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IVc1

Experimental study of towers for high tension lines

Versuche über Hochspannungsleitungsmaste

Estudo experimental de postes para linhas de alta tensão

Etude experimentale de pylônes pour lignes à haute tension

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INTRODUCTION

The fact that the supporting structures for the high tension electric lines represent a large proportion of their total installation costs justifies a thorough study of these structures. Furthermore as large numbers of identical structures are employed, it follows that a small economy achieved in individual elements represents a very large total economy.

This is one of the problems for which it will be possible to obtain important economies through the discussion of the fundamental problem of safety, especially on a statistical basis. The fact that great numbers of structures are being dealt with gives a large volume of information relative to the behaviour of these structures and makes it possible to handle this information on statistical and economic bases. These statistical studies, which have only just been initiated, are not presented here.

The principal difficulties of the application of analytical methods derive from structures in question not being isostatic and their strength being influenced by buckling. The rigorous treatment of buckling is particularly difficult due to the large number of elements and to the difficulty in defining the rigidity of joints.

The experimental methods, especially tests on prototypes, are particularly fruitful in this case, and their drawbacks, principally that of causing the destruction of the structure, in failure tests, are minimized by the fact of having to construct numerous identical structures.

The analytical methods are very useful for a preliminary design, but it is only by experimental tests that convenient information can be obtained of the overall behaviour of a structure and its joints. Experimental tests thus allow a more exact design and they moreover supply information for improving analytical studies.

This paper presents the methods followed at the Laboratório Nacional de Engenharia Civil in carrying out tests on models and prototypes and gives a summary of some of the conclusions derived from the tests (1).

2. *Model tests.*

Towers for two high tension lines (both 150 Kv) were studied by by means of models. For one of the lines two types of towers were studied. Only one type was studied for the other line.

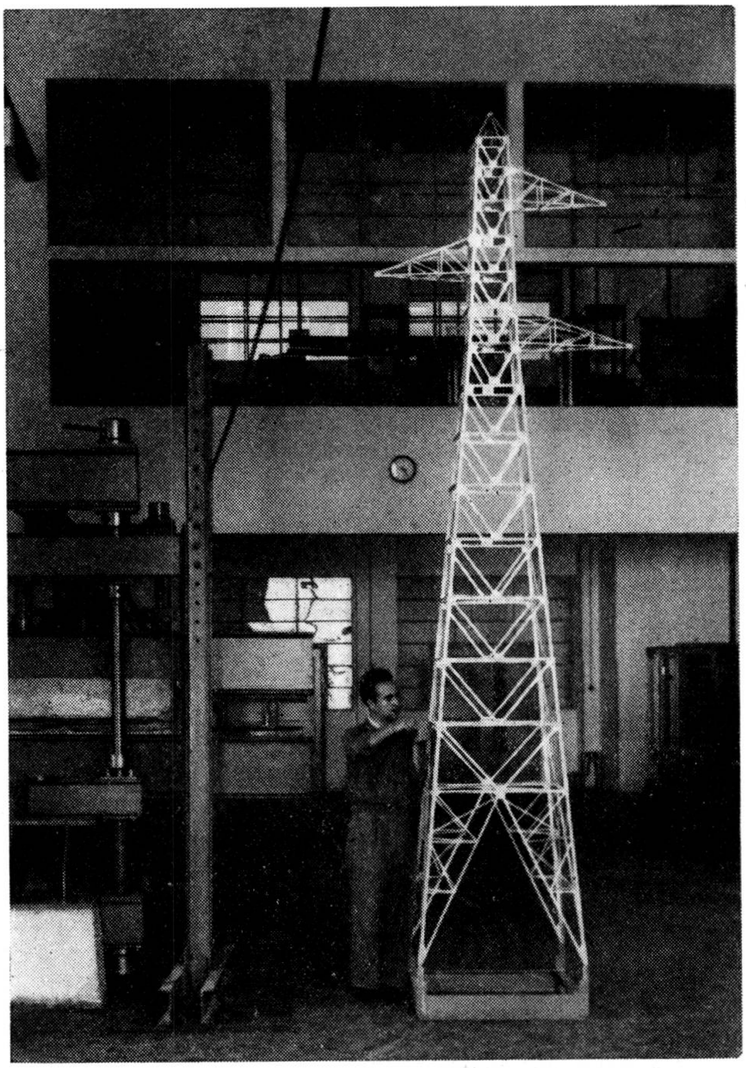

FIG. 1. Model to a scale of 1/6

Three models were built in which perfect geometric similarity in relation to the prototypes was maintained; the two first ones to a scale

(1) The tests were sponsored by the Companhia Nacional de Electricidade that is in charge of high power distribution in the country. The analytical studies and design of the towers was also undertaken by this company.

of 1/6 and the third to a scale of 1/7. Fig. 1 and 2 show two of the models. The model in fig 2 was photographed near by a prototype.

The angle shapes of the models were obtained by bending steel sheets and the joints, made by means of screws, were also reproduced faithfully to scale.

The test method consisted in applying loads and measure the displacements and strains. Fig. 3 shows one of the models during test.

The loads were applied by means of weights, the loading hypotheses considered in the analytical calculation being reproduced. Thus vertical

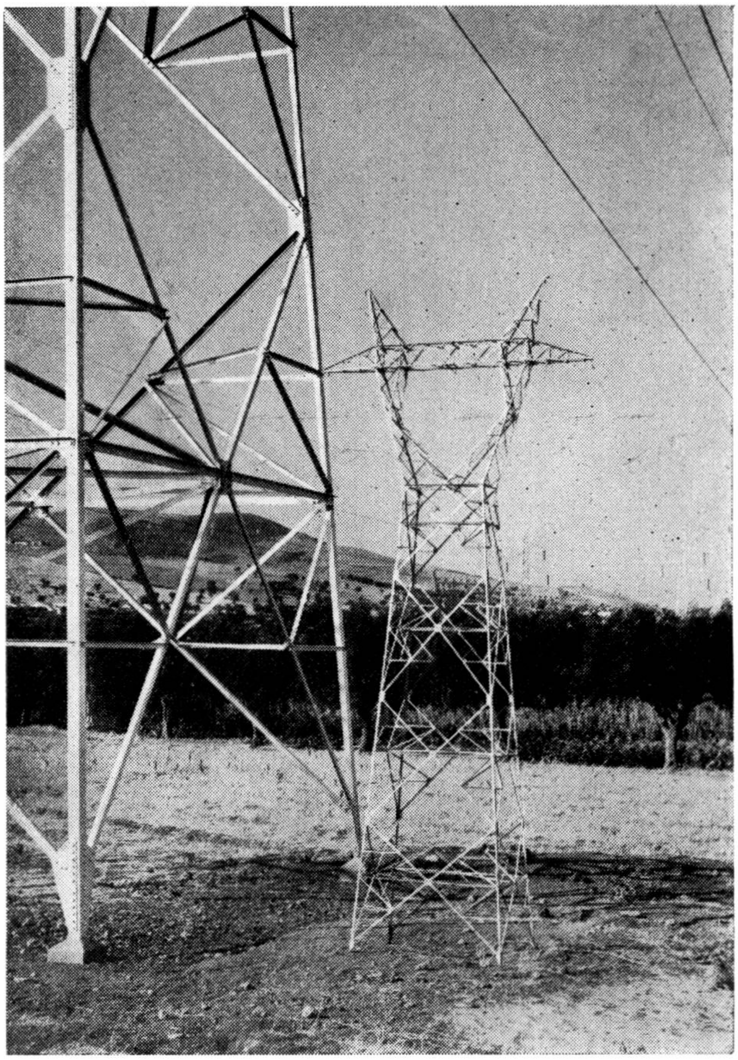

FIG. 2. Model to a scale of 1/7

loads were applied corresponding to the weight of the various elements and horizontal loads which reproduced the loadings due to wind and breakage of cables. The scale of forces was chosen such that the stress limit of proportionality was not exceeded.

The measurement of strains was carried out by Huggenberger strain--meters with an 8 cm base length, three strain-meters being placed at each section, at the vertex and at the edges of the legs. By this means it was possible to determine the strains at the centre of gravity of the sections with perfectly satisfactory accuracy. For the model in which the greatest number of measurements were taken the strains were measured at 48 sections.

For the measurement of displacements deflectometers graduated in 0.1 and 0.01 mm were employed. They were connected to the structure by steel wires.

Taking into consideration the scale of the models and forces, the values determined in the tests were transferred to the actual structure.

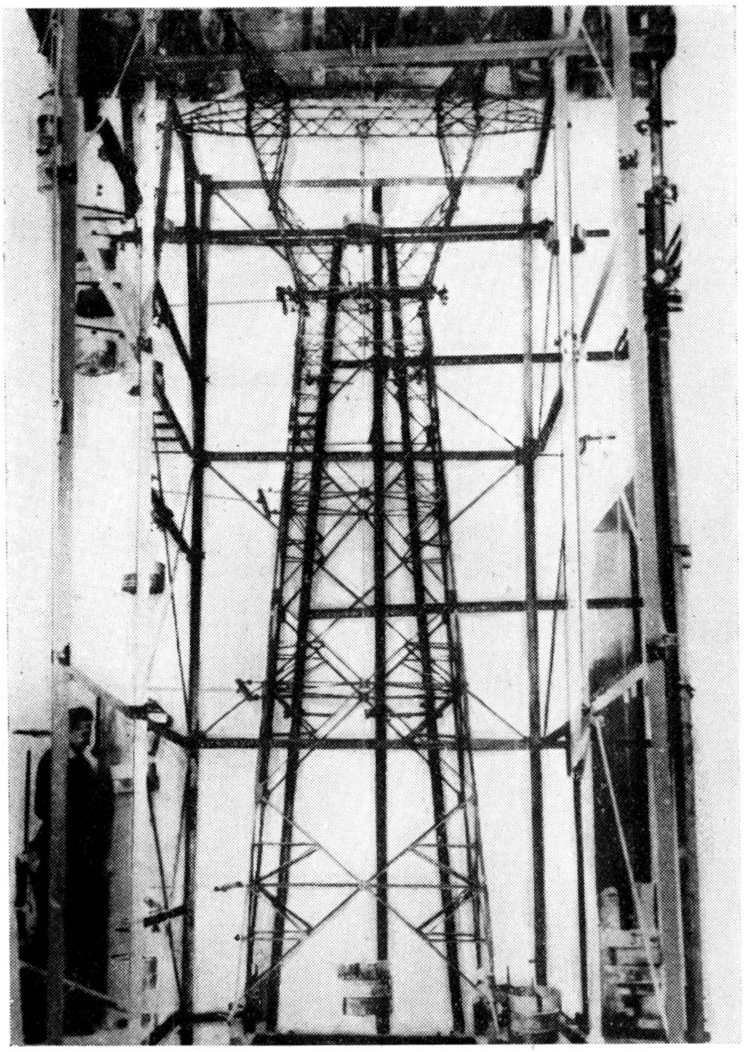

FIG. 3. Model during test

In order to calculate the stresses from the strains measured, accurate determinations of Young's modulus were made on the actual members used in the construction of the models.

From the comparison of the stresses determined analytically and experimentally, fig. 4, it was possible to conclude the following.

For the hypothesis in which the loading was symetrical in relation to the axis of the towers, there was, as was to be expected, relative agreement between the results of the tests and the analytical calculations.

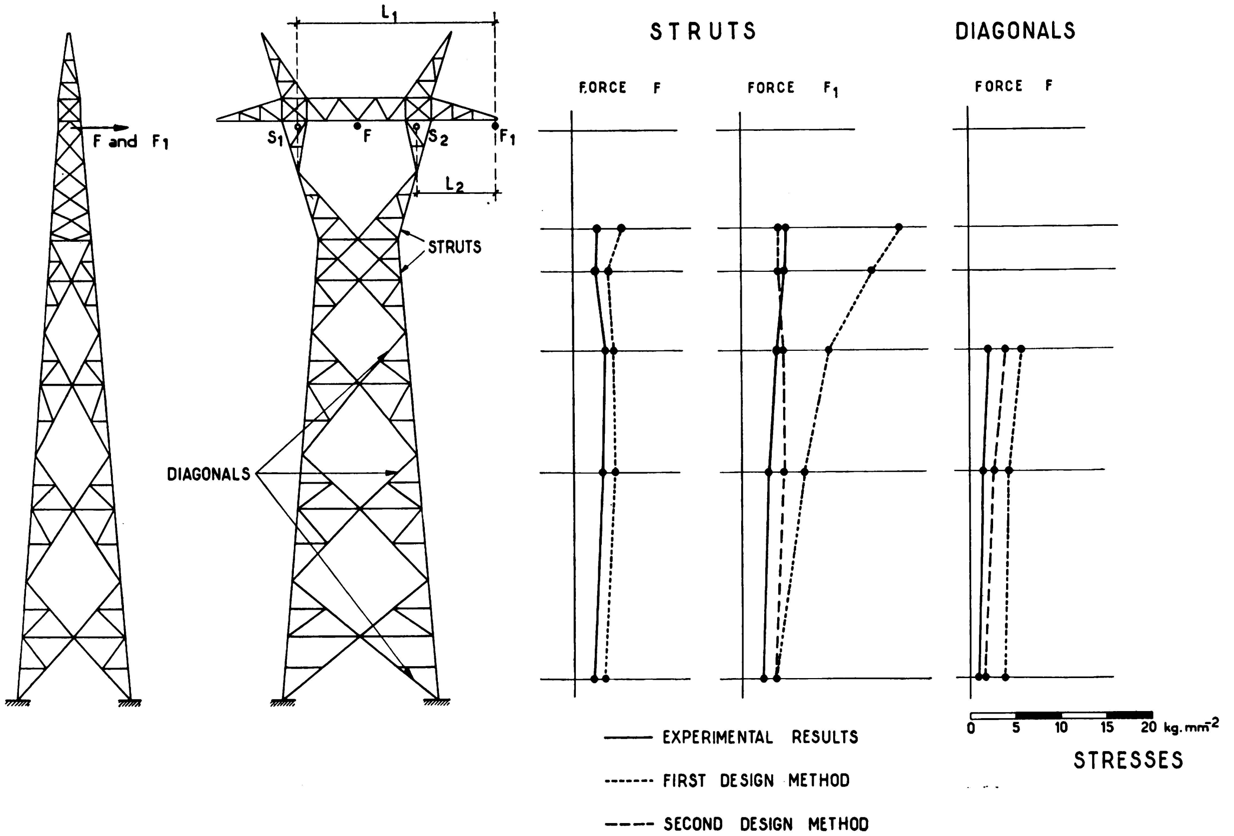

FIG. 4. Results of model test

For the hypothesis in which the loading is not symmetrical and which corresponds to rupture of a lateral cable the stresses determined experimentally differ considerably from the analytical ones. This fact results from the analytical method first adopted being based on unsuitable hypotheses.

In fact, in this first method, to calculate the stresses due to cable rupture the hyperstatic reactions of the frame, which constitutes the upper part of the tower shown in fig. 4, were determined by the current method. The stresses in the members of the tower body were calculated by resolving in an arbitrary way these reactions along the supporting arms of the frame and transmitting the forces thus obtained to the triangular beams which constitute the four sides of the tower body. By this process large stresses were obtained, principally in the struts, whilst, on the other hand, the experimental results showed that the stresses in the struts due to torsion forces were practically negligible.

This led to the adoption of a second calculation method which simply consists in resolving the torsion moment at a given level into forces acting on the sides of the tower, forces which are inversely proportional to the distance between the sides.

The results of this second method are very much closer the experimental results than those of the first method.

For the calculation of the supporting arms of the frame, the application of the force due to rupture of a lateral cable corresponds to the application to each arm of forces S_1 and S_2 (fig. 4). Considering the isostatic system it is clear that $S_1/S_2 = L_2/L_1$. The torsion rigidity of the arms can however lead to values of S_1/S_2 less than the above. For the tower of fig. 4, isostatically, S_1/S_2 is equal to $1/2.5$, whilst experimentally S_1/S_2 was equal to $1/3.5$.

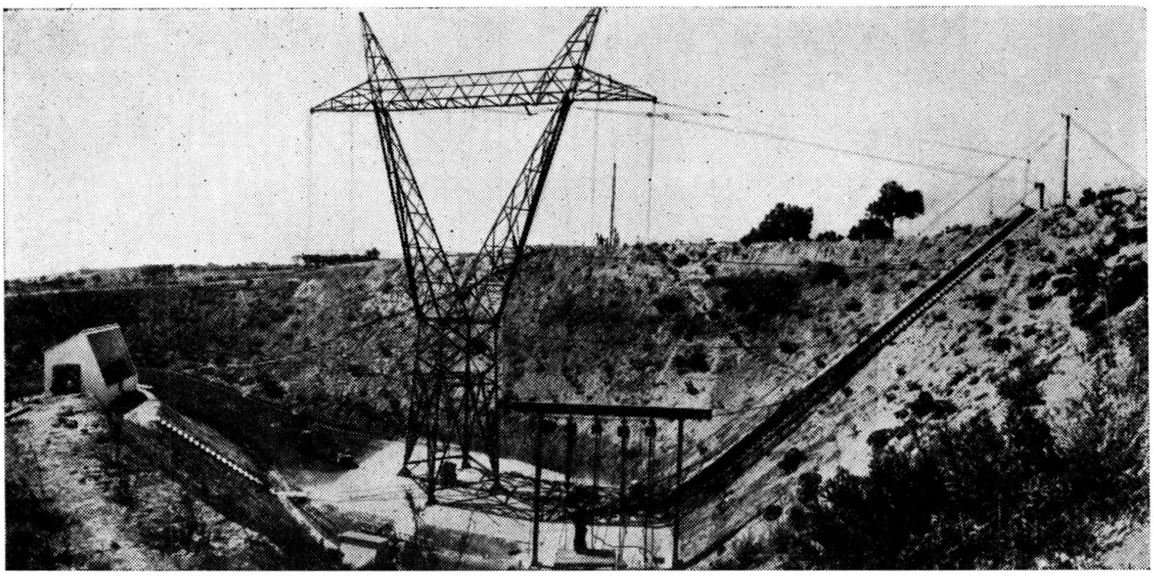

FIG. 5. Installation for the test of prototypes

3. *Prototype tests.*

In order carry out tests on prototypes a special installation was built on the grounds of the Laboratório Nacional de Engenharia Civil, fig. 5, 6 and 7.

The installation consists of a reinforced concrete slab in which steel beams were embedded. The connection of the bases that receive the

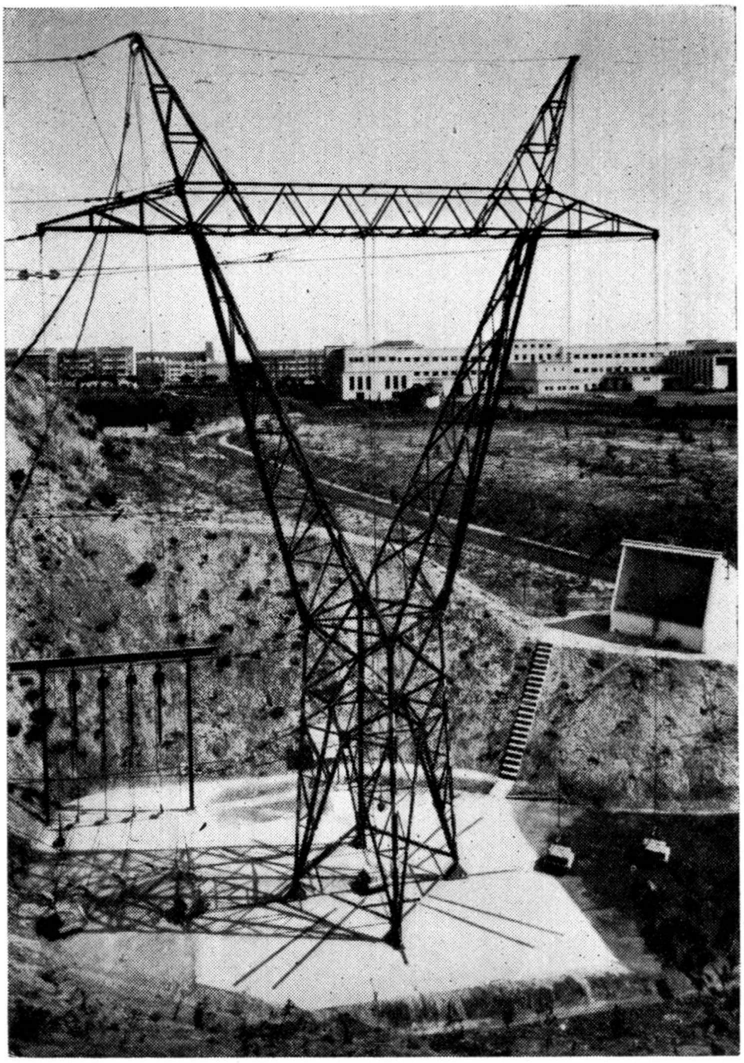

FIG. 6. Installation for the test of prototypes

struts is made through bars that are welded to the upper face of the beams.

The application of the loads is made by means of cables and pulleys. The cables are pulled by chain hoists suspended from a frame.

As horizontal forces had to be applied at a distance of about 20 m from the tower base advantage was taken of the natural configuration of the ground. So it was not necessary to construct large structures to absorb the cable reactions. Use was made of small metallic towers and of a reinforced concrete beam that goes up the side of a slope.

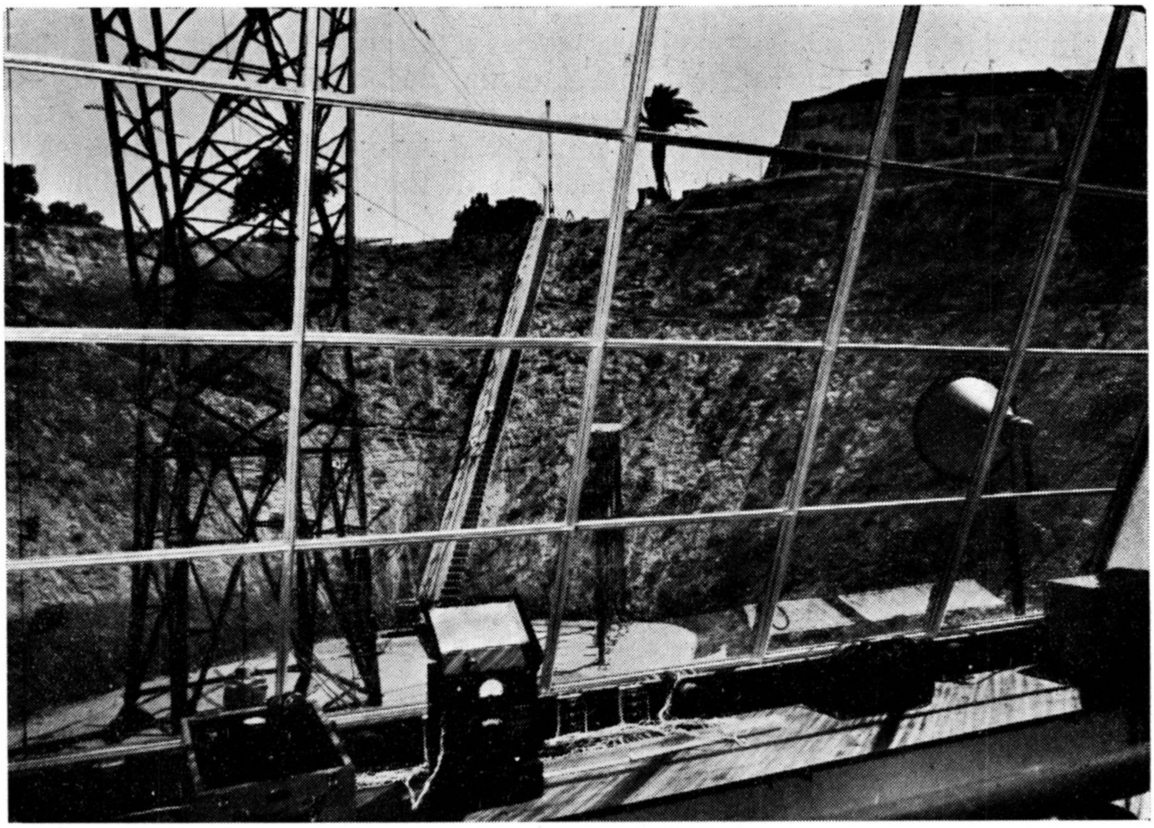

FIG. 7. Observation post

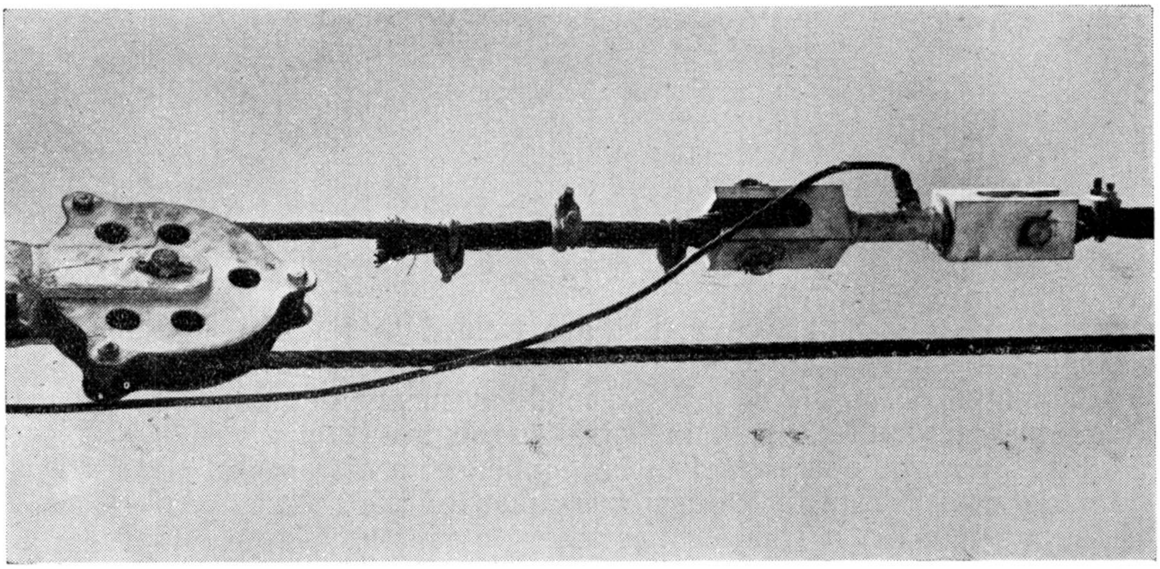

FIG. 8. Dynamometer and cables for the measurement and application of forces

For measurement of the forces specially built dynamometers, fig. 8, are introduced in the cables near the tower under test. The dynamometers have acoustic strain meters inside them. They have capacities from 1.5 to 10 tons and measure the forces with errors of less than 0.5 % of their capacity.

For measurement of displacements simple systems of rules and cursors are employed, the rule being fixed to the ground and the cursor

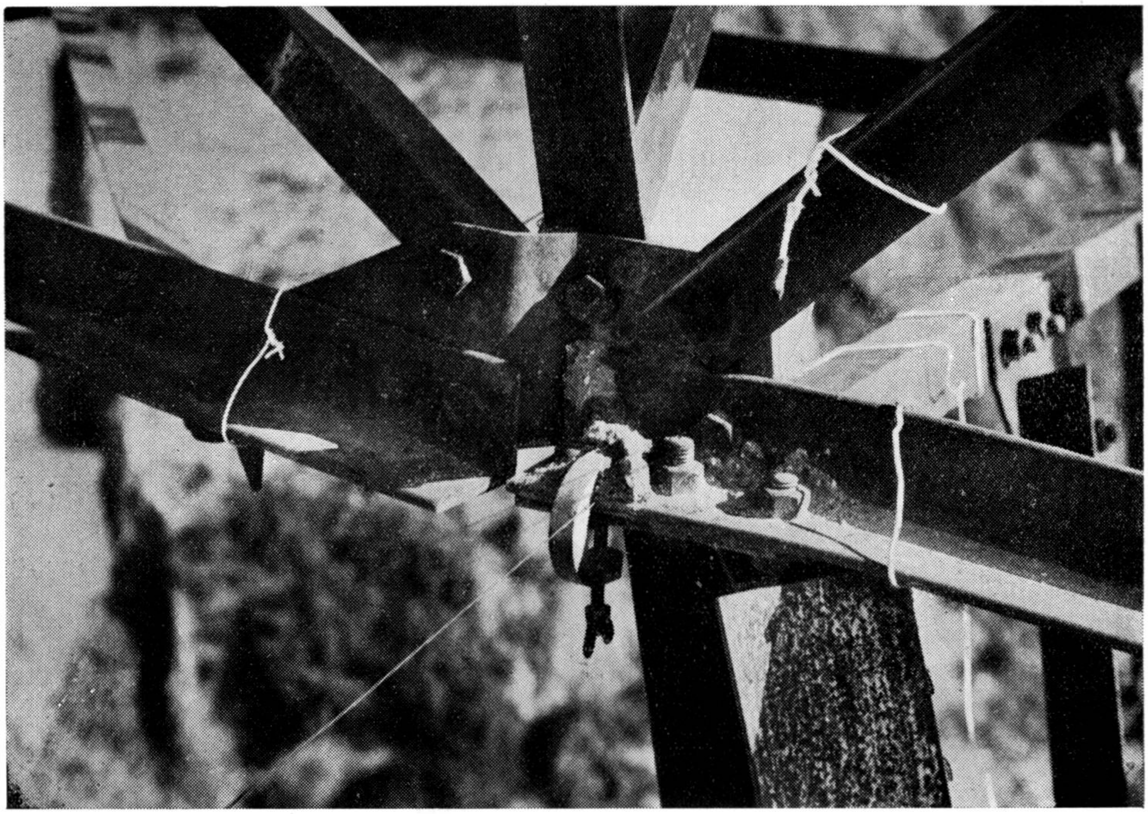

FIG. 9. Rupture of a gusset plate

to a wire connected to the structure under test. To maintain the wire under constant stress, springs having suitable deformability are employed. Optical levelling is used to measure the displacements of the tower base.

Strains are measured by electrical resistance strain gauges protected from hygrometric variations by the method developed by Philips.

A small observation post was built, for the control of the tests, fig. 7. The simple system of manual regulation of the forces by chain hoists proved to be perfectly satisfactory. Instructions to the operators are given by loud-speakers from the observation post.

The first tower tested in this installation was for the 220 KV lines at present being constructed in the country. The base of this tower is rotated through 45° in relation to the direction of the line.

These tests revealed the defficiency of certain types of joints fig. 9, which were thus replaced by more suitable ones.

In these tests the relation between the forces S_1 and S_2 acting on the upper arms was 1/3 whilst isostatically it should be 1/3.3, values which are not very different.

At present tests are being carried out on a type of tower similar to the one above but designed for another type of cable.

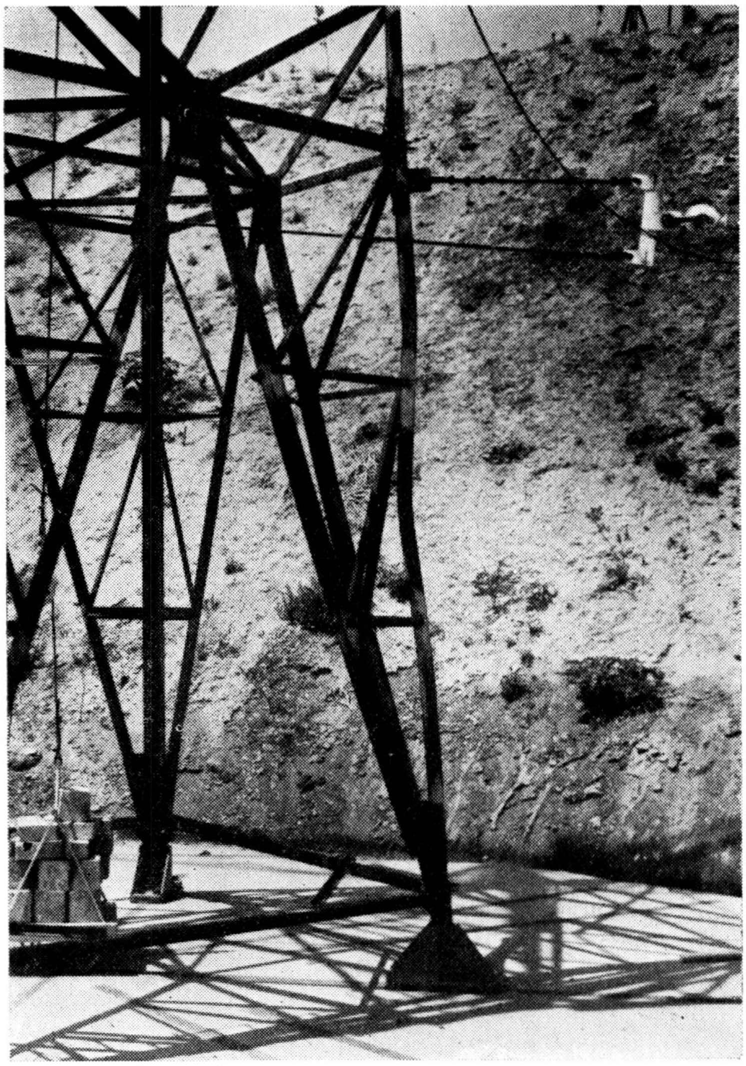

FIG. 10. Buckling of a strut

The failure load for the transverse forces, experimentally determined, was 1.5 times the working load. Failure occurred through the buckling of the strut near the base, fig. 10.

SUMMARY

This paper presents the methods followed at the Laboratório Nacional de Engenharia Civil in carrying out tests on models and prototypes of towers for electric lines of 150 and 220 KV. Some conclusions obtained from the tests are also given briefly.

ZUSAMMENFASSUNG

In der vorliegenden Arbeit werden die Methoden beschrieben, die vom Laboratório Nacional de Engenharia Civil in Lissabon für Versuche mit Modellen und Prototypen zur Prüfung von Leitungsmasten für 150 und 220 KV verwendet wurden. Einige Ergebnisse dieser Untersuchungen sind kurz dargestellt.

RESUMO

Nesta comunicação apresentam-se os métodos seguidos pelo Laboratório Nacional de Engenharia Civil na realização de ensaios sobre modelos e sobre prototipos de postes metálicos para linhas eléctricas de 150 e 220 KV. Referem-se também alguns resultados obtidos nestes ensaios.

RÉSUMÉ

Dans ce rapport sont presentées les méthodes suivies au Laboratório Nacional de Engenharia Civil pour la réalisation d'essais sur modèles et sur prototypes de pylônes métalliques pour lignes électriques de 150 et 220 KV. On y donne aussi quelques résultats obtenus au cours de ces essais.

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IV c 2

Castellated construction

«Ausgezahnte» Stahlbauten

Construção «ameiada»

Construction crènelée

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Scunthorpe

The basic idea.

Methods of achieving economy in steel, with consequent reduction in the deadweight of a structure, while at the same time maintaining or increasing the strength of a given span, have been the objectives of the Consulting Engineer and Architect ever since steel framing as we now know it was accepted as an independent load carrying unit in building construction.

While castellated construction of a type had been used on the Continent to some extent, I feel it is correct to say that this method of construction had not been seriously investigated until 1937, when Geoffrey Murray Boyd was granted a provisional patent and ultimately a British Patent 498,281, and at the same time obtained Patent Cover in several countries outside Britain.

Largerly, no doubt, due to the incidence of the War Period, the foreign cover lapsed but the British Rights have now been extended to 1960.

A series of tests were made to determine the correct shape and size of a castellation to produce a section which would normally be capable of taking a comparatively light load over a span of greater dimensions than average, with reasonable security against high bending stress, and also excessive deflection due to span. At the same time it was desirable that heavier loads could be carried over spans which would normally require a compounded section or the use of a much heavier member. It is, of course, appreciated that this problem of comparative sections is one which is particularly reflected by the much reduced range of beams produced by British Rolling Mills in comparison with the greater availability of range in other countries.

The principle of castellation.

To meet the requirements of depth increase without weight increase, it is necessary to expand a standard rolled steel beam, channel or other suitable section, by a longitudinal cut in such a way, that when the two cut pieces are rejoined the resultant section has a depth which for practical considerations is maintained at one and a half times that of the original section.

This increase in depth (without increase in the weight) must obviously give an improvement in the geometrical properties of the section.

As will be seen from the tables (Plate No. 1) the major moment of inertia is increased by approximately 135 % while the corresponding section modulus is increased by approximately 56 %.

These increases are of considerable value in restricting deflection, while at the same time the resistance of the beam to bending is increased.

It must be emphasised that in the production of the castellation, more is involved than the mere cutting and welding of a beam to a shape which ultimately results in a series of apertures. The cutting must be calculated on a predetermined formula so that the greater capacity of the expanded section can be fully developed for resistance to bending and web buckling.

The line of the original cut in the basic section is so arranged that when the section is expanded and welded together along the neutral axis, the depth of the castellation opening is equal to the depth of the basic section. This arrangement, coupled with a suitable slope to the sides of the castellation opening, ensures full protection against the effects of longitudinal shear at the neutral axis, giving maximum shear resistance throughout the whole section and establishing as high a value as is possible in the capacity of the beam to resist web buckling stresses at the bearings, and at such points where concentrated loading may be applied to the beam.

Consequently it will be seen that the expanded sections have much improved properties to take loads over larger than normal spans without any increase in the weight of steel involved for the beam. In cases where short spans carrying very heavy loads are concerned, the castellated form of construction is not suitable as, in the majority of cases of this character, shear strength is one of the controls, and it will be obvious that the principle of castellation is not so readily applicable, owing to the web apertures. Nevertheless there has been no occasion, even including those cases when tests have been taken to destruction, where shear strength has proved the limiting factor.

Method of manufacture.

The two processes involved are burning and welding which are both within the capabilities of any Structural Fabricating Shop.

Castella Beam	Minimum Equivalent Joist or Joist Compound Section	Saving		Castella Beam		Equivalent Section	
		Wt. in lbs./ft.	Per-centage	Moment of Inertia x-x	Section Modulus x-x	Moment of Inertia x-x	Section Modulus x-x
36 ×7½×95 lbs.	24×7½ joist with 1–12 ×¾ plate on each flange-weight 160 lbs.	65	41	5918	329	4832	379
33 ×7 ×75 lbs.	24×7½ joist with 1–12 ×½ plate on each flange-weight 140 lbs.	65	47	3911	237	3950	316
30 ×7½×89 lbs.	22×7 joist with 1–12 ×⅝ plate on each flange-weight 130 lbs.	41	32	3910	261	3279	282
30 ×6½×65 lbs.	22×7 joist with 1–12 ×½ plate on each flange-weight 120 lbs.	55	46	2863	191	2905	253
27 ×8 ×80 lbs.	22×7 joist with 1–12 ×⅜ plate on each flange-weight 110 lbs.	30	27	3035	225	2531	223
27 ×7 ×75 lbs.	24×7½×95 lbs. R.S.J.	20	21	2699	200	2533	211
27 ×6 ×55 lbs.	24×7½×95 lbs. R.S.J.	40	42	1959	145	2533	211
24 ×8 ×75 lbs.	24×7½×95 lbs. R.S.J.	20	21	2301	192	2533	211
24 ×6 ×62 lbs.	22×7 ×75 lbs. R.S.J.	13	17	1700	142	1677	152
24 ×6 ×50 lbs.	22×7 ×75 lbs. R.S.J.	25	33	1442	120	1677	152
22½×6 ×59 lbs.	22×7 ×75 lbs. R.S.J.	16	21	1477	131	1677	152
22½×6 ×45 lbs.	18×7 ×75 lbs. R.S.J.	30	40	1143	102	1151	128
22½×5 ×42 lbs.	18×6 ×55 lbs. R.S.J.	13	24	1000	89	842	94
21 ×8 ×70 lbs.	20×7½×69 lbs. R.J.S.	19	21	1665	159	1673	167

Castella Beam	Minimum Equivalent Joist or Joist Compound Section	Saving		Castella Beam		Equivalent Section	
		Wt. in lbs./ft.	Per-centage	Moment of Inertia x-x	Section Modulus x-x	Moment of Inertia x-x	Section Modulus x-x
21 ×6 ×57 lbs.	20×6½×65 lbs. R.S.J.	8	12	1253	119	1226	122
21 ×6 ×46 lbs.	20×6½×65 lbs. R.S.J.	19	29	1028	98	1226	122
21 ×5½×40 lbs.	18×6 ×55 lbs. R.S.J.	15	27	883	84	842	94
19½×5 ×35 lbs.	16×6 ×50 lbs. R.S.J.	15	30	662	68	618	77
18 ×8 ×65 lbs.	22×7 ×75 lbs. R.S.J.	10	13	1163	129	1677	152
18 ×6 ×54 lbs.	20×6½×65 lbs. R.S.J.	11	17	893	99	1226	122
18 ×6 ×44 lbs.	18×6 ×55 lbs. R.S.J.	11	20	767	85	842	94
18 ×5 ×32 lbs.	15×6 ×45 lbs. R.S.J.	13	29	516	57	492	66
15 ×6 ×40 lbs.	15×6 ×45 lbs. R.S.J.	5	11	485	65	492	66
15 ×5 ×30 lbs.	15×5 ×42 lbs. R.S.J.	12	29	345	46	428	57
15 ×4½×25 lbs.	10×6 ×40 lbs. R.S.J.	15	38	284	38	205	41
13½×4 ×21 lbs.	10×5 ×30 lbs. R.S.J.	9	30	188	28	146	29
12 ×6 ×35 lbs.	15×5 ×42 lbs. R.S.J.	7	17	266	44	428	57
12 ×4 ×18 lbs.	10×4½×25 lbs. R.S.J.	7	28	129	21·6	122	24 5
10½×4 ×16 lbs.	9×4 ×21 lbs. R.S.J.	5	24	93	17 7	81	18
9 ×3 ×12 lbs.	7×4 ×16 lbs. R.S.J.	4	25	50	11·0	40	11·3
7½×3 ×11 lbs.	7×4 ×16 lbs. R.S.J.	5	31	32	8·4	40	11·3
7½×1¾×6·5 lbs.	5×3 ×11 lbs. R.S.J.	4·5	41	15	4·3	14	5·5

PLATE N° 1

Mechanical longitudinal profile burning mechines are used for the cutting process adjusted by means of steel templates, a separate template being necessary for each depth of section used, and in its preparation appropriate adjustments in profile must be made, so that the resultant cut in the beam is to the correct shape and size. Cutting is continuous with the exception of spaced intervals which must be left uncut (temporarily) in order to avoid undue distortion.

The two resultant pieces are then moved along one castellation or turned end-for-end and assembled by means of tack welding, and then fully welded by the deep penetration electrode technique thus obviating any edge preparation. It is essential to control burning and welding throughout all stages to avoid distortion and the consequent additional operation of straightening. To date no practical alternative method of preparing the two sections has been found to substitute for the gas burning process. The welding is done by manual operation. Alternatives, which would, however, involve a large amount of capital cost, are resistance welding and forge welding.

British Patent 713,794 has, however, been granted to cover a mechanical forge welding process which is also covered by Patents in the majority of countries outside Great Britain.

Variations of the castellated system.

The castellated form of construction can, of course, be applied not only to standard rolled beam sections, but with equal facility to other sections, and in fact to a combination of two varying sections whether of the same type or not. These variations are detailed on Plate No. 2 which also illustrates two types of cruciform sections, the fabrication of which is also simplified by the use of the castellated construction system.

By a variation in procedure, tapered beams can also be formed not only at a lower cost, but with a still further saving in material. This is accomplished quite simply by directing the line of cutting along the web at a pre-determined angle to the flanges, thus by turning one portion end-for-end the tapered section is formed.

It will be realised that the system can provide for an infinite variety of sections in castellated form.

Economies of the system.

Plate N°. 1 is a self-explanatory table which shows in detail the savings in weight of steel by the incorporation of castellated beams of equivalent or greater strength to the minimum equivalent rolled steel joist or joist compound section. These comparisons are, of course, made with British Standard rolled sections.

Plate No. 3 shows the typical load-span relationship of a $36'' \times 7\,{}^{1}/_{2}'' \times \times 95$ lbs. catellated beam as against a British Standard rolled joist $24'' \times 7\,{}^{1}/_{2}'' \times 95$ lbs. and is for convenience shown in graphical form.

The portion of the curve A - B indicates the permissible load governed by the extreme fibre stress of 10 tons per sq. ins.

At point B deflection begins to control the permissible uniformly distributed load as in no case is the load allowed to exceed a value at which the deflection would be more than 1/325 span.

The maximum uniformly distributed load on a castellated beam with its compression flange laterally supported, is controlled by the buckling

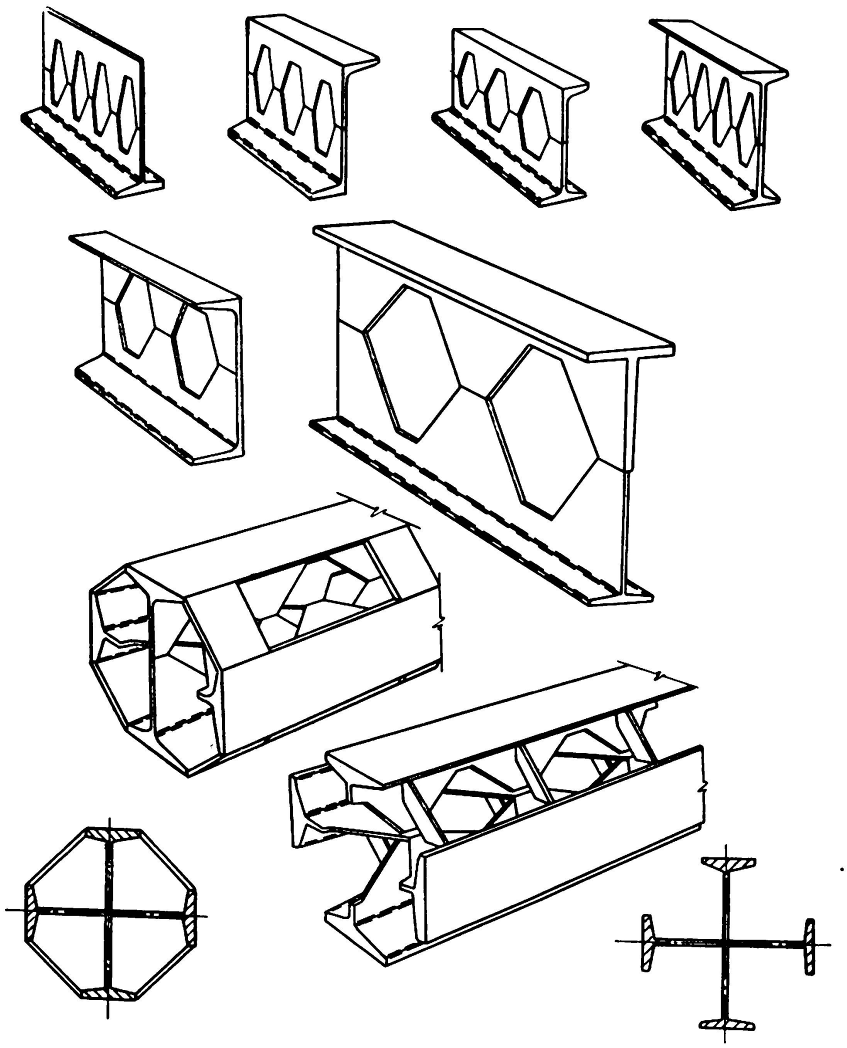

PLATE N° 2. Typical applications of castellation

value of the web. The length of web on the neutral axis of the section between the end of the beam and the first castellation is standard for each section. The safe web bucking value of this portion of web thus controls the maximum permissible end reaction, and hence the allowable load on the beam. This value is shown on the graph by point C, and is the maximum safe load with end castellation left open. The area of web under buckling and hence the maximum permissible load may be increased by filling in the end castellation. This increased value is indicated by point A on the graph.

The values for web buckling are calculated on a minimum bearing length of 1″ for all castellated beams over 13 $^1/_2$″×4″×21 lbs. and on a length of $^1/_2$″ for all smaller sections. In all cases where the safe web bearing load is less than the safe web buckling load point A is established by the former.

The summary of all factors demonstrate that the main applications are in such cases where comparatively light loads have to be taken by means of beams of large spans, or greater length, than would normally be encountered.

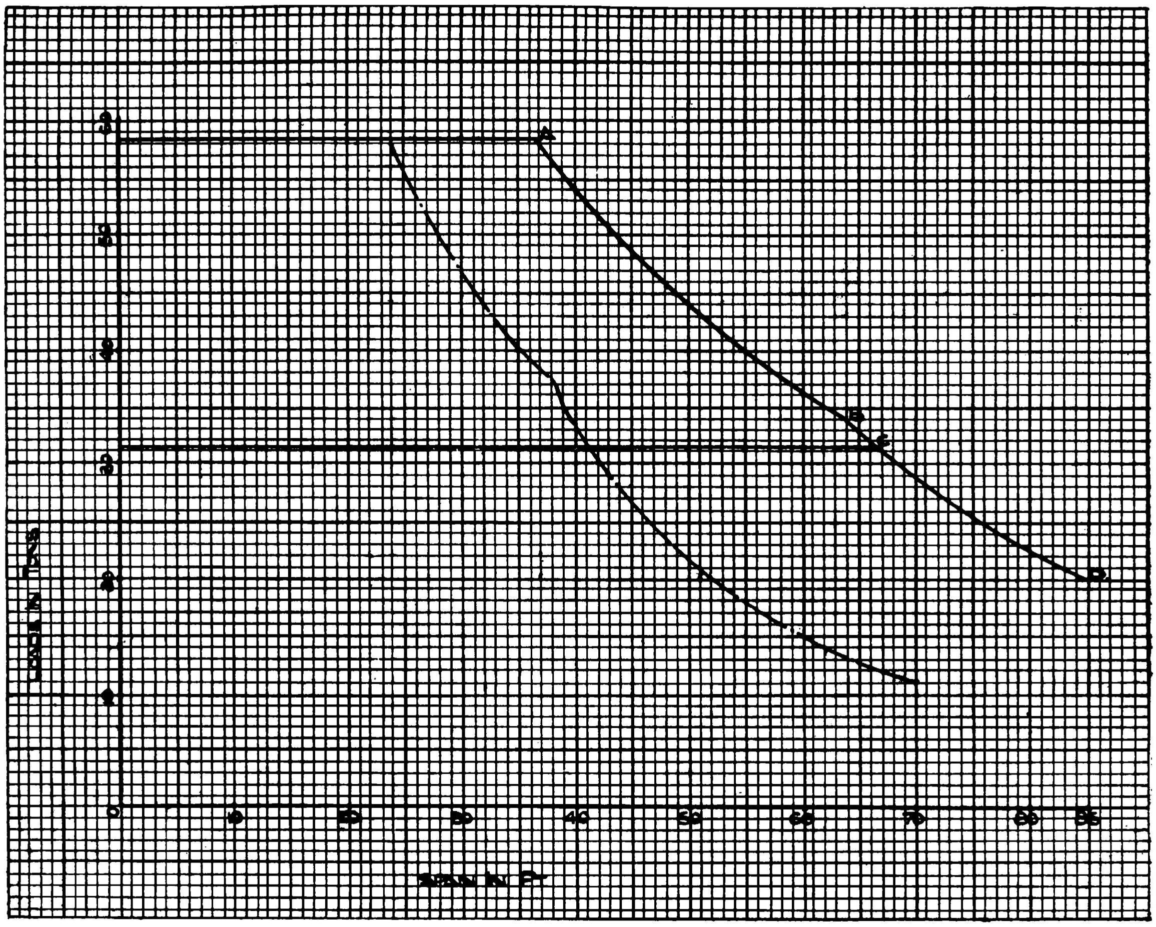

PLATE N° 3. Typical load span relationship between 24″ × 7 ½″ × 95lbs rolled joist (chain dot) and same joist castellated to 36″ × 6 ½″ × 95lbs (full lines)

Broadly speaking, and despite the increased fabrication costs which are involved, it can be stated as a result of experience, that the average saving in weight varies between 11 % and 47 %, and as a rough guide it has been found that the saving in money is approximately half that percentage, i. e. say from 6 % to 24 %.

A specific comparison was made in the case of a typical office type building, consisting of three storeys with flat roof, the overall dimensions of the building being 195 feet long, 50 feet wide centres of stanchions with a storey height of 12 feet, and a total height of 36 feet. The clear span of the floor beams was taken at 50 feet, and the centres of the columns in the length of the building 13 bays at 15 feet.

Only the main floor and roof beams were assumed as being made up in castellated construction. The columns, wall beams and tie beams were allowed for in normal rolled sections.

The following is an analysis of the comparison of the steel weights: — saving in weight of castellated design on the rolled section design – 32.5 %.

Design with Rolled Sections		Design with Castellated sections
200 tons	Total weight of steelwork...	135 tons
15.32 lbs.	Weight per sq. ft. of floor area	10.34 lbs.
1.28	Weight per cu. ft. of building capacity (gross cube of building dimensions on centres)	0.86

In the rolled section design, allowance was made for flange plate curtailment on the 50′0″ span, roof and floor beams. Both the roof and floor beams are compounded in the rolled section design, but there are no compound sections whatever in the castellated design.

Research investigation.

During the course of the development of castellated beams, it became increasingly evident that practical tests should be made on the sections in regard to load capacity, such tests to have particular reference to the effects of web buckling stresses. It is obviously of primary importance to ascertain what stresses and strains arise in a given section from such practical tests as a check on the calculated values, particularly as in the case of castellated beams we are dealing with a section involving certain complexity of stress conditions.

One of the most direct methods of approach to the question of the strains set up in such a section, is by means of the use of electric resistance strain gauges. This method was decided upon after consultation with the United Steel Companies' Research and Development Department, and it was agreed at this stage to carry the tests to destruction.

In regard to web buckling, it will be clear that it must be assumed that some reduction in bearing capacity of the web should be allowed for, relative to a normal rolled steel beam, and this in point of fact is done, as clearly the amount of web available for resistance of buckling stresses over the bearing or at any point where concentrated load may be applied is considerably reduced. Nevertheless a number of tests, all of which have been carried to destruction, have shown that the ability of castellated beams to resist web buckling stresses is higher than might be at first apparent.

The experimental work was conducted primarily to check the theoretical assumptions in this regard, particularly as the primary calculations made in reference to these sections involved a great deal more than a direct application of the ordinary beam theory. A comprehensive set of resistance strain gauges was attached to the test beams so that a final analysis of the results would give a full picture of the behaviour of the sections under load. It is notoriously difficult to simulate the conditions of uniformly distributed load on a beam for test purposes, and in consequence it was decided to use two point loads placed at the quarter points of the span so that the end reaction and the maximum bending moment would be equal to those given by a uniformly distributed load over the whole of the span equal in magnitude to the sum of the point loads.

The top flange of each beam was restrained against lateral failure by a «point contact» lateral support at mid-span so that the test conditions would comply with the requirements of B. S. S. 449/1948 in regard to lateral buckling of the compression flange.

Plate 4 shows the end section of one of the beams after failure had occurred and indicates a typical web buckling failure — the web having

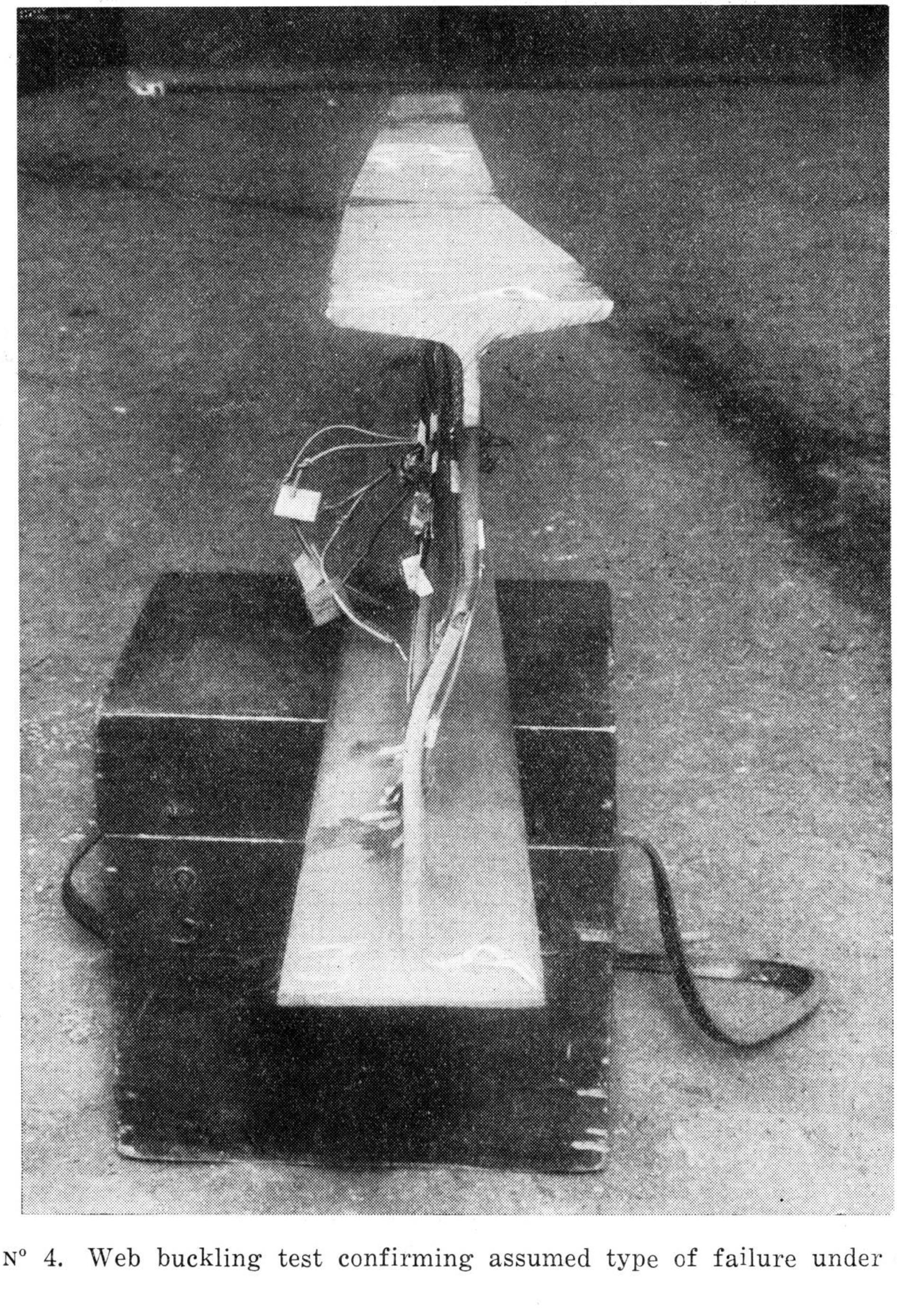

PLATE N° 4. Web buckling test confirming assumed type of failure under overload double curvature as would be expected. All the beams tested under these conditions eventually failed by web buckling, this being the object of the tests.

It may be stated that during the tests in no case was any failure by web buckling observed, until the loads applied to the beams had reached the region of some three and a half times the value which would normally produce a bending stress of 10 tons per sq. in. in the flanges, under conditions where lateral support is provided to the compression flange during loading.

At this stage it is considered that the action at the abutments is in some part that of a homogeneous section, for the other part that of a rigid welded lattice girder. Work is still proceeding on this particular matter as it is felt that theoretical analysis of the beams is not likely to give an accurate reproduction of the true state of stress conditions.

The standard of welding is maintained by the periodic use of Gamma--radiography, particularly to ensure that the welder himself can have visible evidence as to the quality of his work. This opportunity is very much appreciated by the operatives, and there is no doubt that it helps to maintain the highest possible standard.

Two types of isotopes are normally used.

For material up to three quarters of an inch in thickness, a 4000 milicuries Iridium 192 is satisfactory. The pellet is changed every 20 weeks.

A 1000 milicuries cobalt 60 is satisfactory for material over 1 inch in thickness. This pellet only needs changing every 5 to 7 years.

Practical applications.

Although the use of castellated construction is not restricted to steel framed buildings, available space precludes the illustration of more than three specific types.

Plate No. 5 shows the incorporation of castellated beams in a saw tooth pattern roof in a factory for the production of typewriters. The building is 310′ long and the castellation outlines have a considerable effect on the general appearance of the structure, as well as being a light, stiff section for the long members.

Plate No. 6 shows the floor beams in a four storey Technical College in the North of England where the pre-cast floor slabs run on shelf angles which leave at least half the castellation open underneath to permit the passage of service pipes and cables. As the ceiling is suspended at the bottom flange level the finished building is clean in appearance, services are concealed, and floor thicknesses kept to a minimum.

Castellated construction is equally applicable to portal frame or rigid frame industrial buildings, of which that shown in plate No. 7 is typical.

This method of construction has been used with considerable success in light single lane road bridges, a typical example of which is for a roadway 8′6″ wide consisting of seven spans of 50′ the members being $24'' \times 7\,^1/_2''$ Joists castellated to $36'' \times 7\,^1/_2''$.

The system has also been used for light crane bridge girders and for light gantry girders.

Conclusions.

It must be accepted that an approach to the theoretical maximum economy in the use of steel is hindered by the range of hot rolled sections, and the possibility of alternatives such as cold formed sections is limited by practical applications, particularly for the heavier type of structure.

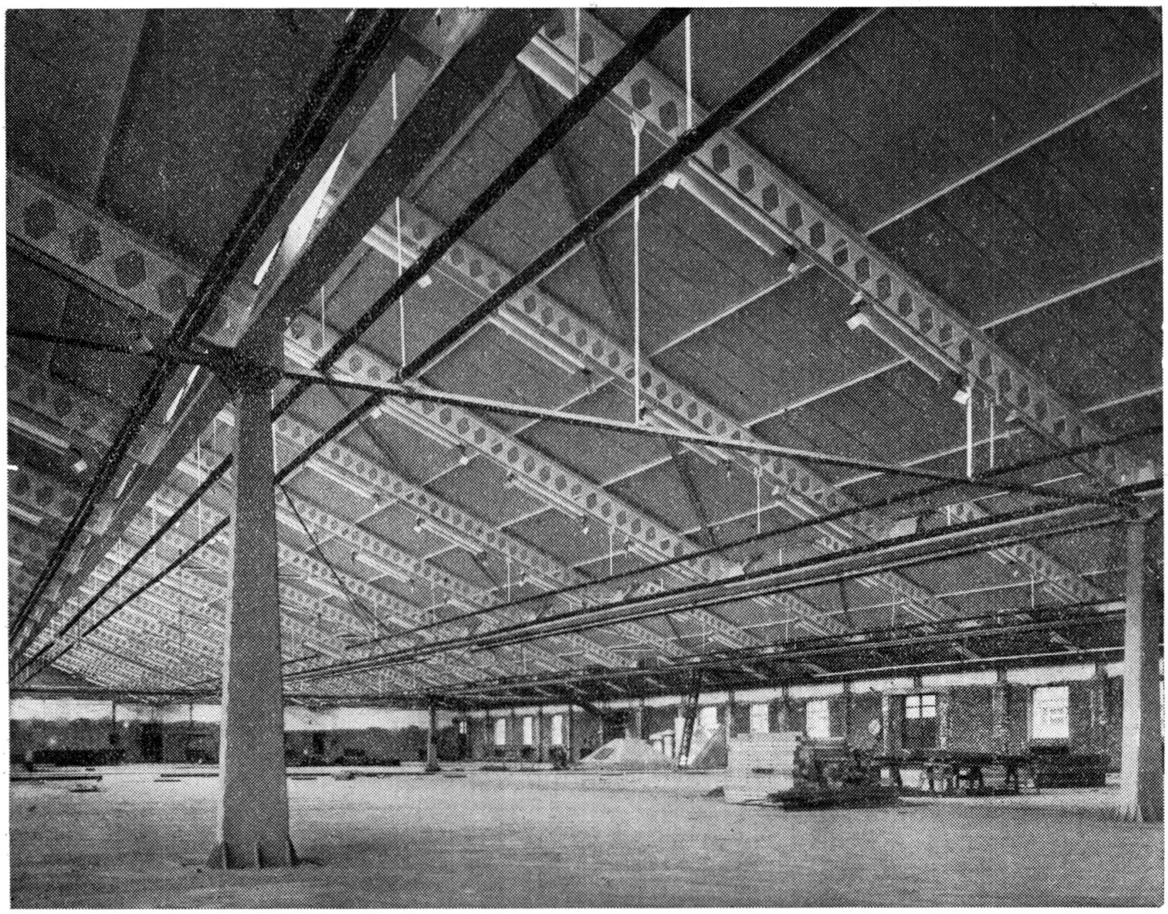

PLATE N° 5

The facility to «stretch» a standard section so as to be equivalent to several higher sections in the standard range means that on a structural contract which would normally require a large number of different sections, the whole requirements can be met with a smaller number of sections, ordered from the mills and expanded to the required depth and strength.

In view of the present heavy demand on the steel industry, this enables Structural Engineers to simplify their orders on the Rolling Mills and so fit them in with rolling programmes resulting in earlier deliveries, and at the same time achieving economies in weight and cost of the finished structure with further resultant savings in foundation costs.

Great efforts are being made throughout the world at the present time to produce structures in steel which possess aesthetic appeal coupled with economy of material and workmanship. Consequently investigation in detail of various up to date constructional methods has been made. Of such methods it may be said that castellated construction is in the forefront.

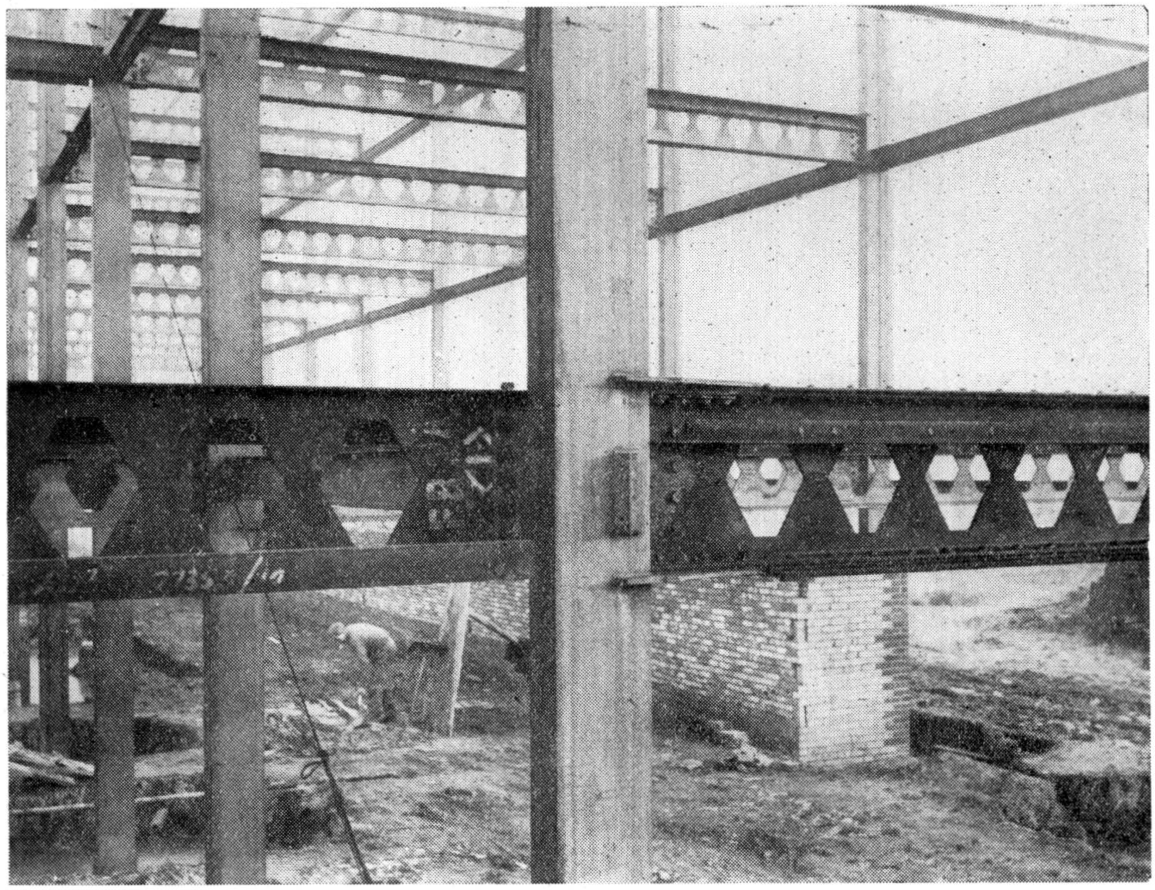

PLATE N° 6

The system possesses many desirable features, being economic in steel, and adaptable in application. The method is well suited for repetition work or for the more specialised treatment required in the individual design of «one off» structures, particularly where callisthenic features are of importance, as the basis of the system is conductive to a light and delicate appearance of the completed frame, and the various components possess a pattern easily incorporated into an aesthetically pleasing whole.

I have endeavoured to show that all these points can be met by the adoption of the castellated method of construction.

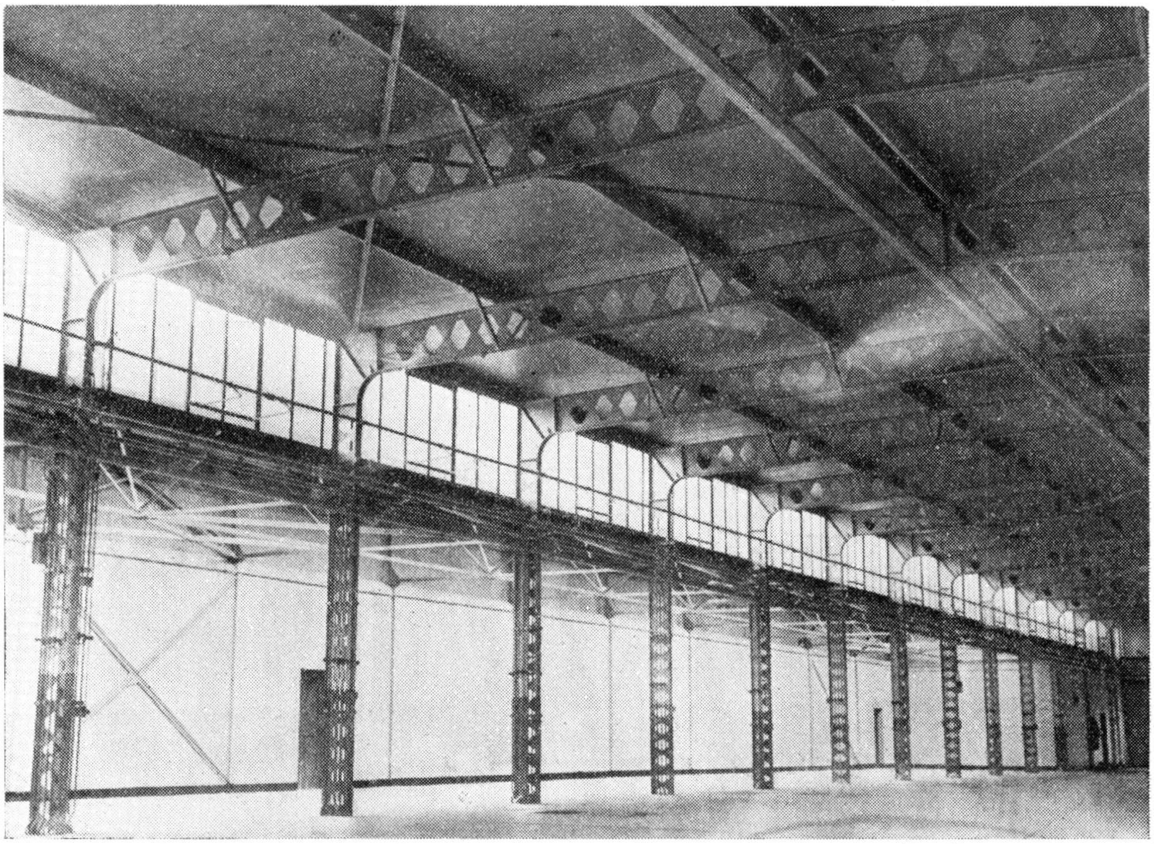

PLATE N° 7

Acknowledgements.

I am indebted to the Directors of United Steel Companies Limited for permission to incorporate information formulated in connection with their «Castella Beams», and to reproduce their copy right photographs.

Also to members of the technical staff of United Steel Structural Company Limited for their collaboration and assistance in the preparation of much of the data which has been included in this paper.

SUMMARY

The principle of castellated construction is that of cutting the web of a steel girder in a series of serrations or teeth, and subsequently welding the pieces together after they have been rearranged so that the depth of the completed member is 50 % greater than that of the original member, with appropriate increases in the major moment of inertia and the section modulus.

The paper incorporates not only the method of manufacture of standard beams, but indicates variations of the principle, whereby combinations of unlike members can be treated in the same way.

Information is given which shows the saving in weight of steel to produce members of equivalent or even greater strength than the original beam, and gives illustrations of practical applications of the castellated system in steel framed buildings.

ZUSAMMENFASSUNG

Das Wesen der ausgezahnten Bauweise besteht in der Trennung eines Stahlträgers im Steg längs einer Zacken- oder Zahnlinie und im nachfolgenden Zusammenschweissen der gegeneinander verschobenen Teile. Die Höhe des zusammengefügten Trägers ist damit 50 % grösser als jene des ursprünglichen und entsprechend erhöht sich auch das Hauptträgheitsmoment und das Widerstandsmoment.

Der Beitrag enthält nicht nur die Herstellungsmethode der Standardträger, sondern auch Abweichungen vom allgemeinen Vorgang, wobei Zusammenstellungen ungleicher Teile auf die gleiche Weise behandelt werden können.

Gegenüberstellungen zeigen die Stahlgewichtsersparnis bei der Herstellung von Trägern gleicher oder sogar grösserer Tragfähigkeit, und in praktischen Beispielen aus dem Stahlhochbau ist die Anwendung dieser Bauweise ersichtlich.

RESUMO

Em princípio, o processo de construção «ameiada» consiste em recortar a alma de um perfilado metálico em forma de dentes, em deslocar seguidamente, uma em relação à outra, as duas peças assim obtidas e soldá-las, ficando finalmente a viga com uma altura de 50 % superior à do perfilado original o que aumenta consideràvelmente os momentos de inércia e resistência da secção.

O autor, além de descrever o método de obtenção de vigas correntes, indica ainda variantes permitindo combinar perfilados diferentes pelo mesmo processo.

Dão-se também indicações acerca da economia de peso realizada em vigas de resistência equivalente ou superior à do perfilado original e descrevem-se aplicações práticas deste sistema em estruturas metálicas.

RÉSUMÉ

Le principe de la construction «crénelée» consiste à découper l'âme d'un profilé métallique en forme de créneaux, à déplacer ensuite l'une par rapport à l'autre les deux pièces ainsi obtenues et à les souder enfin, de façon à obtenir une poutre 50 % plus haute que le profilé original, ce qui permet d'augmenter considérablement les moments d'inertie et de résistance de la section.

L'auteur décrit le mode d'obtention des poutres les plus courantes et indique des variantes permettant de combiner des profilés différents par le même procédé.

Il donne également des renseignements sur l'économie de poids réalisée sur des poutres de résistance égale ou supérieure à celle du profilé original et décrit des applications pratiques de ce système dans des charpentes métalliques.