale à la valeur résultant de l'équation ci-dessus à $\pm 40 \text{ t}$ près.

On peut aussi chercher à déterminer la relation poussée-température par le calcul. Il faut faire alors un calcul complet du pont tenant compte du biais car celui-ci introduit une raideur sensible dans l'ouvrage à cause de l'importance des entretoises sur appui. Le calcul fait (1) avec un module d'élasticité de $3,6 \times 10^6 \text{ t/m}^2$ donne une variation de poussée de $8,6 \text{ t}$ par degré alors que la variation mesurée est de $7,53 \text{ t}$ par degré. Le pont est donc un peu moins raide que ne le laisse supposer le calcul. On pourrait schématiser cela par une diminution du module. Il faudrait pour obtenir une variation de poussée de $7,53 \text{ t}$ par degré, supposer un module de $3,1 \times 10^6 \text{ t/m}^2$. Cette valeur est voisine de celle que l'on admet pour des charges brusquement appliquées. On aurait pu penser que les effets thermiques étaient des phénomènes lents. En fait il n'en est rien ainsi que le montrent d'ailleurs les indications du paragraphe 2-6 ci-dessus.

4 - CONCLUSIONS -

On peut résumer ainsi les enseignements de ces mesures :

4-1- Il n'est pas très aisé de définir la température et la poussée en clé d'un grand ouvrage en béton comme le pont de la RN 7 sur le canal de fuite de DONZERE. La température moyenne ne peut être définie à partir de la longueur du pont qu'à 2 ou 3° près. La poussée ne peut être connue à partir de la température qu'à 40 t près. Dans l'étude des articulations et des joints de chaussée, il convient de tenir compte de ces fluctuations autour des positions théoriques. En particulier, si la poussée d'origine est faible, de l'ordre de 50 t , il faut savoir que des fluctuations aléatoires sont capables d'en changer le signe. Pour un ouvrage de ce type, des poussées beaucoup plus fortes devraient être mises en oeuvre si l'on ne veut pas que les écarts relatifs soient très grands.

(1) Ce calcul a été fait par le Service Central d'Etudes Techniques à l'aide d'un programme de calcul d'un ensemble de barres non articulées (programme PROPAC).

4-2- On peut rendre compte assez fidèlement des phénomènes de dilatation en définissant la température intrinsèque du pont par l'expression :

$$t = 0,70 t_{e-3h} + 0,15 (t_{eM} + t_{em}) - 1j$$

ou t_{e-3h} est la température extérieure à l'ombre trois heures avant l'instant considéré.

t_{eM} et t_{em} les températures extrémales observées dans les dernières vingt-quatre heures.

4-3- Il existe dans l'ouvrage une raideur certaine qui se manifeste par un temps de propagation des efforts entre la clé et les culées, de l'ordre de la dizaine de minutes. Il est malheureusement difficile de savoir dans quelle mesure cette "raideur" résulte des dimensions de l'ouvrage ou du biais qui gêne le mouvement normal des articulations.

4-4- Dans cet ouvrage hyperstatique où la poussée en clé dépend directement de la température, le module d'élasticité qui intervient pour le calcul de la poussée est celui que l'on prend pour des efforts appliqués rapidement, à savoir entre $3,1 \times 10^6$ t/m².

4-5- Il faut signaler enfin que ces études ont permis de définir le niveau convenable de la poussée et de procéder au reclavage définitif du pont. Opération, qui, exécutée en trois phases du 15.4.67 au 29.5.67, a nécessité 20 heures d'interruption de la circulation réparties sur 3 jours. Depuis, le fonctionnement de l'ouvrage est parfaitement normal.

RESUME

Le rapport relate les résultats de mesures de températures, allongements et poussés en clef, effectuées en 1965 et 1966 sur un pont en béton armé de 260 m de portée. Ces mesures ont permis:

- de définir une relation entre la température propre du pont et les températures de l'air ambiant,
- d'analyser d'une manière précise le comportement du pont aux effets thermiques,
- de chiffrer les écarts de poussée en clef réelle par rapport au calcul théorique.

ZUSAMMENFASSUNG

Dieser Bericht zeigt die Ergebnisse der Temperatur-, der Dehnungs- und Scheiteldruckmessungen der Jahre 1965 und 1966 an einer Stahlbetonbrücke von 260 m Spannweite. Die Messungen erlauben:

- eine Beziehung zwischen Eigentemperatur der Brücke und Lufttemperatur aufzustellen.
- genau das Verhalten der Brücke infolge von Temperatureinflüssen zu berechnen.
- die Berechnung der Abweichung des Scheiteldruckes in bezug auf die theoretische Rechnung.

SUMMARY

This report presents the results of measurements carried out in 1965 and 1966 of temperature, deformation and compression at the crown of a reinforced concrete bridge of 260 m span. The measurements were used to obtain the following:

- a relation between bridge temperature and ambient temperature,
- a precise analysis of bridge behaviour due to temperature effects,
- variations in compression in the crown of the bridge from the theoretical values.

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Temperaturspannungen in Spannbetonbrücken

Thermal Stresses in Prestressed Concrete Bridges

Contraintes thermiques dans des ponts en béton précontraint

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1. Vorbemerkungen

Die Temperatureinflüsse auf Tragwerke des Brückenbaues sind ausserordentlich manigfaltiger Natur. Die jahreszeitlichen grossen Temperaturschwankungen beeinflussen eine Brücke gleichmässig über den Querschnitt und erzeugen Beanspruchungen nur soweit die freie Längenänderung behindert ist. Von grösserer Bedeutung sind die relativen momentanen Temperaturunterschiede innerhalb eines Brückenquerschnittes. Solche Temperaturdifferenzen treten in einer Spannbetonbrücke zunächst beim Abbinde- und Erhärtungsprozess des Betons auf, wobei auch nach dem teilweisen Abbau durch Kriechvorgänge Eigenspannungen im Tragwerk zurückbleiben, I 1 I. Dann beginnen die täglichen, durch die Witterung bedingten Temperatureinflüsse, welche zu nichtlinearen Temperaturdifferenzen innerhalb des Tragwerkes führen. Ihre täglich wechselnde Wirksamkeit gefährdet besonders das noch junge Bauwerk durch bedeutende zusätzliche Beanspruchungen sowohl in Quer- wie in Längsrichtung, je nach den Auflagerbedingungen und der Querschnittsgestaltung.

Beim Entwurf von Spannbetonbrücken ist es üblich, in einem zweiten Berechnungsgang für die Zusatzlasten solche Temperaturwirkungen zu berücksichtigen. Dabei werden je nach den klimatischen Verhältnissen vereinfachte Annahmen getroffen, etwa, dass bei einem Kastenquerschnitt in der obersten, gegenüber der untersten Faser der Fahrbahnplatte eine Temperaturdifferenz von $\pm 10^{\circ}\text{C}$ auftreten kann. Diese Annahmen können aber je nach der Ausbildung der Brückenoberfläche ganz erheblich von der Realität abweichen, wie an einem Beispiel gezeigt werden soll.

2. Messungen an der Schanzenstrassenbrücke in Bern

Diese 1959-60 gebaute Brücke weist einen dunklen Asphaltbetonbelag von 6 cm Stärke auf, der direkt auf dem Beton liegt. Sie weist ein nach Süden gerichtetes Gefälle von 5,5 % auf (Bild 1). Gemessen wurde in 4 repräsentativen Punkten eines Querschnittes mittels elektrischer Widerstandstemperturgeber (Bild 2a). Die täglichen Schwankungen unter dem Einfluss der Sonnenstrahlung und der nächtlichen Wärmeabstrahlung in den 4 Messpunkten wurde kontinuierlich über mehrere Monate aufgezeichnet, ebenso die Lufttemperatur (Bilder 3 und 4). Trotz der geringen Anzahl Messpunkte ergibt sich ein instruktiver Aufschluss über die im täglichen Rhythmus besonders bei klarem Wetter auftretenden Temperaturunterschiede innerhalb der Fahrbahnplatte und gegenüber dem Steg des Kastenquerschnittes. Es wird ein Zeitpunkt ausgeprägter momentaner Temperaturdifferenzen herausgegriffen.

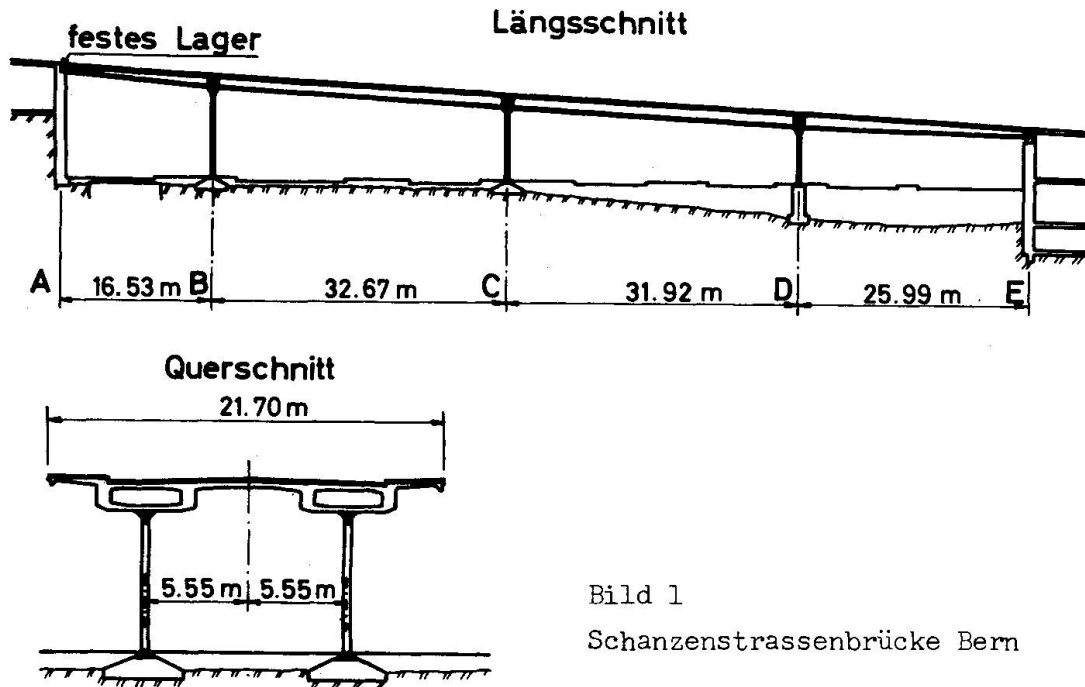


Bild 1

Schanzenstrassenbrücke Bern

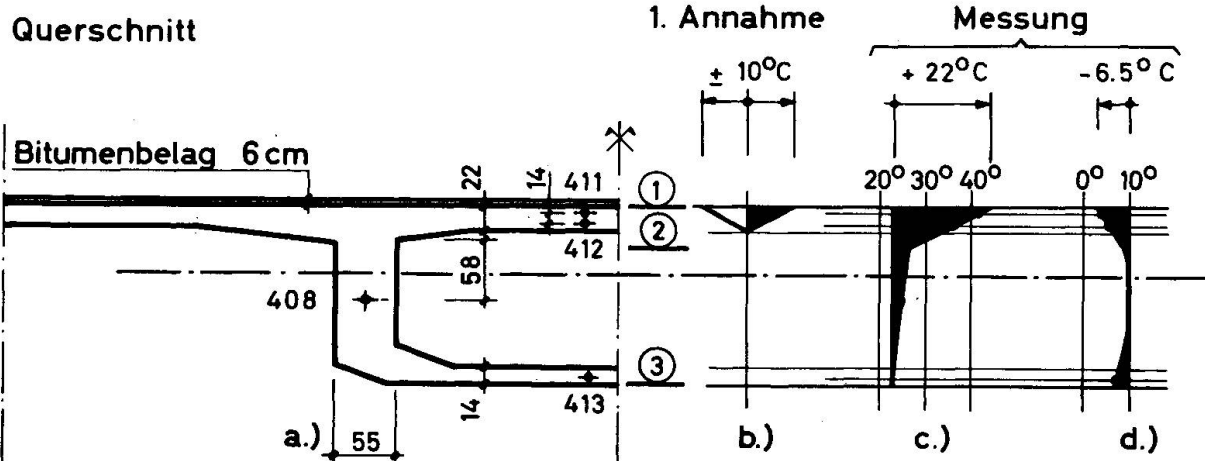


Bild 2

- a. Anordnung der Temperaturmessgeber 411, 412, 408, 413
- b. Angenommene Temperaturdifferenz für stat. Berechnung
- c. Gemessene Temperaturverteilung am 22. Juni 1960 (Bild 3)
- d. Gemessene Temperaturdifferenz am 13. April 1960 (Bild 4)

etwa am 22. Juni 1960. Aufgrund der relativ kurzen Beobachtungsdauer dürfte es sich noch nicht um einen absoluten Extremfall handeln. Es stellt sich aber dabei heraus, dass bei Sonneneinstrahlung erhebliche grössere Werte erreicht werden, als der ursprünglichen statischen Berechnung zu Grunde gelegt wurden. Linear extrapoliert erstreckt sich das Temperaturgefälle von ca. $0,50^\circ \text{C/cm}$ bis in den Steg unterhalb des Ansatzes der Vouten und erreicht über den ganzen Querschnitt insgesamt 22°C (Bild 2c), während der grösste Unterschied der Lufttemperatur an jenem Tage 14°C betrug. Die infolge Abkühlung gemessenen Temperaturunterschiede, z.B. am 13. April 1960 erreichen nicht so grosse absolute Werte und liegen wohl etwa im Rahmen der ursprünglichen Annahme von -10°C .

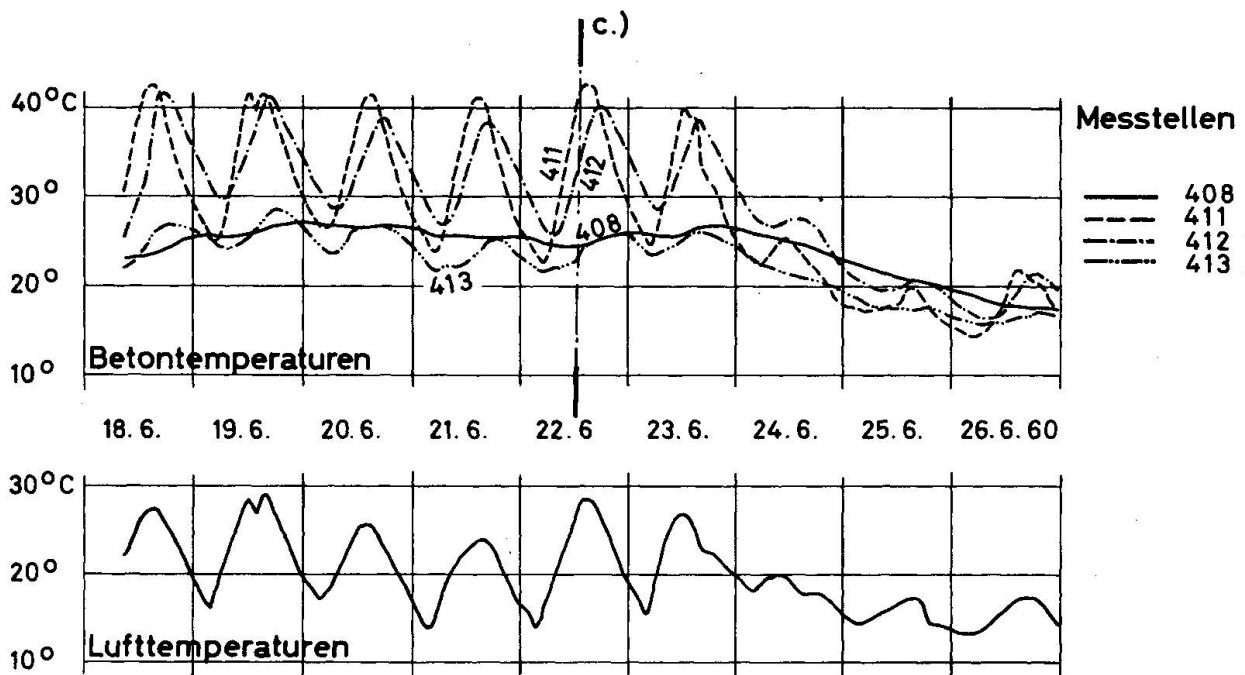


Bild 3 Beton- und Lufttemperaturen Juni 1960

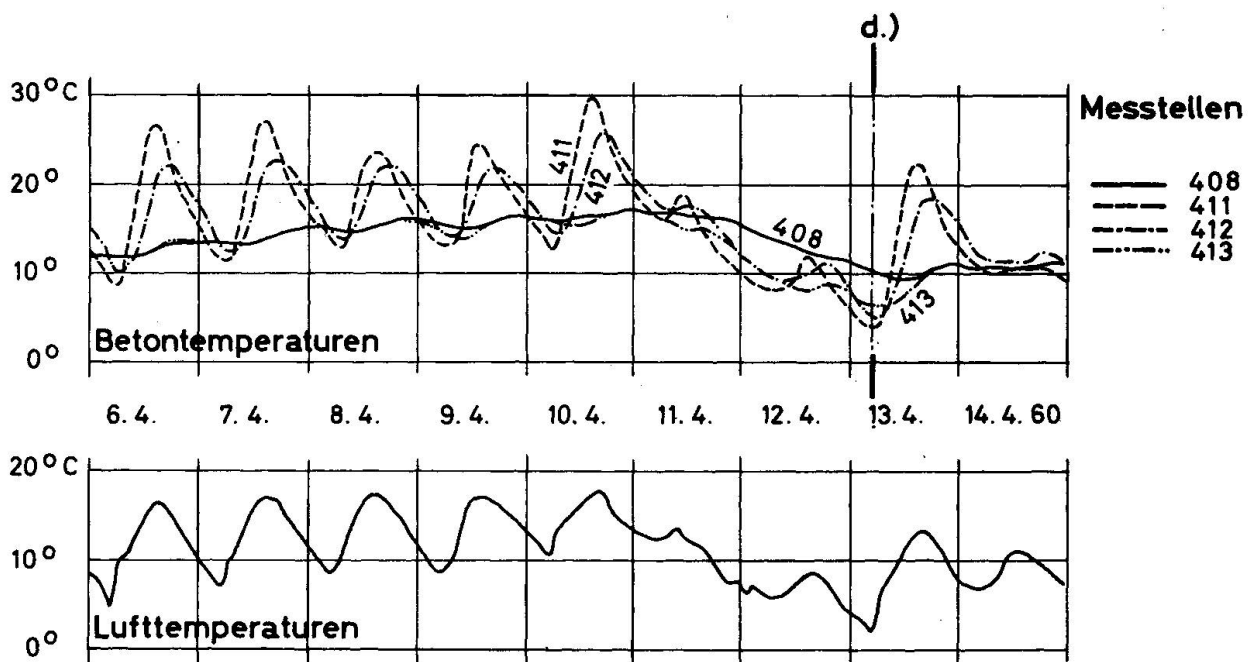


Bild 4 Beton- und Lufttemperaturen April 1960

Da sie aber in der Regel allseitig an der ganzen Querschnittsoberfläche auftreten, nicht nur vornehmlich in der Fahrbahnplatte, sind sie besonders für die Beanspruchungen in Querrichtung zur Brückenachse bedeutsam I 2 I.

3. Normalspannungen infolge Erwärmung der Brückenplatte

Die folgende Betrachtung beschränkt sich auf die Eigenspannungen, welche durch das Temperaturgefälle gemäss Bild 2c in der Brückenlängsrichtung hervorgerufen werden. Diese Eigenspannungen wurden rechnerisch ermittelt, wobei folgende vereinfachende Annahmen getroffen wurden:

- Elastizitätsmodul für kurzzeitige Belastung $E_b = 400'000 \text{ kg/cm}^2$
- Vernachlässigung der schlaffen Armierung
- Analoges Temperaturverlauf über die ganze Brückenlänge und Brückenbreite
- Vernachlässigung der Unsymmetrie infolge Trottoirkonsolen

Die Temperaturdifferenzen erzeugen durch ihren nichtlinearen Verlauf über die ganze Querschnittshöhe Zwängungen innerhalb eines Querschnittes, die sogenannten Eigenspannungen 1. Art, welche nach der von P. Lardy aufgestellten Theorie der Eigenspannungen ermittelt werden I 3 I. Ferner ergeben sich aus der äusseren statischen Unbestimmtheit des Systems positive Biegemomente über die ganze Länge des durchlaufenden Balkens (Bild 5a), welche die Eigenspannungen 2. Art ergeben.

Der Verlauf der resultierenden Eigenspannungen (1. + 2. Art) ist in Bild 5b) für 3 Fasern über die ganze Brückenlänge dargestellt.

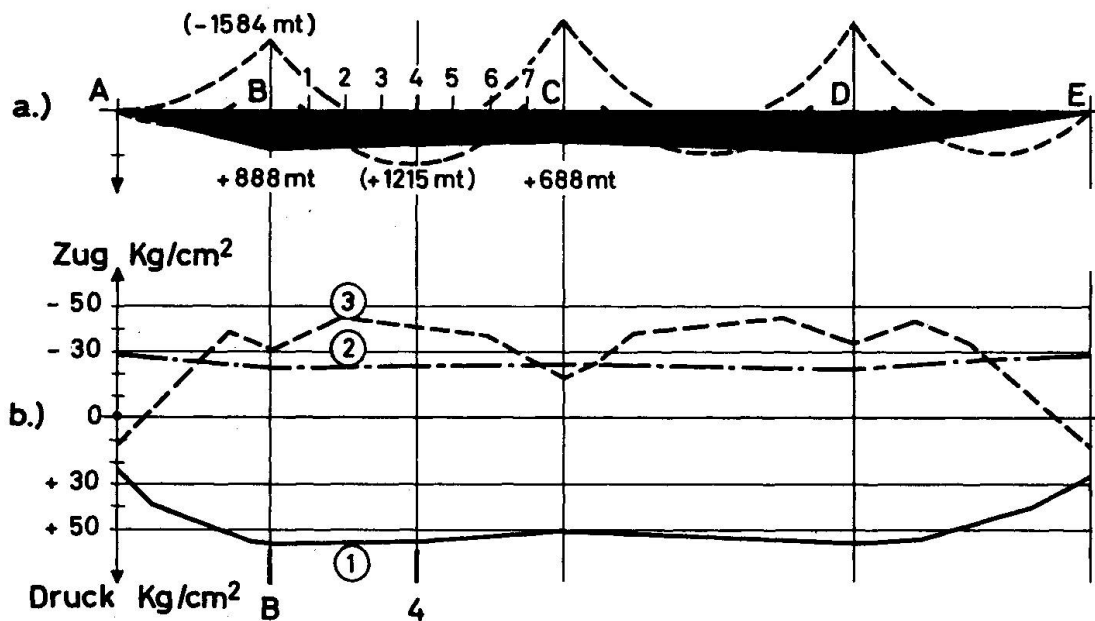


Bild 5 a) Zwängungsmomente am durchlaufenden Balken
b) Resultierende Eigenspannungen für 3 Fasern

- ① Oberste Faser der Fahrbahnplatte
- ② Faser im Steg in der Nähe des Voutenansatzes
- ③ Unterste Faser des Querschnitts

Für zwei charakteristische Querschnitte, einen Feldquerschnitt und einen Stützenquerschnitt sind in Bild 5 die Eigenspannungen 1. und 2. Art getrennt dargestellt (5a und b). Weiter sind in dieser Figur die totalen Eigenspannungen aus Temperatur (c) den Normalspannungen aus den Hauptlasten (Eigengewicht, Vorspannung $T = \infty$) und maximaler resp. minimaler Nutzlast gegenübergestellt.

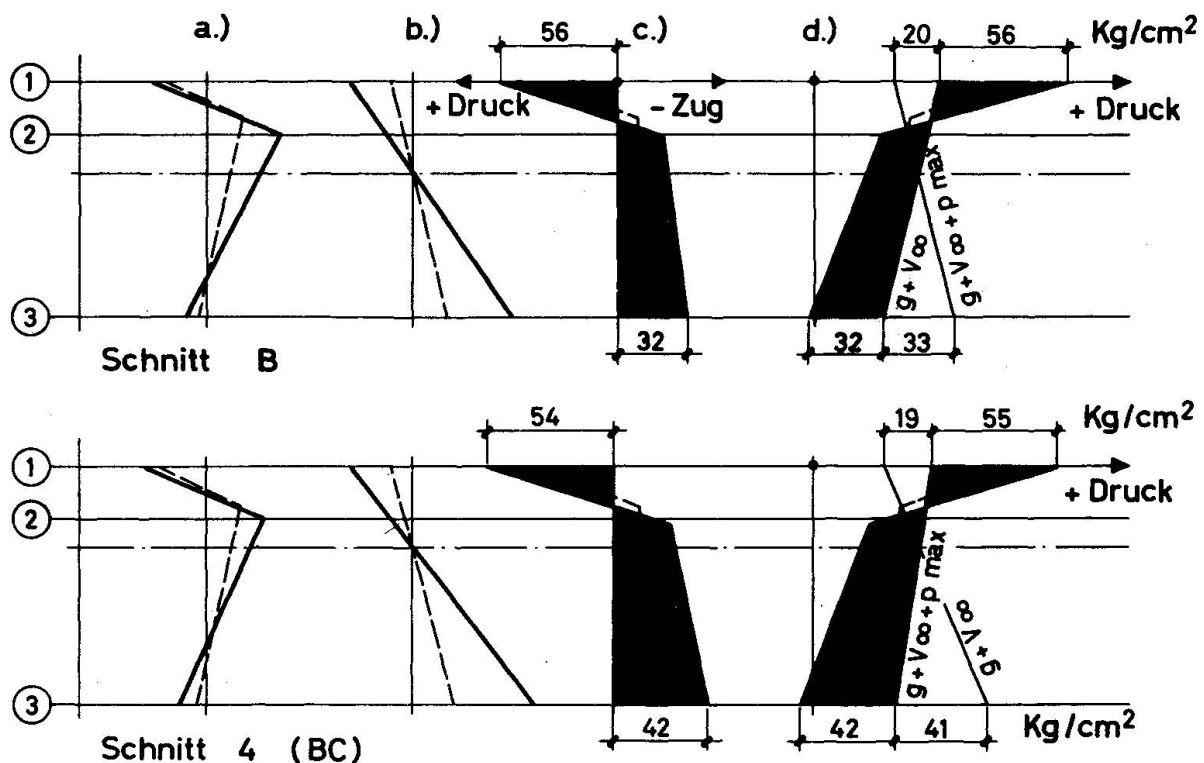


Bild 6 a) Eigenspannungen 1. Art
 b) Eigenspannungen 2. Art
 c) Resultierende Temperaturspannungen 1. und 2. Art
 d) Zusammenstellung mit den Spannungen aus Hauptlasten

Im Feldquerschnitt 4 (BC) treten infolge der gemessenen Temperaturdifferenzen Zugspannungen von rund 42 kg/cm² auf, gegenüber 41 kg/cm² aus maximaler Nutzlast. In der obersten Faser erreichen die Temperaturspannungen fast den dreifachen Wert des Nutzlasteinflusses, nämlich 55 kg/cm².

Im Querschnitt B über der Stütze ist der untersuchte Temperatureinfluss von der gleichen absoluten Grössenordnung wirkt jedoch dem Einfluss der Nutzlast und des Eigengewichtes entgegen, und zwar soweit, dass in der untersten Faser Zugspannungen entstehen können.

Im Steg des Kastenquerschnittes rufen die beobachteten Temperaturunterschiede Zugspannungen in der Grössenordnung von 20 - 25 kg/cm² hervor, wodurch in der Stützennähe die schrägen Hauptzugspannungen ungünstig beeinflusst werden.

Im betrachteten Bauwerk tritt aus den unerwartet hohen Temperaturbeanspruchungen keine Gefährdung ein, weil aus konstruktiven Gründen die Dimensionen reichlich gewählt wurden und daher die zulässigen Spannungen nicht voll ausgenutzt werden konnten. Ferner tritt die rechnerische Nutzlast sehr selten auf und kaum gleichzeitig mit den extremalen Temperaturdifferenzen. Es wurden keine Risse beobachtet.

Bei Tragwerken mit voller Materialausnützung und besonders wenn im Sinne neuerer Empfehlungen auf volle Vorspannung verzichtet wird, dürfte es ratsam sein, vorsichtige Annahmen hinsichtlich der Temperatureinflüsse zu treffen und diese so weit möglich auf Messwerte zu begründen. Es mag daher von Interesse sein, zur Ergänzung der vorstehenden Ausführungen die Resultate ähnlicher Temperaturmessungen an einer Eisenbahnbrücke bekanntzugeben.

4. Temperaturmessungen an der Verbindungsbrücke bei Brugg

Der Querschnitt der untersuchten Spannbetonbrücke ist auf Bild 7a dargestellt. Die vor dem Aufbringen des Schotterbettes gemessenen Temperaturverhältnisse zeigen, dass die festgestellten Differenzen im Beton etwas geringer sind als bei der Schanzenbrücke, obwohl die klimatischen Verhältnisse weitgehend die gleichen waren. Dies dürfte darauf zurückzuführen sein, dass die helle Betonoberfläche weniger Wärmestrahlung absorbierte als der Schwarzelag der Schanzenbrücke. Der Temperaturverlauf über mehrere Tage ist im Bild 8 aufgetragen.

Verbindungsbrücke in Brugg Temperaturverteilung

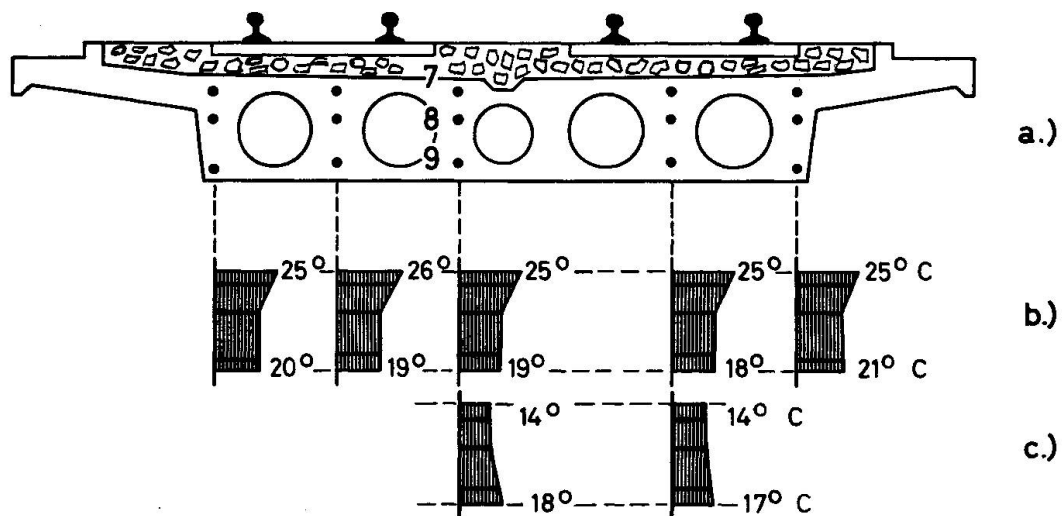


Bild 7 Verteilung der Temperatur im Betonquerschnitt

Verbindungsbrücke in Brugg ohne Schotterbett

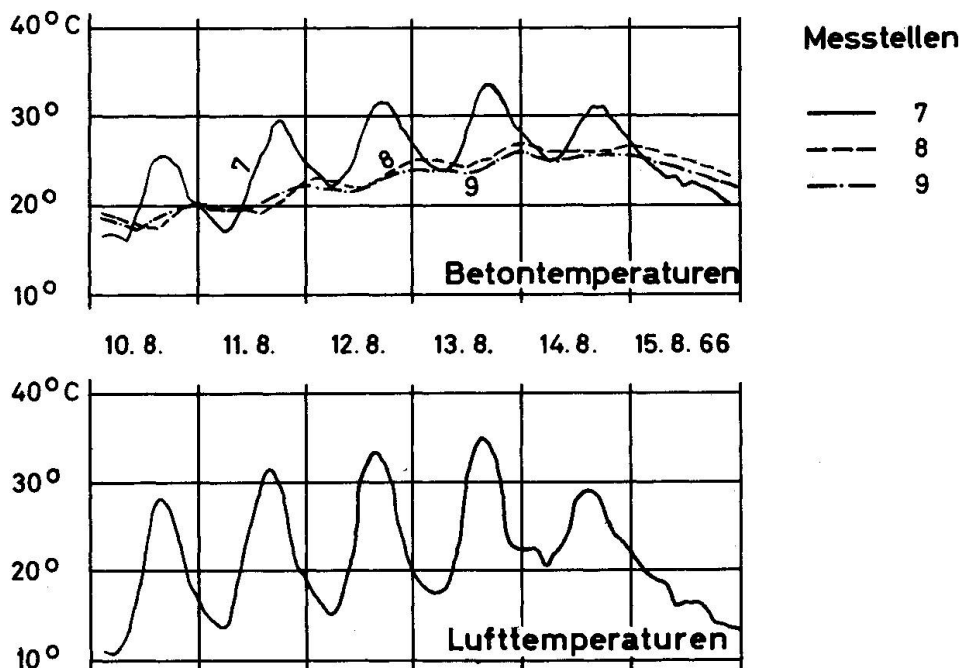


Bild 8 Beton- und Lufttemperaturen im August 1966. Brücke ohne Schotterbett

Die Verteilung der Temperaturen über den Brückenquerschnitt ist aus dem Bild 7b ersichtlich. Die Lufttemperatur an diesem sonnigen Tage stieg von 11°C am frühen Morgen bis auf 28°C am frühen Nachmittag. Es wurde also ein Temperaturunterschied von 17°C erreicht. Die grösste Temperaturdifferenz zwischen der obersten und untersten Messtelle im Beton betrug am Nachmittag maximal 8°C .

Nach Aufbringen des Schotterbetts auf der Brücke wurden die gleichen Messungen an einem sonnigen Tage mit vergleichbaren Verhältnissen durchgeführt. Den Verlauf über mehrere Tage zeigt Bild 9. Obwohl die Temperaturänderungen der Luft im gleichen Rahmen liegen wie bei der ersten Messung nämlich $\Delta t = 17,5^{\circ}\text{C}$ beträgt die Temperaturdifferenz im Beton bei der mit Schotter bedeckten Brücke nur noch $4,5^{\circ}\text{C}$. Es ist bemerkenswert, dass in diesem Falle die höchste Betontemperatur im untern Teil des Querschnitts auftritt. Die Wärmeisolation, die durch das Schotterbett erreicht wird, bewirkt also, dass die Temperaturen an der untern Fläche der Brücke stärker ändernal als direkt unter dem Schotter an der obern Fläche der Betonplatte (Bild 7c).

Verbindungsbrücke in Brugg mit Schotterbett

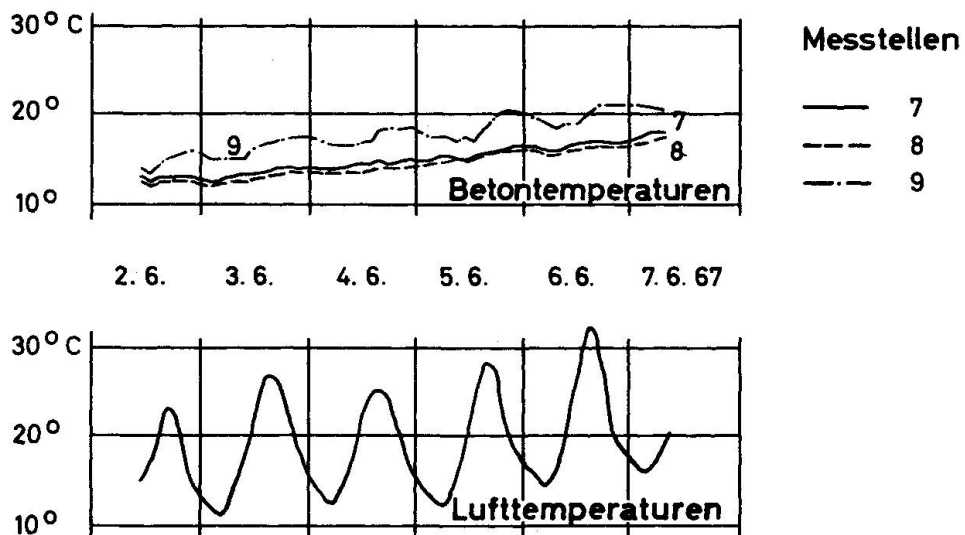


Bild 9 Beton- und Lufttemperaturen im Juni 1967. Brücke mit Schotterbett

Wie aus diesen Untersuchungen hervorgeht, schützt das Schotterbett die Brückenüberbauten wirksam gegen die klimatisch bedingte Wärmeeinstrahlung und vermindert damit wesentlich die zusätzlichen Beanspruchungen, welche durch die Temperaturdifferenzen in der Brückenplatte verursacht werden.

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ZUSAMMENFASSUNG

Im vorliegenden Diskussionsbeitrag wird über einige Temperaturmessungen in Querschnitten von Spannbetonbrücken berichtet. Für eine Strassenbrücke mit Schwarzbelag werden die täglichen Temperaturschwankungen dargestellt, woraus die grösste momentane Temperaturdifferenz zwischen der obersten Faser der Brückenplatte und dem Steg des Hohlquerschnittes entnommen wird. Für diesen Zustand werden die Resultate der Eigenspannungsberechnung in einzelnen Querschnitten des durchlaufenden Trägers dargestellt und gezeigt, dass dieselben den Einfluss der Nutzlast übertreffen können.

SUMMARY

This paper, which is intended as a contribution to the discussion, informs the reader about some results of temperature investigations that have been made in cross sections of prestressed concrete bridges. The daily fluctuations in temperature as they occur in a highway bridge with bituminous coating are plotted, from which the biggest difference in temperature between the top fibre of bridge plate and the web of the hollow section is taken. The calculations of the corresponding residual stresses in characteristic cross sections of the continuous girder appears, that their values preponderate the effect of the live load.

RESUME

L'auteur relate quelques mesures de température effectuées dans des sections de ponts en béton précontraint. Il considère un pont routier à revêtement bitumineux où, à partir des variations journalières de température, il détermine la plus grande différence momentanée entre la fibre supérieure du tablier et l'âme de la poutre caisson continue. Un calcul des contraintes propres correspondantes fait ressortir que celles-ci dépassent largement les valeurs attendues et, dans différentes sections, prédominent sur l'influence de la charge utile.

Effects of Creep, Shrinkage and Temperature on Highway Bridges in the United Kingdom

L'influence du fluage, du retrait et de la température sur les ponts routiers en Grande Bretagne

Der Einfluß des Kriechens, des Schwindens und der Temperatur auf Straßenbrücken in Großbritannien

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1. Introduction

The scarcity of information on the effects of creep, shrinkage and temperature on the behaviour of actual bridges encouraged the Road Research Laboratory, with the co-operation of many enlightened designers, to undertake measurements on certain bridges built in the United Kingdom since 1962. In this way results of direct value to the practising designer were obtained quickly and formed the basis for the development of laboratory and theoretical methods for the estimation of movements. The main conclusions hitherto are summarised in this paper.

2. Early age movements

For the type of mix used in the United Kingdom, an expansion normally occurs during the setting of the concrete up to the time of the peak temperature of hydration, which is usually from 12 to 24 hours after casting. This is followed by a thermal contraction in the hardened concrete together with shrinkage due to moisture loss.

The characteristic temperature and movement curves are illustrated in Fig. 1, which shows the results obtained from a selection of different types of acoustic strain gauge in an insulated concrete block in the laboratory¹. A flint-gravel aggregate was used with a water : cement ratio of 0.4.

The shrinkage movements following the peak hydration temperature are shown to be small but it is worth noting that the thermal contraction of the concrete for the drop in temperature of 20°C following setting, for a coefficient of expansion of 13×10^{-6} , was approximately 260×10^{-6} , which is much greater than the shrinkage during the same period. It follows, therefore, that quite small temperature changes in concrete on the full-scale will produce comparatively large thermal strains between one part and another, which could give rise to cracking a few days after casting.

The full-scale condition is illustrated by the results from a large reinforced cantilever², having a cross section 2.55 m deep by 2.75 m wide at the root, constructed on the M4 Motorway at Chiswick London. Because of the size of the cross section, the temperature at the centre rose more

rapidly than at the outside as indicated by the temperature curves (Fig. 2). The strain gauge at the centre of the cross section, No. 11, indicated a different stress pattern from the remainder which were around the perimeter. During the first day the centre of the section tended to expand more rapidly than the outside and gauge No. 11 indicated compression as the centre was restrained by the remainder of the concrete. During cooling, on the other hand, tension is induced at the centre and compression at the outside, as indicated by the remainder of the gauges. It is likely that there was a residual tensile strain at the centre of the cross section.

In contrast to the large thermal strains soon after casting, the shrinkage strain due to moisture loss, was only about 30×10^{-6} after 1 year from the time of cooling down following casting².

3. Shrinkage arising from moisture loss

On the full-scale structure, there is difficulty in separating out shrinkage from creep, while small specimens cast with a structure indicate more shrinkage than the structure itself because of the size effect^{3,4}. On the Mancunian Way, a continuous prestressed concrete structure⁵, a precast segment, which was reinforced in the same way as the structure, was instrumented with 23 acoustic strain gauges buried in the concrete and left on an exposure site at the Laboratory to indicate shrinkage⁶. The unit is shown in Fig. 3, together with the smaller specimen slabs and columns used for more general studies of shrinkage and creep effects. A limestone aggregate was used for the concrete with a water : cement ratio of 0.38.

Curve 1 (Fig. 4) shows the mean movement of the unit which is actually a small expansion over a period of 5 years from the time of casting. Small specimens on the other hand, shrank up to 300×10^{-6} on outside exposure. It can be concluded, therefore, that the size effect was large. However, for other aggregates on other structures, shrinkage has been indicated in similar experiments. The readings for an unreinforced slab cast with the Western Avenue Extension, a new overhead road in London, and having the same thickness as the root of the cantilever of the structure, indicate a small shrinkage of about 20×10^{-6} after one year (Fig. 5). This is continuing. A sea-dredged gravel was used in the concrete mix with a water : cement ratio of 0.38. The shrinkages of small specimens, $508 \times 102 \times 102$ mm, waterproofed in various ways (Fig. 3), are also given in Fig. 5. It is seen that the size effect is large as the peak shrinkage in an unwaterproofed specimen (curve 5) was 100×10^{-6} in 1 year. The seasonal effect is shown, i.e. the tendency of the concrete to expand in the winter following shrinkage in the summer. This effect is also observed in full-scale structures. (Curves (2) and (3) in Fig. 4; curves (1) and (2) in Fig. 7).

The results of a similar investigation for the new London Bridge⁷ are shown in Fig. 6. Readings from a slab $1.5 \times 1.5 \times 0.3$ m cast with a unit of the structure and reinforced in the same way (curve 1) are compared with readings obtained in the unit itself (curves 2, 3 and 4) and also with small specimens (curves 5, 6 and 7). For this concrete

there is a greater tendency to shrink, the peak reading in the unwater-proofed specimen being 240×10^{-6} in the first year following casting. The mix included a greater fraction of sand than for the mixes for the Mancunian Way and Western Avenue, which is likely to account for the greater shrinkage⁸.

While it is a little early to predict the final conclusions for the magnitude of shrinkage of full-scale bridge structures in the United Kingdom constructed to British Codes of Practice, it is likely that this will be in the range 50 to 200×10^{-6} at 10,000 days with the possible exception of limestone concrete, which may indicate no shrinkage under most exposure conditions. The small shrinkage values observed on the full-scale as compared with other countries may be attributed to the mild and humid British climate.

4. Creep movements

A preliminary analysis of the records of total strain observed at a section of one of the main cantilevers of the Medway Bridge has indicated that the creep factor, following the method of DIN 4227, is likely to be approximately 2.0 at 10,000 days when the shrinkage is 100×10^{-6} . Following the completion of the bridge the total strain for the top and bottom slabs (curves 1 and 2 Fig. 7) show that the sum of the creep and shrinkage is proportional to the logarithm of time, although it must be remembered that the prestressing force is decreasing because of relaxation in the prestressing bars. As the shrinkage of small specimens, $710 \times 152 \times 152$ mm, housed within the box girders of the bridge (curve 3), is nearly equal to the sum of the elastic strain, shrinkage and creep for the loaded bottom slab, it is confirmed that the size effect between small specimens and the full-scale structure is large.

For the Mancunian Way (Fig. 4), a preliminary analysis shows that $\phi = 1.0$ approximately at 10,000 days and again total strain appears to be proportional to the logarithm of time. There was a static period during the first winter but the low value of ϕ is attributed to the use of limestone aggregate for the concrete. Flint-gravel was used for the Medway Bridge. The preliminary analysis of the results from other bridges suggests that ϕ is likely to be in the range 1.0-2.0 for the British climate at 10,000 days.

5. Correlation of movements between small specimens and the full-scale

It has been found that small specimens kept outside are useful in predicting movements on the full-scale as the effect of climate is allowed for, although the movements in the specimens are larger because of the size effect. To correct for the size effect, attempts were made to reduce the magnitude of the movements in the specimens by waterproofing them. In this way creep and shrinkage on the full-scale could be predicted by loading small specimens.

Using specimens $508 \times 102 \times 102$ mm, waterproofing was first carried out by leaving strips untreated down two opposite faces in such a way that the volume/surface area ratio was the same as that of the

structure. It was found that these specimens moved more than the structure (curve 4, Fig. 5). Waterproofing was later carried out by banding the specimens with a waterproofing membrane around their centres, in such a way that the depth of waterproofing was equal to the mean thickness of the box sections of the bridge. This gave similar shrinkage movements (curve 3 as compared with curve 1, Fig. 5 and curves 5 and 6 as opposed to curves 1, 2, 3 and 4, Fig. 6). It was inferred that because shrinkage movements were the same, creep movements in loaded specimens would also be the same.

Having achieved similar movements in the specimens to those in the full-scale structure, a method was devised for the determination of stress in concrete structures⁹. Specimens were loaded in creep rigs (Fig. 8) to give the same strain at all times as in an element of the structure by incrementally adjusting the load in the rigs. The stress in the specimen was then the same as that in the structure (Fig. 9).

6. Temperature Movements and Stresses

Temperature movement in bridges can be divided into two main components. Axial movement along the neutral axis and rotational movement caused by thermal bending. The resultant thermal stresses, f , developed in a beam with the temperature, T , varying as a function of its depth, $T(z)$, is given by:-

$$f = \alpha E T(z) + \frac{1}{A} \int \alpha E T(z) b(z) dz + \frac{z}{I} \int \alpha E T(z) b(z) z dz \quad (1)$$

where α	=	thermal coefficient of expansion
E	=	elastic modulus
$T(z)$	=	temperature change as a function of beam depth
A	=	cross sectional area of beam
$b(z)$	=	Width of the beam as a function of its depth
I	=	second moment of area of the section about the transverse axis through its centroid.

The datum for assessment of axial and bending thermal strains is the strain distribution derived from the actual temperature distribution, and is given by the first term. This implies that non-linearity of thermal gradients introduces stresses which are algebraically summed with axial and rotational stresses.

The middle term, divided by E , determines the axial movement: it disappears from equation (1) when axial movement is completely restrained. The last term divided by E , calculates the rotational movement: it drops out when the beam is restrained in bending. The full-scale investigation has yielded the following general conclusions:-

- 6.1 The thermal coefficient of expansion, determined from measurements of movements at expansion joints and the distribution of temperatures in cross sections of bridge superstructures varied between $(7.5 \pm 1) \times 10^{-6}$ for the Mancunian Way (limestone aggregate concrete) to $(13 \pm 1) \times 10^{-6}$ per $^{\circ}\text{C}$ (Hammersmith Flyover, flint-gravel aggregate concrete). Concretes having granite aggregates gave intermediate values. Tests on small specimens gave a similar range of values.
- 6.2 Over a 24 hour period, the mean temperature of the superstructure is lowest at 08.00 ± 1 hour (GMT) and the overall length is a minimum; the superstructure reaches its highest mean temperature and maximum length at 18.00 ± 1 hour (GMT).
- 6.3 The mean temperature of the superstructure at about 08.00 is usually within $\pm 3^{\circ}\text{C}$ of the air temperature in the shade at the same time. This is the only time when the bridge temperature correlates regularly with the air temperature and this fact is worth noting for setting bearings and expansion joints. Because of this correlation it follows that seasonal and weekly fluctuations in bridge temperature follow closely the changes in the mean air temperature¹⁰. Also at this time the temperature is approximately uniform through the superstructure. Because of thermal lag in the mass of the bridge structure the diurnal range of concrete bridge movements in the United Kingdom is unlikely to exceed the equivalent of a 6° change in bridge temperature above the minimum at 0800 hrs even when the diurnal fluctuation is 20°C above the same minimum.
- 6.4 Both experimental and theoretical results indicate that the range of extreme mean temperatures of the superstructure is approximately the same as the range of extreme shade temperature in the same area¹¹. It should be emphasized that this conclusion applies only to the extreme values and cannot be used to determine mean temperatures of the superstructure at a particular time. However, it does enable the range of longitudinal movements of a bridge in a particular location to be calculated from the data on shade air temperature for that location. The actual values for the range will depend on the period to be considered in design and the choice of period will be governed by the seriousness of the consequence of failure. In the United Kingdom, main members of the superstructure will usually be assumed to have a design life of 120 years and it will be appropriate when guarding against structural damage to consider what the extreme bridge temperature range over this period is likely to be. For the south-eastern part of the country, a range of -20°C to $+40^{\circ}\text{C}$ is not likely to be exceeded. When damage due to temperature extremes is likely to be confined to non-structural finishes or to components whose failure have no serious consequences, then a shorter period of say 20 years may be acceptable. The corresponding range of extreme mean temperatures then becomes -15°C to 35°C for design purpose. This introduces a probabilistic concept into the specification of extreme temperature ranges, but the amount of data available is inadequate for a precise statement of probability of

unservicability or failure.

6.5 A theoretical method can be used for estimating extreme ranges of mean temperatures of superstructures¹¹. It can also be used for predicting extreme temperature distributions and gradients across a section of a box girder. The method assumes that the mean temperature across a section is a minimum at 08.00 (GMT) and that there is no temperature gradient at this time. These assumptions are justified by the observations in paragraphs 6.1 and 6.2 above. Starting from 08.00, the temperature distribution in the concrete is then calculated for subsequent intervals of time by solving iteratively the differential equation for the transient conduction of heat within a solid. The boundary conditions at the upper surface of the bridge deck are given by the energy balance between incoming solar radiation, the heat transfer between deck surface and the air, and the conduction of heat into the surface of the deck. On the underside of the superstructure, the heat lost to the air by convection is equated to the heat conducted into the concrete. The heat exchange between the internal surfaces of the box section is taken into account. To solve the differential equation numerically, requires that the variation of maximum solar radiation and maximum air temperature with time, for one day, is known and that the absolute maximum for both occur on the same day. This coincidence has not yet been recorded, so that there will be a tendency for thermal ranges and gradients to be overestimated by the theoretical method.

6.6 Rotational movements give rise to such effects as additional longitudinal movements away from the neutral axis, e.g. on joint sealants; the drooping of cantilevers; and the inducing of significant hogging moments on hot days. The magnitude of thermal rotational movement is determined from the third term in equation (1) and the resulting stresses for various degrees of restraint are shown in Fig. 10 for a box section, on a very hot afternoon. In continuous structures, the hogging tendency is restrained by continuity at the supports, and this leads to the redistribution of moments shown in Fig. 11. The resultant stresses, determined by integration of equation (1), are shown for Sections B and C in the same Figure. At Section A, conditions approach the distribution of stress without restraints, shown in Fig. 10; whilst at Sections D and E conditions approximate to the case of full restraint in bending, with expansion allowed.

Experimental results and calculations indicate that the thermal stresses in box structures, both longitudinally and transversely, are nearly as large as the stresses induced by the design loading. They may, therefore, have a significant effect on the design of concrete sections and their reinforcement.

7. Acknowledgements

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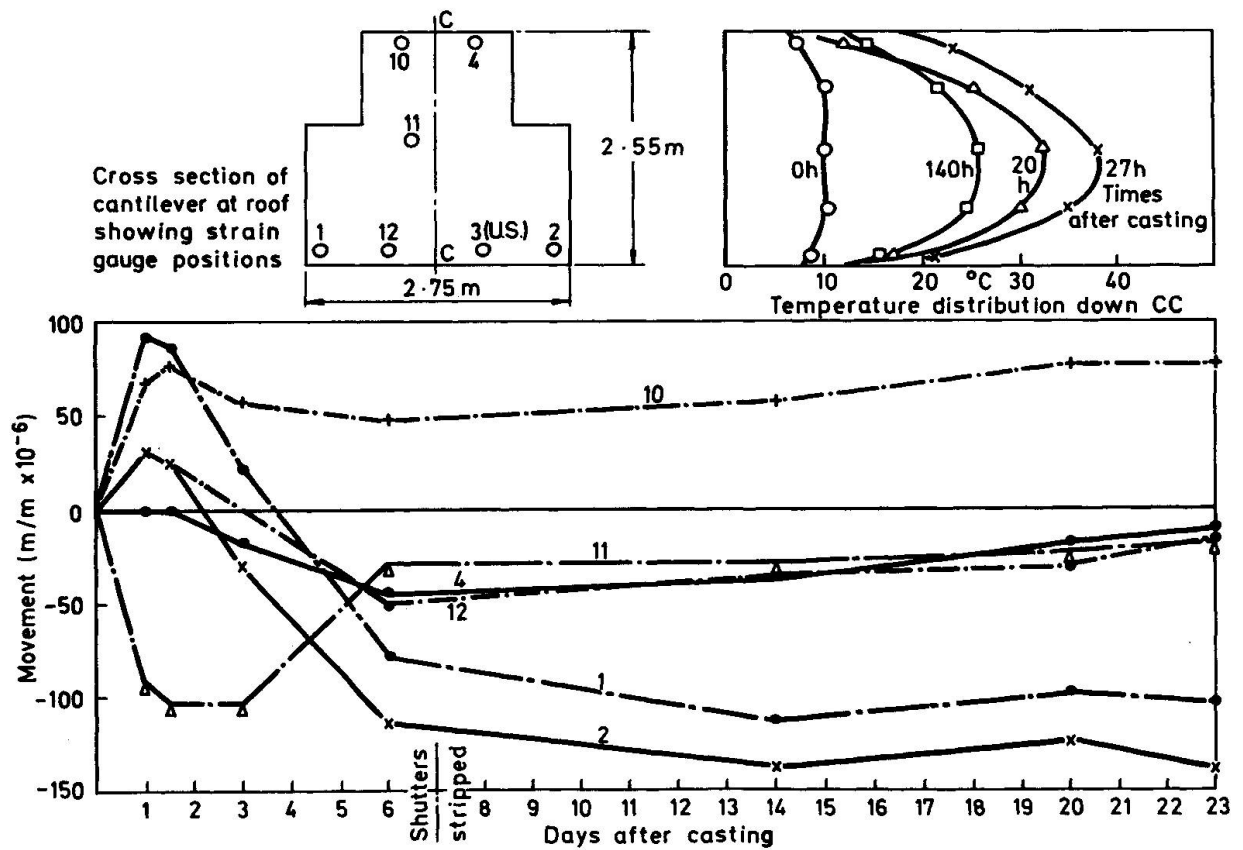
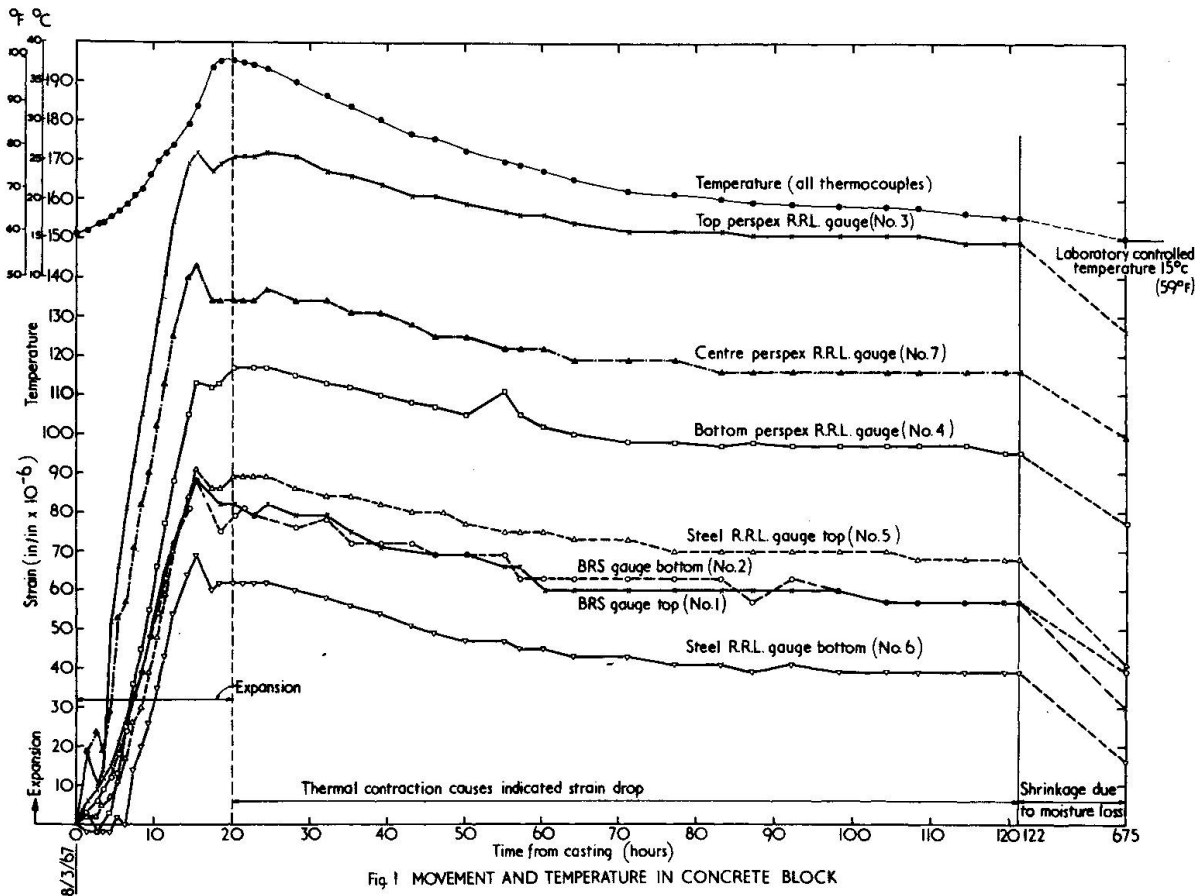




Fig. 3: Unit from Mancunian way and creep and shrinkage specimens

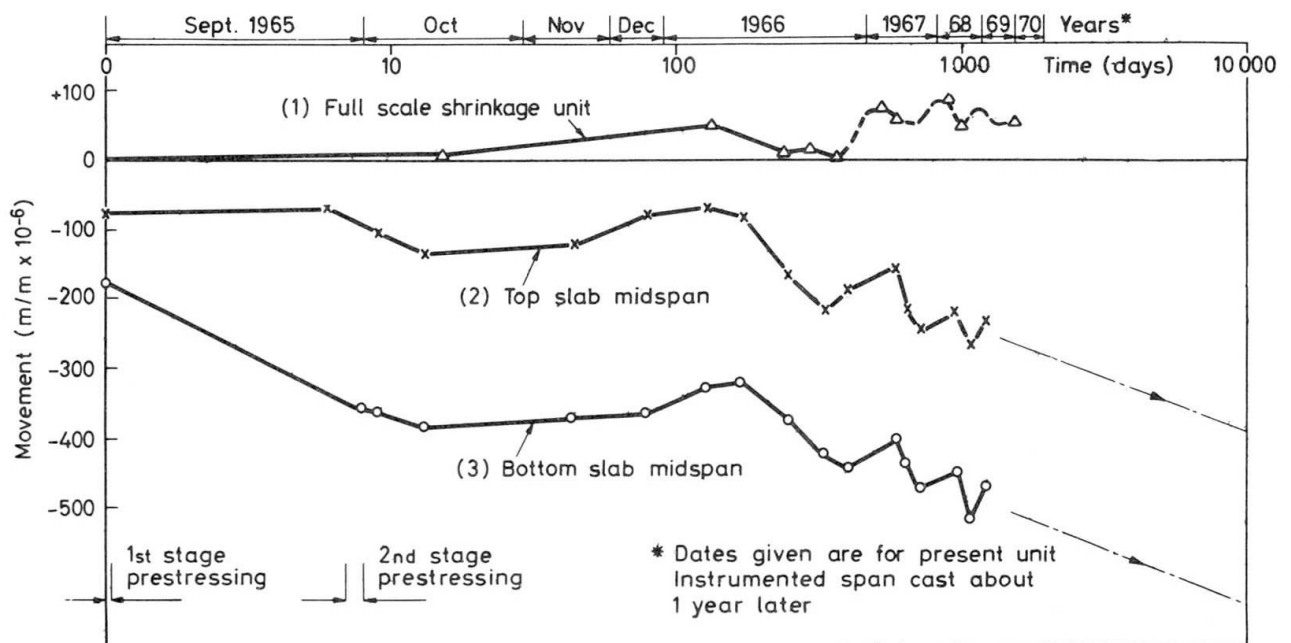
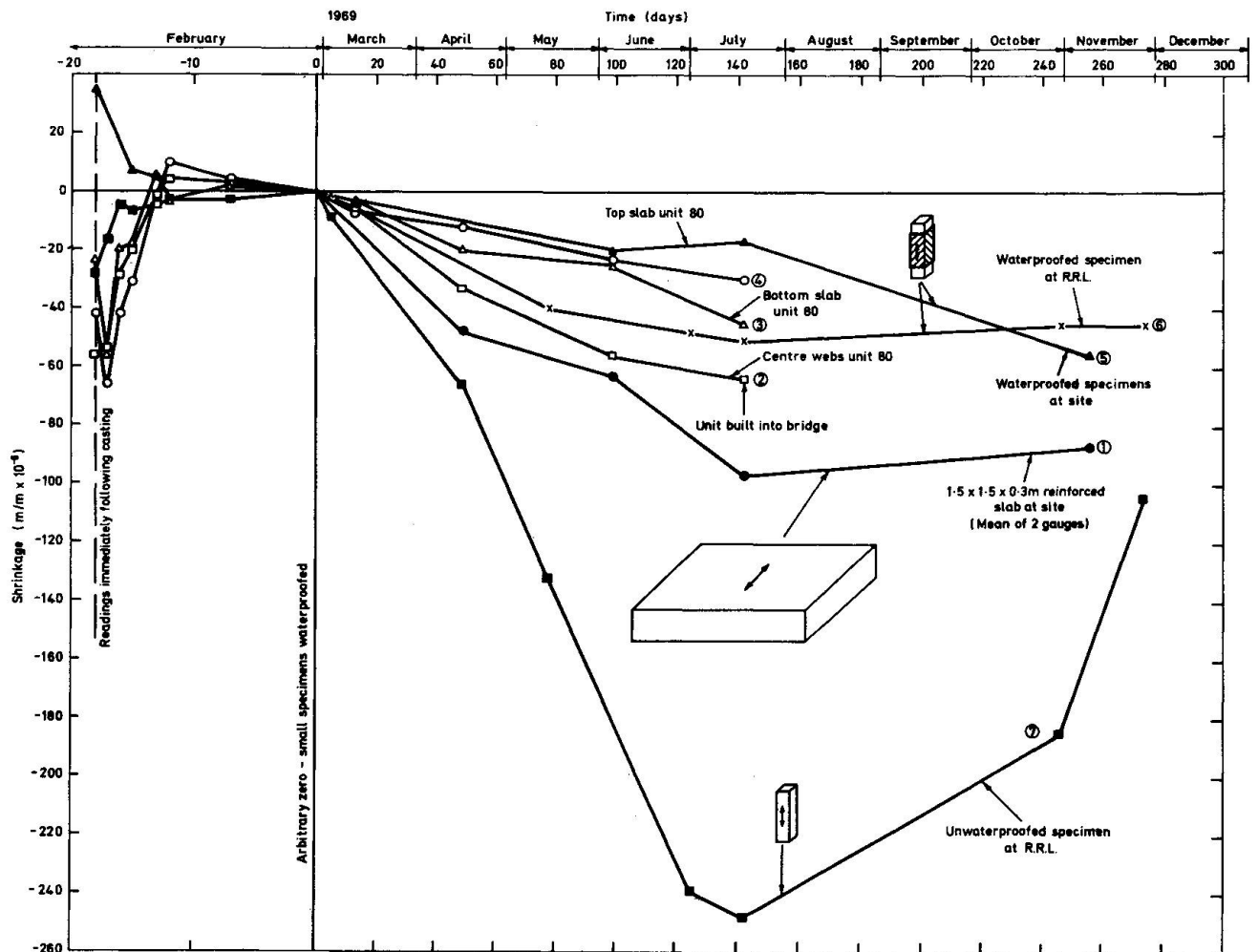
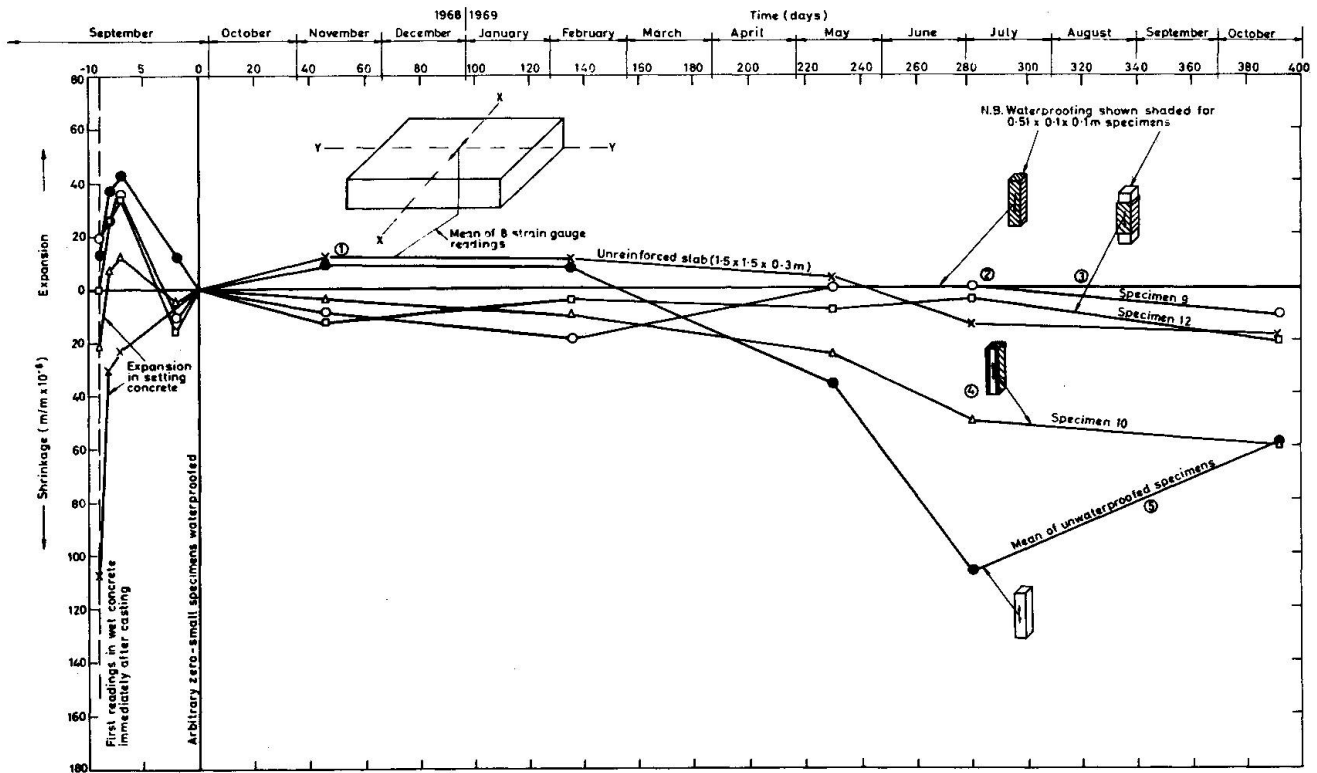


Fig. 4. MOVEMENTS IN SHRINKAGE UNIT AND STRUCTURE OF MANCUNIAN WAY



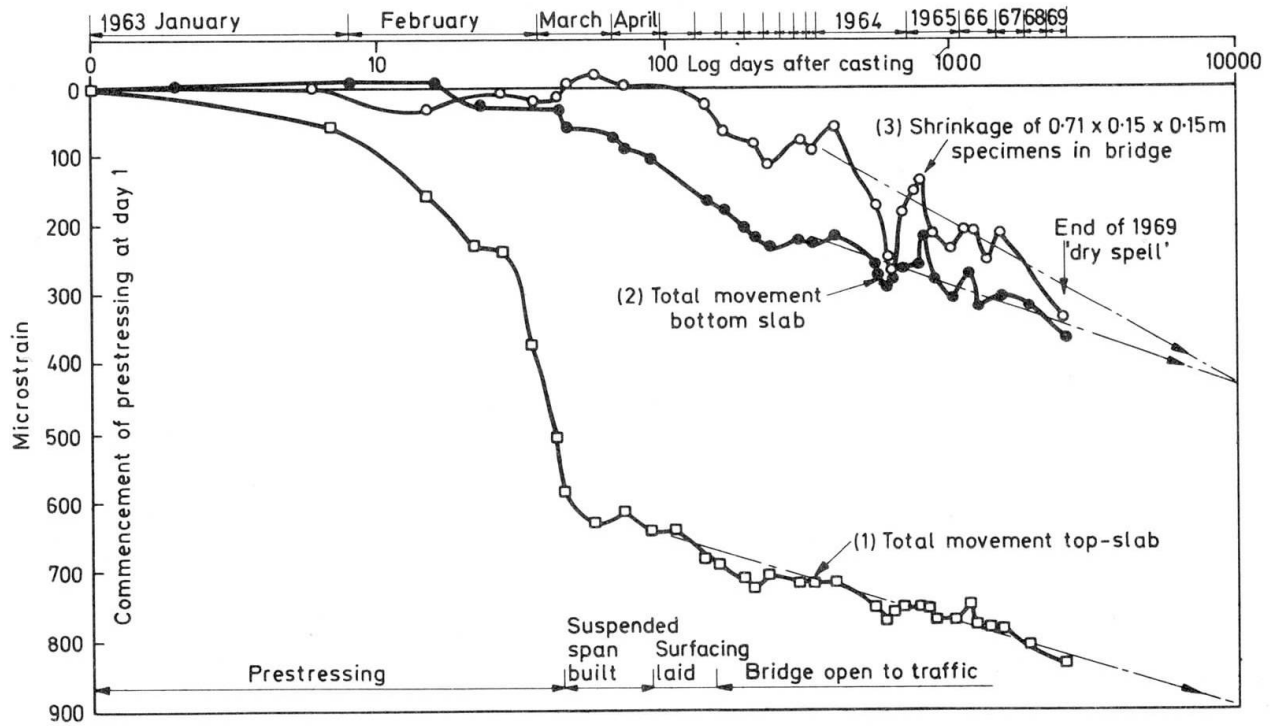


Fig. 7. MOVEMENTS FOR MEDWAY BRIDGE

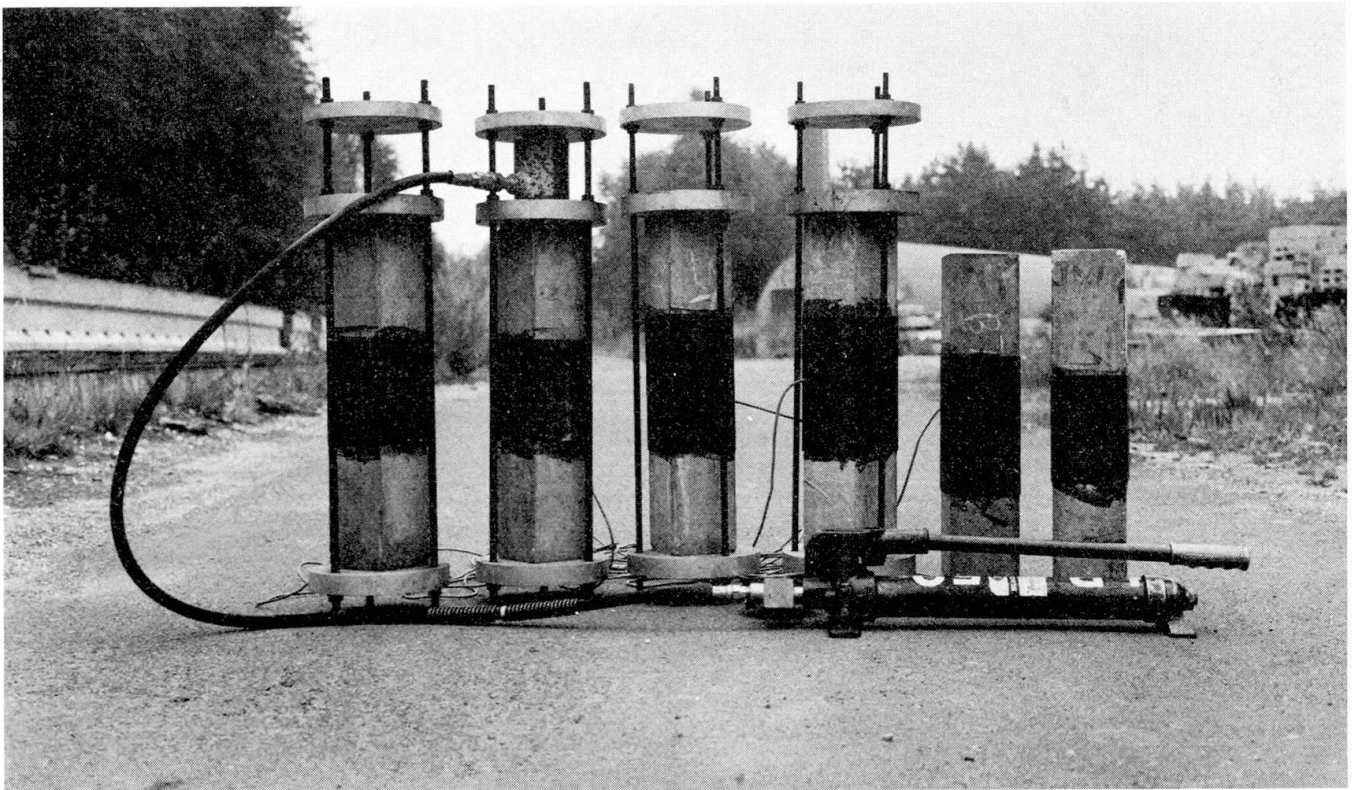


Fig. 8: Creep rigs and specimens

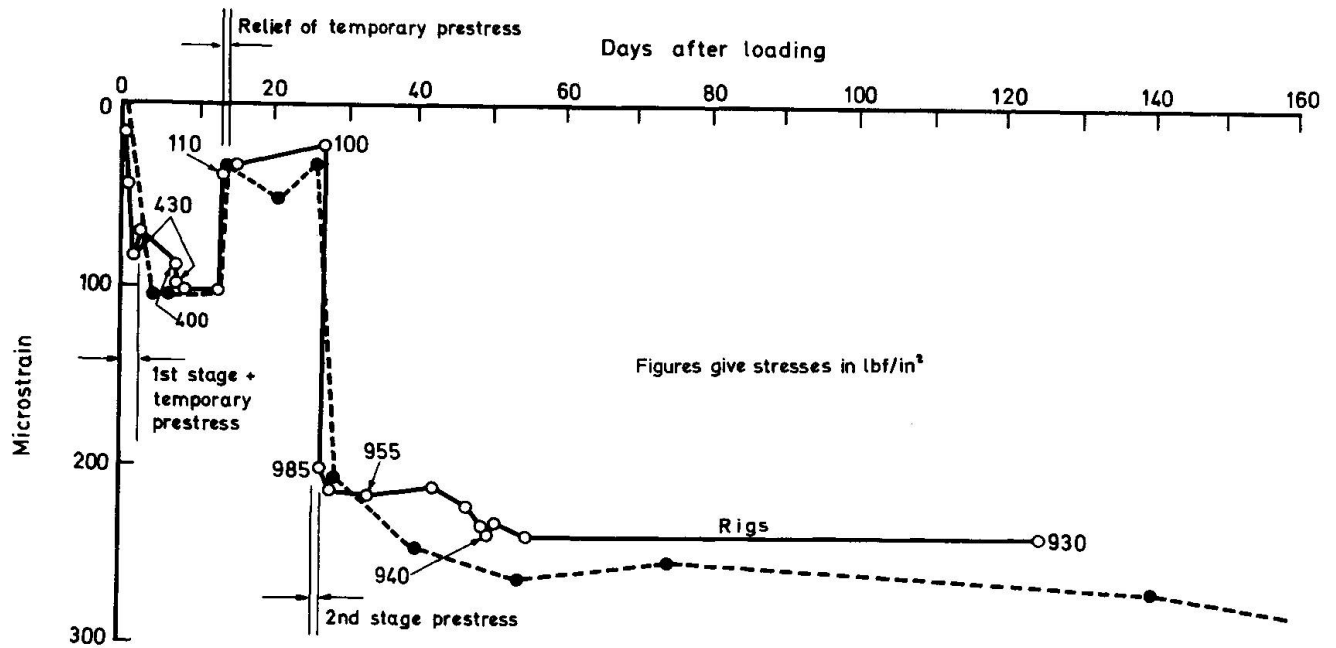


Fig. 9. SECOND ASSESSMENT OF STRESS IN LOWER SLAB

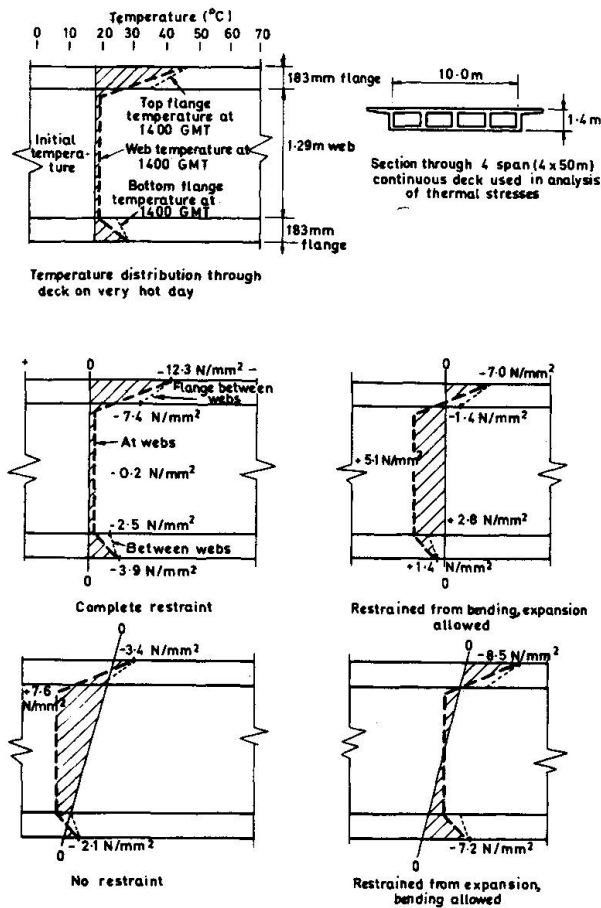


Fig. 10. STRESSES DUE TO TEMPERATURE

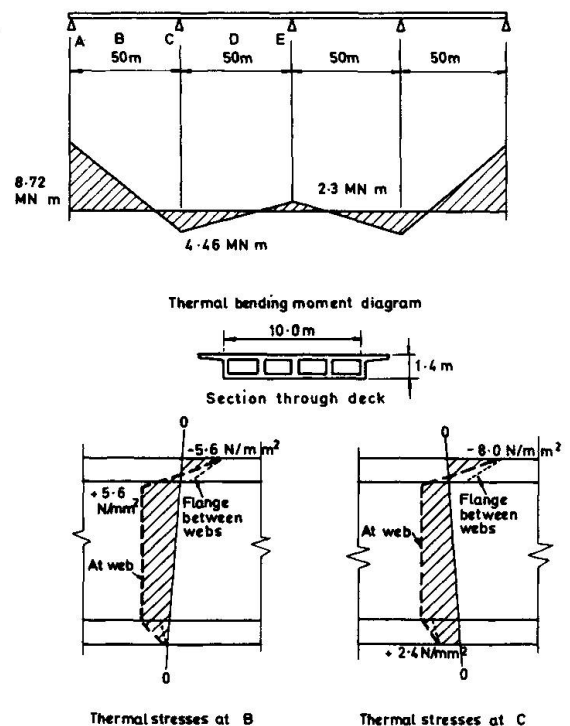


Fig. 11. THERMAL STRESSES IN CONTINUOUS MULTI-CELLULAR BOX DECK

SUMMARY

Measurements of shrinkage, creep and temperature made on bridges in the U.K. are discussed. It is shown that the magnitude of the thermal movements on setting may well exceed that due to the later shrinkage arising from moisture loss. The creep factor ϕ to DIN 4227, has been found to be in the range 1.0 to 2.0 at 10,000 days and the total movement was proportional to the logarithm of time. Correlation has been achieved between small specimens and the structure by partially waterproofing the specimens. The range of thermal movement in bridges is described and it is shown that thermal stresses are sometimes comparable with stresses due to live loads.

RESUME

Les auteurs décrivent des mesures de retrait, de fluage et de température faites sur des ponts au Royaume Uni. Ils démontrent que la grandeur des mouvements pendant le durcissement peut bien dépasser ceux dûs au retrait ultérieur qui résulte d'une perte d'eau. Le facteur du retrait ϕ selon DIN 4227 se trouvait entre 1,0 et 2,0 à 10.000 jours et le mouvement total était proportionnel au logarithme du temps. Pour pouvoir faire une corrélation entre les petites éprouvettes et la structure on a dû imperméabiliser partiellement les éprouvettes. On décrit l'étendue du mouvement thermique et on démontre que les contraintes thermiques sont quelquefois comparables aux contraintes dues aux surcharges.

ZUSAMMENFASSUNG

Besprochen werden Kriech-, Schwind- und Temperaturmessungen an Strassenbrücken in Grossbritannien. Es wird gezeigt, dass die Höhe der thermischen Bewegungen während des Erhärtens jene übersteigt, die aus dem Schwinden infolge Feuchtigkeitsverlust auftreten. Der Kriechmodul ϕ nach DIN 4227 ergab sich zu 1,0 bis 2,0 bei 10 000 Tagen und die gesamte Bewegung war proportional zum Logarithmus der Zeit. Korrelation zwischen kleinen Proben und dem Bauwerk wurde durch teilweise Wasserdichtung der Proben erreicht. Der Bereich der thermischen Bewegungen in Brücken wird beschrieben und gezeigt, dass die Spannungen durch Temperatureinflüsse manchmal mit jenen aus Nutzlast verglichen werden können.

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Mesures de température et de déformations sur un pont bâti en encorbellement

Temperatur- und Verformungsmessungen an einer im Freivorbau erstellten Brücke

Temperature and Deformation Measurements in a Cantilever Constructed Bridge

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Introduction

Le pont est réalisé par une poutre continue sur quatre appuis, avec portée de 45 m, 90 m, 45 m. La section varie entre 5, 10 m de hauteur sur les appuis intermédiaires, et 1,80 m sur les appuis latéraux et au milieu (*)

Il n'y a pas de poutres transversales, autre que sur les appuis; la section est en caisson, avec quatre nervures longitudinales.

La construction se fait en encorbellement; comme les appuis sur les piles sont réalisés au moyen de rotules, on a placé un appui supplémentaire en acier à 5 m de distance de l'axe de l'appui sur pile, pour avoir les deux appuis nécessaires pour la construction, avant que la travée ait trouvé son support définitif sur l'appui latéral.

Le système de précontrainte est réalisé par des câbles de 1/2" de diamètre, système Freyssinet.

A la fin du mois de Juillet on avait achevé la première moitié de l'ouvrage, et on est maintenant en train de partir de l'autre côté du fleuve (Fig.1).

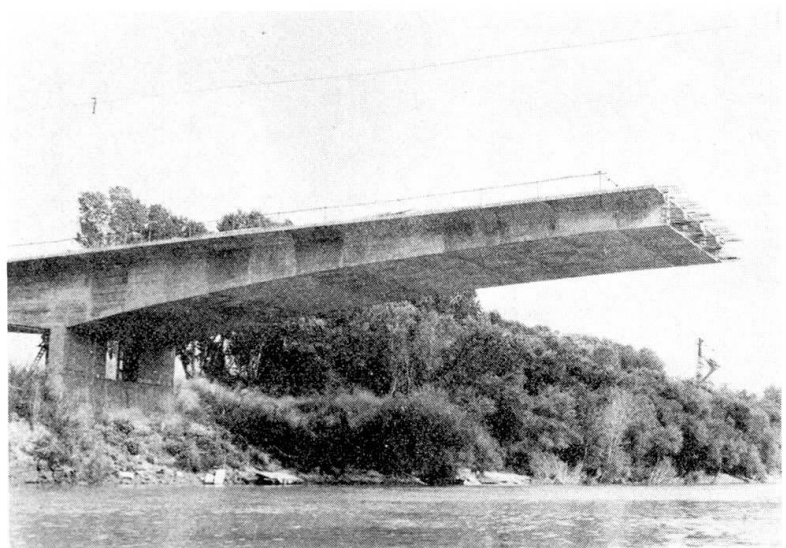


Fig. 1

La deuxième moitié de l'ouvrage va être bétonnée et précontrainte suivant

(*) Le pont est sur le Tibre à 30 km environ au nord de Rome et est actuellement en construction de part de l'entreprise Baldelli.

la même succession; le dernier voussoir va être collé en correspondance de la clé du pont à l'aide d'une résine epoxy à fin de permettre une précontrainte rapide et de éviter que les mouvements thermiques relatifs des deux consoles puissent en dommager un béton frais.

Programme de recherches

On a pour but de contribuer à la détermination expérimentale des paramètres suivants, nécessaires au calcul et à l'étude du comportement de structures de ce type:

- 1) rétrait local du béton dans l'ouvrage dans des conditions de température et humidité relatives variables mais connues;
- 2) fluage local du béton dû aux contraintes;
- 3) déplacements verticaux (flèches) conséquent aux effets de l'élasticité, du rétrait, du fluage, de la relaxation et de la température;
- 4) distribution et variation de la température à l'intérieur de l'ouvrage;
- 5) frottement dans les câbles.

Appareillage de mesure

Pour les mesures de déformation unitaire, on a renoncé à l'emploi de strain gages électriques, puisque les mesures doivent se prolonger deux ans au minimum et la stabilité du zéro ne peut pas être assurée, notamment dans les défavorables conditions ambiantales sur un ouvrage, hors du laboratoire.

On a ainsi préféré l'emploi d'une jauge mécanique de déformation, le Tenso-tast de Huggenberger, qui, en bonnes conditions peut assurer la mesure avec des erreurs qui ne dépassent pas les 5 millièmes de millimètre.

Puisque la grandeur des déformations locales à mesurer est de 100 à 1000 millièmes environ, et la base de mesure est de 100 mm au maximum (100 mm avec rallonge) on a préparé des rallonges en fer galvanisé fixées au béton; comme on voit à la figure 2: la mesure se fait entre deux repères dont l'un est fixé au béton et l'autre se trouve à l'extrémité d'une barre solidale avec l'autre extrême; la con-

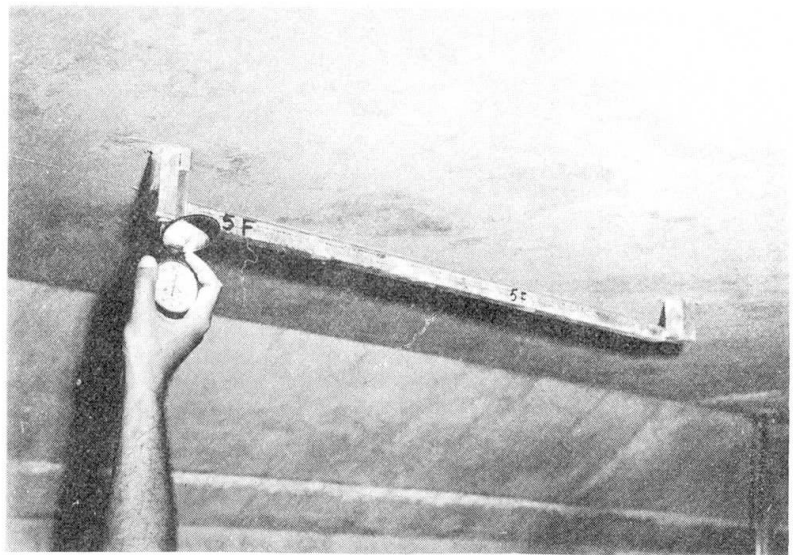


Fig. 2

nexion entre les deux parties est réalisée par un ressort laminaire en acier dur.

La base de mesure ainsi réalisée est de 1 m de longueur; on a la précision voulue, aussi bien que l'avantage de lire des informations en μ/m .

Evidemment il y a un certain effet de température, puisque la barre suit le changement plus rapidement que le béton; on a vu que cet effet est suffisamment petit.

Avec ces instruments, qui ont été perfectionnés avec des essais préliminaires en laboratoire, avec de très bons résultats, on mesure les grandeurs relatives aux points 1) et 2) ci-dessus.

Pour les mesures des flèches verticales on a employé deux techniques différentes, se contrôlant l'une l'autre.

La première, couramment employée dans les essais de ce type, est celle du nivellement de précision, soit sur l'extrados du tablier, par stade en invar, soit à l'intrados avec appuis fixe de l'instrument optique, et repères fixés à la semelle du tablier.

Avec ces mesures on a deux désavantages:

- on doit employer deux opérateurs, pendant plusieurs heures chaque fois;
- dans le temps qui est nécessaire à compléter un nivellement, on a vu que la déformation due aux changements de température peut atteindre plusieurs millimètres, surtout par beau temps, en plein soleil; de cette façon on perd une grande partie de la précision de mesure.

La deuxième technique a été mise au point exprès pour cette recherche et pourtant a elle même un certain intérêt expérimental.

On emploie des vases communicants à mercure (Fig.3) dont le niveau est observé soit à l'aide d'un jauge mécanique à centième de millimètre, soit à l'aide d'un instrument électrique à induction, qui commande par l'intermédiaire d'un amplificateur, un enregistreur à plusieurs canaux.

On a choisi d'employer un double système de mesure afin de contrôler, pendant la longue période des mesures, la dérive du zéro instrumental du système électrique, qui, quoique réduite dans les instruments de ce type, ne peut pas être considérée comme négligeable.

Le palpeur du jauge mécanique est abaissé à la main sur une plaque en perspex, flottante sur le mercure; on est sûr d'avertir le point de contact avec une erreur qui ne dépasse pas les 3-4 centièmes de millimètre, comme on a contrôlé préalablement en laboratoire.

La communication entre les différentes vases est réalisée par un tuyau en polyéthylène de 4 mm de diamètre interne, suspendu à la paroi d'une des nervures de l'ouvrage.

La mesure mécanique se fait en plaçant le jauge, à fur et à mesure sur les différentes vases; la position du zéro du jauge est contrôlée par des repères solides avec la paroi de l'ouvrage.

Une des vases est placé sur la pile, et l'on rapporte au niveau de celle ci, les mesures effectuées aux autres endroits.

La mesure de la température est obtenue à l'aide de thermocouples, et d'un enregistreur multiple.

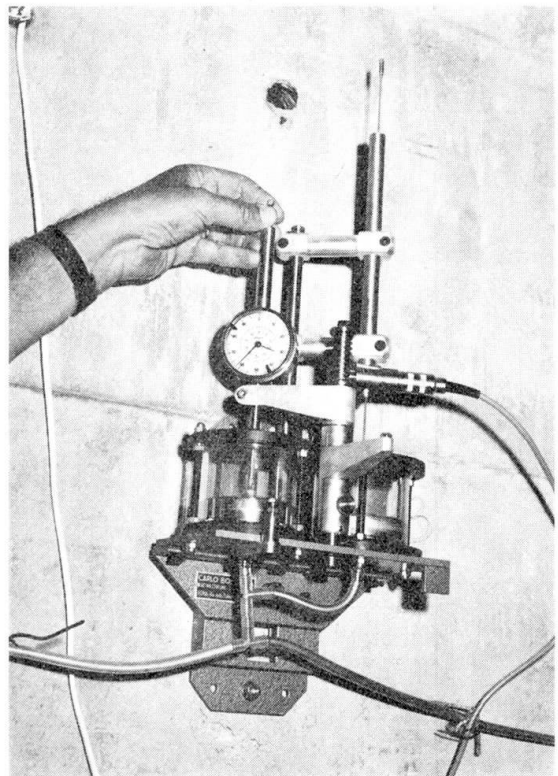


Fig. 3

Les thermo couples sont placées à l'intérieur du béton à trois centimètres environ des surfaces.

On mesure aussi l'humidité et la température de l'air à l'intérieur des caissons.

Les pertes des tensions dans les câbles, lors de la précontrainte initiale, ont été mesurées à l'aide de dynamomètres placés entre la poutre en béton et le verin aux deux extrémités des câbles.

Les dynamomètres ont été enlevés après mesure, et on a précontraint à nouveau les câbles.

La figure 4 représente le schéma du pont et la distribution des instruments.

Prémiers résultats

Dans les diagrammes qui suivent sont consignés des exemples des premiers résultats, obtenus depuis le commencement du bétonnage (janvier) jusqu'à l'achèvement de la première moitié de l'ouvrage (juillet).

La distribution des températures à l'intérieur du béton, mesurée pendant un mois dans plusieurs endroits de la même section, est bien loin de la linéarité sur la hauteur de la section; de ce qu'on peut conclure que les variations journalières de la température provoquent des contraintes non négligeables même dans des structures statiquement déterminées.

Les déplacements verticaux (flèches), pendant la construction, ont suivi suffisamment de près les valeurs théoriques de calcul; il y a quelques différences, que puissent s'expliquer, en partie, par le fait que les temps d'exécution ont subi des retards par respect à ceux prévus dans le projet.

De cela suit que les déformations visqueuses et de rétrait ont eu lieu pendant une plus longue durée; d'autre part, le béton était moyennement plus durci de ce qu'on avait prévu.

Les deux effets ne se compensent qu'en partie.

Les déformations locales de rétrait et fluage, compte tenu de celles élastiques, ont été généralement plus grandes de celle de calcul (de 20-25% environ).

Le fluage et le rétrait ont été calculés selon les recommandations du C.E.B. respectivement avec un coefficient $\varphi_{\infty} = 1,8$ et $\epsilon_{\infty} = 3.10^{-4}$.

Dans le diagramme 6 est consigné la variation des déplacements verticaux, enregistrée dans les deux jours plus chauds de la saison courante (5 et 6 août).

De ce diagramme on observe la grande valeur absolue de la déformation thermique journalière.

Les températures dans la section, qui étaient presque uniformes (25° C environ) au lever du soleil ont enregistré une valeur maximum de 42,5° C à l'extrados du tablier, à 16,30 heures.

A la même heure, la température de l'air à l'intérieur du caisson était de 31° C.

Le bulletin météo donnait 37° C de température maximum à Rome.

A 16^h15 la configuration déformée du pont était celle indiquée, avec une flèche de 42 mm à l'extrémité de la console.

Une telle flèche serait provoquée par une charge de 35 tonnes, concentrée à l'extrémité de la console.

De tout cela peut on déduire que, spécialement dans une structure hyperstatique, telle que va devenir la poutre une fois achevée, les contraintes dues à la variation journalière de température le long de la hauteur de la poutre sont assez importantes.

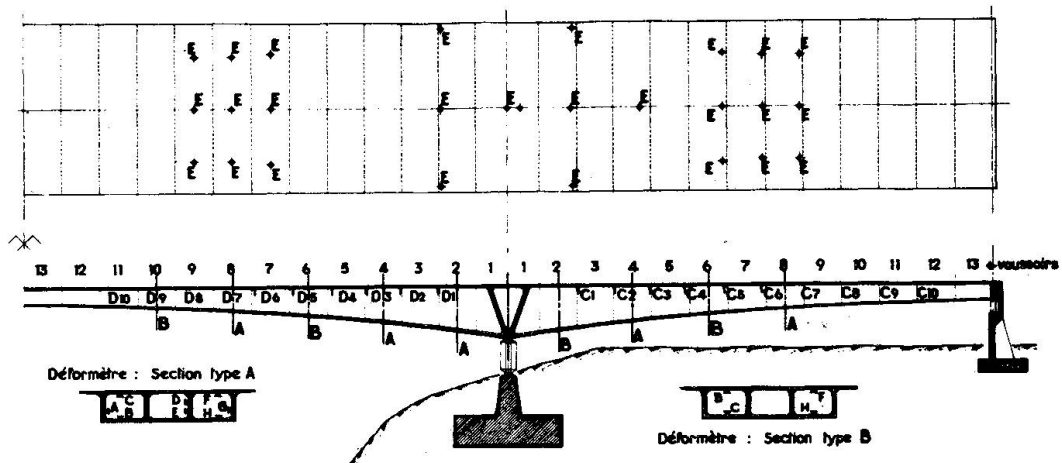


Fig. 4 - Bases de mesure du nivellement optique et schéma des bases pour déformètre

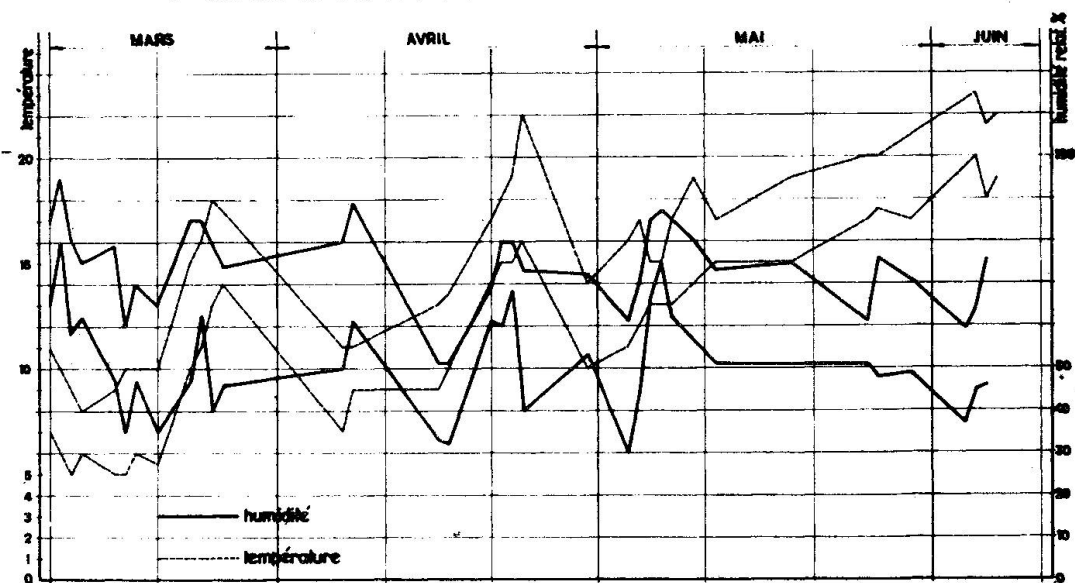
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Fig. 5 — Température et humidité relative maxima et minima journalières de l'air à l'intérieur de l'ouvrage

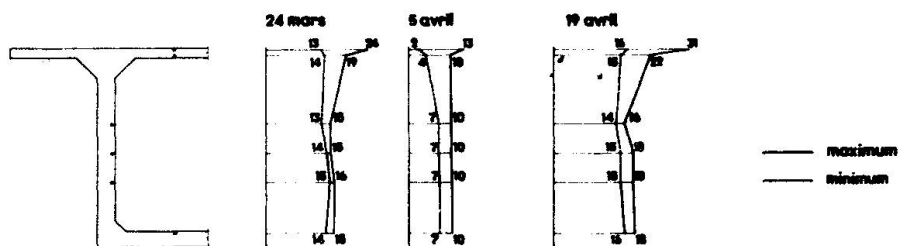
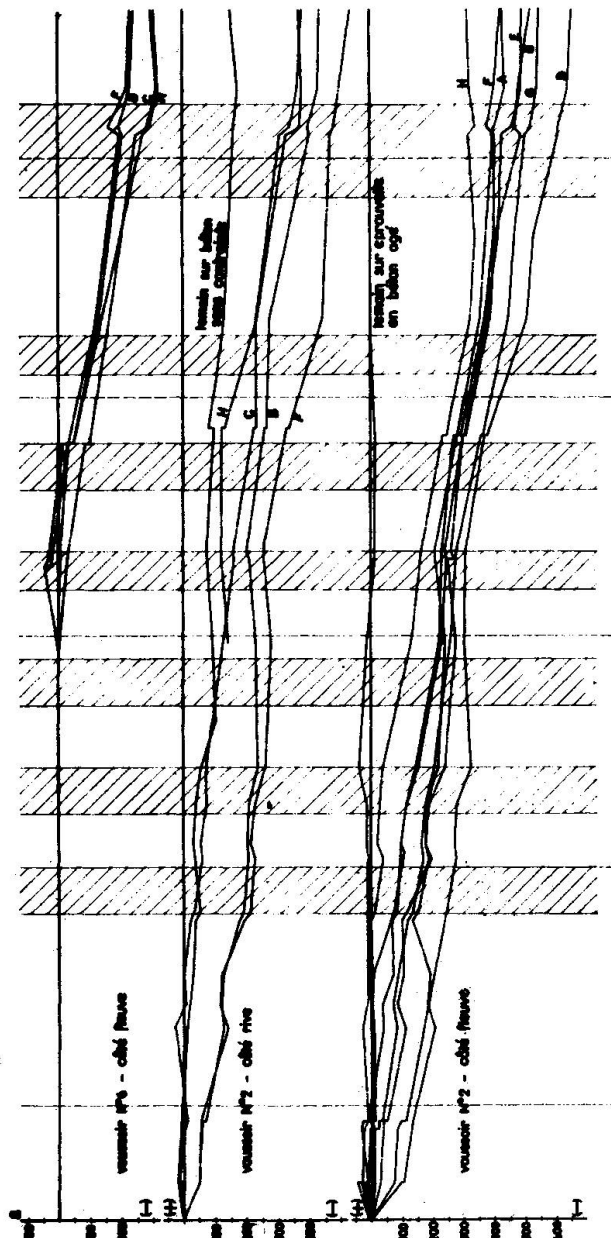
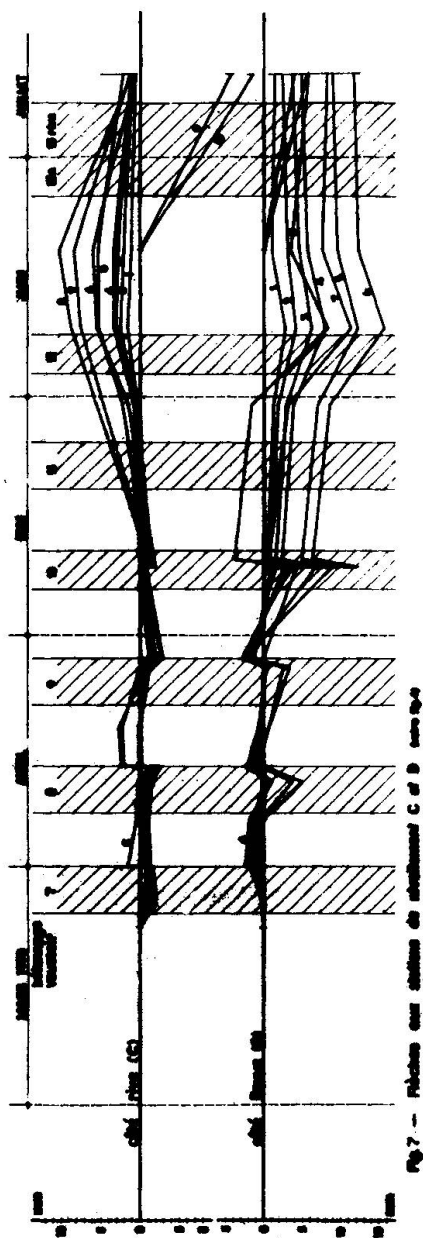


Fig.6 — Températures maxima et minima dans un jour, à l'intérieur du béton



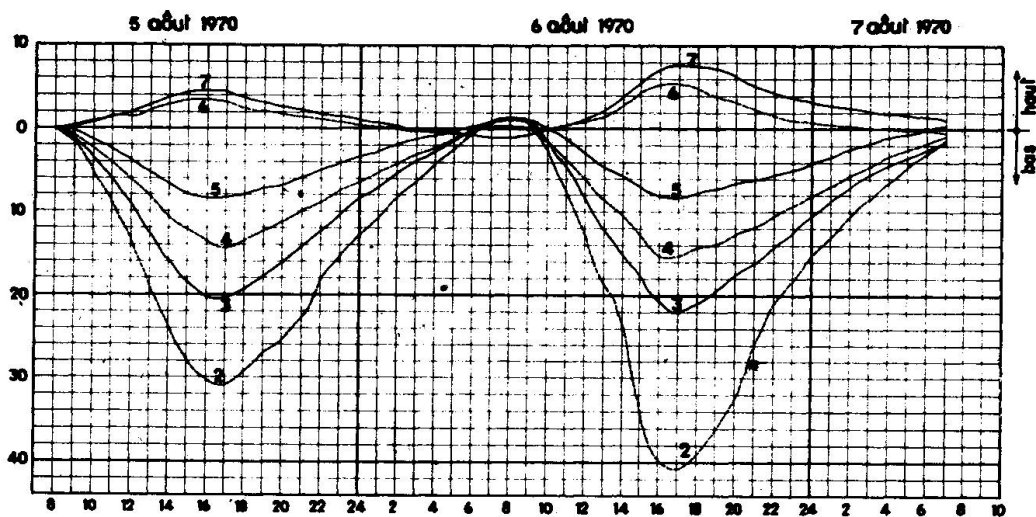


Fig. 9 — Flèches dues à la température les jours 5, 6 et 7 août 1970, enregistrées avec nivelles à mercure. (voir Fig. 11)

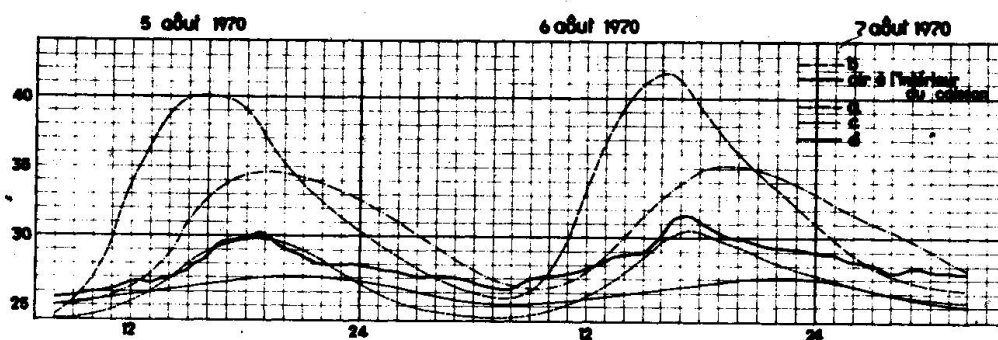


Fig. 10 — Températures à l'intérieur du béton les jours 5, 6 et 7 août (voir Fig. 11)

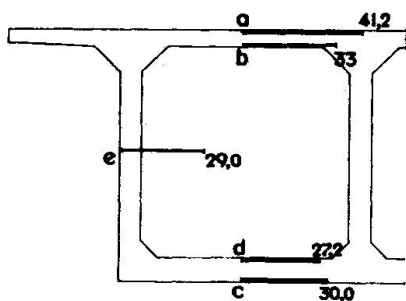
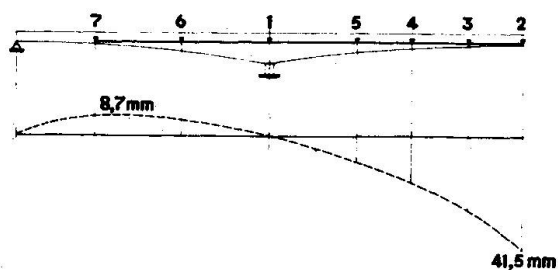


Fig. 11 — Distribution de la température à 16^h15 du 6 août



— Disposition des nivelles à mercure et flèches dues à la température (à 16^h15 du 6 août)

Les mesures de tension dans les câbles ont montré des pertes par frottement très élevées, dues surtout aux irrégularités de mise en oeuvre, conséquentes au système de construction; on a mesuré les valeurs suivantes:

- Câble longueur 74 m, tangente finale 5° : les pertes par frottement au milieu sont 17% respect à la valeur de mise en tension;
- Câble longueur 90 m, tangente finale 0° : les pertes sont 19%;
- Câble longueur 90 m, tangente finale 7° : les pertes sont 25%.

RESUME

On a mesuré les déformations unitaires, les déplacements verticaux, les températures à l'intérieur du béton pendant les premiers six mois de construction d'un pont en béton précontraint, sur le schéma de poutre continue à trois travées, bâti en encorbellement. Les déformations unitaires dues au rétrait et au fluage sont un peu plus grandes que celles prévues. On a remarqué la grande sensibilité de la structure aux distributions non uniformes de la température sur la hauteur de la section.

On décrit l'appareillage qui a été étudié pour ces mesures.

ZUSAMMENFASSUNG

Es wurden die Dehnungen, die vertikalen Verschiebungen und die Temperaturen im Beton während der ersten sechs Monate nach Fertigstellung einer im Freivorbau erstellten Spannbetonbrücke mit drei Feldweiten gemessen. Die Dehnungen, die durch das Kriechen und Schwinden verursacht werden, waren etwas höher als vorgesehen. Es wurde eine bedeutende Empfindlichkeit gegenüber nicht gleichmässigen Temperaturänderungen über den Querschnitt festgestellt.

Es werden im einzelnen die eigens für diese Forschungen entwickelten Instrumente beschrieben.

SUMMARY

Measurements of strain, temperature and vertical deflection were carried out for six months after the construction of a three span, prestressed concrete, cantilever constructed bridge. Strains resulting from creep and shrinkage of the concrete were slightly larger than the design values. The measurements were found to be particularly sensitive to non-uniform variations in temperature over the cross section.

Details of the special instrumentation are given.

Creep Effects in some Arch and Cantilever Bridges

Effets du fluage sur quelques ponts en arc et en encorbellement

Kriechwirkungen in einigen Bogen- und Freivorbaubrücken

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1 – INTRODUCTION

The techniques followed by the Laboratório Nacional de Engenharia Civil for the observation of structures, mainly bridges and dams, have been published at several instances (1 to 6). The present paper simply calls the attention to the techniques used for the measurement of strains and stresses, particularly taking into account the effects of creep.

For the accurate measurement of strains and stresses in reinforced and prestressed concrete structures acoustic strain gauges embedded in the concrete are used. These strain gauges have cylindrical body and heads and total lengths varying between 21 and 34 cm. They yield accuracies of the order of 5×10^{-6} . (4)

When measuring strains the main difficulty consists in distinguishing the part due to stresses from the part due to shrinkage and temperature. Also the evolution of the mechanical properties with time - creep effect - is in general difficult to follow and to interpret. In order to overcome this difficulty the strain gauges are embedded in the structure at three different conditions: i) active gauges, ii) compensating gauges and iii) controlled

gauges, fig. 1.

The active gauges measure the total strains in the structure. The compensating gauges are introduced inside double wall cylindrical boxes, built of thin copper sheet. Thus they are subtracted from the action of stresses. The controlled gauges are also inserted into similar double wall cylindrical boxes. However metallic cushions filled with oil placed at the top of the boxes allow to apply known stresses on the concrete cylinders where the strain gauges are embedded. By controlling the pressure of the oil in the cushion, mechanical tests of the concrete inside the structure, particularly creep tests, can be performed.

By the simultaneous use of an active and a controlled gauge it is even possible to perform a direct measurement of stresses, not involving the knowledge of the mechanical properties of the concrete. In fact, if the pressure of the oil in the cushion is varied in such a way that the strains indicated by both the active and the controlled gauges are permanently equal, the pressure applied corresponds without further corrections to the existing stresses.

In practice instead of varying the pressure on the controlled gauge it is preferable to apply a constant pressure, which corresponds to performing a creep test. This procedure, as compared with the usual technique of performing creep tests in the laboratory, has the great advantage that a very close similitude between the conditions around the active and the controlled gauge is automatically obtained. In this way, the creep corrections to be introduced in the active gauges can be very accurately computed. Thus the overall accuracy of the results is great, as shall be seen.

In order to reach this high accuracy, the thermal and hygrometric conditions around the active, the compensating, and the controlled gauges must be as similar as possible. Consequently, in the case of structures formed by walls, the cylindrical boxes indicated above are substituted by simple thermal and hygrometric insulations, as indicated in fig. 2.

2 – RESULTS OF TESTS

Figs. 3 to 5 show the results of creep tests performed in three bridges: two arch bridges and a prestressed cantilever bridge. These creep tests refer to controlled gauges and were run during periods varying from 500 to 2000 days.

The creep tests at Abreiro bridge were performed from 1956 to 1958, during 900 days, by applying a constant stress of 45 kgf/cm^2 , fig. 3.

The creep tests at Arrabida bridge were performed in 1962 and 1963, during 500 days, by applying a constant stress of 80 kgf/cm^2 , fig. 4. The measurement of strains and displacements is still being continued at the present date. However the creep tests were discontinued in 1963 due to rupture of the metallic cushions of the controlled gauges.

The creep tests at the north viaduct of the Tagus River bridge started in 1964 and are still in progress. The results presented refer to a period of about 2000 days. The stress applied is 100 kgf/cm^2 , fig. 5.

Fig. 6 indicates the results obtained by a group of strain gauges at section S of one arch of Arrabida Bridge. As can be seen the creep test was performed at a stress near to the one due to the applied loads.

The diagrams of the evolution of strains, shrinkage and creep in the prestressed viaducts of the Tagus River bridge are presented in (6).

Table I indicates for the different bridges: the mean value of the resistance of the concrete at 28 days, obtained by testing 20 cm cubes; the instantaneous modulus of elasticity; the creep coefficient corresponding to 500 days, φ_e ; and the 500 days modulus of elasticity, obtained by dividing the instantaneous modulus of elasticity by $1 + \varphi_e$.

The creep coefficient relates the strains due to creep ϵ_f , with the instantaneous ones due to the applied stress, ϵ_0 . Linear creep is thus assumed.

The number of compression tests performed, from which the indicated mean values were computed, varied from about 20 in the case of Abreiro bridge to several hundred in the case of Arrabida bridge.

It is interesting to compare the results presented in Table I with those that would be obtained by using the expressions recommended by FIP-CEB (7).

Assuming that the experimental modulus of elasticity, E_e , corresponds to the secant modulus, E_s , as defined by FIP-CEB, R12,222 and that this last one is 10% lower than the tangent modulus, E_t ; adopting expression

$$E_t = 21000 \sqrt{R'_b}$$

where R'_b is the resistance in cylinders expressed in kgf/cm^2 ; and further taking this resistance as 0.85 of the resistance in cubes R'_{bc} ; it comes

$$E_s = 0.9 \times 21000 \sqrt{0.85 R'_{bc}} = 17500 \sqrt{R'_{bc}}$$

The values of E_e , E_s and E_s/E_e are indicated in Table II.

For Arrabida and Tagus River bridges the computed values differ respectively plus and minus 8% from the experimental ones. In case of Abreiro bridge the computed value is 36% higher than the experimental one. It must be referred that in this case the modulus of elasticity considerably increased in time and reached the mean value of $2.7 \times 10^5 \text{ kgf/cm}^2$ at the age of 300 days.

As regards creep, FIP-CEB Recommendations give the creep coefficient ψ_t by the product of 5 coefficients

$$\psi_t = k_c k_d k_b k_e k_t$$

which, respectively, consider

- k_c - relative humidity of the air
- k_d - age at loading
- k_b - amount of cement and water/cement ratio
- k_e - fictitious thickness
- k_t - time and fictitious thickness.

For the cases under consideration these coefficients take the values indicated in Table II and lead to the values ψ_t also indicated in the same table. The agreement between computed and measured values is very good for Arrabida and Tagus River bridges and poor for Abreiro bridge. Attention is called to the fact that in this case the coefficient k_t (influence of

time) takes a very low value due to the great thickness of the arch, about 100 cm.

In all the cases, the stresses at different sections of the bridges were computed by using the results of the active gauges, duly compensated by the controlled gauges.

In Abreiro bridge, the accuracy with which it was possible to compute the stresses was fairly good. However, due to technique improvements this accuracy was considerably increased in Arrabida and Tagus River bridges.

For instance, in the case of Arrabida bridge the axial forces computed from the measured stresses satisfy the conditions of equilibrium with a mean error of 2% and maximum errors of about 4%. In this bridge a total of 214 gauges were used to determine stresses at 8 sections. It is interesting to note that due to creep there was an important evolution of the bending moments in the sections of the arches. The evolution in time of the displacements agrees with the measured evolution of strains.

As is well known, creep effects are very important in cantilever prestressed concrete bridges. The measurements performed at the Tagus River bridge allowed an understanding of the general behaviour of the structure and contributed to an accurate forecasting of the evolution of the cantilever displacements in time.

3 – CONCLUSIONS

For studying the behaviour of reinforced and prestressed concrete structures accurate measurement of strains and stresses is needed. The measurement of stresses is particularly difficult due to creep effects.

Thus, for the observation of structures, the simultaneous use of active, compensating, and controlled gauges is advisable, the last ones allowing to perform compression tests, particularly creep tests, in the structure, in just the same conditions as those under which the other gauges are operating.

The results on creep obtained at Arrabida arch bridge and at the prestressed cantilever viaduct of Tagus River bridge agree closely with the creep expression presented at the "FIP-CEB Recommendations". For Abreiro

bridge the agreement is much worse.

In these three bridges the measurement of stresses was performed along several years with high accuracy. The verification of equilibrium conditions allowed to check this accuracy.

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TABLE I

BRIDGE	28 DAYS MEAN RESISTANCE R_{bc} (kgf/cm ²)	INSTANTANEOUS MODULUS OF ELASTICITY E_e (kgf/cm ²)	500 DAYS CREEP COEFFICIENT φ_e —	500 DAYS MODULUS OF ELASTICITY E_{500} (kgf/cm ²)
ABREIRO	3.0 10 ²	2.2 10 ⁵	0.80	1.2 10 ⁵
ARRABIDA	5.6 10 ²	3.8 10 ⁵	0.75	2.2 10 ⁵
TAGUS RIVER	3.9 10 ²	3.7 10 ⁵	1.72	1.4 10 ⁵

TABLE II

	MODULUS OF ELASTICITY			CREEP COEFFICIENT		
	MEASURED E_e (kgf/cm ²)	COMPUTED E_s (kgf/cm ²)	$\frac{\text{COMPUTED}}{\text{MEASURED}}$ E_s/E_e	MEASURED φ_e	COMPUTED $k_c k_d k_b k_e k_t = \varphi_t$	$\frac{\text{COMPUTED}}{\text{MEASURED}}$ φ_t/φ_e
BRIDGE			—	—	—	—
ABREIRO	2.2 10 ⁵	3.0 10 ⁵	1.36	0.80	2.3 × 0.8 × 1.0 × 0.7 × 0.2 = 0.3	0.4
ARRABIDA	3.8 10 ⁵	4.1 10 ⁵	1.08	0.75	2.1 × 0.7 × 0.9 × 0.75 × 0.7 = 0.7	0.93
TAGUS RIVER	3.7 10 ⁵	3.4 10 ⁵	0.92	1.72	2.3 × 1.0 × 1.4 × 0.75 × 0.7 = 1.7	0.98

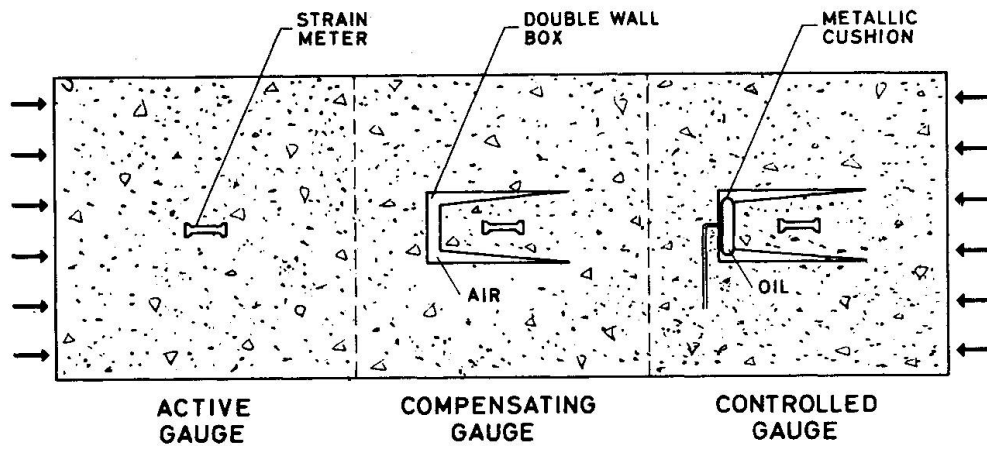


Fig. 1

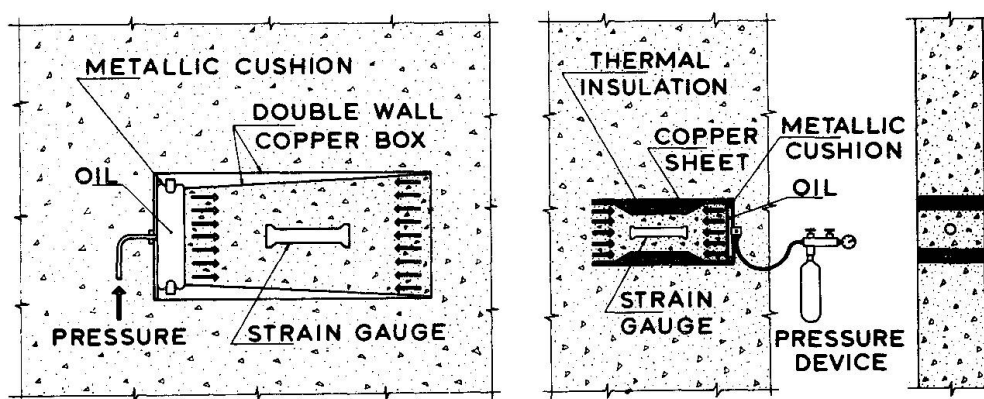


Fig. 2

ABREIRO BRIDGE

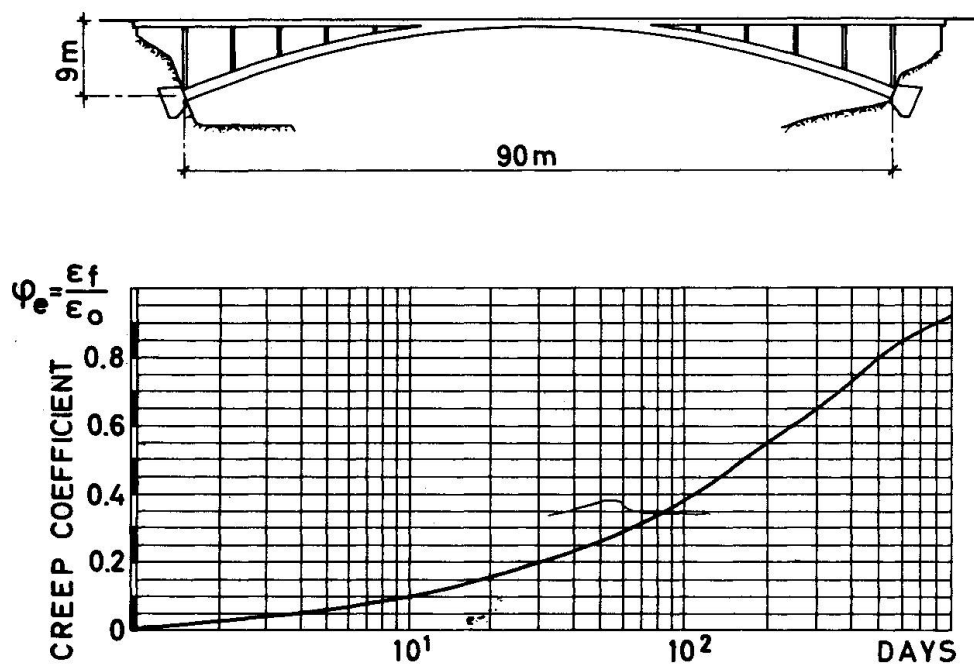


Fig. 3

ARRABIDA BRIDGE

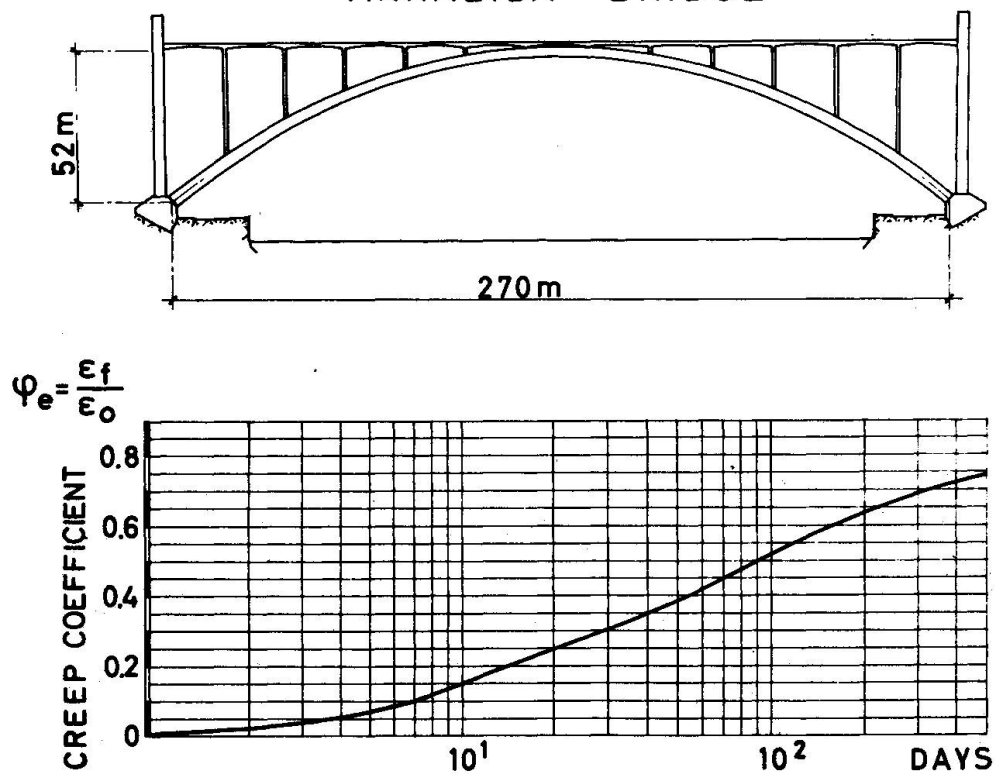


Fig. 4

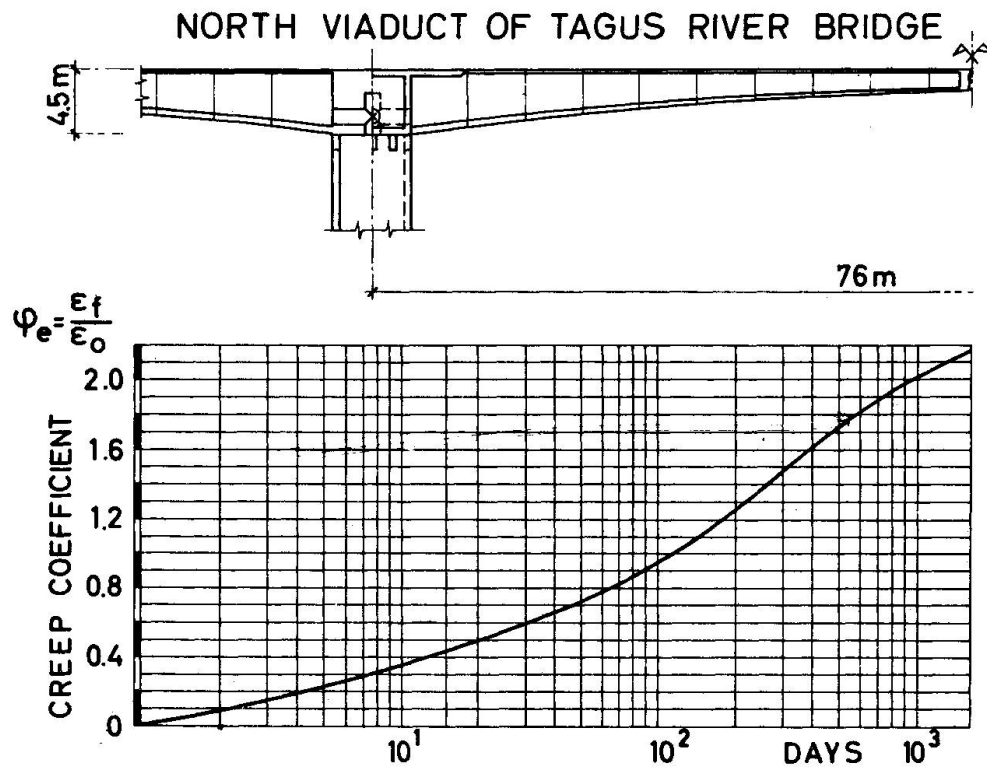


Fig. 5

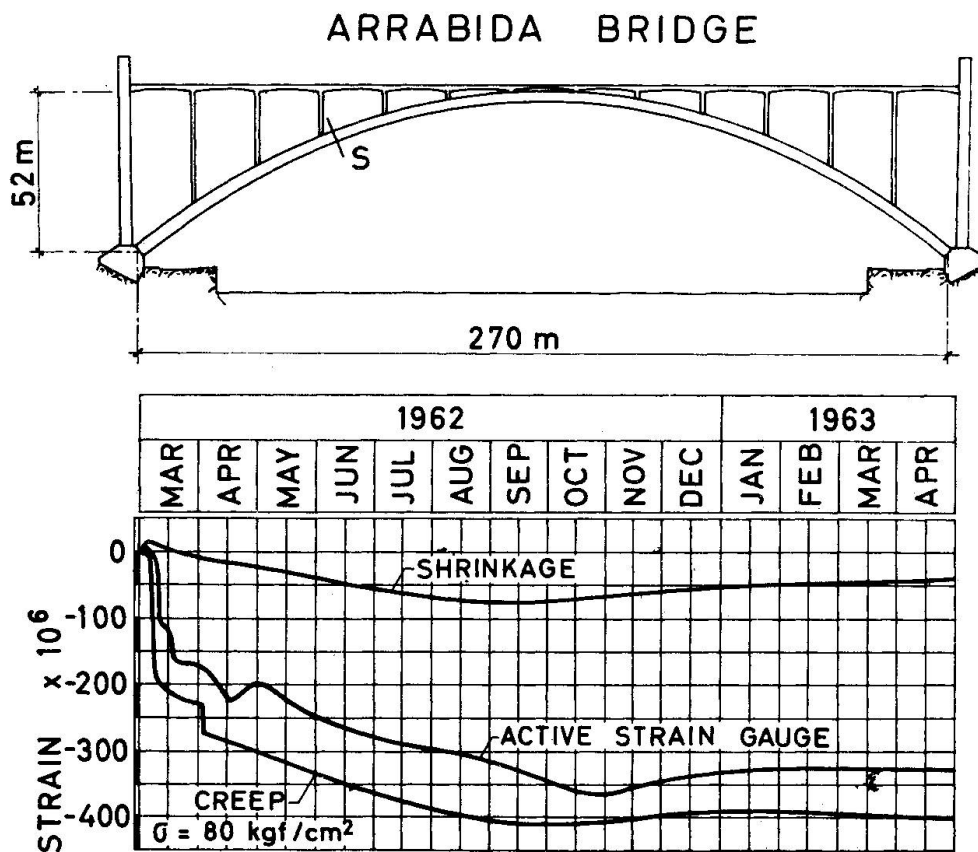


Fig. 6

SUMMARY

The techniques followed at LNEC, Lisbon, for the in situ measurement of strains and stresses in reinforced concrete structures are briefly described.

The results of creep tests performed in three bridges are presented and interpreted.

Attention is called to the advantages of performing creep tests in the bridges using controlled gauges.

RESUME

On décrit sommairement les techniques suivies au LNEC pour mesurer in situ les déformations et les contraintes des ouvrages en béton armé.

Les résultats des essais de fluage effectués sur trois ponts sont présentés et interprétés.

On souligne les avantages d'effectuer des essais de fluage à l'aide d'extensomètres contrôlés.

ZUSAMMENFASSUNG

Es werden die Verformungs- und Spannungsmessungstechniken kurz beschrieben, die im LNEC, Lissabon, für Ortsmessungen angewandt werden.

Die an drei Brücken durchgeführten Kriechversuche werden dargestellt und ausgewertet.

Es werden auch die Vorteile hervorgehoben, die sich aus mit kontrollierten Dehnungsmessern durchgeführten Kriechversuchen ergeben.

Measurement Results on Behavior and Effect of Temperature Change on R.C. Structure

Résultats des mesures du comportement d'une structure en béton armé, et influence du changement de température

Meßergebnisse über Verhalten und Wirkung von Temperaturwechseln in Stahlbetonbauten

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Introduction

This report comprises the results of measurements and analyses of temperature changes and behavior (movement) made on the roof slab and frame of the General Testing Laboratory of Ohbayashi-Gumi Engineering Research Institute.

Construction of the laboratory was completed in December 1965. Measurements of changes in temperature distribution and behavior of this building were commenced in May 1967 and are being continued even now.

At that time the measurement were started, there were frequent occurrences of trouble with waterproofing on roof slabs and judging it of value to conduct investigations on actual building in order to eliminate this trouble, it was decided to carry out the temperature change and behavior measurements described. Although emphasis was laid on measurements at only. The roof slab in the beginning, it was realized that a solution could not be obtained unless the accompanying behavior of the frame was known, then a change was made mid-way in the program to include investigations of the frame.

The Institute is situated at suburbs of Tokyo at lat. $35^{\circ}47'N$ and long. $135^{\circ}03'E$, approximately 40 km northwest from the city center. Photo.1 shows a bird's eye view and Fig. 1 is the layout of the laboratories which is at a location where urbanization has not progressed relatively. As can be seen from these illustrations there is not very much shading by buildings and trees, and especially with regard to the building under observation the sun shines evenly on it with results showing a uniform amount of sunlight on the surface of the roof slab, a condition found extremely good for carrying out measurements.

The building on which measurements are being made is the General Testing Building A in Fig. 1, divided into four sections by expansion joints. Measurements reported herein were performed on the block marked . Briefly described, the building consists of precast concrete curtain walls (SHOCKBETON), floor slab concrete finished with vinyl acetate resin coating and ceilings of plaster board finished with paint.

The roof waterproofing consisted only of a coating about 0.5 mm thick of chlorosulfonated polyethylene (hypalon) which was considerably deteriorated at the time measurements were begun so that concrete was in a condition close to total exposure.

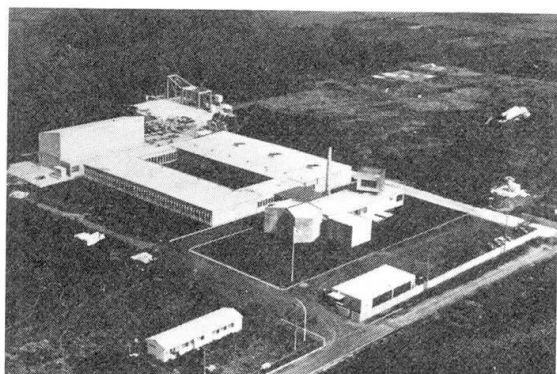
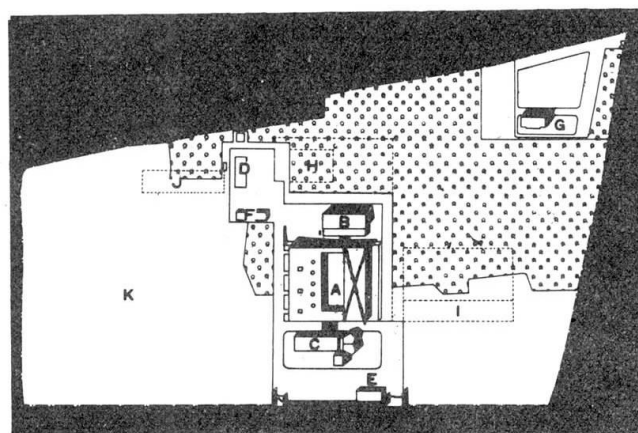


Photo. 1 Bird's-eye view

Fig. 1
General layout

- | | | |
|--|--|--|
| A General testing building | D Multi-purpose testing table | H Large scale structure testing building |
| B Structural strength and vibration testing building | E Garage and guard's room | I Administration building |
| C Acoustic and air conditioning testing building | F Construction method & equipment testing room | J Workshop |
| | G Apartment house for staff families | K Outdoor testing area |

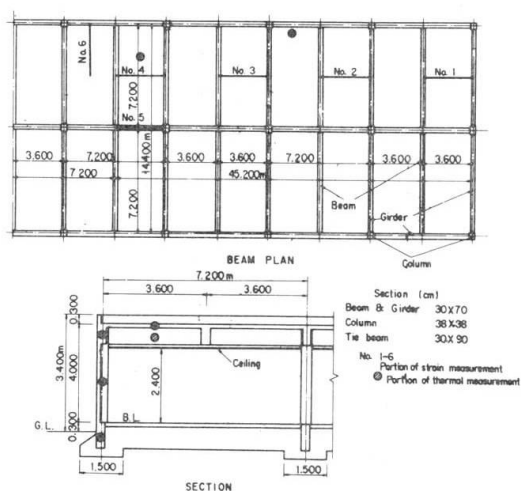


Fig. 2 Beam plan and section.

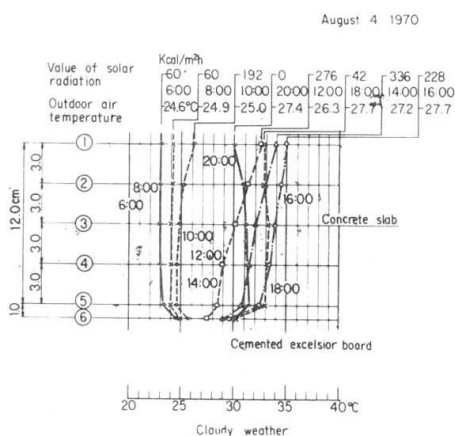
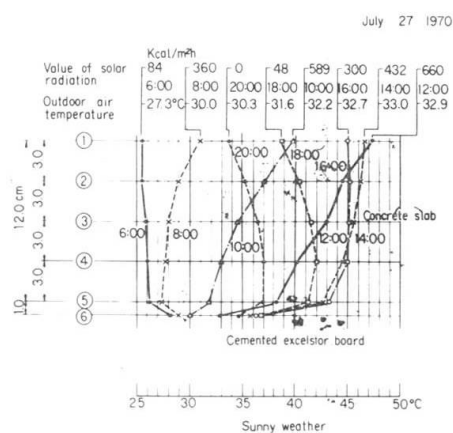
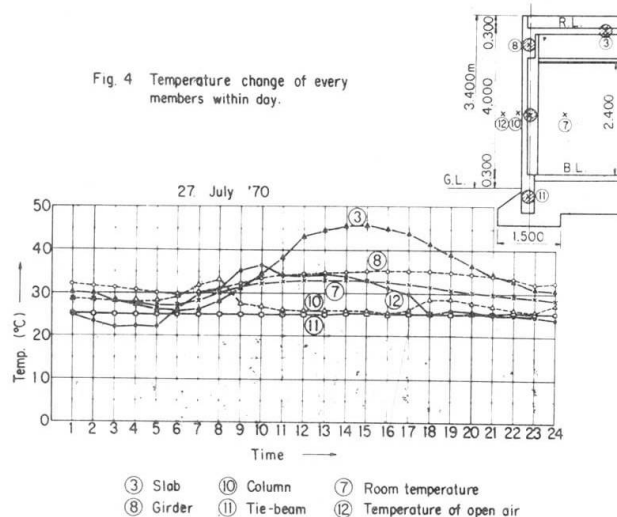


Fig. 3 Temperature change of roof slab within day at every time.

Heating and cooling of the interior are carried out with package type air conditioning units placed at several locations within the building.

Results of Measurements of Temperature Distribution and Behavior

Fig. 2 gives the beam plan and section of the building in which locations of temperature and strain measurements respectively are indicated.

(A) Measurement of temperature distribution change.

(1) Roof slab temperature change within day.

Examples of the conditions of change at the surface and concrete in the roof slab for sunny and cloudy weather are in Fig. 3. Temperature changes are indicated for every two hours, the min. temperature found to occur at around 6 a.m, heat being stored until max. are reached at 2 to 4 p.m. It should be noted that cemented excelsior boards are fixed to the underside of the roof slab of this building.

(2) Temperature changes of roof slab, beam, column and tie-beam within day.

The temperature changes at the centers of the various members are indicated in Fig. 4. In summer according to this figure, tie-beam (11) shows no change while Column (10) to the east side is struck by sunlight from around 5.00 to 6.00 A.M. so that its temperature is raised. From 8 a.m. the temperature drops (approximately 7°C) due to air-conditioning in the interior and is maintained evenly until around the time (5 p.m.) when cooling is shut off after that the temperature begins to rise again. Beam (8) is not affected by cooling, but its temperature difference with tie-beam being approximately 5 to 7°C while the difference with the center of the slab is approximately 10°C.

These temperatures were noted with automatic recorders connected by lead wires to thermocouple and the thermocouple sheet (copper-constantan) embedded in holes drilled into concrete at the various locations indicated in Fig. 3 and Fig. 4.

(B) Measurement of strain change and its results.

The places at which strain change was measured were the 6 locations numbered 1 through 6 in Fig. 2. As can be seen from the beam plan there are beams at the middle of girders in span direction, the intervals being 3.60 m while the spacing between girders are 7.20 m, there being 7 spans. The plan is a rectangle 50.40 m (ridge direction) x 14.40 m (span direction).

Although displacement should be measured for girder span, since it was difficult to obtain 16 fused-quartz glass tubes for 7.20 m long at that time, measurements initiated to made on 3.60 m that is one-half of span.

Details of device of strain measurement are given in Fig. 5. Photo. 2 shows the manner in which this apparatus is installed. In this method, quartz glass tube with a measurement length of 3.60 m is used with other end fixed and another end free, a differential transformer being mounted to the free end through which au-

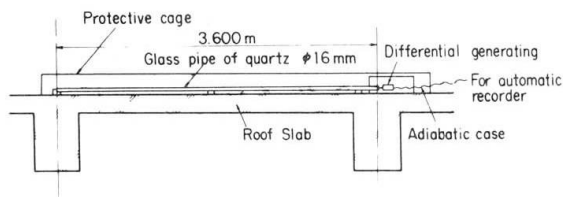


Fig. 5 Device of strain measurement

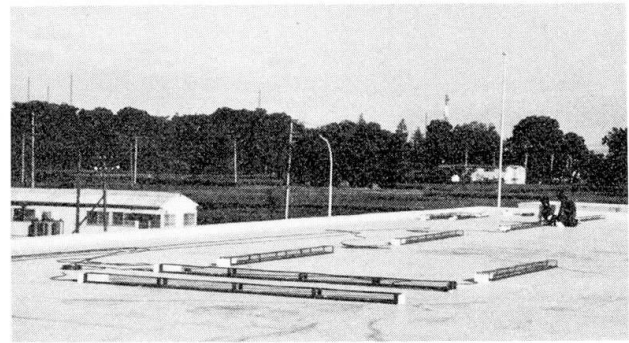


Photo. 2 The whole view of strain measurements

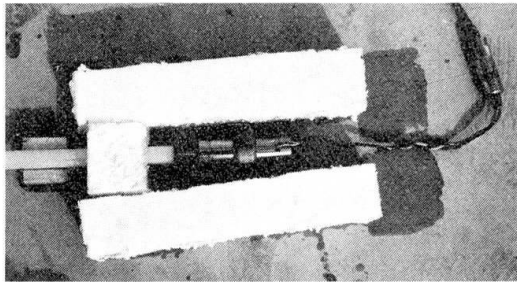


Photo. 3 Insulating case and differential transformer

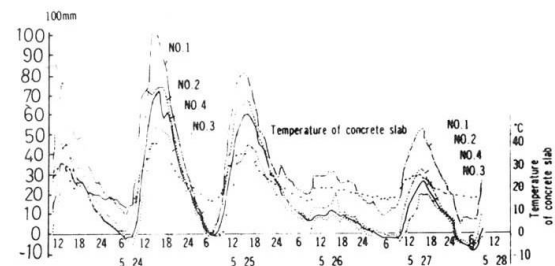


Fig. 6 Strain changes between girders. 24 ~ 28 MAY. 1968

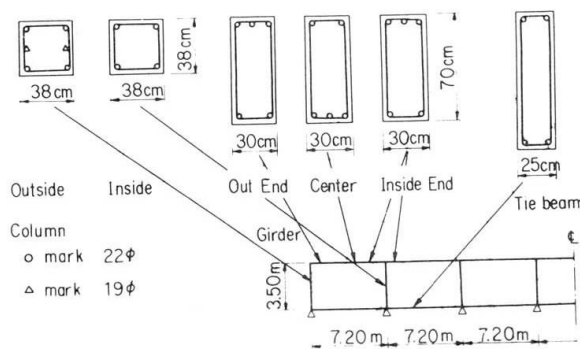


Fig. 9 Section List.

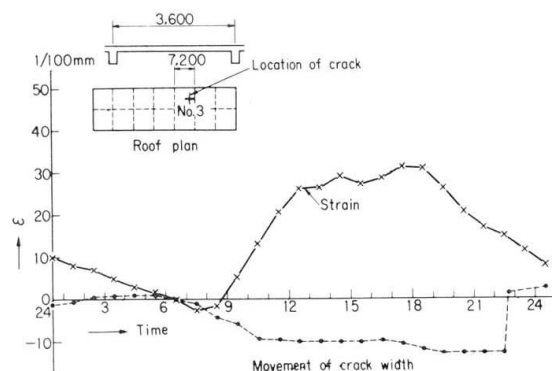


Fig. 8 Behavior on strain of bay between girders and width of crack.

Table 1. Typical values (dI & ΔI) at 6:00 and 16:00 o'clock on the every season (1968~1969)

Season Location Time	SPRING (1968) (25 MAY)				SUMMER (1968) (1 AUG.)				AUTUMN (1968) (1 NOV.)				WINTER (1969) (15 JAN.)			
	dI mm		ΔI mm		dI mm		ΔI mm		dI mm		ΔI mm		dI mm		ΔI mm	
	16.00	6.00	16.00	6.00	16.00	6.00	16.00	6.00	16.00	6.00	16.00	6.00	16.00	6.00	16.00	6.00
NO. 1	0.07	-0.79	1.62	0.88	0.14	1.48	-0.34	-1.08	1.48	-0.81	-1.43	1.24				
NO. 2	0.07	-0.68	1.50	0.69	0.21	0.96	-0.30	-0.95	1.30							
NO. 3	0.54	0	1.08													
NO. 4	0.02	-0.67	1.38	0.67	0.25	0.84	-0.40	-0.92	1.04	-0.78	-1.18	0.80				
NO. 5				0.64	0.25	0.78	-0.03	-0.54	1.02	-0.48	-0.85	0.74				
NO. 6				0.90	0.19	1.42	-0.42	-1.12	1.42	-0.92	-1.46	1.08				
Temp. °C	36.7	14.1		42.7	26.2		22.0	7.6		10.5	-2.0					

Time of measured standard is am. 8.00 25 July 1968 and temperature is 28 °C.

The dI is values of actual measurement for $L=3.60$ m.

The ΔI is displacement between one day in the case of calculated values as $L=7.20$ m

The temperature is position into concrete below 6cm from surface.

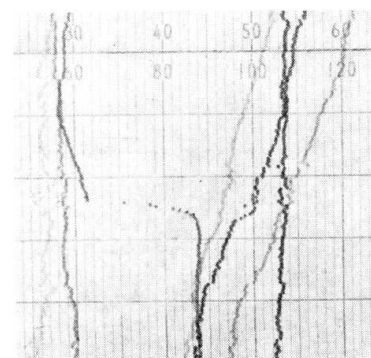


Photo. 4 Crack width of opening up by recording paper

tomatic recording is carried out. Since the differential transformer is easily affected by heat, an insulating case of urethane is provided around it (see photo. 3) to eliminate the influence of heat while moreover a dummy is provided for making corrections.

Fig. 6 gives the results of measurements made on the locations from No. 1 through No. 4 employing this apparatus. It can be seen from this figure that there are great changes in strain at the outer end portion. The temperature changes at the center of the concrete roof slab also recorded showing strain to vary accompanying temperature changes at that time.

In this amount of strain, since five years have elapsed from the time of completion of the building, minute cracks have been formed and all factors such as drying shrinkage, creep and thermal behavior are needless to say contained in the figures.

Temperature changes and strain changes for 10-day periods of each season (summer, autumn, winter) in illustrated form are as given in Fig. 7. Representative values by season taken from this figure and sought as behavior instead of strain quantities at 0600 and 1600 hours are shown in table 1.

(C) Measurement of crack width.

Setting the matter of crack formation lay aside, the behavior in widths of cracks already formed were measured. Crack were found to be around 0.5 mm in width. The location of the crack is shown in Fig. 8. Strain measurement apparatus were installed astride these cracks with gage lengths of 10 cm and the behavior recorded. As a reference, the strain in Span No. 3 was also measured. The behavior of the two are given in Fig. 8. From this it is seen there is no effect on span strain although crack width is contracted when the concrete is expanded. During seasons of daytime outdoor of above 30°C the crack try to open up in the night-time (around 2100 to 2200 hours) when heat is being released, but are resisted by bond of reinforcing steel so that an equilibrium condition is maintained — when the equilibrium can no longer be maintained the phenomenon of an instantaneous opening up is seen. Although the amount of this opening up is about 0.12 mm it is believed local stresses are of considerable magnitude in comparison with the length of the opening measured. Photographic evidence of this record is shown in Phto. 4.

Analyses by Thermal Stresses of Frame

Calculations based on thermal stresses were made for the time being assuming the frame to be elasticity. The cross sections of the various members are indicated in Fig. 9. Based on these and assuming a difference of $\pm 30^{\circ}\text{C}$ with the standard temperature, stress and deformation are sought as shown in Fig. 10, where $E = 2.1 \times 10^5 \text{ kg/cm}^2$ and coefficient of thermal expansion of concrete $\alpha = 1.2 \times 10^{-5} \text{ m/m}^{\circ}\text{C}$. As a reference, the stress employed in designing is also given, the calculation being made for seismic coefficient of $K = 0.2$.

From the results it is seen that thermal stresses are large when there is a temperature difference of 30°C.

Next, for crack formation at outer columns, inner columns and outside and inside ends of beams, reinforcing steel strain of 0.1%, and at yield of reinforcing steel, the bending moments and curvature of the various members were sought and are given as shown in Table 2. The relationships of bending moment with curvature

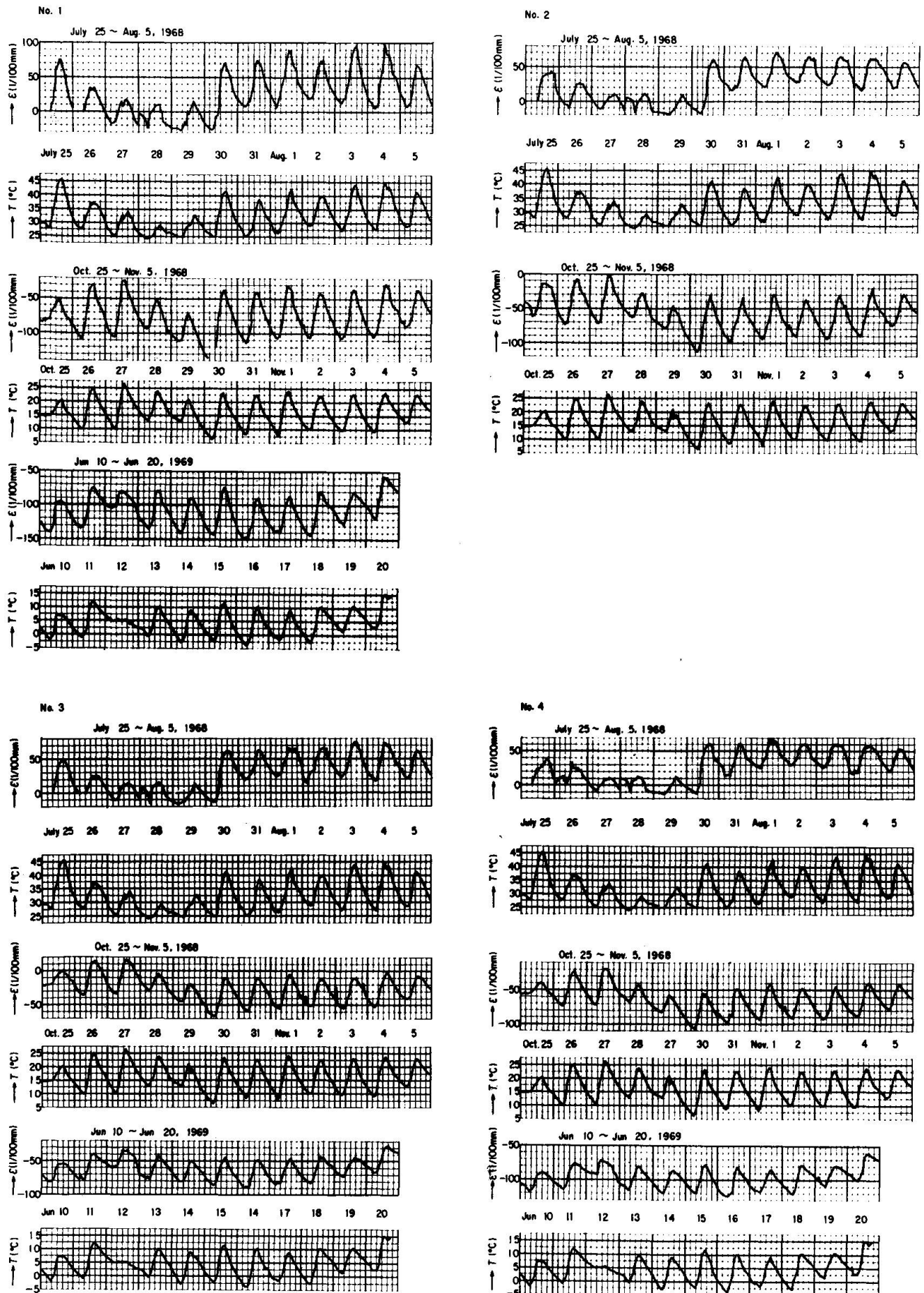


Fig. 7 Strain change of strain and temperature change in slab between seasons.

are indicated in Fig. 11. From these it is seen there is slight lowering of rigidity in columns, but a tendency for considerable lowering in beams.

From the calculation results it is seen that the first cracks are produced at the foots of columns, and the temperature difference sought from these calculated values is 9.3°C . In other words, this frame will enter the plastic range if a temperature difference of more than 9.3°C occurs and the stresses in Fig. 11 will be more or less changed.

The assumptions used in the calculations were:

Yield point of reinforcing steel:	$s\sigma_y = 3,000 \text{ kg/cm}^2$
Yield strain of reinforcing steel:	$s\varepsilon_y = 0.142\%$
Young's modulus of reinforcing steel parallel to ε axis after yield:	$sE = 2.1 \times 10^6 \text{ kg/cm}^2$
Concrete strength:	$c\sigma_b = 200 \text{ kg/cm}^2$
Strain of concrete:	$c\varepsilon_b = 0.2\%$

Closure

This study was made to seek stresses from elastic analyses stressing actual measurements with cross-sectional properties of the various members examined for elastoplasticity. In future studies it is necessary for elastoplasticity of the frame to be investigated for further clarification.

On the other hand, from the above results, when designing is carried out for waterproofing of roof slabs where heat is transmitted directly down to concrete it becomes necessary for stress analyses to be made considering the thermal properties anticipated of the various members of the frame:

As a method of waterproofing, it is observed that the temperature of concrete is maintained at a relatively constant level throughout the year if insulating material is inserted between the waterproofing and concrete and this is shown in Fig. 12. For comparison purposes, a case of rubber sheet waterproofing with no insulating material is also indicated.

In order to understand basic matters, besides drying shrinkage tests of concrete conducted in the laboratory, it is strongly felt that gaining a grasp of the physical properties of concrete members under various circumstances applying temperature factors corresponding to outdoor and indoor and indoor temperatures to concrete specimens is a matter of great importance.

In closing, the authors wish to express their deepest gratitude to Prof. H. Umemura of the University of Tokyo for his guidance in conduction of the tests.

Reference

Kiyoshi Muto: Plastic Design of Reinforced Concrete Structures, Vol. II, Earthquake-Resistance Design Series, Maruzen Co. (in Japanese)

Table - 2

Occasion	Member	Outside Column ($\sigma_0=6.2$)	Inside Column ($\sigma_0=10.4$)	Girder ($\sigma_0=0$)
When crack occurred concrete	Bending moment M : ton-m	4.3	4.3	9.1
	Resistant moment $C = \frac{M}{bd^2}$ kg/cm ²	7.9	7.9	6.2
	Curvature $\frac{1}{\rho}$	0.036×10^{-2}	0.038×10^{-2}	0.031×10^{-2}
When strain of reinforced bar developed 0.1 %	Bending moment M : ton-m	11.1	10.4	15.3
	Resistant moment $C = \frac{M}{bd^2}$ kg/cm ²	20.2	19.0	10.4
	Curvature $\frac{1}{\rho}$	0.145×10^{-2}	0.146×10^{-2}	0.134×10^{-2}
When reinforced bar reached yield	Bending moment M : ton-m	13.6	12.5	21.7
	Resistant moment $C = \frac{M}{bd^2}$ kg/cm ²	24.7	22.8	14.8
	Curvature $\frac{1}{\rho}$	0.204×10^{-2}	0.205×10^{-2}	0.188×10^{-2}

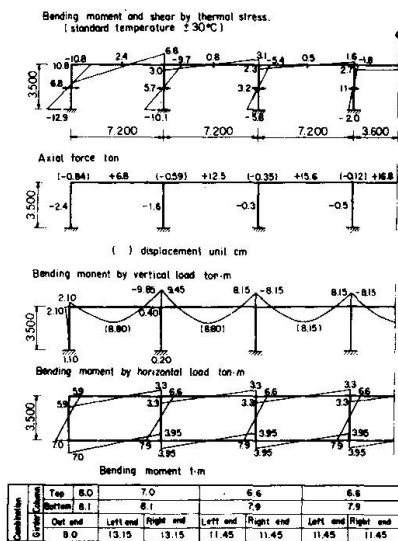
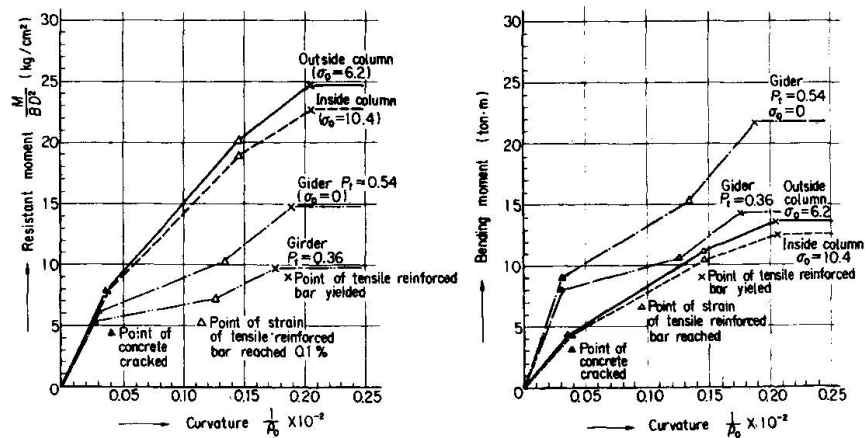
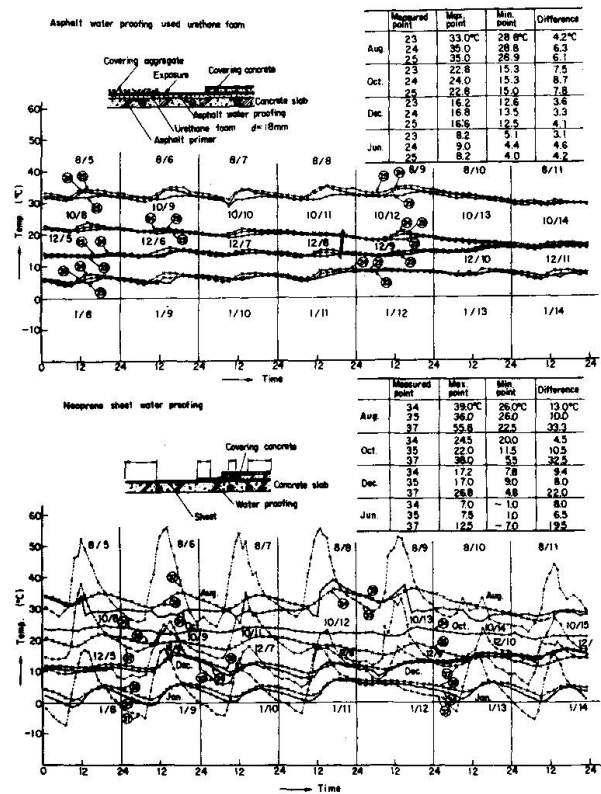


Fig. 10 Stress diagram.



SUMMARY

The behavior of expansion and contraction of a reinforced concrete building have been measured from May 1967 and is being continued still now. The results of observations showed that behavior of R. C. structure was related to the temperature changes in concrete, and the instantaneous increasing of crack width was observed in night-time especially when outdoor temperature of daytime raised more than 30°C. Some other analysis were examined on the observed data.

RESUME

On a effectué des mesures d'extension et de contraction sur un bâtiment de béton armé à partir de mai 1967. Les résultats des observations montrent que le comportement de la structure de béton armé est lié à la variation de la température dans le béton. En particulier, quand la température de l'air extérieur dépassait 30°C de jour, on a observé de nuit que la largeur de la fissure augmentait. On a tenté l'analyse des résultats des mesures.

ZUSAMMENFASSUNG

Seit Mai 1967 wird das Verhalten der Ausdehnung und des Zusammenziehens eines Stahlbetongebäudes gemessen. Die Beobachtungen zeigten, dass das Verhalten des Stahlbetongebäudes in Beziehung zur Temperaturänderung im Beton stand. Insbesondere wurde eine Zunahme der Rissweiten in der Nacht beobachtet, wenn die Aussen-temperatur tagsüber über 30°C gestiegen war. Weitere Untersuchungen aufgrund der beobachteten Werte wurden vorgenommen.

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Measurements and Observations of Creep and Shrinkage in Prestressed Reinforced Concrete Bridges

Mesures et observations des effets du fluage et du retrait sur les ponts en béton précontraint

Kriech- und Schwindmessungen sowie -beobachtungen an Spannbetonbrücken

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1. Introduction

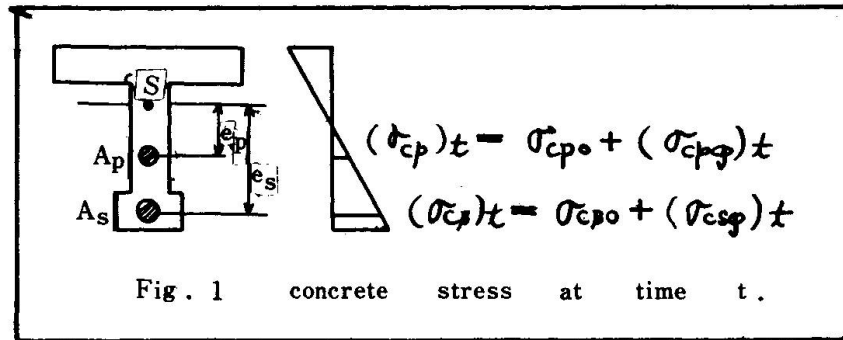
The "Prestressed Reinforced Concrete (PRC)" system ⁽¹⁾ is defined as a system in which relatively small prestressing forces are introduced into reinforced concrete (RC) to reduce the opening of cracks. In this case, the RC is designed to have the required safety factor against collapse. The conditions to be met for tolerable crack opening in PRC is the same as that in RC. In other words, PRC is a partially prestressed concrete with steel reinforcements.

The term "Prestressed Reinforced Concrete (PRC)" was proposed by the senior author's team in 1961 to distinguish PRC from the full or limited prestress system used to date. The main difference between the two systems is as follows; the basic system of PRC is reinforced concrete while that of PC is plain concrete in the absence of prestressing forces.

In this paper, the behaviors of creep and shrinkage in PRC system is discussed on the basis of the proposed method of analysis and measurements on Kamihimekawa bridge which was designed as the proposed system.

2. Derivation of the Equations for Creep and Shrinkage Stresses in PRC system

The following equations are obtained for creep and shrinkage stresses in PRC system which section has comparatively large sectional area of tendon.



Concrete stresses at time t are shown in Fig. 1 and concrete strain due to creep and shrinkage at the position of reinforcement for dt time can be expressed by

$$\{(\sigma_{cs})t/E_c + w/\varphi\} dy$$

On the other hand, the axial stress due to the restraint of reinforcements can be obtained as follows:

$$-\{(\sigma_{cs})t + wE_c/\varphi\} dy$$

In the similar manner, the following equations can be derived at the position of tendon:

$$\left. \begin{aligned} d(\sigma_{cs})t/dy + \alpha_{ss}\{(\sigma_{cs})t + wE_c/\varphi\} + \alpha_{sp}\{(\sigma_{cp})t + wE_c/\varphi\} &= 0 \\ d(\sigma_{cp})t/dy + \alpha_{ps}\{(\sigma_{cs})t + wE_c/\varphi\} + \alpha_{pp}\{(\sigma_{cp})t + wE_c/\varphi\} &= 0 \end{aligned} \right\} (1)$$

where $\alpha_{ss} = nA_s/A_i(1 + e_s^2/\rho_i^2)$, $\alpha_{sp} = n_pA_p/A_i(1 + e_se_p/\rho_i^2)$

$$\left. \begin{aligned} \alpha_{ps} &= nA_s/A_i(1 + e_se_p/\rho_i^2), \quad \alpha_{pp} = n_pA_p/A_i(1 + e_p^2/\rho_i^2) \end{aligned} \right\} (2)$$

where ρ_i is radius of gyration about A_i .

Substituting the initial conditions that $(\sigma_{cs})_{t=0} = \sigma_{cs0}$

and $(\sigma_{cp})_{t=0} = \sigma_{cp0}$ into Eq. (1)

$$\left. \begin{aligned} \sigma_{cs} &= K_1 e^{Ay} + K_2 e^{By} - wE_c/\varphi, \quad \sigma_{cp} = K_3 e^{Ay} + K_4 e^{By} - wE_c/\varphi \\ \therefore \sigma_{csg} &= \sigma_{cs} - \sigma_{cs0}, \quad \sigma_{cpg} = \sigma_{cp} - \sigma_{cp0} \end{aligned} \right\} (3)$$

$$\begin{aligned}
\text{where } A, B &= \frac{\alpha_{ss} + \alpha_{pp}}{2} \pm \sqrt{\left(\frac{\alpha_{ss} + \alpha_{pp}}{2}\right)^2 - (\alpha_{ss}\alpha_{pp} - \alpha_{sp}\alpha_{ps})} \\
K_1 &= \frac{1}{A-B} \left\{ (A + \alpha_{pp})(\sigma_{cs0} + w E_c / \varphi) - \alpha_{sp}(\sigma_{cp0} + w E_c / \varphi) \right\} \\
K_2 &= \frac{-1}{A-B} \left\{ (B + \alpha_{pp})(\sigma_{cs0} + w E_c / \varphi) - \alpha_{sp}(\sigma_{cp0} + w E_c / \varphi) \right\} \\
K_3 &= \frac{1}{A-B} \left\{ (A + \alpha_{ss})(\sigma_{cp0} + w E_c / \varphi) - \alpha_{ps}(\sigma_{cs0} + w E_c / \varphi) \right\} \\
K_4 &= \frac{-1}{A-B} \left\{ (B + \alpha_{ss})(\sigma_{cp0} + w E_c / \varphi) - \alpha_{ps}(\sigma_{cs0} + w E_c / \varphi) \right\}
\end{aligned} \quad (4)$$

Solving for steel stress

$$\sigma_{sy} = - \int_0^y n \left\{ (1 - \alpha_{ss})(\sigma_{cs} + w E_c / \varphi) - \alpha_{sp}(\sigma_{cp} + w E_c / \varphi) \right\} dy$$

There results

$$\begin{aligned}
\sigma_{sy} &= n(1 - \alpha_{ss}) T_1 - n \alpha_{sp} T_2 \\
\sigma_{py} &= n \alpha_{ps} T_1 - n(1 - \alpha_{pp}) T_2
\end{aligned} \quad (5)$$

where $T_1 = \frac{K_1}{A}(1 - e^{Ay}) + \frac{K_2}{B}(1 - e^{By})$, $T_2 = \frac{K_3}{A}(1 - e^{Ay}) + \frac{K_4}{B}(1 - e^{By})$

The increment of radius due to creep and shrinkage can be expressed as

$$\left\{ \sigma_{sy} / E_s - \sigma_{py} / E_p \right\} / (e_s - e_p)$$

Therefore, the deflection and the angle of slope are obtained by integrating the above term throughout the span length.

The above derivations are based on the uncracked whole concrete section.

However, for the cracked sections, the deflection and the increment of radius due to creep and shrinkage can be also obtained by substituting the section modulus and stresses, computed from the concrete section with the neglected tension zone, into above equations.

3. Kamihimekawa Bridge

The Kamihimekawa bridge is the first PRC (prestressed reinforced concrete) bridge constructed in Japan.

The bridge has the main span length of 48m forming a -shaped rigid frame structure with the total span length of 80m. The carriageway is 7m wide and horizontally curved with the radius of 400m, Fig. 2.

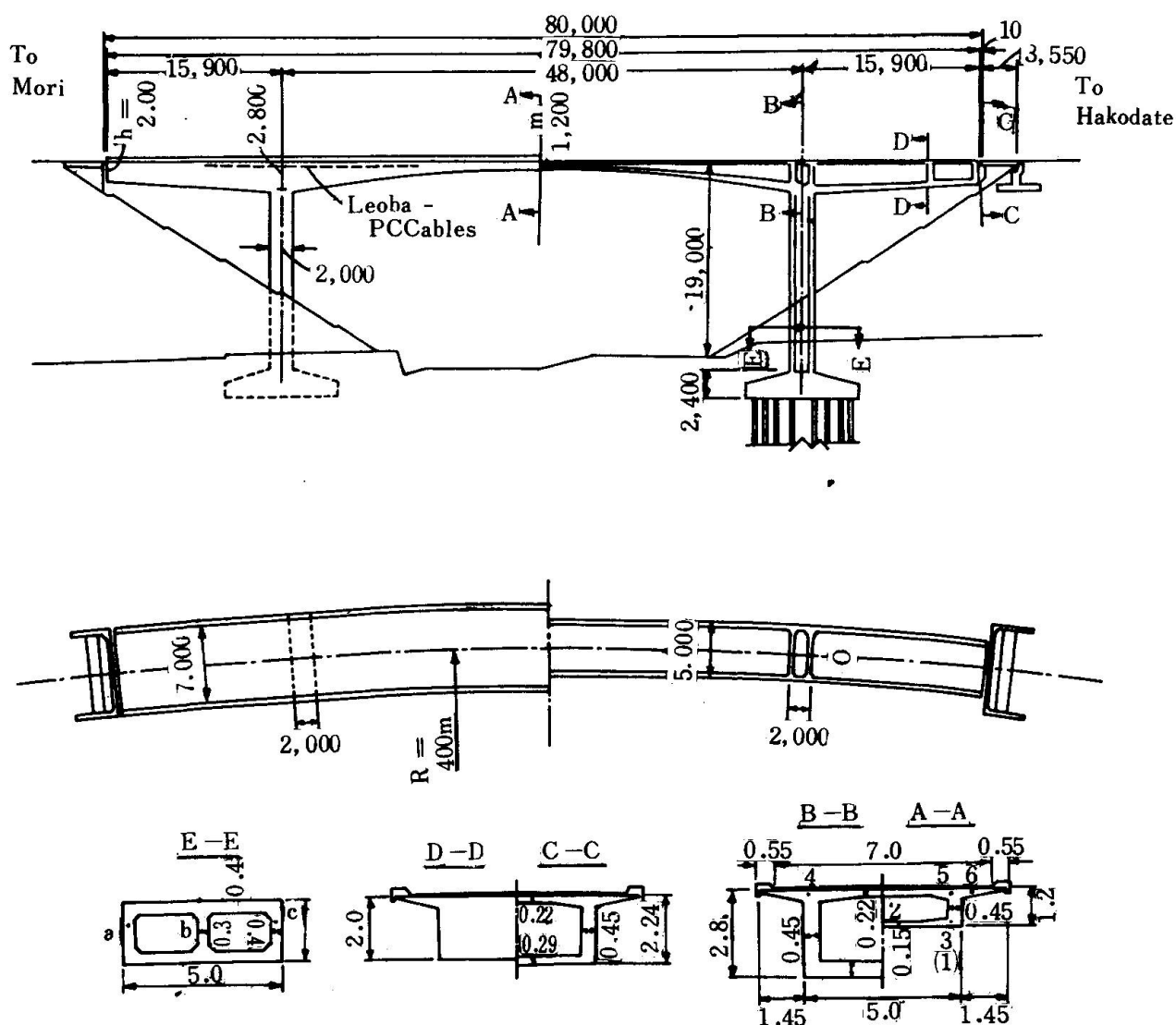


Fig. 2 Kamihimekawa Bridge (Japan)

Each half span and the cantilever girder were constructed by successive-corbellings from the piers in a symmetrical manner. The static calculations of

the structure were performed for the two states of structural systems, i.e. a system of cantilevers during the construction and that of rigid frame after closing.

The structure was designed as a RC system, using 32mm cold-twisted bars of steel SDC 40 as the main reinforcement and concrete of $\sigma_{28}=360 \text{ kg/cm}^2$ quality for the main girder and $\sigma_{28}=210$ for the piers.

The total of 470 ton of prestressing force were longitudinally introduced during the corbelling construction to the portions of deck girders of 22m length in order to control crack widths, Fig. 2. The prestressing was performed in accordance with the Leoba system in the two webs of box girder by means of prestressing tendons which consist of 6 x S66 (16/8) and 2 x S24 (12/5)

The super structures was built by corbelling in successive stages without centering. The length of the successive advances, which numbered 20 in all, was 3.0m. In accordance with the progress of the girders, the non-prestressed main bars $\phi 32$ were screwed to the next bars of 6m length by means of FY-couplers.

The FY couplar is a special type of screw-threaded coupler which was developed by the senior author's team and experimentally tested to ensure the transmitting of the entire strength of the bar to be joined.

The main structure was completed in November, 1965 and the bridge was opened to traffic in June, 1966.

4. Measurements of Creep and Shrinkage in Kamihimekawa Bridge

The strain due to shrinkage are measured at the section D-D of side span, shown in Fig. 2, where the prestressing force is not introduced.

Fig. 3 shows the results of the investigations on shrinkage.

The dotted points indicate the calculated values and folded lines show the measured values.

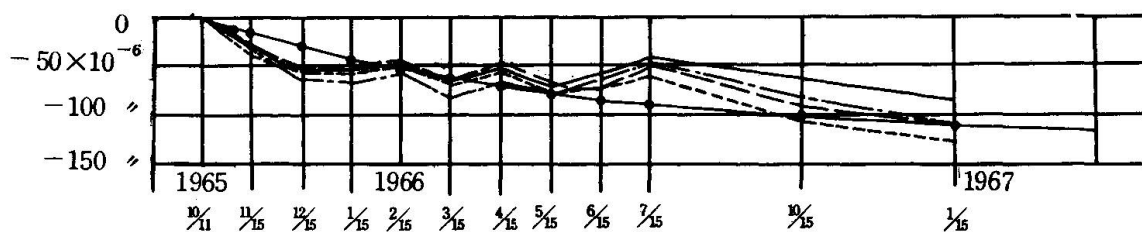


Fig. 3 E_{el} at section D-D

E_{el} : elastic strain

E_g : creep strain

E_w : shrinkage strain

The shrinkage strains observed for the first three months are greater than the calculated ones, since the developments of shrinkage is almost same as the assumption that $\epsilon_s = 13 \times 10^{-5}$.

Fig. 4 shows the total value of elastic creep and shrinkage strains, and Fig. 5 shows the total value of elastic, creep, shrinkage and secondary strains.

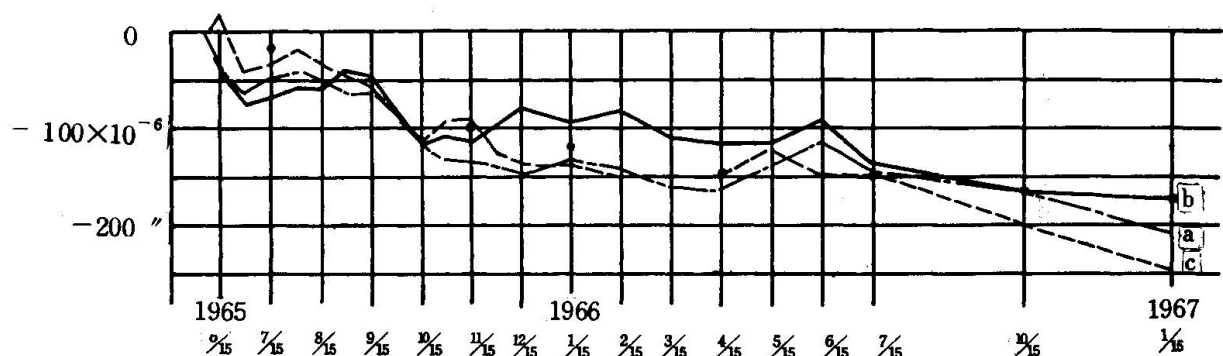


Fig. 4 $E_{el} + E_g + E_w$ at section E-E

The observed final values are almost same as the calculated value based on the assumptions that $\phi_{\infty} = 2.5$ and $\epsilon_s = 13 \times 10^{-5}$.

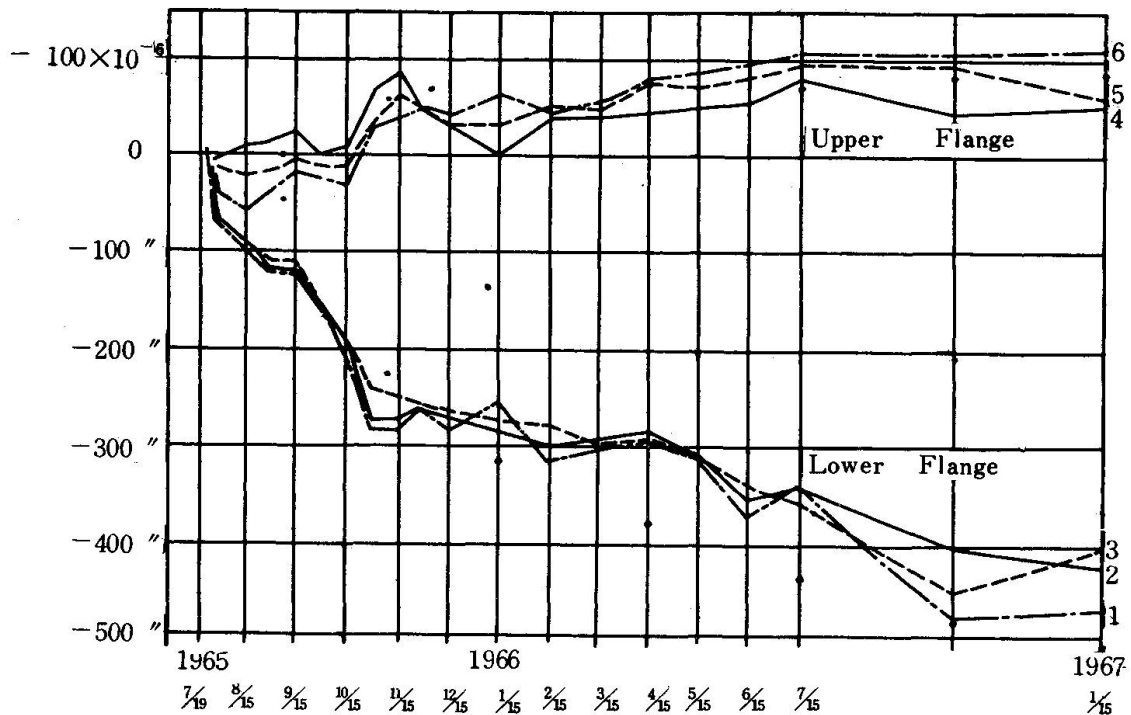


Fig. 5 $\epsilon_{el} + \epsilon_g + \epsilon_w$ at section A-A

5. Acknowledgement

The authors are grateful to Dr. Y. Kakuta for his assistance in the investigations and are also indebted to Dr. M. Hayashi, director of the Civil Engineering Research Institute / Hokkaido Development Bureau, for his efforts in the development of the PRC system.

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SUMMARY

The behaviors of creep and shrinkage in PRC system is described on the basis of the proposed method of analysis and measurements on Kamihimekawa bridge which was designed and constructed as the proposed system.

RESUME

On décrit les influences du fluage et du retrait dans le béton armé précontraint (PRC), sur la base des calculs et des mesures du pont de Kamihimekawa conçu et exécuté selon le système proposé.

ZUSAMMENFASSUNG

Beschrieben wird das Kriech- und Schwindverhalten in bewehrten Spannbetonbauten aufgrund eines vorgeschlagenen Rechenverfahrens sowie Messungen an der Kamihimekawa Brücke, welche nach diesem Verfahren entworfen und ausgeführt worden ist.

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Déformations de retrait et de fluage du béton

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En liaison avec la nature physique du béton se trouvent les déformations de retrait et de fluage.

Conformément à la méthode, qui est actuellement généralement admise, la mesure du retrait ne commence à peine qu'après le décoffrage des échantillons (24 ou 48 heures après leur élaboration).

On néglige les déformations jusqu'à cet in-

stant comme insignifiantes et sans importance pour les dimensions des éléments de béton.

La figure 1 représente le développement du retrait initial d'un ciment portland en fonction de la variation du rapport eau/ciment, à partir de la première jusqu'à la 48-ème heure après la préparation des échantillons.

La grandeur du retrait initial en comparaison avec le retrait après la 48-ème heure représente de 35 à 55% du retrait total du 28-ème jour et il ne doit pas être négligé,

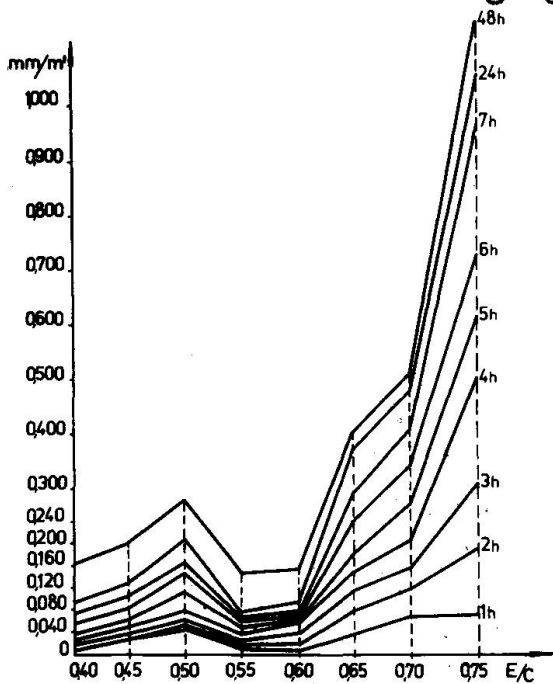


fig.1

étant donné qu'il se développe après que le mortier de ciment ait obtenu une forme déterminée.

En liaison avec l'évolution quantitative du retrait hors de humectation et séchage périodiques les données indiquées sur les deux figures suivantes présentent de l'intérêt.

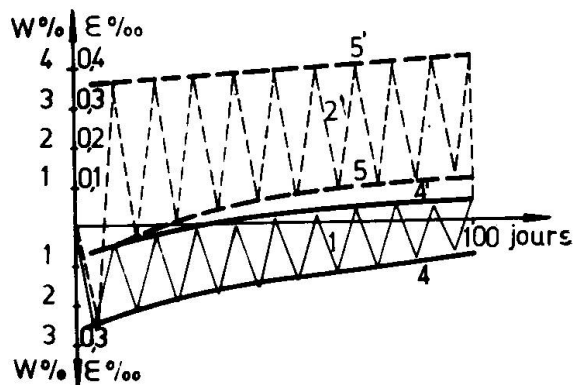


fig.2

Pour les cycles de courte durée de 5 et 15 jours, comme ça se voit sur la figure 2 qui se rapporte au cycle de 5 jours, le processus de gonflement augmente continuellement, tandis que le processus de retrait diminue continuellement (lignes 4 et 4'). L'Humidité pesante dans les échantillons croît respectivement sans cesse (lignes 5 et 5').

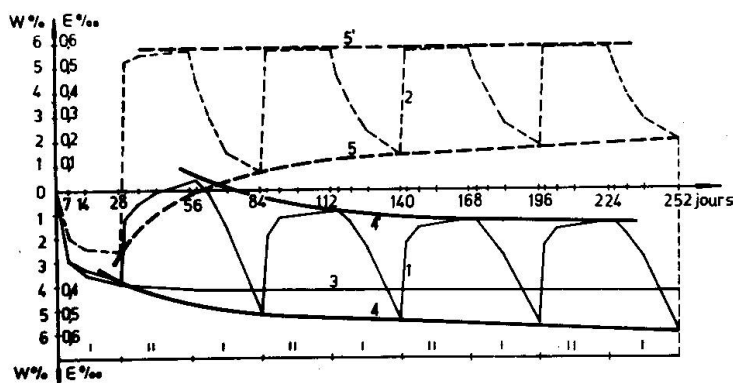


fig.3

humidité de 55% (ligne 3).

Pour les cycles de longue durée de 28 et 56 jours, figure 3, quoique l'humidité pesante présente un accroissement, le gonflement diminue tandis que le retrait augmente continuellement en dépassant celui du mortier de ciment, soumis à un retrait continu avec une hu-

midité de 55% (ligne 3). Un intérêt beaucoup plus grand au point de vue théorique et pratique représente l'influence de l'humidité du béton sur la caractéristique quantitative du fluage.

Nos études ont été effectuées dans notre laboratoire sur deux séries d'échantillons du même ciment et avec la même composition du béton. Les échantillons de la première série sont saturés en eau et isolés avant la charge, tandis que ces de la deuxième série sont séchés pendant 72 heures jusqu'à l'obtention d'un poids constant et isolés avant la charge. La charge pour chaque série a lieu respectivement à l'âge de 3, 7 et 28 jours. Les résultats sont donnés sur la figure 4. Il en ressort

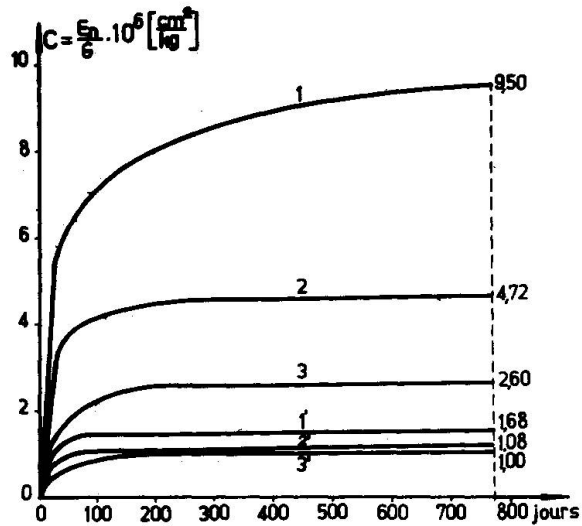


fig. 4

clairement que les échantillons saturés en eau ont un fluage plusieurs fois plus grand. En outre, le fluage diminue avec l'augmentation de l'âge du béton.

L'isolement des échantillons dans les deux séries permet d'éliminer l'influence du retrait et des forces capillaires sur la grandeur du fluage.

Differential Creep, Shrinkage and Stress Redistribution in Composite Prestressed Concrete Beams

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In the preliminary publication, a study is contained on differential shrinkage (1). In addition to the theories by Birkeland, Evans and Branson (Ref.4-6 of paper (1)) that of Evans and Parker (2) should be mentioned. All these theories are based on the same assumptions i.e. that the free shrinkage of the prestressed and non-prestressed components, acting at the respective centroids, are known and that the resulting strain distribution is straight. Thus, if the free shrinkage and/or creep strains of the prestressed component is ϵ_{pf} and that of the non-prestressed component is ϵ_{nf} , it is possible to obtain the position of the resulting strain distribution from the difference of these two free strains $\Delta\epsilon = \epsilon_{nf} - \epsilon_{pf}$, as seen from Fig.1 in which the strain distributions due to (i) the free and (ii) the resultant strains are plotted. It is shown in this figure that the actual resulting strain at the centroid of the prestressed component is $\epsilon_{pf} + \Delta\epsilon_p$ and that at the centroid of the non-prestressed component amounts to $\epsilon_{nf} + \Delta\epsilon_n$, where $\Delta\epsilon_p = \Delta\epsilon/K_1$ and $\Delta\epsilon_n = \Delta\epsilon/K_2$. The two constants K_1 and K_2 depend solely on the properties of the sections and E-values and amount to:

$$K_1 = A_{pc}/n_o \cdot A_n + (A_p \cdot e_o^2)/(I_p + n_o \cdot I_n) \text{ and } K_2 = A_{pc}/A_p + (n_o \cdot A_n \cdot e_o^2)/(I_p + n_o \cdot I_n)$$

In these equations A_p , A_n and A_{pc} are the respective cross sectional areas of the prestressed, non-prestressed and composite sections; I_p and I_n are the corresponding I-values and $n_o = E_{cn}/E_{pn}$ is the ratio of E_c -values of the two components, whereas e_o is the vertical distance between the two centroids. The remaining strains can be computed from the geometrical conditions when the strains at the two centroids are known. This relationship has been published in (3) but was used already in paper (4) at the IABSE Congress Stockholm 1960. Fig.2, taken from this paper, shows comparative results of three different cross section. It is seen that only with a cross section according to example No.1

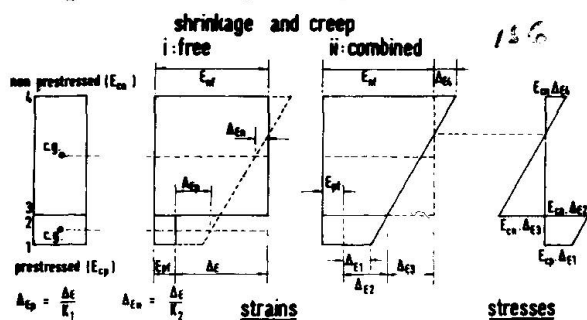


Fig.1

Diagrams of strains and stresses due to differential shrinkage and creep.

similar results are obtained to those, presented by the authors in paper (1), resulting in additional tensile stresses due to differential shrinkage at the outer tensile face of the section. With examples 2 and 3, compressive stresses

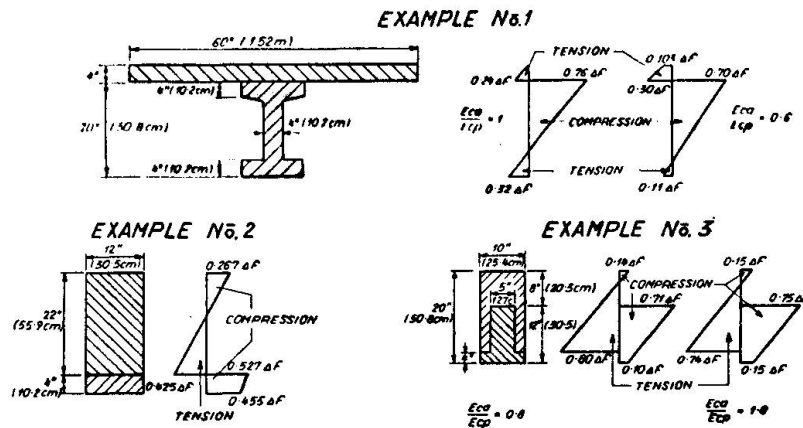


Fig.2. Examples: Stresses due to differential shrinkage, as shown at the IABSE Congress Stockholm, 1960.

are induced by differential shrinkage, whereas tensile stresses occur at the lower face of the non-prestressed component. This results in a stress redistribution owing to the greater stress difference at the adjoining faces of the prestressed and non-prestressed components, as discussed in paper (4). In this case cracks became visible at a lower nominal tensile stress at the outer tensile face than is the case in similar homogeneous sections.

In order to clarify the stress redistribution and to ascertain the magnitude of differential shrinkage and creep in the prestressed and non-prestressed components, tests were carried out at DUKE University, North Carolina, U.S.A. in 1967/68 which are to be described in detail elsewhere (5). Here, only some important results are illustrated. Fig.3 shows particulars of the specimens. It was endeavoured to study the extreme cases at which either mainly creep or shrinkage takes place ('C' and 'S' beams); the former was obtained when the added concrete was cast upon the prestressed concrete on release of the prestress after the plank had been moist cured to avoid shrinkage. For the 'S' beams, relating to differential shrinkage, the added concrete was placed much later, after a substantial part of shrinkage and creep of the prestressed component had already taken place. The third case, relating to stress redistribution ('R' beams), was investigated in such a way that the two components were separately

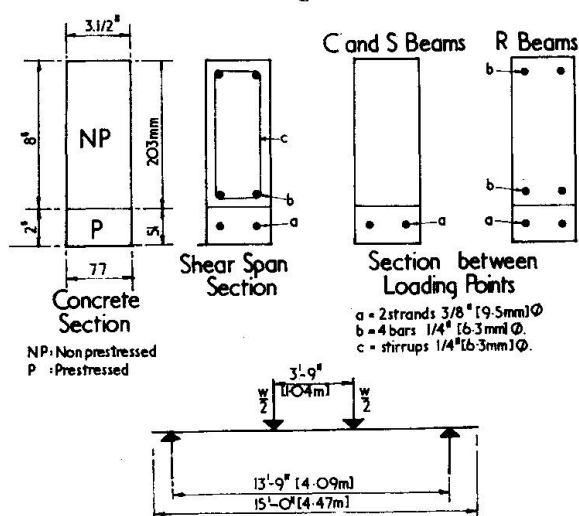


Fig.3. Specimens at tests DUKE University.

precast and then glued together which allowed strain measurements. Stirrups were provided in the shear spans of the non-prestressed component only and the reinforcement 'b' was limited to the shear spans in the 'C' and 'S' beams, but provided along the entire length in the 'R' beams.

Some of the results are shown in figure 4. Generally, with the 'C' beams the precompression was greatly reduced by the creep in the prestressed plank (which was further increased by differential creep), whereas with the 'S' beams the shrinkage stresses in the added concrete and the compressive stresses in the prestressed component are increased. The loads at which visible

Specimen	Nominal Differential Stresses	Self Load [W=0]	Microcracking		Visible cracking
			N.P. Concrete	P. Concrete	
C1 [creep only]			W=4500 		W=5600
S1 [shrinkage only]			W=3500 		W=5500
R3 [glued 12 days before testing]			W=3500 	W=5000 	W=6500

Fig.4. Some Results, DUKE University Tests 1967/68.
(Stresses in psi. (lbf/in.²); Loads in lbf).

cracks occurred were approximately the same for the 'C' and 'S' beams, although the theoretical stress conditions were completely different in both cases. With the 'S' beams there were relatively high nominal concrete stresses in the non-prestressed component already under self load, when microcracks developed, as can be seen from the photograph Fig.5. This was obtained from a photoelastic coating, using a method, as described in paper (6). With the 'C' beams microcracks occurred at a later stage.

In the 'R' beams microcracks developed first in the non-prestressed concrete at the same load at which they had already become visible in the prestressed plank of the 'S' beams. Microcracking in the prestressed concrete occurred at a load slightly less than that at which they had become visible with the 'C' and 'S' beams. However, the load at which the cracks became visible in the 'R' beam was appreciably higher than those in the 'C' and 'S' beams. These studies have shown that redistribution of stresses may cause visible cracking at relatively low nominal tensile stresses at the outer face, if there are very high nominal tensile stresses in the non-prestressed component.



Fig.5. Microcracking in 'S' beam under self load.

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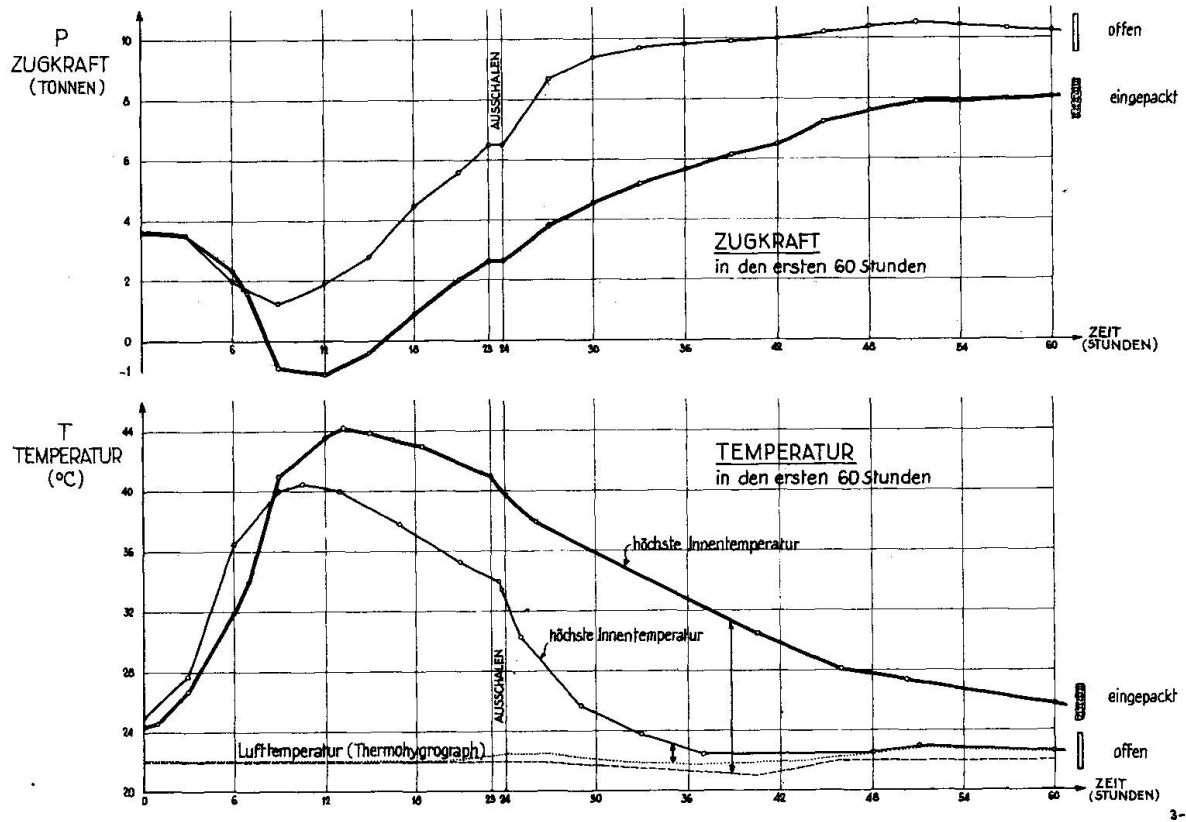
Isolierende Abdeckmatten beeinflussen das Verhalten des Betons

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Zur Verbesserung der Betonqualität benutzen wir seit 1963 Matten zum Abdecken des frisch eingebrachten Betons. Diese Matten bestehen aus zwei äusseren Lagen starker Plastic-Folie und einer wärmeisolierenden Schaumstoffeinlage. Wir erreichen damit, dass die Wärme, welche der Zement beim Abbinden entwickelt, zurückgehalten wird und das Wasser im Beton nur langsam entweichen kann.

Wir wollten die wirtschaftlichen Vorteile, die sich aus diesem Verfahren ergeben (etappenweiser Einsatz von Schalung und Gerüst), vom Fertigteilbau auf den Ortbetonbau übertragen. Da wir Temperaturen von über 50°C gemessen hatten, stellten sich uns die Fragen: Was geschieht beim Ortbeton, wenn wir eine neue Betonieretappe an eine alte anschliessen? Besteht beim Abkühlen wegen der Behinderung des Schwindens durch die Haftung die Gefahr von Rissen im frischen Beton? Wie gross sind diese Schwindkräfte? Wie wirkt sich das Abdecken überhaupt auf die Eigenschaften des Betons aus?

Es wurden zwei Versuchskörper, die nahe der Kontaktfläche zum alten Beton aus dem frischen Beton herausgeschnitten gedacht sind, praktisch unter der Presse betoniert. Beide Proben wurden nach 24 Stunden ausgeschalt. Ein Körper war jedoch vor und nach dem Ausschalen mit Matten eingepackt. Die Abmessungen der Betonkörper betrugen 20/20/220 cm. Sie waren armiert mit 4 Stäben von 12 mm Durchmesser. Diese Stähle waren in den Endplatten verschweisst. Die Pressenkraft wurde mit einem sogenannten "hydropacer" gesteuert, und zwar so, dass die Länge der Körper konstant blieb. Die Kraft, die nötig war, um gegen die natürliche Längenänderung aus Temperatur und Schwinden die Bedingung "Länge bleibt konstant" zu erzwingen, wurde fortlaufend gemessen. Temperaturen und relative Luftfeuchtigkeit wurden ebenfalls registriert. An separat erstellten Prüfkörpern konnte die Entwicklung von Druckfestigkeit, Biegezugfestigkeit und Schwinden verfolgt werden.



3- 7776

Bild 1

Bild 1 zeigt die Entwicklung der Temperatur in den Versuchskörpern und den Verlauf der Pressenkraft. Beim nachbehandelten Körper steigt die Betontemperatur höher an, sinkt dann jedoch langsamer gegen die Raumtemperatur als beim nicht nachbehandelten Beton. Dieser Temperaturentwicklung entsprechen die Kräfte: Grösserer Druckanteil und sanfterer Zug-Anstieg beim nachbehandelten Beton.

Im Bild 2 sind die Werte für die Betondruckfestigkeit dargestellt. Der schnellere Anstieg beim abgedeckten Beton gestattet früheres Ausschalen und Entfernen der Gerüste. Beachtlich sind die höheren Werte für die Biegezugfestigkeit und der erfreulich kleinere Schwindkoeffizient (Resultate von Prismen 12/12/36 cm)

Im Bild 3 sehen Sie - für den Zeitpunkt 60 Stunden nach dem Betonieren - die ungefähren Zugspannungen ($\sim \sigma_z$), errechnet aus der gemessenen Kraft. Die nächste Kolonne zeigt die Biegezugfestigkeit (β_{Bz}) nach 60 Stunden. Wenn die Zugfestigkeit mit der halben Biegezugfestigkeit eingesetzt wird, ergibt sich - als ein Mass der erwirkten Qualitätsverbesserung - die

Sicherheit $\langle s \rangle$ gegen einen Anriss $\frac{\beta_{Bz}}{\sigma_z} \approx 1$ resp. 1,45.

Dieser Wert steigt nach 92 Stunden auf ca 1 : 1,57.

Zum Abschluss des Versuches ermittelte man die Restkraft, indem der Versuchskörper bis zum ersten Riss gezogen wurde:

nach 14 Tagen	Anrisslast	Zugkraft	Kraftreserve
offen	12,8 t	11,5 t	1,3 t
eingepackt	17,0 t	10,0 t	7,0 t

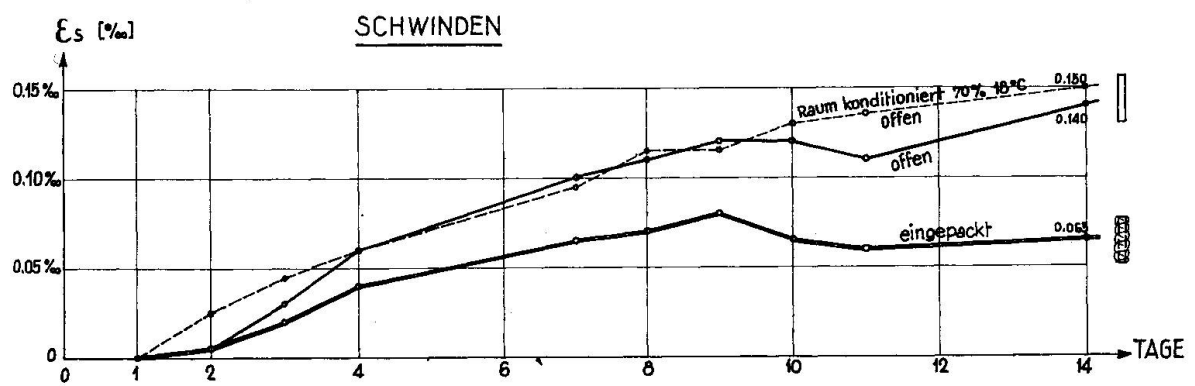
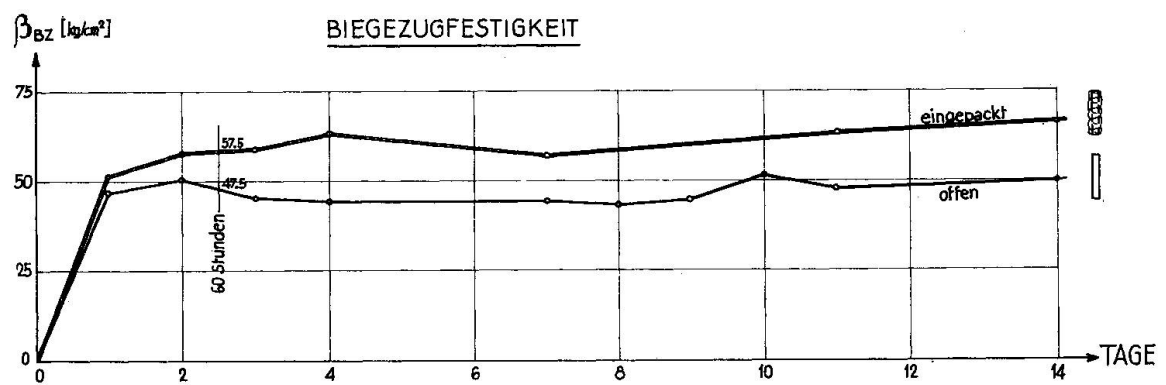
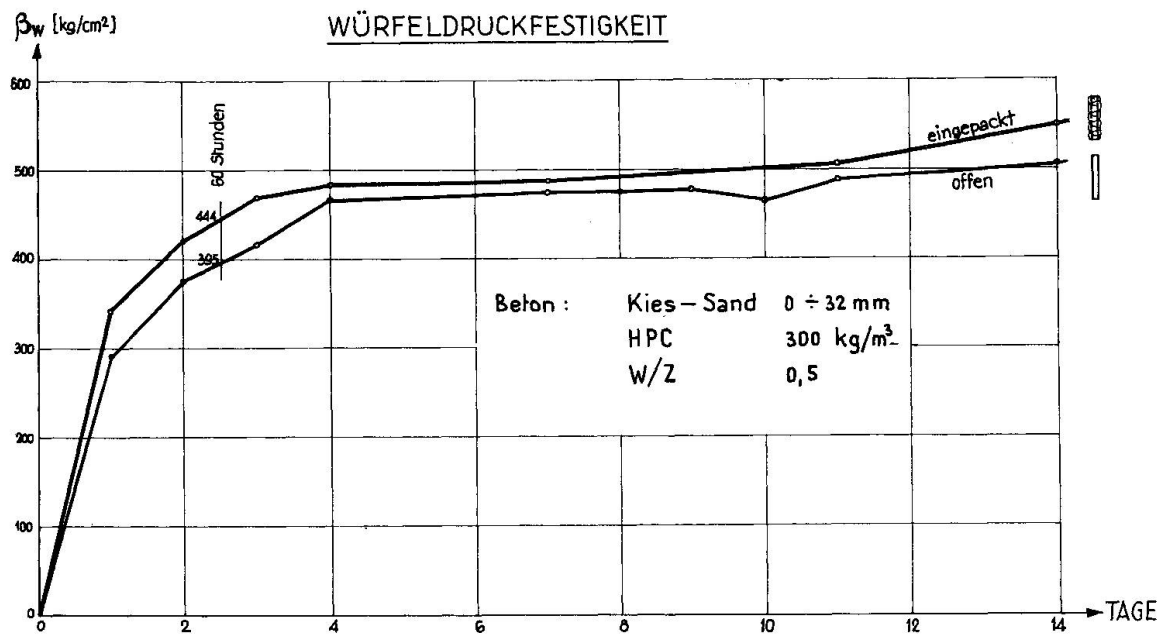


Bild 2

BETONKÖRPER 200/20/20 CM	$\sim \sigma_z$ AN SÄULE GEMESSEN [kg/cm ²]	β_{Bz} AN PRISMEN GEMESSEN [kg/cm ²]	β_z $\sim \frac{\beta_{Bz}}{2}$ [kg/cm ²]	$\langle S \rangle$ $= \frac{\beta_z}{\sigma_z}$
IN FREIER NORMALER RAUMATMOSPHÄRE 	25	50	25	1.00
MIT RÜCKSTAU VON WÄRME UND WASSER 	20	58	29	1.45

Bild 3: Sicherheit $\langle s \rangle$ gegen Anriss nach 60 Stunden (angenähert)

Die Eidg. Materialprüfungs- und Versuchsanstalt (EMPA) führte die Versuche durch. Es sind nur zwei Stichproben. Sie bestätigen jedoch die positiven Erfahrungen der Praxis.

Die Kosten für die abgedeckte Betonoberfläche belaufen sich auf ca Sfr. 1.50 pro m². Bei Industriebauten, Brückenplatten, ja sogar Wohnbauten übersteigen die wirtschaftlichen Vorteile diesen Betrag. Die Qualitätsverbesserung bei Wannen, Behältern, Schalen, Betonstrassen und Flugpisten geldmässig zu erfassen, ist schon schwieriger. Es ist oft angezeigt, dieses Verfahren und seine Vorteile über einen Umweg in die Praxis einzuführen. Dieser Umweg ist der Winterbeton. Das Abdecken der Einbringstellen und der Armierungen mit den Matten verhütet, dass Schnee in die Schalungen fällt und sich dort festsetzt. Den mit Beton auszufüllenden Raum kann man in der Nacht vorher beheizen: so werden auch die Eisen warmgehalten. Die Mattenabdeckung wird dann nur an der eigentlichen Einbringstelle für das Betonieren kurze Zeit geöffnet.

Measurements of Temperature Changes of Concrete on Nusle Valley Bridge

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The bridge over the Nusle valley in Prague is a five-span framework, the length of the spans being 68,25, three-fold 115,50 and 68,25 meters respectively (1). Due to the arrangement of the intermediate supports it was possible to construct the load-bearing structure in one unit 485 meters long. The frame cross-beam is formed by a thin-walled bar having a closed cross section with two-side corbelled out footpaths (Fig. 1). The double level disposition made it possible for the underground to pass through the inside of the bridge. On the roadway there are placed two three-lane carriageways of the North-South expressway.

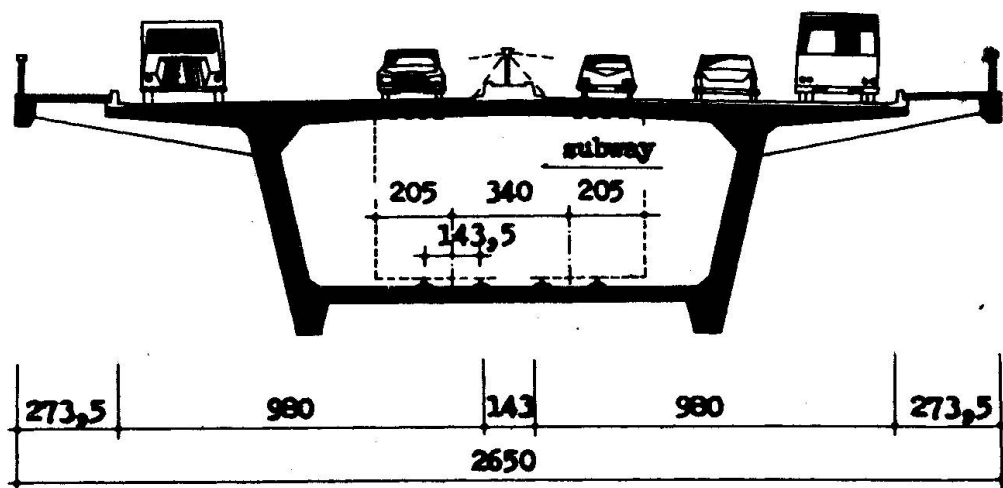


Fig.1 Cross section of the bridge

The composition of the concrete mixture was as follows:

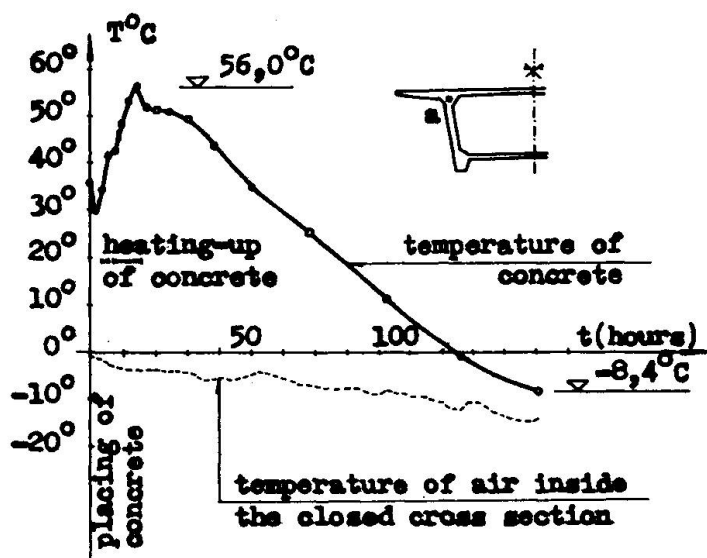


Fig. 2 Time variation of temperature in point a

cement mark 450	425 kg
plasticizer S	2,8 l
water	150 l
aggregates	1845 kg

For temperature measurements 139 thermometers were placed during cantilever concreting the last two lamellas of the second span. From Fig. 2 the time variation of temperature after placing the concrete in the frame corner where the wall and the plate of roadway are connected (point a) in the middle of the last lamella can be seen. The maximum temperature measured was 56,0°C at the end of heating-up of concrete which was needed owing to severe frosts in December 1969. After 6 days the temperature in this point sank to 8,4°C below zero.

The lamellas of the load-bearing structure were concreted stage by stage so that after placing the reinforcement of the lower slab and walls the lower slab was concreted, after one day the concreting of the walls followed and after intermission of one week during which the falsework of the inner space was erected and the reinforcement of the roadway deck placed the floor system was finished. In Fig. 3 can be seen the temperature of concrete 10 hours after concreting the lower slab. Fig. 4 shows the temperature 20 hours after finishing concreting the walls and 44 hours after concreting the lower slab. The maximum temperature was measured in the centre of the wall by the value

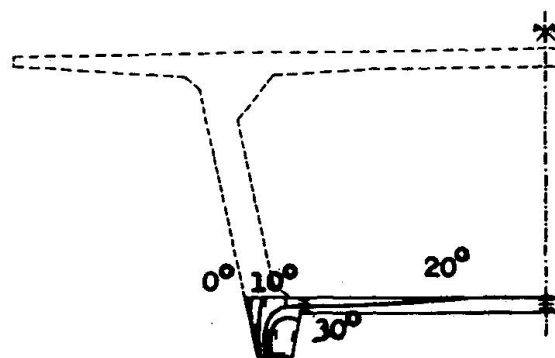


Fig. 3

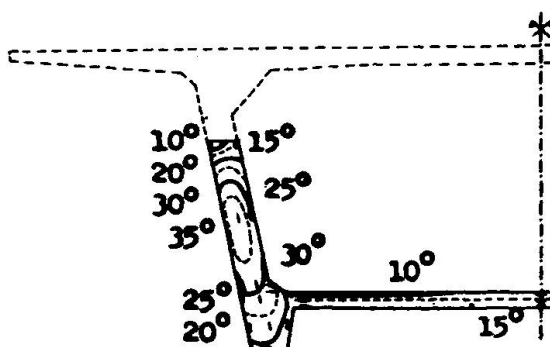


Fig. 4

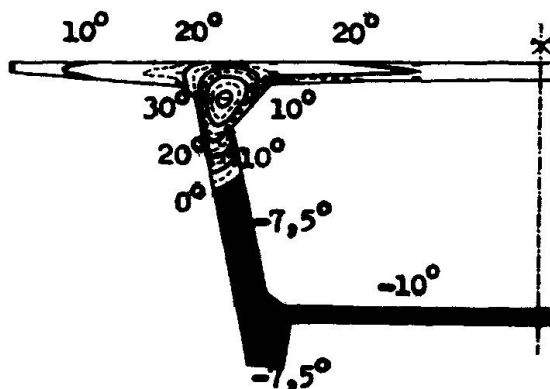


Fig. 5

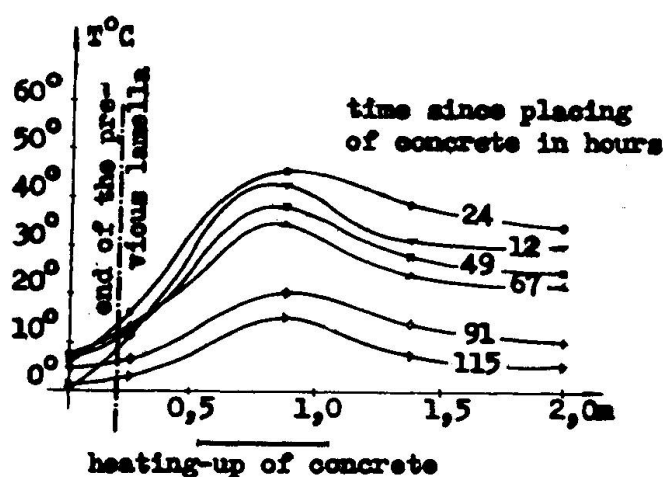


Fig.6

8,4°C below zero. In Fig. 6 the variation of temperature of concrete in the wall in the longitudinal direction can be seen.

The results of measurements show the great effect of the heating-up of concrete in the time after the concrete was placed and the influence of the frozen concrete of the previous lamella.

References

1. Michálek, V., Bridge over the Nusle valley in Prague, publication for the Sixth FIP Congress in Prague, Prague, 1970.

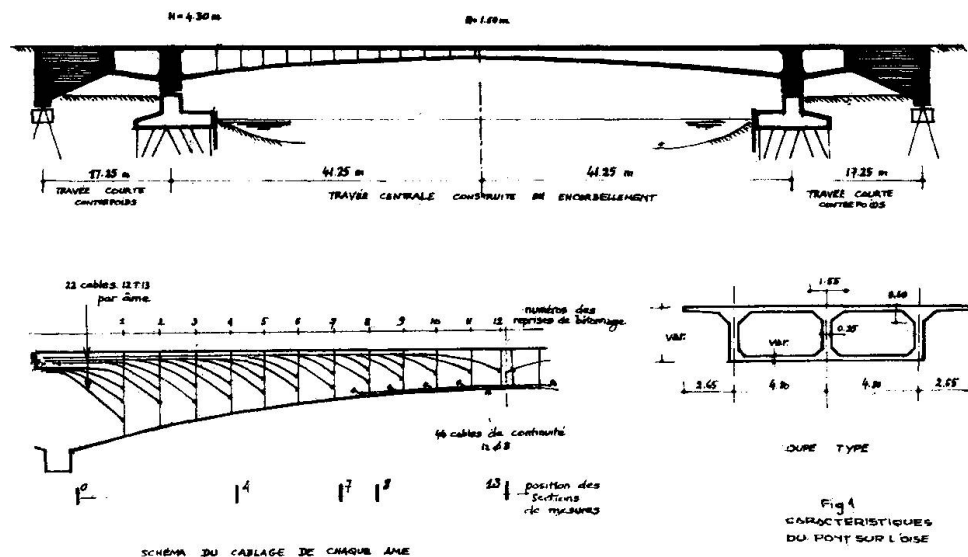
of 38,6°C. In Fig. 5 we can see the temperature one day after placing the concrete of the roadway plate and one week after finishing the concreting of the walls. The position of the thermometers can be seen too, marked by short commas. The grey part of the cross section already had the temperature below zero. The maximum temperature was measured in the centre of the frame corner (point a in Fig. 2) and was 51,1°C while the air temperature inside the closed cross section was

Mesures de déformations faites pendant la construction d'un pont en béton précontraint par encorbellement

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1.) Mesures effectuées au pont sur l'Oise, autoroute A1

1.1 La structure du pont étudié comporte une poutre-caisson à trois âmes de 82 m de portée solidaire de deux travées courtes formant contrepoids d'équilibrage (Fig.1). Chaque moitié de la grande travée a été construite en encorbellement à partir d'une travée-contrepoids. Chaque tronçon d'encorbellement avait 3,14 m de longueur, et sa construction demandait une semaine au total.

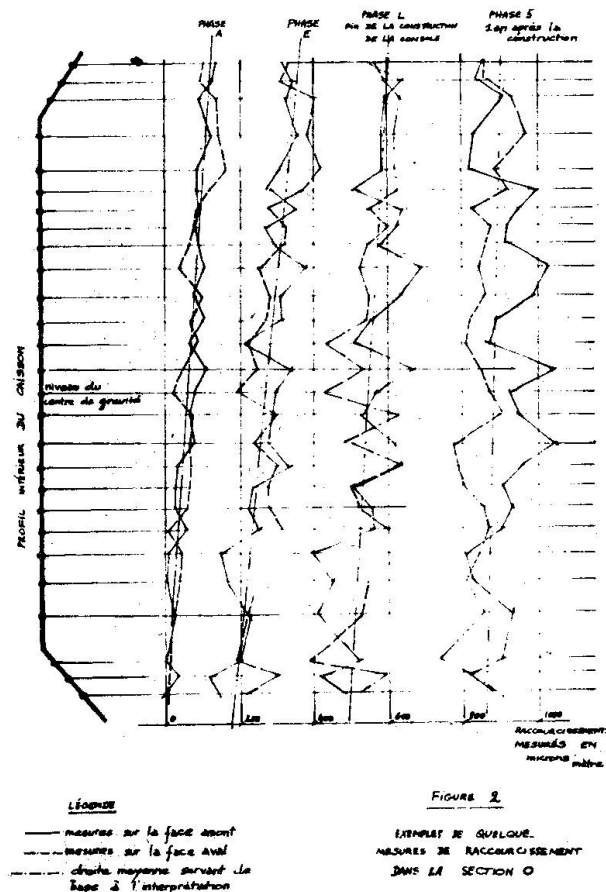


La précontrainte est assurée par deux groupes de câbles. En cours de construction, chaque tronçon d'encorbellement était relié aux parties déjà construites par des câbles STUP 12 T 13. Au moment du clavage des deux consoles, des câbles 12 ϕ 8 disposés dans le hourdis inférieur de la zone centrale de l'arche ont été mis en tension, pour assurer la continuité de l'ensemble.

1.2 Des mesures de déformations ont été faites dans plusieurs sections de l'ouvrage (à l'encastrement, au milieu et dans deux sections intermédiaires). On a collé quelques heures après coulage du béton, sur les deux faces de l'âme centrale du caisson, des billes disposées en deux files verticales à la maille de 20 x 20 cm environ. On a ensuite mesuré au comparateur les variations de la distance entre les deux billes situées sur une même horizontale. Ces mesures ont été faites à chaque stade de la construction par encorbellement, puis à intervalles réguliers pendant quatre ans.

On a mesuré également, à l'aide de thermocouples noyés dans le béton, la température moyenne du béton. On a ensuite ramené les déformations mesurées à celles qui auraient été lues si la température du béton était resté fixe (température origine).

1.3 Si on admet l'hypothèse de la conservation des sections planes, on a donc des indications précises sur l'évolution dans quelques sections du raccourcissement au niveau du centre de gravité et de la courbure de flexion (Fig. 2).



1.4 En plus des mesures sur l'ouvrage, on a fait des mesures en laboratoire du retrait et du fluage sous charge sur des éprouvettes de béton qui avaient été conservées sur le chantier même jusqu'au moment de leur mise en charge.

2.) Estimation des contraintes réelles et des déformations de l'ouvrage

2.1 Les charges verticales supportées par l'ouvrage sont bien connues puisqu'il s'agit essentiellement de poids de béton ou de poids de superstructures.

Par contre, les efforts de précontrainte sont plus difficiles à connaître exactement à chaque stade de la construction. Ils dépendent en effet des pertes réelles de précontraintes dans les câbles.

L'influence du coefficient de frottement du câble dans sa gaine à la mise en tension est faible, car les câbles sont courts et peu courbes. Les mesures du raccourcissement unitaire au niveau des câbles nous donnent directement l'effet du fluage et du retrait sur les efforts de précontrainte.

On a mesuré en laboratoire la relaxation d'un toron constitutif d'une câble 12 T 13 pour une tension très voisine de cette atteinte effectivement à la mise en précontrainte. Mais seule une part de cette relaxation s'est effectuée dans les câbles de l'ouvrage réel, parce que les câbles étaient injectés.

En cours de construction une faible variation des efforts de précontrainte entraîne une forte variation du moment de flexion subi par l'ouvrage, (parce que les effets contraires du poids propre et de la précontrainte se compensent presque). Il est donc possible, par comparaison avec les raccourcissements mesurés, d'apprécier le bien fondé des hypothèses faites sur les valeurs de la précontrainte.

2.2 On a utilisé pour évaluer les déformations unitaires, la méthode simplifiée indiquée par le CEB. Dans cette méthode, les déformations unitaires sont commandées par des paramètres faisant intervenir la composition du béton, la plus petite dimension transversale de la pièce étudiée, les conditions hygrométriques l'âge du béton au moment du chargement et le temps écoulé depuis le chargement.

Pour faciliter l'interprétation, nous avons supposé que l'hygrométrie du béton restait constante aux alentours de 70 % (ouvrage au-dessus d'un fleuve); nous avons admis le même loi que le CEB pour tenir compte de l'influence de l'âge du béton au moment du chargement et nous avons déduit des mesures expérimentales en laboratoire, la valeur du module de déformation sous charges instantanées. Ainsi, les déformations unitaires ne dépendent plus que de deux inconnues restant à déterminer par l'interprétation à savoir le coefficient de fluage et la loi d'évolution du fluage dans le temps depuis le chargement.

3.) Interprétation des mesures

3.1 Comme nous l'avons vu précédemment, les mesures nous donnent dans plusieurs sections de l'ouvrage et à des époques déterminées, le raccourcissement au centre de gravité, et la courbure de flexion.

D'autre part, les lois du CEB telles que nous les avons simplifiées nous permettent de relier les déformations aux contraintes, à condition de fixer deux paramètres (coefficient de fluage, évolution du fluage dans le temps).

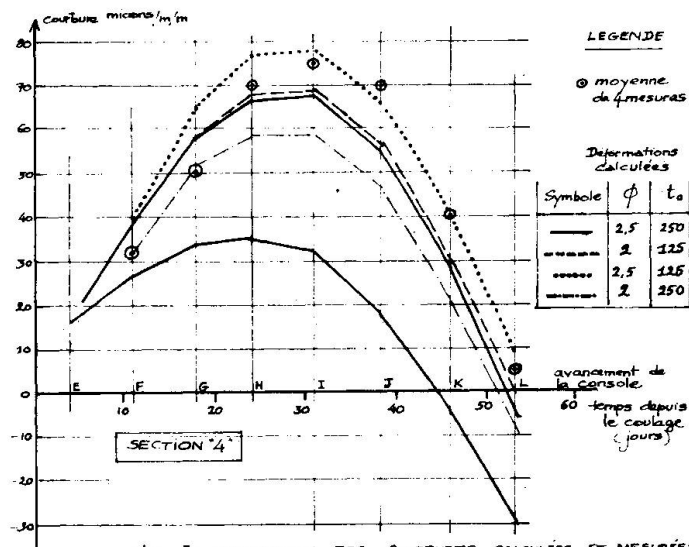
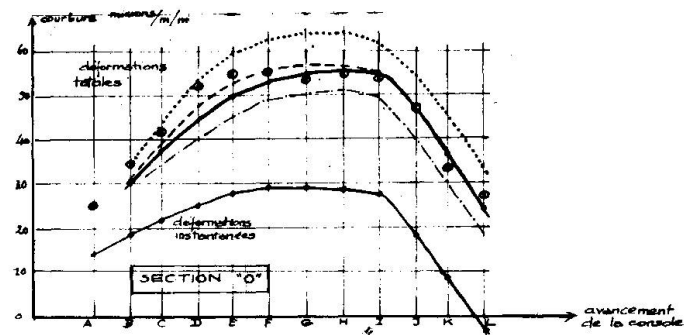
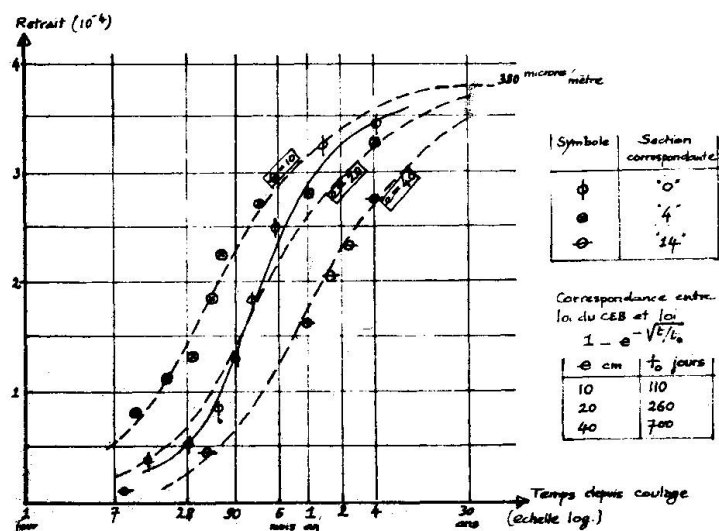


FIG. 3 COMPARAISON DES COURBURES CALCULÉES ET MESURÉES

FIG. 4 VALEURS DU RETRAIT SÉQUENTES DE L'ÉTUDE D'INTERPRÉTATION
COMPARAISON AVEC LES LOIS D'ÉVOLUTION DU CEB

La comparaison des courbures mesurées et des courbures déduites de l'étude de la structure nous permet de fixer des valeurs plausibles de ces deux paramètres (Fig. 3).

3.2 Les raccourcissements au niveau du centre de gravité sont la somme des raccourcissements de retrait et des raccourcissements dus aux efforts de précontraintes eux-mêmes.

A partir des valeurs précédemment admises, on peut en déduire par soustraction une loi d'évolution du retrait dont la valeur finale est comparable à celle trouvée par des mesures expérimentales directes sur des éprouvettes de même béton et dont l'évolution est comparable à l'évolution du fluage (Fig. 4).

Ceci permet donc de contrôler les hypothèses et déductions précédemment faites.

4.) Résultats de l'interprétation des mesures

4.1 Après de nombreux tâtonnements, il nous a été possible de trouver une interprétation cohérente pour l'ensemble des mesures pendant et après la construction sur plusieurs sections de l'ouvrage en adoptant les valeurs approximatives suivantes :

Module instantané du béton 400 000 Kg/cm²

Coefficient de fluage 2,5

Retrait final dans l'ouvrage 380×10^{-6}

Expression mathématique représentant l'évolution du fluage et du retrait dans le temps $V = 1 - e^{-\left(\frac{t_n - t_i}{t_0}\right)^{0,5}}$

où t_n est l'âge du béton au moment où l'on considère le fluage, t_i est l'âge du béton au chargement, t_0 compris entre 120 et 250 jours

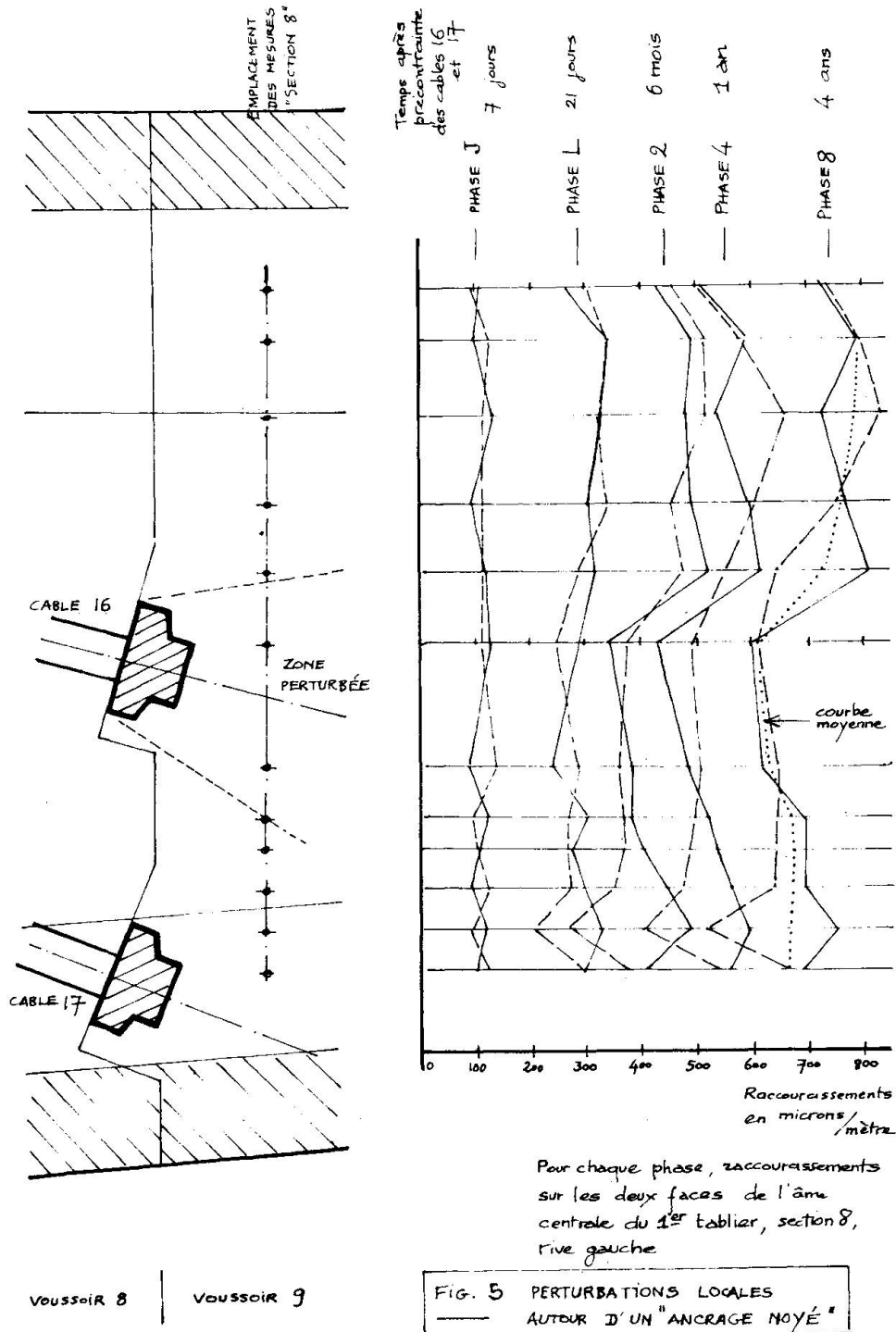
4.2 Si l'on modifie de plus de 15 % la valeur de l'un quelconque des paramètres indiqués ci-dessus, il devient impossible d'avoir une explication cohérente des phénomènes observés. Ceci ne permet pas d'affirmer que la valeur de chacun de ces paramètres est exacte, mais seulement que tous ces paramètres, pris conjointement, définissent une relation contrainte-déformation qui conduit à une très bonne image des faits observés.

4.3 On remarquera par ailleurs que la loi d'évolution proposée pour le retrait et le fluage correspond approximativement à la loi proposée par le CEB pour des épaisseurs de 20 cm (réédition 1970). Les Rédacteurs de l'Instruction française sur l'emploi du béton précontraint dans les ouvrages d'art (1965) avaient proposé déjà une loi très voisine corrigeant sur ce point les Recommandations du CEB dans leur première rédaction (1966).

5.) Déformations localisées autour d'un ancrage noyé de précontrainte

L'une des sections de mesure se trouve placée immédiatement en arrière d'un ancrage de précontrainte, dont le câble avait été tendu avant le bétonnage du tronçon étudié. Par fluage, le béton a continué à se déformer immédiatement sous l'ancrage. Les parties de béton situées au-delà du joint de coulage ont suivi cette déformation et il en est résulté une nette décompression locale, dont on peut suivre l'évolution dans le temps (Fig. 5).

Cette étude met en évidence la finesse des résultats de mesure qui ont été obtenus. Rappelons que la méthode de mesure a été mise au point par le Centre Expérimental du Bâtiment et des Travaux Publics, sous la direction de M. MAMILLAN et a déjà été employée avec succès sur des ponts au Maroc et en France. La campagne des mesures sur le pont sur l'Oise a été faite par cet organisme à la demande des Ponts et Chaussées de l'Oise. M. BELMAIN, Ingénieur des Ponts et Chaussées, nous a autorisés à présenter ces résultats et nous l'en remercions vivement.



I

Comments to Theme I
Remarques sur le thème I
Bemerkungen zum Thema I

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As one of the instigators of this conference, it seems appropriate to follow Professor Wastlund's remarks with a short summary of what I thought might result from our discussions here.

Many writers have illustrated the damaging failures that can result from designs which do not make adequate provision for the effects of creep, shrinkage and temperature changes. Whole buildings have literally torn themselves apart. Precast post-tensioned tees have had the ends of the webs pulled away from the rest of the beams. Supporting shelves have been separated from the sides of main girders. Columns have bowed, cracked and completely failed. The list could be greatly extended and usually results in excessive expenditures where fairly inexpensive precautions could have originally been adopted.

There seem to be at least three procedures that can be followed. One is to cut structures into independent, completely separated units of moderate length within which experience has demonstrated that the volumetric changes can be accommodated. Another is to attempt to estimate the magnitude of the forces developed by such volumetric reductions and provide reinforcement to transfer these forces properly from one end to the other. Then there are unusual solutions such as precompressing the structure by means of a prestressed peripheral beam endeavoring to match the precompression with the internal tensions from the effects of creep, shrinkage and temperature change. The first is almost entirely a matter of experience and judgment. It is hoped that the last two can supplement such experience and judgment with definite mathematical techniques.

The design of structural frameworks to accommodate gravity loads has become a very elaborate technical procedure. Whether slope deflections, moment distribution or matrices are used, engineers all over the world can communicate freely, knowing that the techniques themselves are well understood and that one need only discuss the numerical details. Shelves full of text books, building codes, the work of ACI 318, CEB/FIP, and hosts of others all deal with intricate and highly detailed procedures for properly providing support for gravity loads. These references also caution that the effects of creep, shrinkage, temperature changes shall be properly accounted for. Even such climatological phenomena as snow, wind, earthquake, and now blast have good appraisals of the loading conditions and use pretty much the same procedures and strength levels so designers can communicate very well.

For the effects of creep, shrinkage and temperature, codes and authorities usually dismiss the entire subject with a caution to "consider them."

Although my personal contact with the problem goes back longer than I care to disclose, it was in 1941 when designing a storage structure for an ordnance depot involving double 20-ton cranes in a building 540 by 1470 feet (165 x 448 meters) that we were really put on our mettle to develop mathematical techniques to deal with creep, shrinkage and temperature. This study arose not so much from the technical aspects as from the fact that the completed design was over a million dollars less expensive than competing designs and so it became imperative to investigate some of these unusual situations instead of relying entirely upon past experience. They worked!

Even after the parameters have been isolated and their relative effects evaluated and after coefficients for the volumetric changes have been determined on moderate-size specimens under strict laboratory control, there is still the problem of determining what are appropriate values for inclusion in design calculations since exposures vary within the length of the structure. Should the type of mix, kind of aggregate, make of cement, order of placement or any other parameters be found to contribute heavily, they should be taken into account. If they play a minor part, they might be combined into a single factor.

As an example, knowing the behavior of a cylinder or bar of concrete in a closely controlled storage, what should one use for concrete placed at different times under different temperature and humidity conditions (over which the designer has little control), assembled rigidly into a completed structure, exposed conceivably to ice and snow at one end and possibly the direct rays of the sun on the other?

To get a step further, if the bases of all of the columns in a one-story structure were on ball bearings, the volumetric changes of the roof system would have little effect. It is only when the columns are anchored and forced to bend that they provide restraint for the shrinking roof system. Need the characteristics of the subgrade be determined? If the column can tilt its foundation more readily than it can itself be sprung, the restraint is lessened. If the spandrel beams are free to slide and roll over on stiffly anchored columns, relief will be afforded.

The hope, then, was that, by comparing the experiences of a considerable number of design engineers, it would be possible to establish the more important parameters, gain some idea of the probable spread in practical structures (possibly having to divide into different climatic zones), somewhat resolve the questions of tilting, springing, bowing and so forth and conceivably aim for an eventual mathematical procedure that could be as well agreed upon as moment distribution. Such a method would considerably simplify the work not only of the designer but of the code writer and particularly of insurers who must pass upon the responsibilities for faulty designs.

These were the aims and goals with which this seminar set out. You can judge the progress made. It was excellent at the companion ACI meeting in New York last spring, as Bob Philleo will report this afternoon.

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