THE USE OF STEEL TO B.S. 968 : 1962 IN THE ALL-WELDED FRAME OF A 19-STOREY BUILDING

G. COOPER
B.E.(HONS), F.N.Z.I.E., M.N.Z.I.E.

Synopsis:

This paper describes the use of steel to B.S. 968:1962 in the all-welded frame of a 19-storey building, the first large-scale use of this steel in New Zealand. The reasons for the adoption of an all-welded steel frame in this material and the problems associated with the shop and site welding are described, in addition to details of joints, and problems of material supply. The project has demonstrated that with the proper technical and physical resources on the part of the fabricator, and the appropriate control, steel to B.S. 968:1962 in both plate and rolled sections is a practical and economical building material worthy of serious consideration by all structural designing engineers.

Senior Partner, W.G. Morrison and Partners,
Consulting Engineers, Wellington,
New Zealand.

1. INTRODUCTION

1.1. General

Consideration of the ways in which structural sections and plate may be combined in a structure shows that the logical conclusion is to dispense with secondary site welding or bolting of members and to site weld beams directly to the columns. This direct method of assembling members is not uncommon, but as applied to a multi-storey frame it presents a challenge to the fabricator, the designing engineer, and the metallurgist.

The general acceptance and use of steel to B.S. 968:1962 (1) has no doubt been retarded by the failures experienced in Australia (2) and the U.K. These failures have engendered respect both for the precautions one must take with the material and for the consequences if one does not. Any prejudice which now exists could arise, in the author's opinion, only from a lack of knowledge of the material and of the necessary fabrication procedures. Precautions are necessary, as indeed they are necessary with any steel. It is probable, where heavy sections are involved, that at least the same precautions are required, and possibly greater problems of fabrication and erection are involved, in the use of a steel of lower tensile strength.

1.2. The Scope of the Project

The ultimate development is a 19-storey office building incorporating a five-storey parking building. The project incorporates approximately 240,000 sq. ft. gross of office accommodation and approximately 90,000 sq. ft. gross of parking area. Each typical upper floor of the office tower has a gross area of approximately 13,800 sq. ft. in dimensions 252 ft. by 54 ft. 6 in.

A total of approximately 1,800 tons of structural steel is incorporated in the project, of which approximately 1,500 tons is in high yield stress rolled sections and plate to B.S. 968:1962, and the balance in mild steel rolled sections and plate to B.S. 15:1961 (4).

2. THE CHOICE OF STEEL TO B.S. 968:1962

The use of a high yield stress steel appeared at the design stage to have one basic advantage over the use of mild steel (or N.D.1 quality steel): cost. Since the structural steel section sizes throughout the building are generally governed by stress considerations, the use of a high yield stress steel to B.S. 968:1962, with correspondingly higher allowable stresses, indicates an overall weight saving against steel to B.S.15 of between 20 and 30%. In addition, the reduced weight of sections significantly affects the amount of preparation and welding required for beam-column junctions. Secondary effects are possible savings in concrete and boxing, resulting from reduced member sizes, and reduced foundations.

On the basis of the known cost differences between steel to B.S. 968 and steel to B.S.15 (more probably N.D.1 quality to B.S. 2762 (6)) and an overall weight saving of, say, 25%, it appeared that a cost saving on the structural steel frame would result from the use of steel to B.S. 968. It was decided to use steel to B.S. 968:1962, with the assumption that other problems associated with this decision could be satisfactorily resolved.

3. DESIGN FEATURES

An attempt was made to achieve structural symmetry within the limits of the owner's requirements and the configuration of the boundaries.

To eliminate the unsymmetrical stiffening effect of the partitions, all partitions have been separated from the structure by vertical seismic gaps of widths not less than three times the interstory deflections calculated for the design lateral forces. A complete horizontal separation has also been provided at the top of all permanent partitions at beam soffit level. In addition, all windows have generally been provided with sub-frames to ensure independent structural frame distortion under earthquake loading. Secondary non-structural damage is very much an open question. There is a significant lack of experience of the level of damage to be expected, and of how to design either for no secondary damage or for a certain level of damage.

All structural steel members are encased in concrete for fire protection and to achieve adequate deflection stiffness under the design loading.

4. IMPACT STRENGTH OF ROLLED SECTIONS TO B.S. 968 : 1962

At the outset, for economic reasons, no serious consideration was given to the use of plate only in built-up sections, even though B.S. 968:1962 contains no provisions for low-temperature impact properties for rolled sections. The basic concern in the decision to adopt rolled sections was, however, the lack of guaranteed impact properties and also the directional properties of rolled sections. (The joint details contemplated, in which it was proposed that beams be butt welded to column flanges, relied for satisfactory performance on the transfer of beam flange loads through column flanges perpendicular to the direction of rolling.) The designers were also very conscious of the failures experienced in Australia and the U.K., and of the fact that very little experience existed on the welding of rolled sections to B.S. 968. Having regard for the problems involved in producing a notch ductile material in rolled sections, it was decided that rolled sections to B.S. 968:1962 should be ordered with the additional requirement of guaranteed minimum impact properties. A guaranteed level of impact strength, which might seem unnecessary for a building was required for two reasons:

First, the building is in a region of high seismicity, with all member sizes primarily controlled by earthquake forces. Brittle fracture under these conditions was considered a distinct possibility. Second, although the steel frame is encased in concrete, the frame would be exposed to ambient temperatures for a considerable time during construction. In Wellington, ambient temperatures frequently fall below 38°F and sometimes below 35°F (the lowest recorded temperature is 28.6°F.)

Opinions may vary. For earthquake-resistant building frames the author is convinced that a guaranteed minimum level of impact strength is an important and necessary requirement of the material supply specification. In rolled sections, testpieces may be taken from several locations and notched in various directions relative to the direction of rolling. The results of such tests on any particular section will vary, as also will tests on similar section sizes out of the same cast. No one test could be taken to truly represent the material rolled; certainly the steel manufacturers would not claim that it did. In addition, one impact test is a long way short of a complete transition curve. However, a practical degree of control is necessary, and if the basis of testing is standardised, a comparative measure is established against which the performance of the material can be judged. The main concern was whether the steel manufacturers would undertake to supply rolled sections having guaranteed minimum impact properties.

5. STRUCTURAL STEEL SUPPLY

In retrospect the supply of structural steel to an acceptable specification proved to be one of the largest problems.

The original supply specification, dated January 1965, was based on B.S.968:1962, including amendments 1 and 2, with the additional requirement that impact tests be carried out on rolled sections to meet the requirements for plate, namely 20 ft-lb at -15°C with no single value less than 15 ft-lb. No thickness limit was specified. The supply specification also contained a note that the buyer was prepared to discuss the impact provisions with the manufacturer. In June 1965, amendment 3 to B.S.968:1962 appeared. This introduced a minimum impact specification (by agreement with the Purchaser) applying to rolled sections up to and including a thickness limit of $\frac{3}{4}$ in. (average impact value not less than 20 ft-lb at 0°C.) No minimum single value was specified. In July 1965 press criticism of the B.S.968:1962 specification appeared, centred around the failure of heavy rolled sections proposed for use in the Kleinwort-Benson building in Fenchurch Street, London.

Amendment 6 to B.S.449:1959, dated 29 June 1966 subsequently appeared confirming the designers' original basic requirement of 20 ft-lb at 0°C for all rolled sections, except that no single minimum value is specified. To the author's knowledge there has been no further amendment to B.S.968:1962 since amendment 3. For all sections exceeding $\frac{3}{4}$ in. thickness, the impact values to be obtained have still to be agreed between the manufacturer and the purchaser. It must be assumed that the increased requirements of the amendment to B.S.449 can now be met by the manufacturers.

6. MECHANICAL AND CHEMICAL PROPERTIES OF THE STEEL SUPPLIED

The mechanical and chemical properties of all column and beam sections supplied to B.S.968:1962 are shown in Table 1. The mechanical properties and chemical analyses were generally within the specification, but in some cases were on the maximum limit of the specification. The properties of all plate supplied to B.S.968:1962 were satisfactory. No major surface rolling defects appeared. Careful observation for these was maintained, however, and each defect was evaluated in relation to its size and orientation to the design function of the member concerned. Rejectable defects in a premium grade material are a cause for great concern when replacement involves a considerable time delay and a lesser grade material cannot in all circumstances be used as an alternative.

7. JOINT DETAILS

Five basic joint details were involved in the frame (Figs. 2 and 3 are typical.) These details were decided upon in collaboration with the consulting metallurgists in the first instance, and later with the fabricator. Generally at all beam-column junctions the columns were plated across the toes of the flanges. This applied in all cases of the exterior beam-column and beam-column-spandrel connections, where longitudinal earthquake forces were involved, and in some of the interior beam-column junctions where the x-x properties of the column were insufficient to accommodate transverse earthquake forces. In all plated joints the column stiffners (being the continuation of the beam flanges) penetrate the column plates for field butt welding directly onto the longitudinal beam flanges. All columns are orientated to accommodate transverse moments about the x-x asis. All transverse beams have been butt welded directly onto the colum flanges. Where it was necessary to haunch the transverse beams at their ends, the haunch plates were shop welded to the beams and haunch plates in turn field butt welded directly onto the column flanges. All beams and spandrels were field bolted to web erection cleats shop welded to the columns. These full depth web cleats were used as backing bars to the field web butt welds to the columns.

In general the main butt welds in the beam flanges were prepared for two-thirds downhand welding and one-third overhead, the backing runs being laid first on the inside faces to enable gouging to precede the placing of the outside runs.

8. FABRICATION AND ERECTION

The structural steelwork specification, based on the provisions of B.S.2642:1965 (11), which was then in draft form, embraced all aspects of welding equipment, electrodes and storage, supervision, operator testing, welding procedure, workmanship and assembly, preheating, weather protection, repairs, non-destructive testing etc. The fabrication and erection problems discussed below were directly related to the aspects of fit-up, electrodes, preheat temperatures, joint restraint, and supervision. At the time tenders were called no New Zealand fabricator had prior experience of welding rolled sections to B.S. 968, although some experience existed on the welding of plate to B.S.968.

The main difficulty experienced in the shop fabrication was the standard of boilermaking required to obtain correct fit-up of parts when rolling tolerances of the heavy column sections were on the outer limits. Tailor-made stiffeners were often necessary.

Fit-up of joints was one of the site problems. Column rolling tolerance, beam length tolerance, and plumbing the building resulted in a considerable variation in root gaps. Some joints were too tight and had to be re-prepared, and others had too wide a root gap. In the latter case the preparation was either built up or a backing rod was inserted in the gap and later gouged out. In some instances column flange toe-in resulted in gaps varying across the width of the beam flange. These problems resulted in some welds having insufficient weld metal placed from the first side, and cracking was a possibility during back-gouging. In addition the amount of overhead welding was greatly increased.

The main problem experienced on the site was with the transverse beam to column butt welds. Cracks developed in positions 1 and 2 as shown in Fig. 4. Initially cracks were experienced in the web welds, generally associated with lamella tearing of the column flange material. Repairs were best accomplished by removing about 18 in. of beam section to allow good access for gouging into the column flange to remove the cracked area. Block filling of the web butt welds onto the erection cleats as backing bars was later adopted in lieu of bead runs. All of the cracks in positions 1 and 2 started with a toe crack leading into the column flange material and tearing at times to a considerable depth in a lamella fashion. The directional properties of the flanges of the rolled columns were clearly demonstrated. No provision was made in the supply specification for tests transverse to the direction of rolling. The cracks indicate that the column material has virtually no elongation in this direction. Tests carried out on B.S.968 plate proposed for utilisation in a similar manner showed the ultimate tensile stress to be almost the same as the specified yield point. It is not surprising, therefore that when a crack is initiated, stresses due to welding restraint open the crack to large proportions in the direction of rolling. Repairs to beam flange welds cracked into the column flange were also accomplished by removing about 18 in. of beam section. Welding the gouge sealed the "end grain" in the column, and the section of beam was replaced and welded onto the metal fill in the column. This method of repair proved satisfactory but time consuming. The total number of cracks in both web and flange butt welds was less than 3% of the welds made. This proportion of cracking is not severe in the type of joint adopted, and is somewhat less than might reasonably be expected.

The electrodes used for most of the site welding had a yield point of 26 to 30 ton/in^2 although in the latter stages these were unavailable and electrodes having a yield point of 30 to 35 ton/in^2 were used. The former electrodes were considered most satisfactory, although no difficulties arose with the latter.

From the designers' viewpoint, the problem experienced with cracking could be largely overcome by welding haunch plates or beam stubs onto the face of column flanges in the shop under much better control and fit-up than is possible on site. From the inspection viewpoint the location of cracks presented no serious difficulty. Since the cracks are toe cracks from the top run of weld metal progressing by virtue of the built-in stress in the weld, they open quite noticeably. Most of the cracks were initially identified by the operators.

Initially, during site erection and welding the specified positional tolerances of the steel members were exceeded in places. A programme of check measuring was established to determine the effect of weld shrinkage on root gaps and column verticality, and to determine welding procedure. The outcome was that generally positional and level accuracy was within $\frac{1}{2}$ in, although local positional errors up to 1 in. were measured. Overall verticality was within 1 in. In retrospect, concrete encasement sizes could have been greater to allow increased positional tolerances in the steel frame. In spite of the best endeavours, errors occur. The obvious solution is, within limits, not to adjust the steel work but to absorb the errors in the encasement concrete. Accurate rigging prior to welding followed by a rigid welding sequence is vital for the maintenance of close positional tolerances.

9. CONCLUSION

Amendment 6 to B.S.449:1959 specifies minimum impact requirements for rolled sections to B.S.968 as a precaution against brittle fracture for welded elements subject to tension. The implication is that the specified impact values can be achieved by the steel makers. The provisions of B.S. 968:1962, incorporating amendment 3, however, leave the impact values for sections above $\frac{3}{4}$ in. thick open for agreement with the manufacturer. The B.S.968 specification, in the author's opinion, should state what values can be achieved for sections above $\frac{3}{4}$ in. thick and reiterate the provisions of amendment 6 to B.S.449. The position regarding impact values of rolled sections to B.S.968 is far from satisfactory. Unless a buyer has rights of rejection against an impact specification he will tend towards the use of N.D. quality steel to B.S.2762. The only other alternative is to use built-up sections out of plate to B.S.968. Economic considerations would not favour this alternative.

An error of judgement may have been made in the decision to incorporate by reference, the provisions of B.S.968:1962 in the material supply specification. There is no reason why a buyer should not write his own material specification and invite quotations in the normal manner from as many different sources he may choose. The ideal arrangement in the author's opinion would be for the buyer to negotiate directly with a particular steel mill selected on the basis of a performance specification.

It is not possible to state that the overall cost was less than would have resulted from the use of N.D.1 steel and B.S.15 steel. In the author's opinion, the alternative solution may have been cheaper. It is possible however, that this first use in New Zealand of high yield stress steel on this scale in an all-welded frame has carried an unwarranted cost contingency. The experience gained on this project should result in lower costs in future similar work. This condition must be satisfied if steel to B.S.968 is to gain popularity in New Zealand.

The successful use of the material demands:

(a) A fabricator with the necessary technical and physical resources and a genuine intention of executing the work strictly in accordance with the specification. The fabricator's price must match this intention.

(b) Co-operation between the engineer, the metallurgist, and the fabricator at all times (preferably including the design stage.)

(c) An understanding by all parties of the others' problems and sufficient flexibility to admit alternatives when the circumstances dictate.

(d) A clear understanding by the material supplier of the material specification, and sufficient technical resources to ensure that it is complied with.

(e) Education of operators to the requirements of the material.

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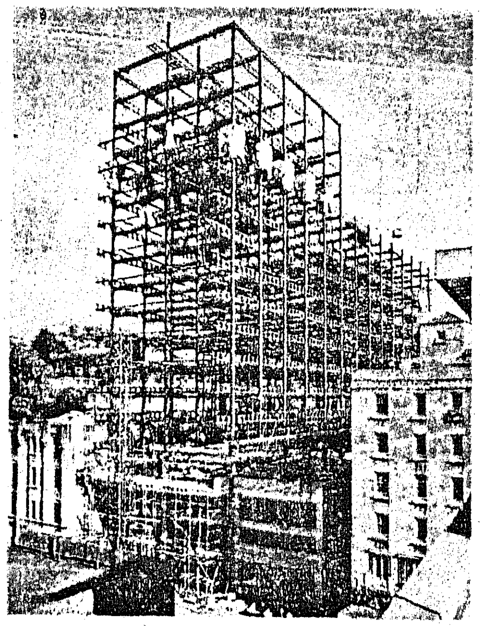

FIGURE 1. THE BUILDING DURING CONSTRUCTION

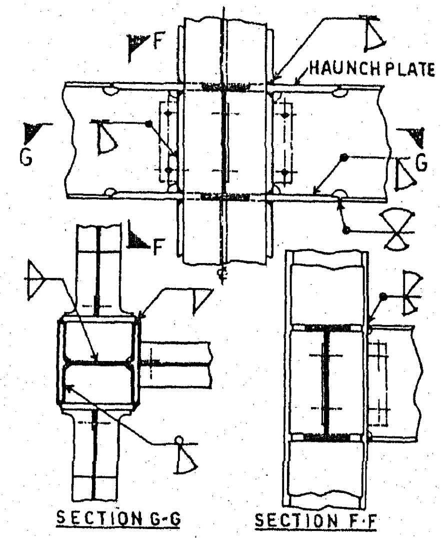

FIGURE 2. EXTERIOR BEAM TO COLUMN JOINT.

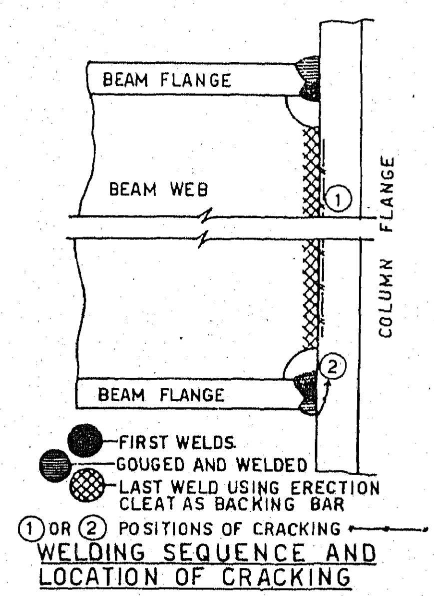

FIGURE 4.

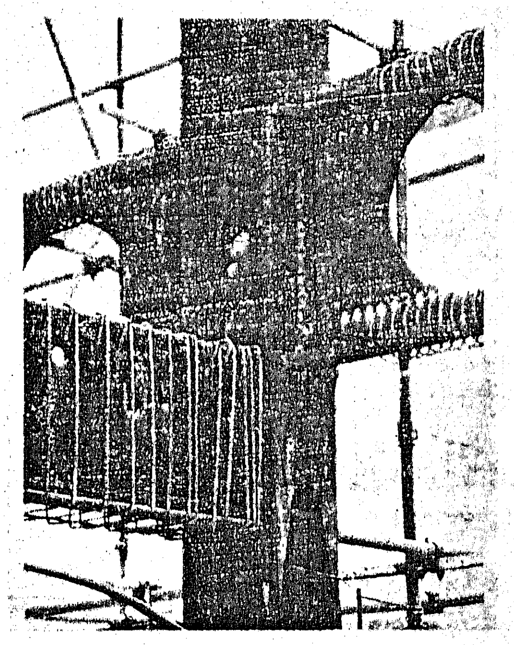

FIGURE 3 EXTERIOR BEAM TO COLUMN SPANDREL JOINT.

	Column or beam size	No. of bars	Charpy impact values ft. lbs at 0° centigrade: Overall average	Minimum average on 3 tests	Maximum average on 3 tests	Minimum single value	Maximum single value	Chemical analysis (ladle): C. max/min	Si. max/min	Mn. max/min	S. max/min	P. max/min	Cr. max/min	Mn+Cr max.	Yeild. tons/sq. inch max/min	Tensile tons/sq. inch max/min	Elong. % max/min
COLUMNS	14'×16'×314 lb	5	20·3	18·6	22	14	25	·220 / ·210	·044 / ·040	1·46 / 1·44	·044 / ·032	·027 / ·018	·039 / ·030	1·499	27·3 / 24·0	36·8 / 34·7	23 / 20
	14'×16'×228 lb	5	27·9	26·3	29·6	24	33	·220 / ·220	·046 / ·040	1·50 / 1·48	·030 / ·024	·024 / ·022	·042 / ·030	1·530	28·5 / 26·8	38·8 / 36·8	23 / 20
	14'×16'×193 lb	23	30·7	26·6	36·6	24	44	·220 / ·200	·046 / ·040	1·50 / 1·30	·032 / ·024	·032 / ·018	·042 / ·030	1·530	27·3 / 25·3	38·8 / 35·9	25 / 19
	14'×16'×158 lb	47	28·4	25·0	36·6	14	44	·220 / ·200	·044 / ·028	1·50 / 1·36	·036 / ·030	·024 / ·018	·050 / ·032	1·539	28·5 / 24·2	39·0 / 35·0	24 / 20
	14'×14½'×136 lb	56	31·8	26·0	43·3	18	51	·190 / ·180	·040 / ·034	1·46 / 1·24	·030 / ·028	·040 / ·026	·036 / ·023	1·496	27·4 / 26·0	39·0 / 36·4	23 / 20
	14'×14½'×103 lb	28	46·3	37·3	55·3	33	59	·180 / ·180	·030 / ·026	1·40 / 1·38	·036 / ·028	·020 / ·020	·012 / ·011	1·441	26·8 / 25·0	38·6 / 34·4	24 / 20
	14'×14½'×87 lb	28	79·0	79·0	79·0	71	87	·200	·030	1·36	·034	·024	·026	1·386	27·5 / 26·8	38·4 / 37·0	21 / 20
	12'×12'×79 lb	42	48·6	30·3	70·6	20	75	·190 / ·180	·036 / ·022	1·44 / 1·36	·036 / ·026	·027 / ·022	·041 / ·027	1·472	27·8 / 24·8	37·2 / 34·9	23 / 22
	12'×12'×65 lb	28	81·6	81·6	81·6	79	87								25·7	34·9	23
	10'×10'×49 lb	28	82·0	82·0	82·0	81	83	·160	·022	1·38	·040	·016	·018	1·398	24·6	32·4	24
	8'×8'×31 lb	24	36·0	36·0	36·0	30	45	·180	—	1·37	·039	·022	·030	1·400	25·2	37·2	28
BEAMS	21'×13'×127 lb	20	16·0	(ONE	SET)	13	22	·190	·036	1·34	·038	·032	·023	1·363	25·0	35·8	25
	21'×13'×112 lb	122	24·0	22·0	26·0	13	27	·180 / ·180	·038 / ·032	1·30 / 1·28	·048 / ·034	·026 / ·020	·033 / ·028	1·333	26·8 / 25·0	36·9 / 33·9	25 / 23
	21'×8¼'×82 lb	98	25·8	18·0	39·0	17	41	·190 / ·160	·038 / ·024	1·24 / 1·12	·036 / ·024	·025 / ·018	·030 / ·024	1·249	27·0 / 24·4	37·4 / 33·9	25 / 21
	21'×8¼'×73 lb	75	41·7	18·0	55·6	17	81	·190 / ·170	·040 / ·024	1·42 / 1·12	·040 / ·024	·035 / ·018	·036 / ·025	1·478	28·6 / 24·4	38·8 / 33·4	25 / 21
	21'×8¼'×62 lb	28	51·0	(ONE	SET)	37	60	·180	·040	1·34	·040	·018	·030	1·370	26·0	36·4	23
	21'×8¼'×55 lb	28	67·6	(ONE	SET)	66	70	·170	·032	1·26	·042	·018	·032	1·292	25·0	34·0	23
	18'×7½'×55 lb	28	57·0	56·0	58·0	53	66	·180 / ·160	·036 / ·024	1·38 / 1·32	·048 / ·032	·026 / ·026	·039 / ·020	1·419	26·2 / 26·0	37·6 / 36·0	23 / 22
	16'×7'×45 lb	28	48·3	ONE	SET	45	54	·160	·024	1·32	·032	·026	·020	1·340	25·8	35·4	23
	10'×5¾'×29 lb	118	54·4	27·3	79·6	25	90	·195 / ·150	·045 / ·012	1·59 / 1·25	·048 / ·033	·039 / ·018	·034 / ·022	1·500	29·3 / 25·8	38·7 / 34·8	26 / 19
	10'×5¾'×25 lb	27	59·0	46·0	72·0	43	73	·195 / ·185	·030 / ·025	1·50 / 1·34	·033 / ·026	·021 / ·020	—	1·500	28·5 / 26·4	39·0 / 34·1	22 / 16
	9'×4'×21 lb	531	47·4	32·0	70·0	29	80	·190 / ·170	·050 / ·040	1·38 / 1·29	·039 / ·023	·033 / ·029	·038 / ·019	1·416	28·7 / 25·9	38·1 / 34·6	24 / 20

TABLE: 1. TEST RESULTS FROM MILL CERTIFICATES FOR COLUMNS AND BEAMS TO B.S. 968, 1962.