BEHAVIOUR MODIFICATION OF SPACE TRUSSES

by

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Abstract

Behaviour Modification of Space Trusses

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Today space trusses are designed using linear elastic theory, and the load carrying capacity of the truss is considered to be limited by the failure of a few critical members.

Because of high redundancy, and due to practical constraints in selecting member sizes, only a portion of the member strength is used. Attempts have been made to utilize the remaining strength of the members by providing a means of transferring load from critical compression members to the understressed members. However because of the brittle buckling characteristics of compression elements, when they fall in the economic range of slenderness, postbuckling reserve capacity of compression members cannot be developed. Load transfer must thus be made before compression members reach their capacity.

One part of this study is concerned with the modification of space truss behaviour by introducing nonlinearity in the chord members prior to attaining their maximum load. The amount of nonlinearity required to increase the load capacity of the truss by an economically justifiable amount has been studied, and the feasibility of the proposed system was confirmed by tests on the

elements and on an assembled full scale truss.

As an alternative, transfer of load from critical chord elements to the understressed chord elements can be achieved by the selective removal of internal bracing members. A means to determine the optimum selection of diagonals to be removed has been established.

The above techniques were applied to four space trusses which were similar in size and proportions, but which varied in their support conditions and the distribution of member sizes. It was further considered whether the combination of the two techniques, of nonlinear chord behaviour and diagonal removal, could further improve the capacity, or whether each, when optimum, leads to the same load carrying capacity.

Acknowledgement

The author is grateful to Professor Cedric Marsh for accepting the task of supervising the direction, organization, and scope of this research, and wishes sincerely to thank him for his guidance, criticism, and enthusiasm.

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Special thanks is extended to my wife, Zhaleh, my daughter Ahoo, and my son Aidin for their understanding and patience.

CONTENTS:

Notation

A=	area
α=	factor, showing the point of deviation from linear elastic behaviour in stress strain curve
b=	number of bars
break point=	in "SPAN" program, a stage where one or more members reach their capacity
β=	factor, giving amount of nonlinear strain
chord=	members in horizontal planes of space truss
D=	outer diameter
Δ=	deflection of truss
δ=	axial shortening
d=	inner diameter of tube
diagonal=	diagonal bracing members
E=	modulus of elasticity
e=	depth of groove
ε=	strain
ε_e=	elastic strain
FLD=	force limiting device
F_u=	ultimate strength
F_y=	yield strength
γ=	factor of over-design of compression members
I=	moment of inertia
j=	number of joints
kN/J=	kilo newton/joint
L=	length

LOF= lack of fit

Nonlinearity= a member behaviour in which the force/axial change of length is not linear

N_x= number of members to receive imposed lack of fit

P_c= buckling load of member, (compression capacity)

P_t= tension capacity of member

P_u= ultimate axial load

Q= total load

q= load at each joint

R= number of reactions

"SPAN"= structural analysis computer program

σ= stress

σ_c= compressive stress

σ_e= Euler stress

σ_t= tensile stress

strut= compression member

t= wall thickness of tube

tie= tension member

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CHAPTER 1

SPACE TRUSS DESIGN

1.1 INTRODUCTION

A space truss is a three-dimensional structural system assembled of linear elements so arranged that forces are transferred in a three dimensional manner. Some of the characteristics which make space trusses popular are:

- Their ability to create multipurpose architectural spaces and column free areas.

- Their light-weight and hence reduced susceptibility to earthquakes.

- Their use of small elements which facilitates mass production, transportation and handling at site, and their assembly without highly-skilled workers.

- Their aesthetic appeal, visual elegance, with interesting geometric patterns.

They have been used for exhibition halls, gymnasia, auditoria, swimming pools, aircraft hangars, and anywhere that large unobstructed spaces are required. During the past five decades space trusses have been developed for the construction market, and companies all over the world have patents and trade marks for their fabrication and assembly.

This study deals particularly with double layer grids, which consist of upper and lower planes of square grids, offset relative to each other, inter-connected by inclined members. Fig. 1.1 shows such a space truss.

Because these space trusses are best utilized when they are square in plan and supports are widely spread in both directions, the study is confined to this case.

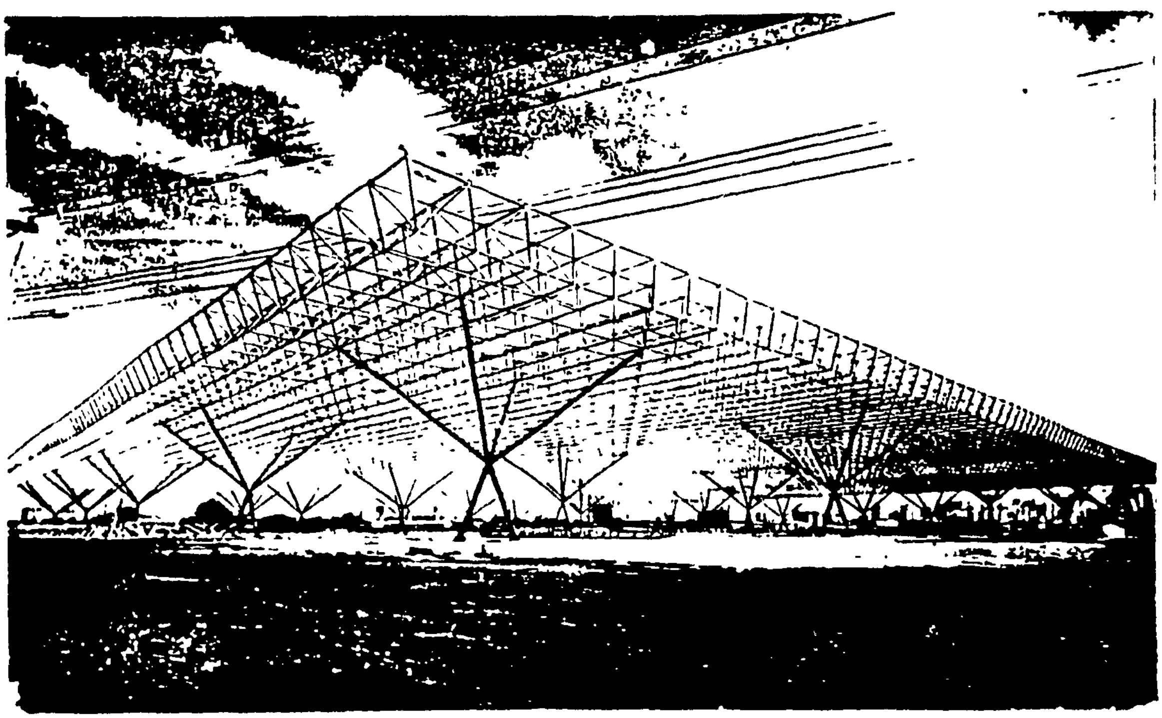


Fig. 1.1 A Space Truss

Parque Anhembi Exhibition Hall in Sao Paulo Brazil, double layer grid aluminum space truss, column spacing 60 m, covered area 62500 m^2, designed by C. Marsh [21].

In the beginning, the analysis of these unconventional large span structures was not easy using the conventional methods of structural analysis. In the first international conference on space structures which was held in the University of Surrey, Guildford, England on September 1966, elastic behaviour of space structures was presented. In these structures the hierarchy of members is not clearly defined, thus the structures cannot be subdivided for the purpose of analysis, and must be treated in their entirety. In these years computer time was expensive, and the speed and memory capacity was low to handle the large number of members involved. To avoid the large number of equations continuum methods were introduced, and Renton [34] compared methods of discrete and continuum analysis.

Space trusses with parallel square grids do not possess torsional stiffness and the pattern of failure is different from that of the plates, the behaviour being more like that of grillages, Marsh [22].

Flower and Schmidt discussed a method to convert a truss into an equivalent truss with fewer members, and hence with reduced degrees of freedom, but whose behaviour roughly corresponds to the behaviour of the initial truss, [4].

At the second international conference on space structures held at the University of Surrey on September 1976, some studies were reported on the ultimate strength of space trusses. The necessity of nonlinear behaviour for compression members was emphasized. Marsh [27] presented an approximate upper bound analysis for orthogonal grid space trusses which requires elasto-plastic behaviour of members.

In order to utilize the unused capacity of truss members, brittle failure must be prevented. For tension members this can be arranged by providing means to ensure that they yield over the full length thereby providing ductile behaviour, but the compression members, especially when they fall in the economic range of slenderness, buckle in a brittle manner without developing any post buckling strength. In some cases this behaviour causes the actual load capacity of the complete truss to be less than its capacity predicted by elastic analysis, Schmidt et al, [36].

Parke [46] suggested that the compression members be over designed to ensure that the tension members, which have a ductile behaviour, yield first, but even in this case the collapse load finally depends on the brittle failure of a compression member.

Schmidt, Morgan and Clarkson, [37] presented results of tests on trusses in which compression members were over designed, and some improvement in ductility has been reported. However, safe, effective and economic levels of over design factors have yet to be determined. Also, in cases where upwards wind loading is comparable to downwards gravity loading, it would seem difficult to ensure ductile behaviour for both failure modes.

Raissi Fard and Marsh, [33] showed that the nonlinear behaviour of eccentrically loaded diagonals can effect a redistribution of force in the chords thereby increasing the load capacity of the truss. The idea of managing force distribution in the chords by providing nonlinear behaviour in the diagonals led to the idea of removing certain diagonals in order to provide an "assured path" for the flow of forces. An optimum method for the selection of the diagonals to be removed was yet to be determined.

Redistribution of force by means of force-limiting devices (FLD's) was suggested by Schmidt and Hanaor [35]. Such devices, ideally, would yield under a predetermined force and would maintain this limited force under increasing deflection thus providing the strut with elastic-plastic load-deflection characteristic. Only critical members need to be protected by FLD. Considering the possibility of load reversal, in some cases it may be advisable to attach FLD to critical members in the nominal tensile chords as well. Their tests of a determinate truss with FLD's gave good agreement with theoretical predictions. However for an indeterminate truss, the initial failure was approximately 75% of the predicted load. They explained that joint rotation and corresponding change in member stiffness were probably the cause of the variation.

Prestressing by imposing a lack of fit was proposed by Hanaor and Levy [9]. Instead of letting a random lack of fit (LOF) produce random initial bar forces, a known LOF is used to produce an initial tension in critical compression members. Theoretically, it is possible to achieve in this way the same increase in ultimate load strength that would be possible if compression members behaved elasto-plastically. It turns out that diagonal members are most suitable for this purpose which coincides with Marsh's results. However, in case of load reversal this method is not applicable.

At the third international conference on space structures, held at the University of Surrey on September 1984, the behaviour modification of space trusses by nonlinear member behaviour was recognized by several authors, but no attempt was made to quantify the influence of nonlinear behaviour of the members on the overall behaviour of trusses.

In the present study behaviour modification of space trusses has been examined in the following sequences:

- Unutilized capacity of space truss with elastic analysis.

- Methods to exploit the unutilized capacity, which fall in two main categories.

a: Nonlinear force/change-of-length relationship.

b- Elastic force redistribution.

- Influence of nonlinear force/change-of-length relationship, with particular attention to the extent of the nonlinearity required to effect a useful improvement.

- Practical development of nonlinear member behaviour.

- Elastic force redistribution by removing selected diagonals, with the intention to find an optimum selection procedure.

- Combining diagonal removal and nonlinear member behaviour.

- Space truss systems on the market.

- Conclusion.

1.2 SPACE TRUSS ANALYSIS

Space trusses are currently analyzed using linear elastic theory and the load carrying capacity is usually considered to be limited by the first member or set of members to fail. The struts and ties are treated as straight, axially loaded pin ended members, for which the relationship between force and change of length is linear up to buckling in compression members or yielding in tension members. In the quest to draw on the capacity of the structure beyond this assumed limit it is necessary to examine the behaviour of struts and ties.

Tension members, ideally would yield overall, but may rupture in a brittle manner at the net section or at the connection. For slender compression members there is a plateau at the maximum load in the force/change-of-length curve, but, when the buckling stress is greater than one half of the yield stress, failure is sudden. Fig.1.3 is a theoretical graph, relating axial force to axial shortening based on the assumption that yielding in the extreme compression fibre in a straight pin ended column limits the capacity (see appendix A). This graph is idealistic but it serves the purpose of showing how the plateau diminishes with decreasing slenderness.
In the practical range of slenderness, the behaviour of the strut is brittle, and the collapse of the system can be initiated by buckling of a few members, Schmidt et al [36].

To give the system a capacity beyond the linear elastic range it is necessary to provide a means of transferring load from the critical members to understressed elements, before the critical members fail.

For the analysis of space structures the computer program "SPAN" was used. This program was first written at the University of Waterloo by Peter Kneen specifically for linear analysis of offset square-on-square space trusses, [15], and has been modified and extended in the Centre for Building Studies, Concordia University, to handle nonlinear behaviour. The sequence of analysis is: a unit load is applied and member forces computed, P_1. The ratio of member capacity, P, to the force P_1 is calculated, $P/P_1=k_1$; the lowest value of k_1 is found; this is the elastic load capacity of the structure. For elasto plastic behaviour all the forces, P_1, are multiplied by k_1 and subtracted from the member capacities, leaving $(P-P_1k_1)$, which gives the remaining capacities of the members. For one or more members this capacity is zero. These members are removed.

Using the residual capacities, the preceding analysis is repeated to give the increment of loading k_2 which will bring the next members up to their limiting capacities. These members are then removed, and the cycle repeated until the structure is no longer stable; the matrix becomes singular, representing a fully plastic mechanism. The total load at failure is then $\Sigma_{i=1}^{n} k_i$, where n is the number of cycles of analysis.

This procedure is based on the assumption that the members which have reached their capacity can sustain their maximum load for an indefinite axial strain, ie. they are elasto-plastic, Fig. 1.2(b). In reality, after a compression member reaches its capacity, it unloads as in Fig. 1.2(c). This is accommodated in the program by replacing the area of the member which has failed by a negative area, chosen to give an appropriate unloading slope. Analysis using the model has demonstrated that there is essentially no post buckling capacity in space trusses when a primary compression member is the first to fail.

To avoid this brittle failure the members must possess a nonlinear force/change of length characteristic in the loading range, as illustrated in Fig. 1.2(d). This behaviour is modeled by changing the original area, A, to $(A)(E_2/E_1)$ where E_1 is the modulus of elasticity of the segment OA and E_2 is the slope of the segment AB.

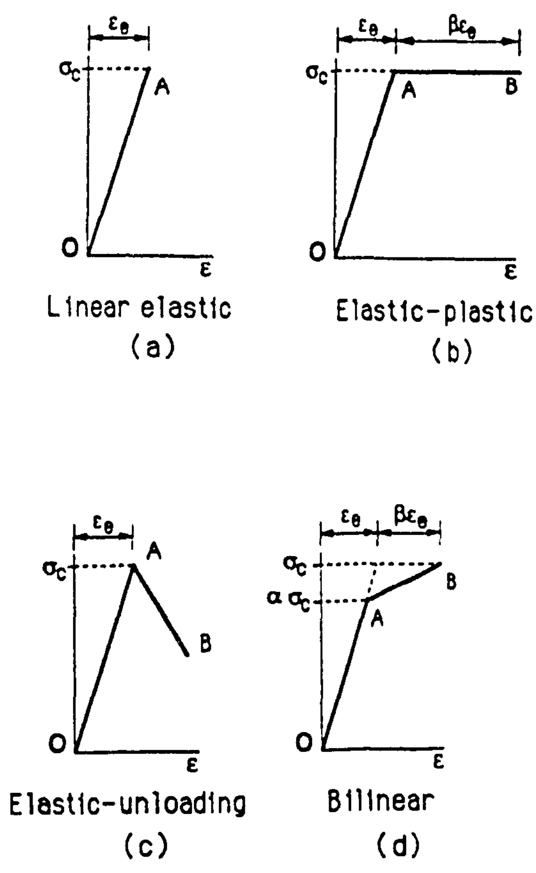


Fig. 1.2, Member behaviour models

σ_y = yield stress
σ_e = Euler stress
σ = axial stress
ε = effective axial strain
E = Young's modulus
A = cross-sectional area
P = axial force

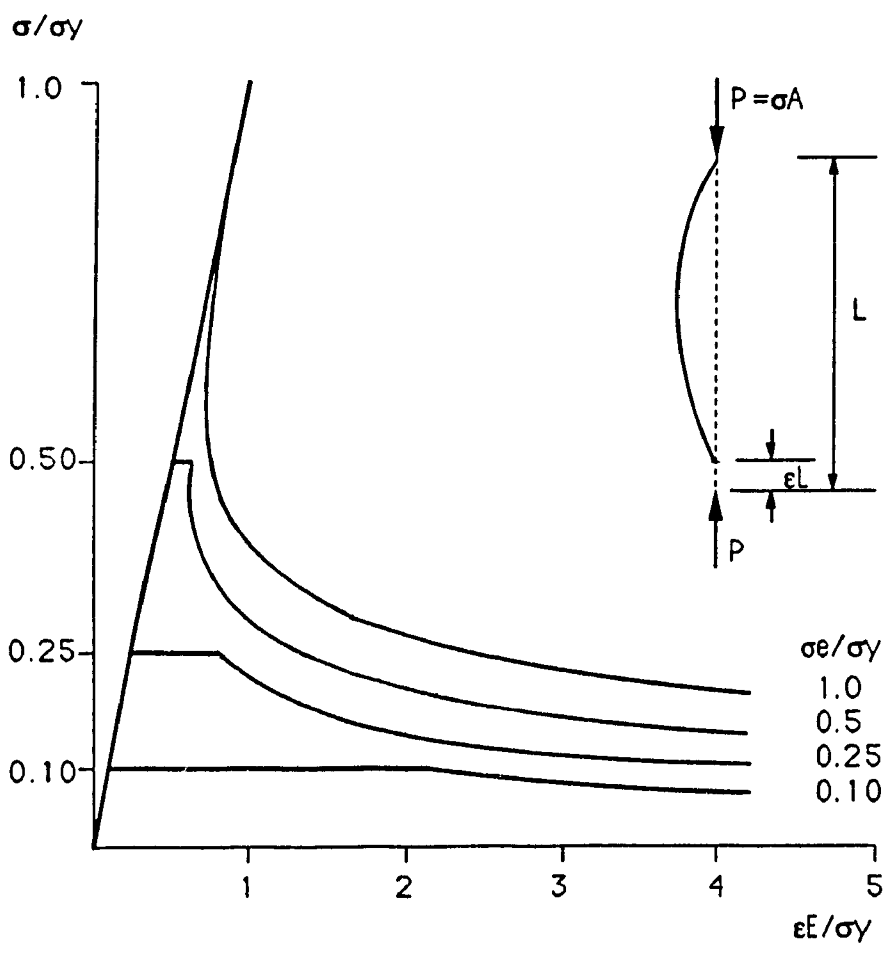


Fig. 1.3, Mean axial stress versus effective axial strain for axially loaded struts

CHAPTER 2

PATTERN OF FORCE DISTRIBUTION AND THE AMOUNT OF UNUTILIZED CAPACITY OF SPACE TRUSS MEMBERS

2.1 INTRODUCTION

On one hand, an ideal space truss would have many different sizes of members, every member proportioned according to the force it sustains. This would lead to a minimum-weight structure, though not necessarily to minimum cost. On the other hand, a space truss might have only one member size to facilitate mass production, and to simplify supervision at the factory and at the site. This would minimize the cost of labour. In practice the aim is to use that number of member sizes which optimizes the total cost. The optimum weight obtained by linear elastic analysis of a truss with many sizes of bars was studied by Kneen [16].

In the present study the proposed methods of behaviour modification are applied to trusses with uniform member sizes and to trusses with a limited number of different sizes. The effect of boundary conditions is also studied.

To find out how much unused capacity remains in the members of a typical space truss system, and to establish the effect of nonlinear member behaviour, four arrangements were analyzed, Fig. 2.1.

Each truss is a double-layer, offset-square-on square grid spanning 60 m in each direction and a node spacing of 3.75 m in each direction. The truss depth of 5.0 m gives a span/depth ratio of 12, which is low for typical space trusses but was chosen to ensure that deflection limits were not exceeded.

All four types are targeted to support a nominal uniformly distributed load of 0.8 kPa, factored by 1.5, giving a required ultimate load of 1.2 kPa. Trusses Type 1 and 2 have uniform member sizes, Trusses Type 3 and 4 use different member sizes. Type 1 and 3 are simply supported at four corners while Type 2 and 4 are continuous over the supports.

The truss system is composed of chords and diagonals made of round tubular sections of aluminum alloy 6061-T6, with ultimate stress, F_u= 260 MPa and yield stress, F_y=240 MPa. The design of joint pieces is not included in this study. The proportion was chosen to represent a realistic large span structure carrying a typical load with limited deformation.

It is difficult to establish the optimum number of different sizes, but it is unlikely to be less than three, so this number was chosen to permit comparison between the merits of the different optimizing procedures to be examined. The selection of the actual three sizes to be used is also an optimization problem but this has yet to be pursued. The member sizes were chosen for their capacity as chords according to CAN3-S157-M83, Strength Design in Aluminum. For trusses Type 1 and 2 the member size is dictated by the most highly loaded chord caused by the specified loading; this is size L in Table 2.1 For trusses Type 3 and 4 three member sizes are used, L being the largest. The medium size, M, was chosen to have a compression capacity 1/2 of L, and the small size, S, was chosen to have a compression capacity 1/5 of L, when used as a chord member. The ratio of diameter/thickness was 30 for all sizes. The same member sizes were used for the diagonals, which are longer than the chords. The limiting maximum slenderness of 120 was satisfied by diagonal size S. The dimensions and capacities of these sizes of members are given in Table 2.1.

Table 2.1, member sizes

		Size L	Size M	Size S
	D, mm	240	180	140
	t, mm	8	6	4.6
	r, mm	82	61.5	47.9
	A, mm^2	5830	3280	1960
Chords: L=3750 mm				
	λ	46	61	78
	σ_c, MPa	180	168	113
	P_c, kN	1050	550	220
	P_t, kN	1050	590	350
Diagonals: L=5660 mm				
	λ	69	92	118
	σ_c, MPa	145	82	48
	P_c, kN	840	270	90
	P_t, kN	1050	590	350

Because of the very high load in the diagonals at each support, this member was treated as an exception and was assigned a size sufficient to avoid failure. There are support arrangements which avoid this problem, but, they are not discussed here.

The capacity of tension members and compression members is limited to 0.75 F_yA, a value chosen to allow for the influence of end connections.

Member sizes, load capacities and the force distribution given by one cycle of elastic analysis, for the four types of truss, are described below:

2.2 TRUSS TYPE 1

Truss type 1 is simply supported, with uniform member size. The geometry of the truss is shown in Fig. 2.1. A preliminary analysis gave the required size of the large chord member which is L in the Table 2.1, where its capacity when used as an internal diagonal is also given.

The load capacity of the truss is 1.25 kPa, governed by compression failure in the top chord at the midspan of the boundary. The weight of the truss is equivalent to a load of 0.42 kPa giving the ratio: load/weight =2.98.

Fig. 2.2 shows the forces in the top chords which are parallel to the line A-A. For the chords on the column-line A-A the forces vary from a maximum compression of 1050 kN to a minimum compression of 570 kN, while for the chords on the centre line (B-B) the forces vary from a tension of 5 kN to a compression of 20 kN. The forces for chords parallel to A-A are shown for sections D-D and A-C. Similarly Fig. 1.6 shows the force distribution in the bottom chords parallel to A-A along the column-line (A-A), centre-line (B-B), and also for sections D-D and A-C. Fig. 1.8 and 1.9 show the forces in the main diagonal members.

The pattern of force distribution in top chords, bottom chords and diagonals shows that there exists a considerable amount of unutilized capacity in the members. Fig. 2.4 illustrates that 128 chords have been loaded with less than 10% of their capacity, while only 12 chords are loaded to over 90% of their capacity. 28% of the capacity of the chords, and 7% of the capacity of the diagonal members has been utilized. On the average, only 18% of the total capacity of the truss has been utilized.

At the column-line the maximum deflection, due to the factored load, is 470 mm, ie span/128. At the service load this becomes span/190. The centre-line remains almost straight with a slight reverse curvature.

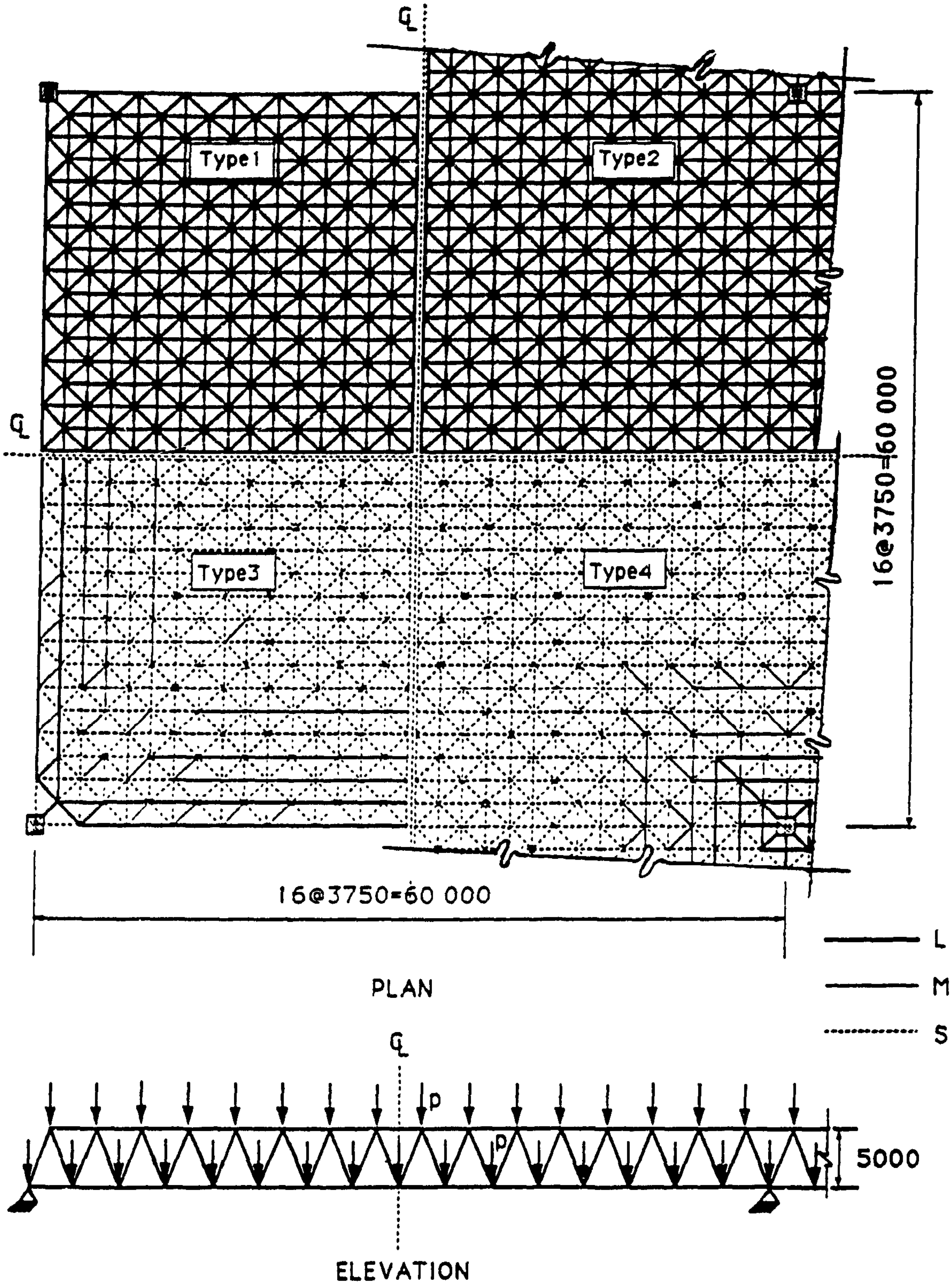


Fig. 2.1, Truss arrangement

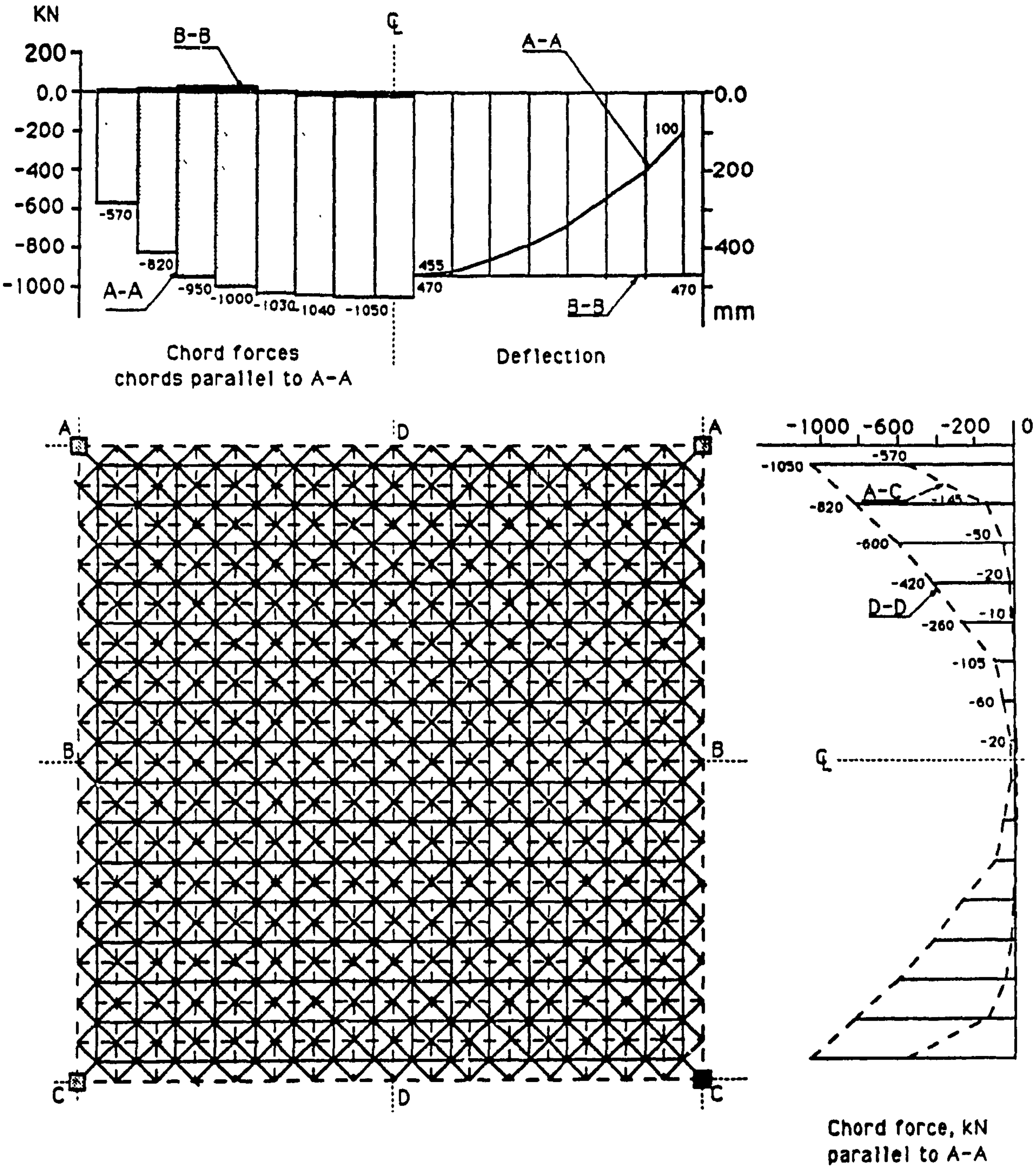


Fig. 2.2 Truss Type 1, top chord force and joint displacement

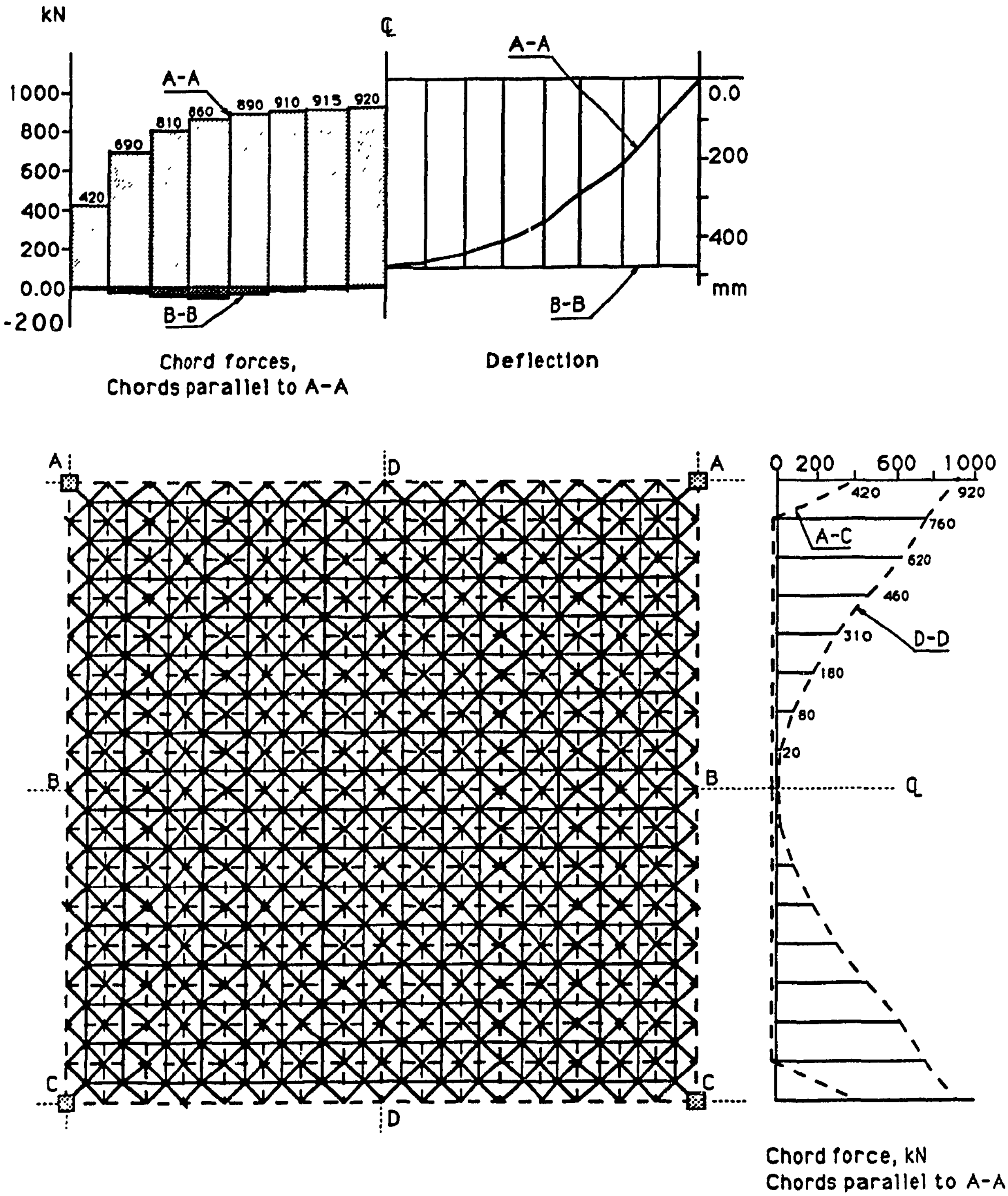


Fig. 2.3 Truss Type 1, bottom chord force and joint displacement

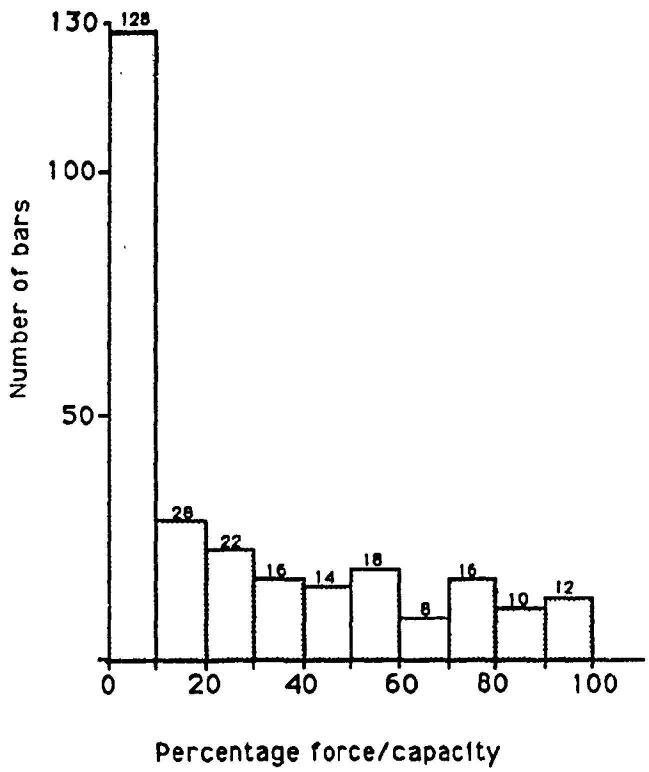


Fig. 2.4, Truss Type 1, chord force distribution

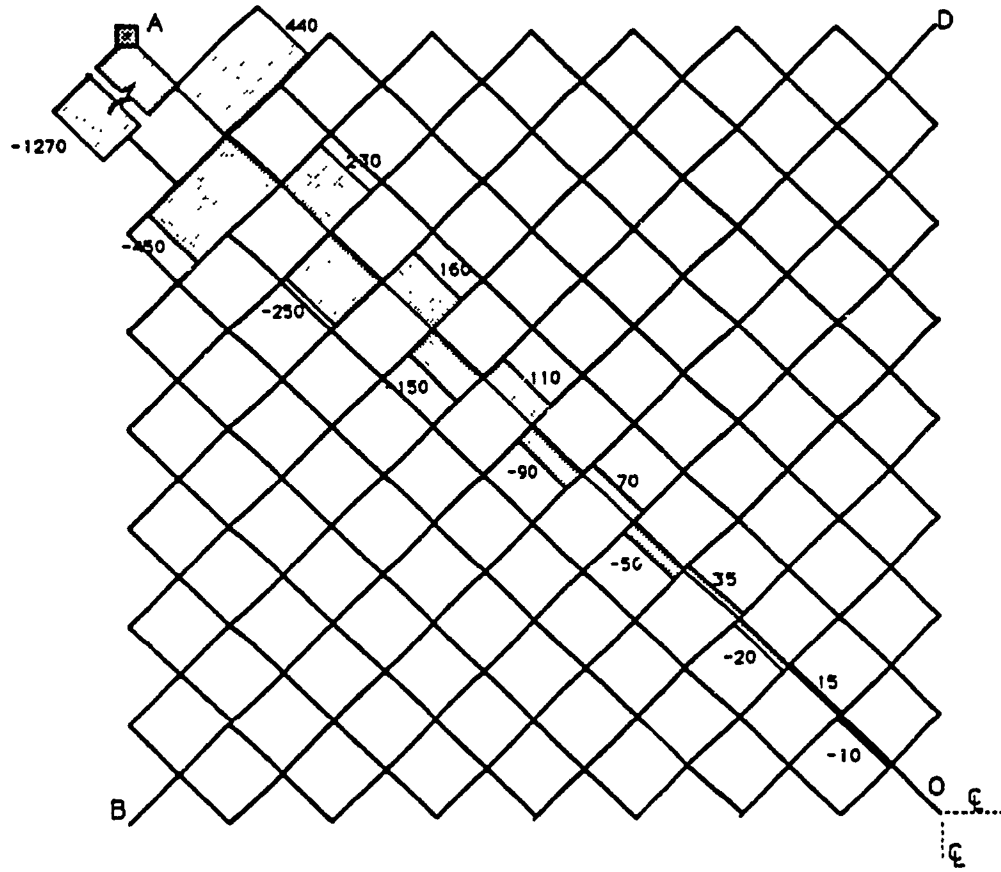


Fig. 2.5, Truss Type 1, force distribution in diagonal members

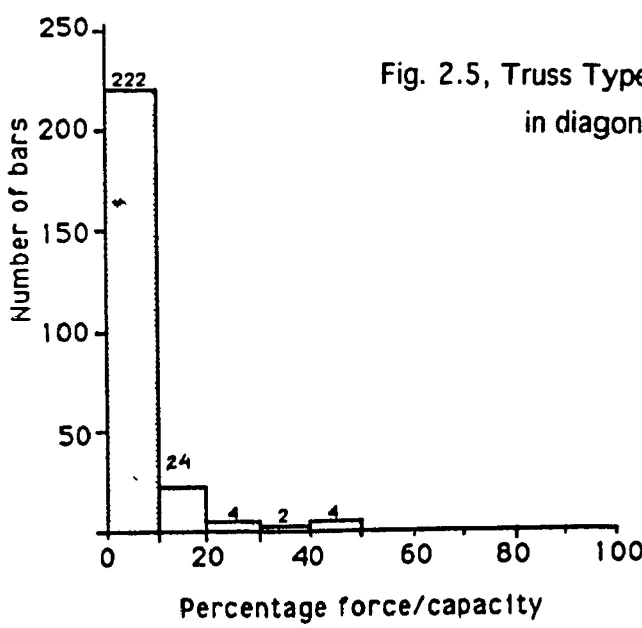


Fig. 2.6, Truss Type 1, percentage force distribution in diagonal members

2.3TRUSS TYPE 2

Truss Type 2 is continuous over the supports with a uniform member size. The geometry of the truss is shown in Fig. 2.1. The same member size as that for the simple truss Type 1 is used.

This change in support condition increased the load capacity to 1.3 kPa, but failure now occurred in the top chord at the support. The weight of the truss remains equivalent to a load of 0.42 kPa, the same as the truss Type 1, because of the same member size and the same geometry, giving the ratio: load/weight =3.1. The forces in the top chords and bottom chords parallel to A-A, and the diagonals are shown in the Fig. 2.7, 2.8 and 2.11. For top chords along the column line (A-A) the forces vary from a tension of 1050 kN, at the support, to a compression of 275 kN at midspan, while at the centre line (B-B) the forces vary from 20 kN to -30 kN. Transition between the lines A-A and B-B are shown with A-C and D-D Fig. 2.7 and 2.8. At the boundary from A towards B the member forces parallel to A-A vary from 1050 kN to 20 kN, and at the centre this variation is from -270 kN to -30 kN. The Fig. 2.9 and 2.10 show the amount of utilized capacity of the chords and the diagonal members. 12 % of the capacity of the chord members and 8% of the capacity of diagonal members are utilized. On the average, only 10% of the total capacity of the truss are utilized.

The Fig. 2.7 and the Fig. 2.8 show also the manner in which the truss deforms under the uniformly distributed loading. The centre-line of the truss is still almost straight. The maximum deflection at the midpoint of the truss is 115 mm under the factored load, giving a service load deflection of span/780.

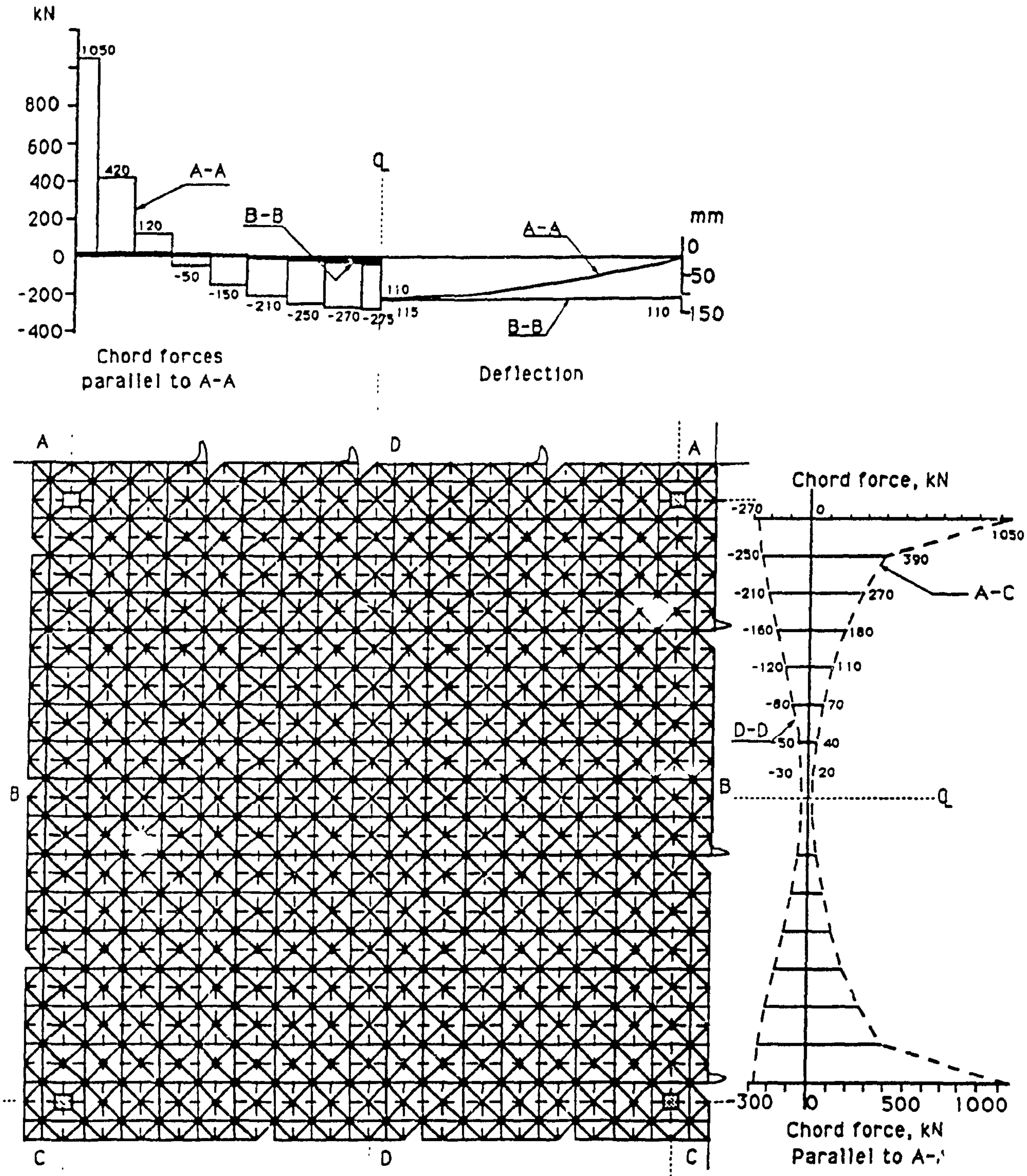
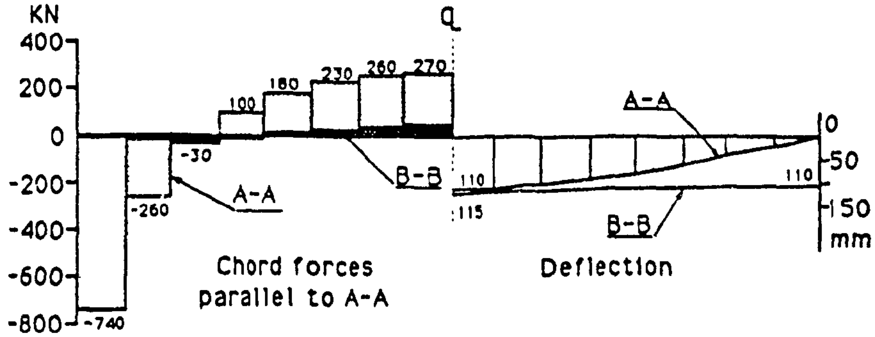


Fig. 2.7. Truss Type 2, top chord force and joint displacement



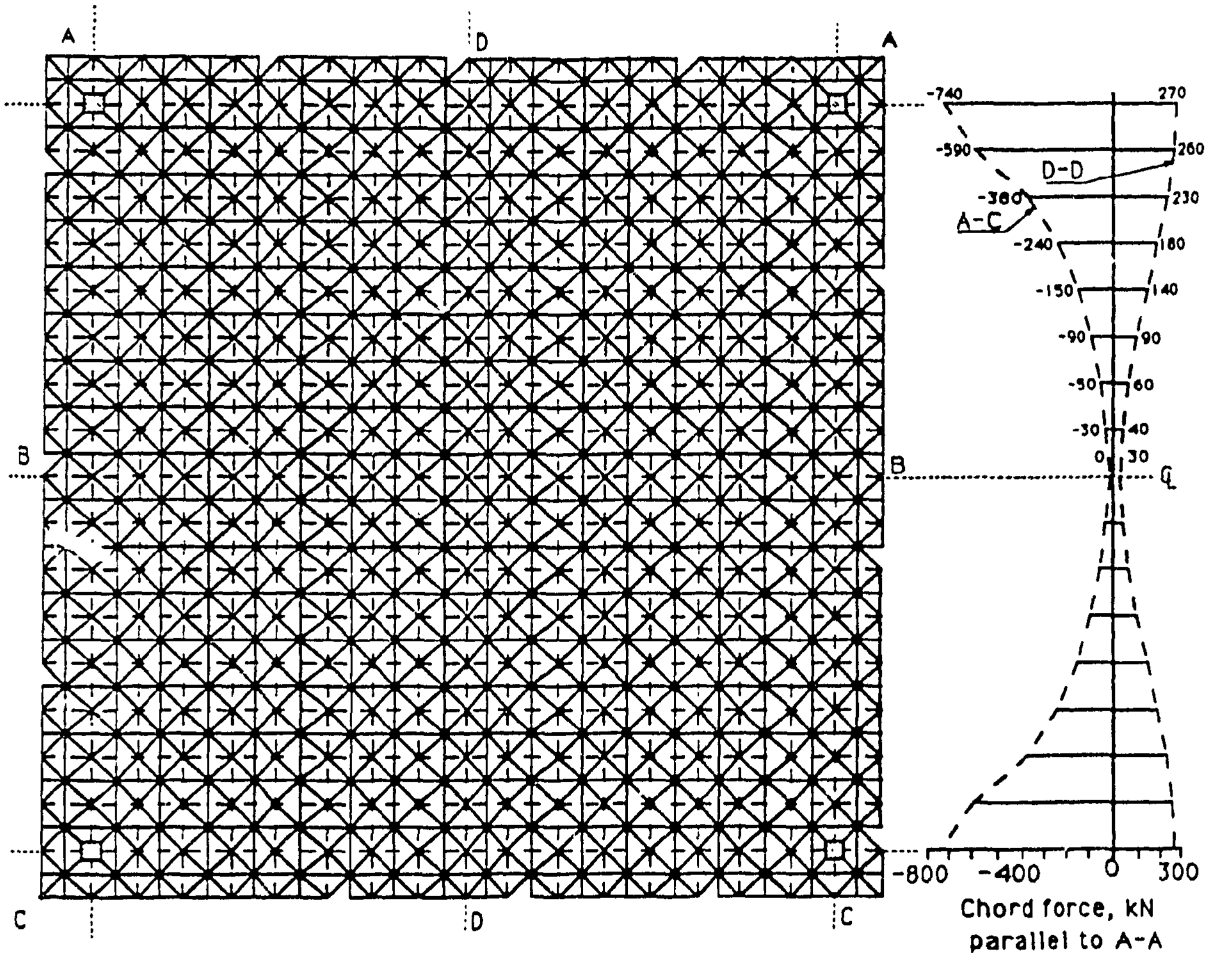


Fig. 2.8, Truss Type 2, bottom chord force
and joint displacement

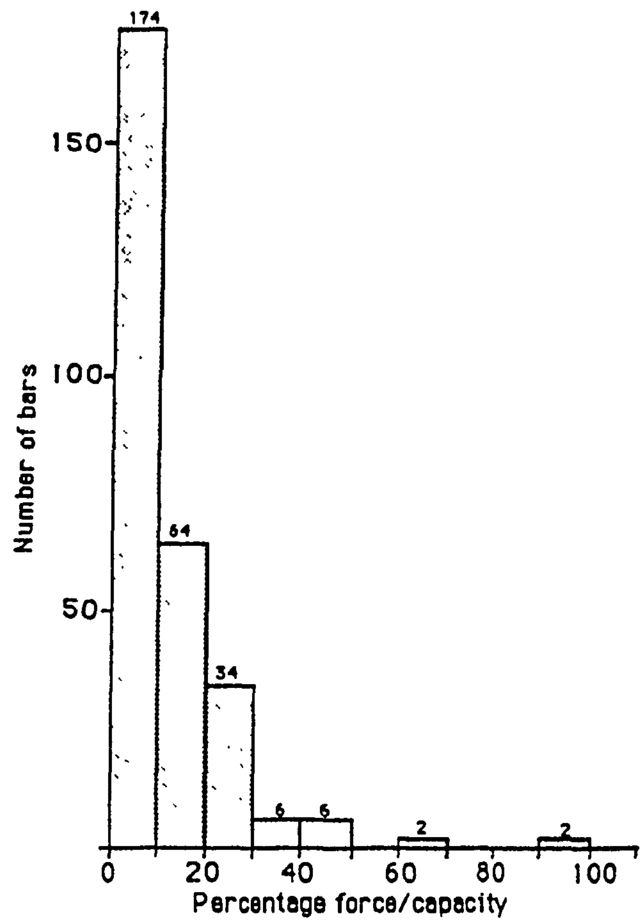


Fig. 2.9, Truss Type 2, force distribution percentage in chord members

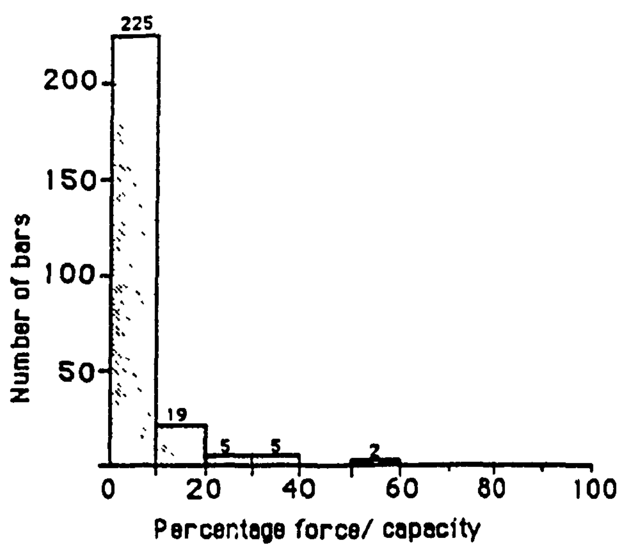
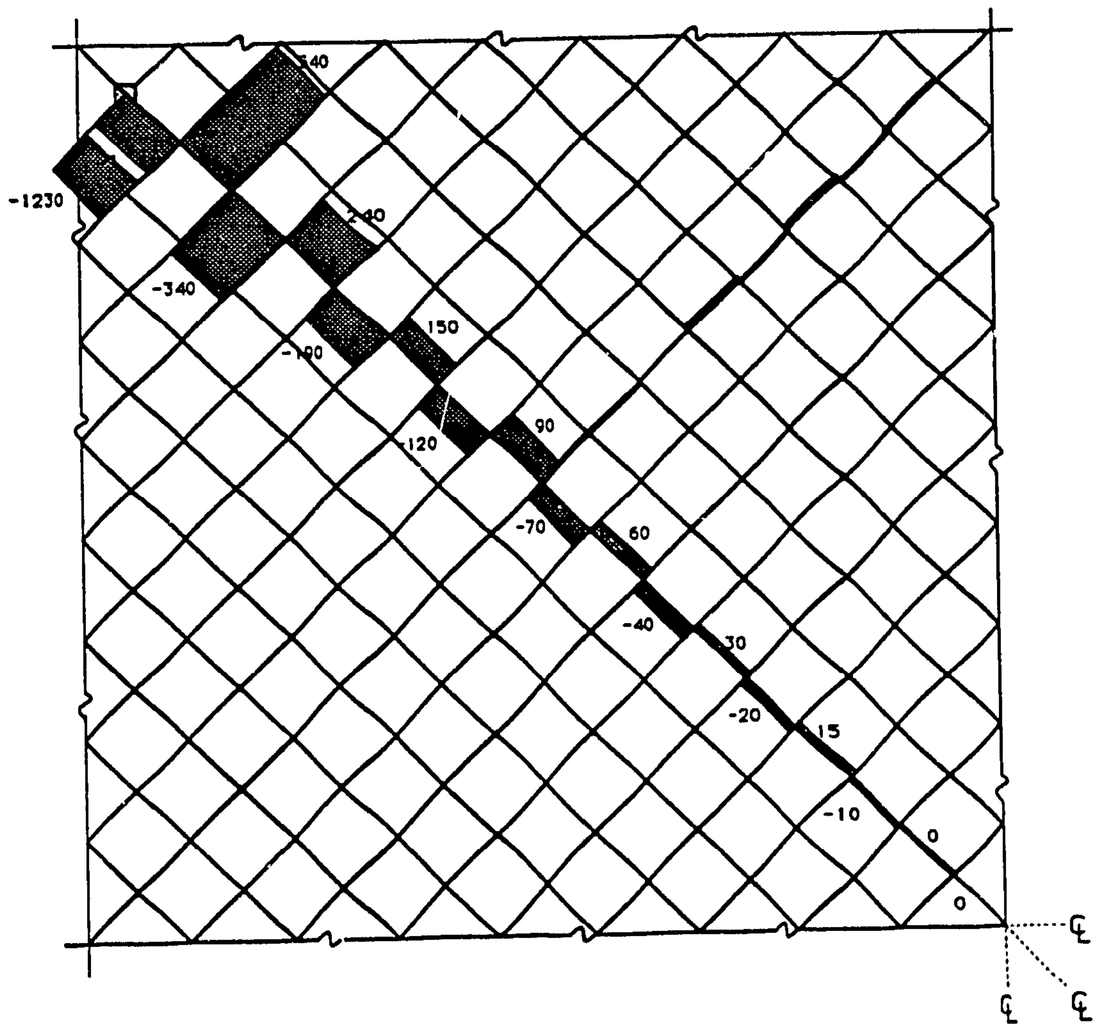


Fig. 2.10, Truss Type 2, force distribution percentage in diagonal members



PLAN

Fig. 2.11, Truss Type 2, force distribution in diagonal members

2.4 TRUSS TYPE 3

Truss Type 3 is simply supported, and makes use of the three member sizes in Table 2.1. The geometry is as for the truss Type 1 shown in Fig. 2.1.

To make the best use of the three member sizes the procedure was as follows:

Assigned the small size (S) to all the members, and increased the load until a member (S) failed. The failed members were changed to the medium size (M), and the analysis was repeated to establish which members now were the first to fail and be replaced by the next larger size. This was repeated until the first failure occurred in the largest chord section (L).

The capacity was limited by the failure in compression of the top chord at the centre of the column-line at an applied load of 0.936 kPa. This value is 22% less than the target value, but, in conducting the analysis, it was decided that the member sizes would be held constant and the load capacity would vary. The weight of the truss was 0.162 kPa giving a ratio: load / weight= 5.78. Fig. 2.12 shows the distribution of member sizes in a quarter plan of the truss.

The force distribution in the top chords is shown in the Fig. 2.13; forces in the chords parallel to A-A, along the column-line, vary from a compressive force of 1050 kN at the midspan to 370 kN at the support. In the same direction but along the centre-line (B-B) chord forces vary from a compressive force of 90 kN at the centre to 10 kN at the column-line. The cross sections D-D and A-C show the transition of member forces from the column-line towards the centre-line. The pattern of force distribution has not changed greatly compared with the truss Type 1, but the lighter chord accepts lower forces, and hence the reduction in overall capacity. Fig. 2.14 shows the force distribution for the bottom chord. The force distribution in the main diagonals is presented in Fig. 2.15.

Fig. 2.16 illustrates that 44 chord members are loaded with less than 10% of their capacity while only 16 members are loaded to over 90% of their capacity. As for the diagonals, 125 members are loaded to less than 10% of their capacity while only 8 members are loaded to above 70% of their capacity. 42% of the capacity of the chords and 18% of the capacity of the diagonals has been utilized. On the average, only 30% of the total capacity of the truss has been utilized.

The Fig. 2.13 and 2.14, also show the manner in which the truss deforms. The centre-line of the truss is not as straight as is truss Type 1, but the relative deformation is still small.

At the limiting load the maximum deflection at the centre of the column-line is 480 mm, and at the centre of the span is 570 mm. For service load the maximum deflection will be span/158.

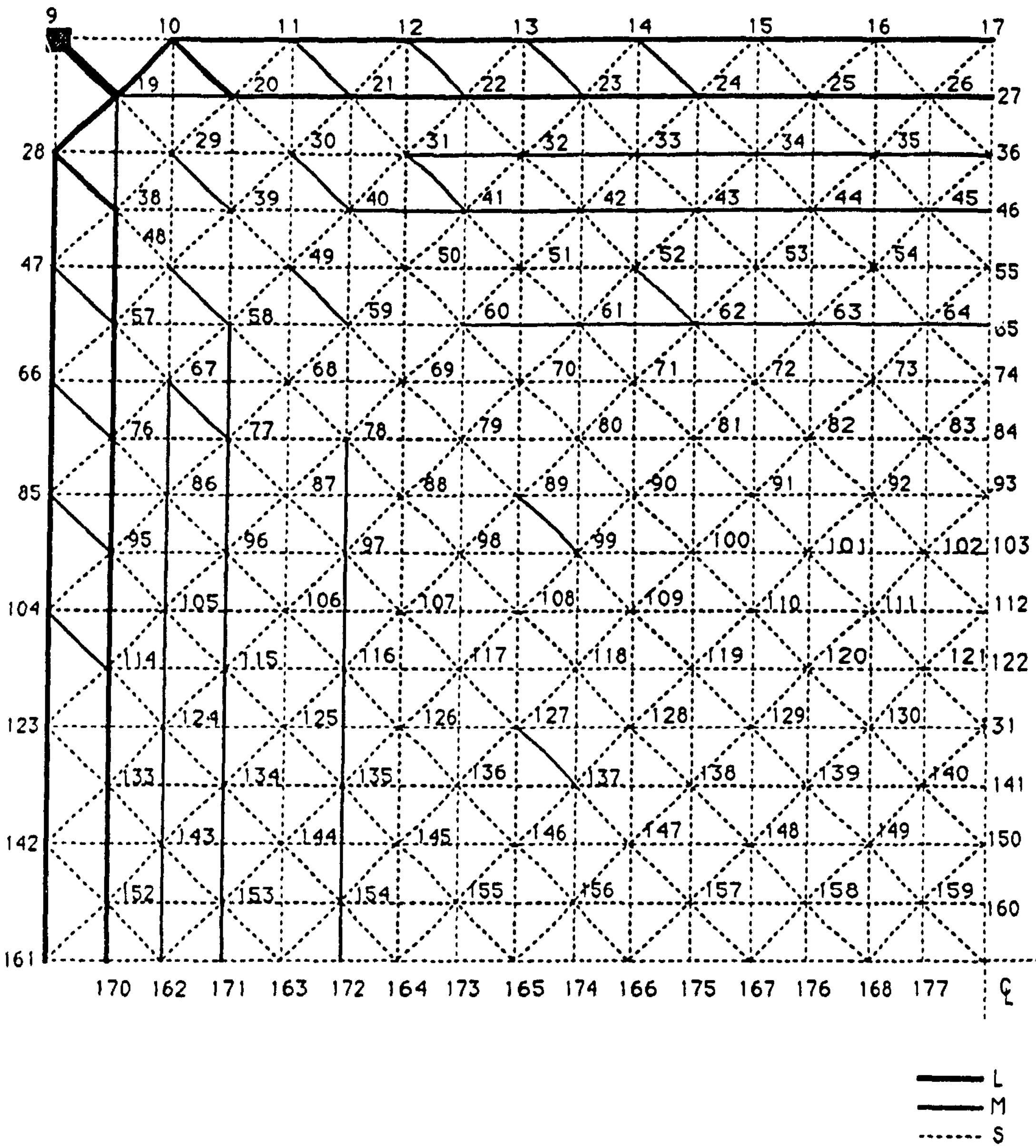


Fig. 2.12, Truss Type 3, distribution of member sizes

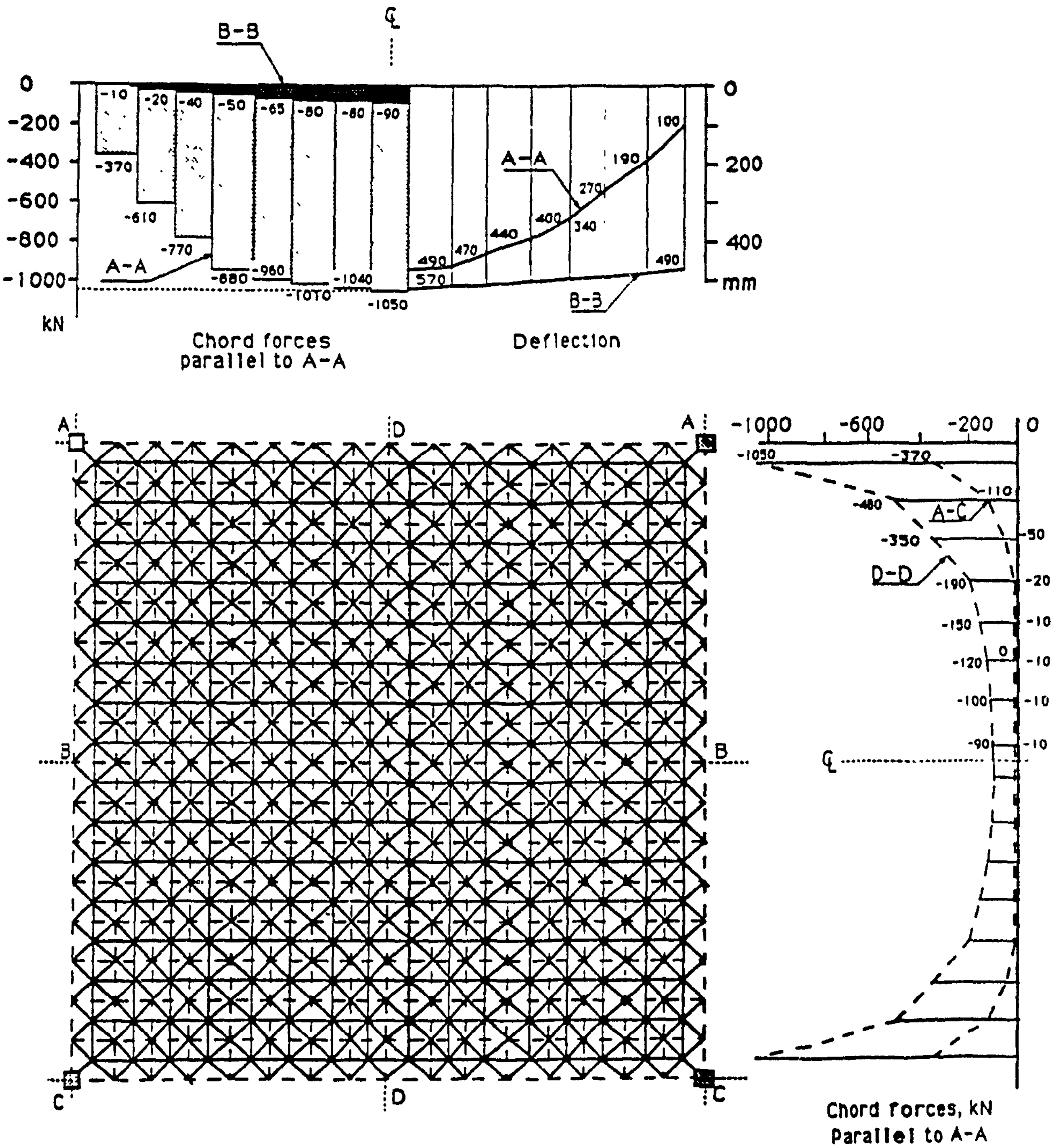


Fig. 2.13, Truss Type 3, top chord forces and joint displacements

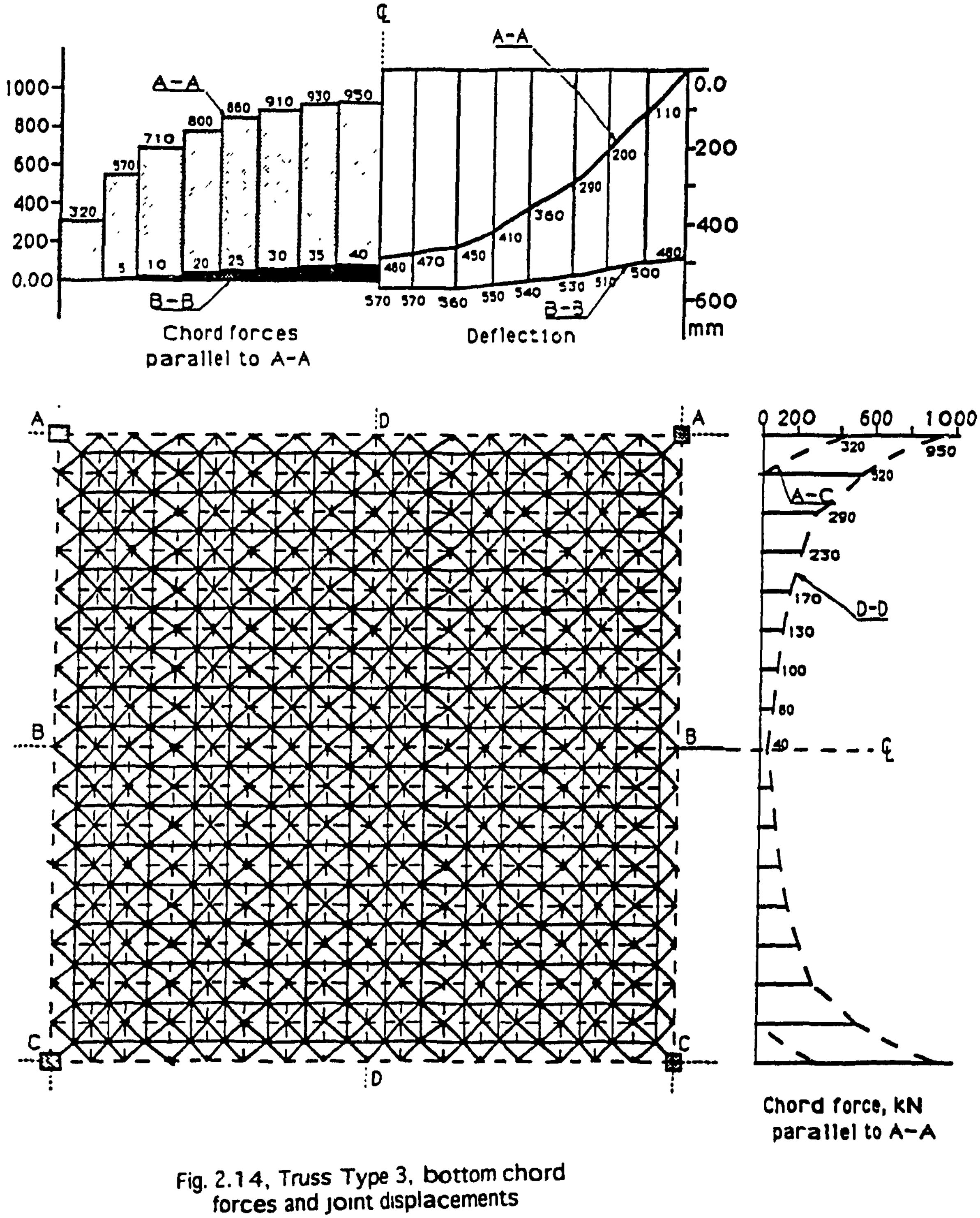


Fig. 2.14, Truss Type 3, bottom chord forces and joint displacements

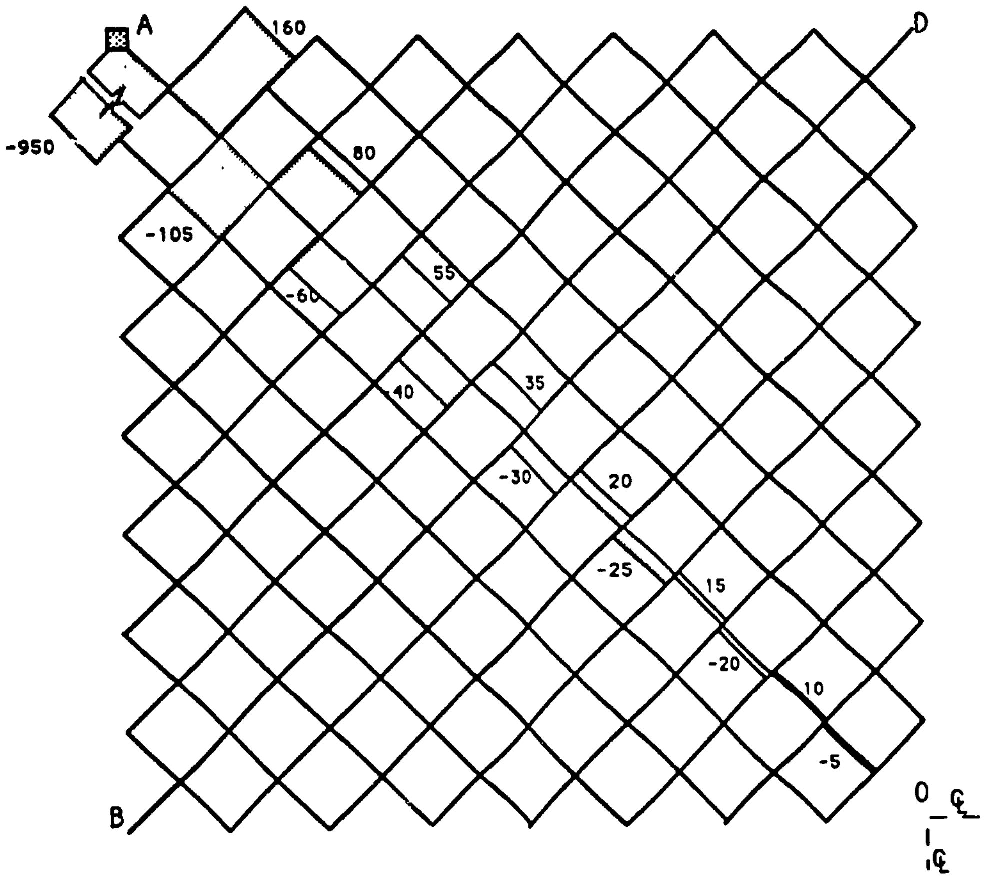


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Fig. 2.15, Truss Type 3, forces distribution in diagonal members

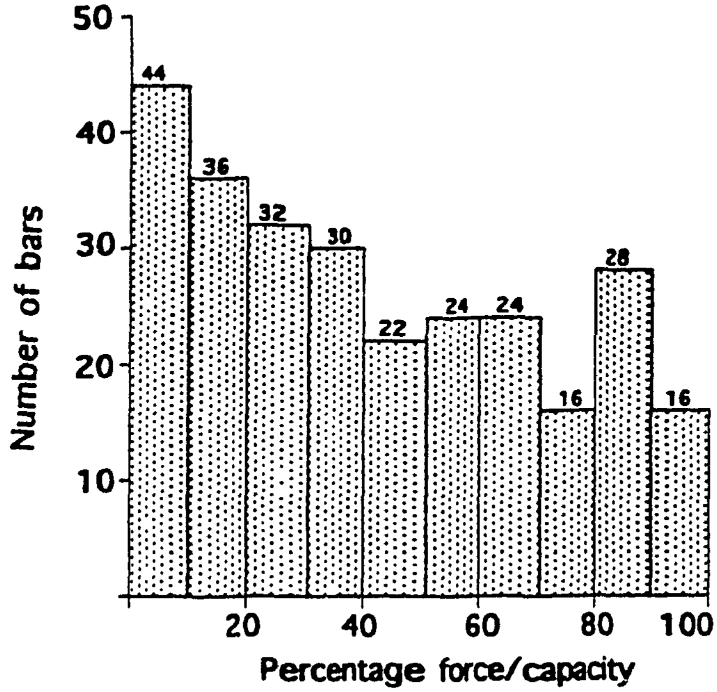


Fig. 2.16, Truss Type 3, force distribution
in chord members

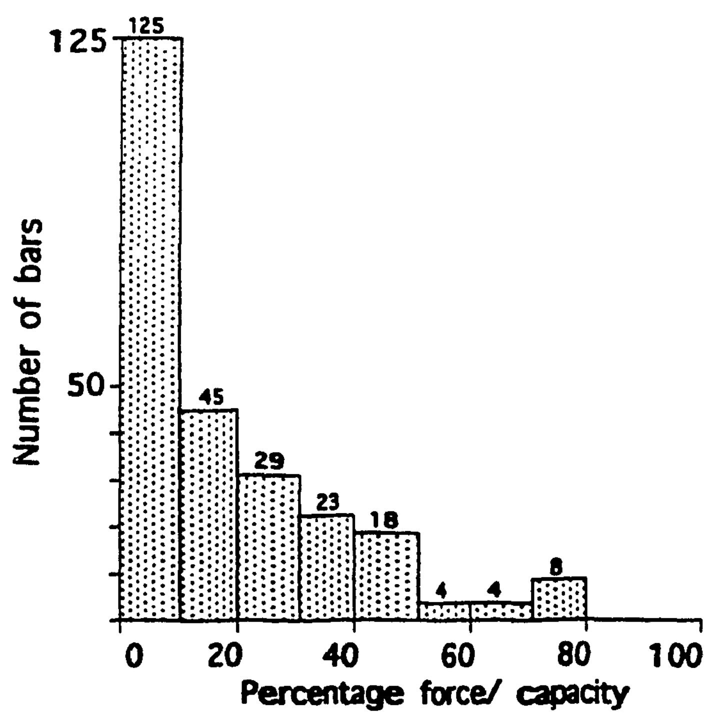


Fig. 2.17, Truss Type 3, force distribution
in diagonal members

2.5 TRUSS TYPE 4

Truss Type 4 is continuous over the supports, with three member sizes. The geometry of the truss is as for the truss Type 2 shown in Fig. 2.1. The same three member sizes as in the truss Type 3 were used and the same procedure was followed to assign the member sizes. The quarter plan, Fig. 2.18, gives the distribution of member sizes.

The capacity of the truss is 1.21 kPa limited by failure in tension of the top chords adjacent to the supports. The weight of the truss is 0.152 kN giving a ratio: load/weight=7.83.

The force distribution in the top chords, is shown in the Fig. 2.19. Forces in chords parallel to A-A, along the column-line, vary from a tensile force of 1050 kN at the support to a compressive force of 210 kN at midspan. In the same direction, along the centre-line (B-B), chord forces vary from a tensile force of 40 kN to a compressive force of 40 kN. The cross sections D-D and A-C show the transition of forces in the members parallel to A-A from the column-line towards the centre-line, which indicate that the force distribution pattern has not changed compared to the truss Type 2, except that the forces are lower.

Fig. 2.20 shows the member force distribution in the bottom chords. The force distribution in the diagonals is given in Fig. 2.21. Fig. 2.22 illustrates that 66 chord members are loaded with less than 10% of their capacity while only 4 members are loaded to over 90% of their capacity. As for the diagonals, 111 members are loaded to less than 10% of their capacity while only 2 members are loaded to above 90% of their capacity, (Fig. 2.23). 31% of the capacity of the chords and 23% of the capacity of the diagonals has been utilized. On the average, only 27% of the total capacity of the truss has been utilized.

Fig. 2.19 and 2.20 also show the manner in which the truss deforms. The centre-line of the truss has a slight curvature as in truss Type 2, but the deformation is still small. At the limiting load the maximum deflection at the centre of the truss is 211 mm, and at the centre of column-line 187 mm. For service load the maximum deflection will be span/427.

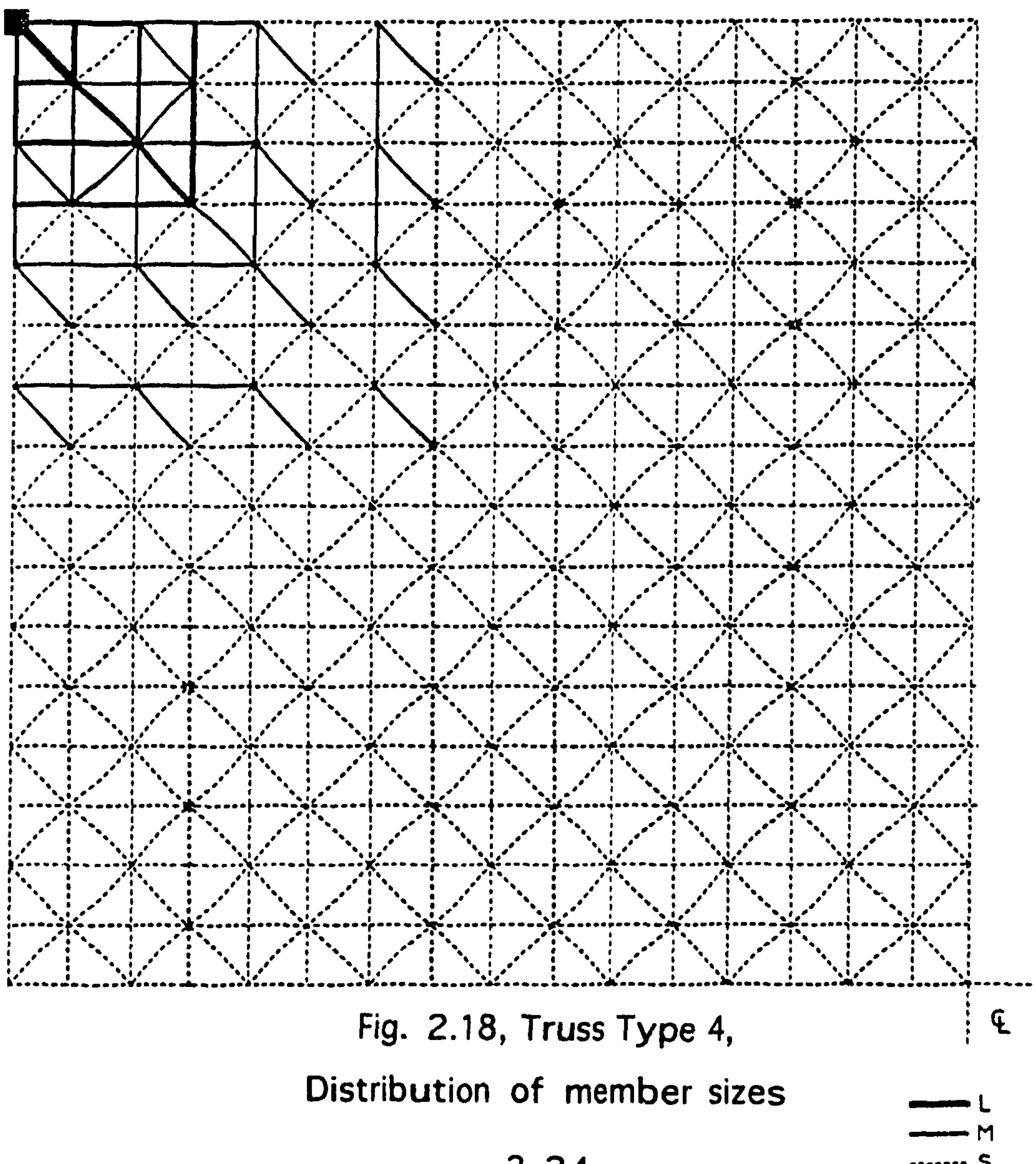


Fig. 2.18, Truss Type 4,

Distribution of member sizes

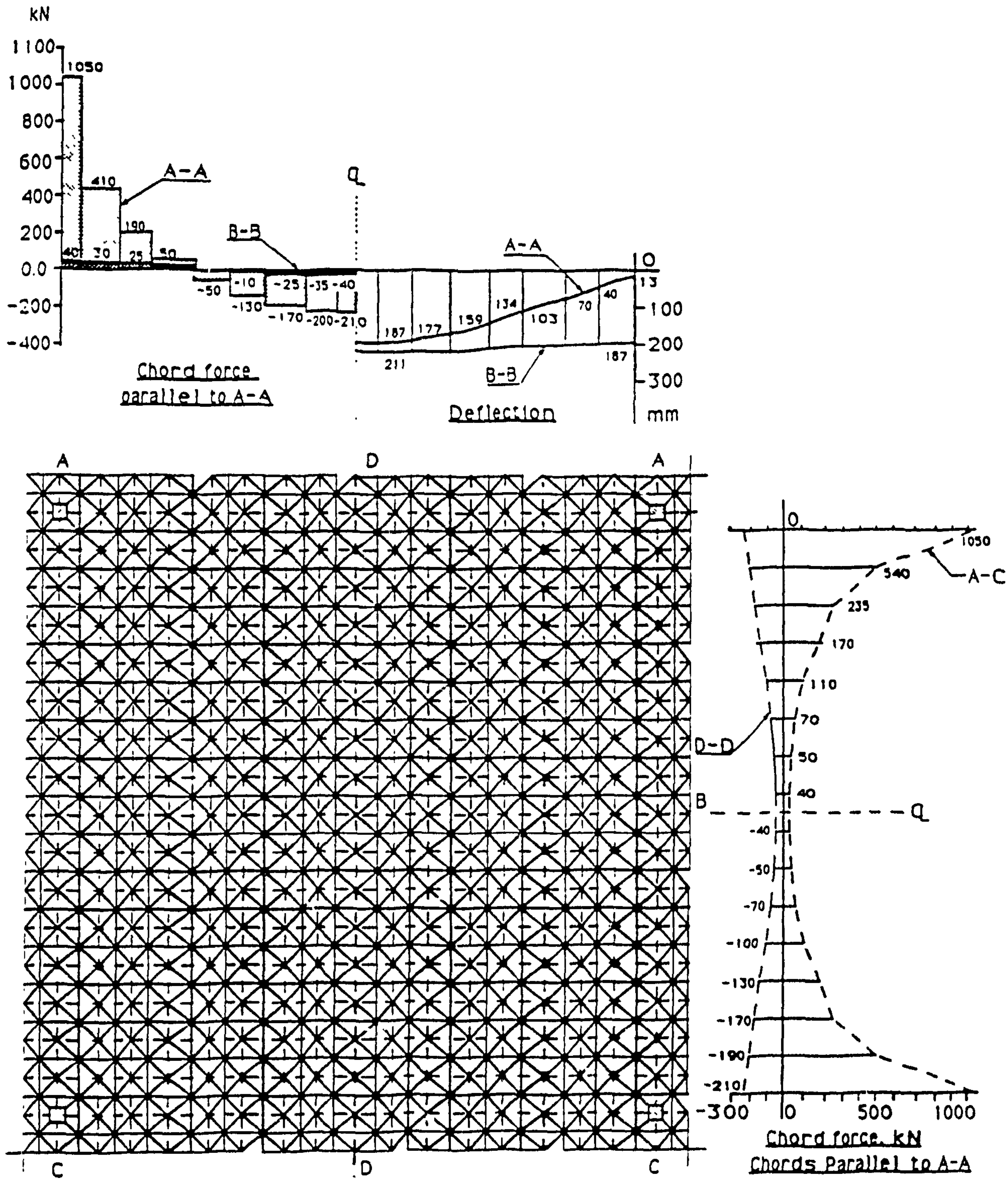


Fig. 2.19. Truss Type 4, top chord forces and joint displacements

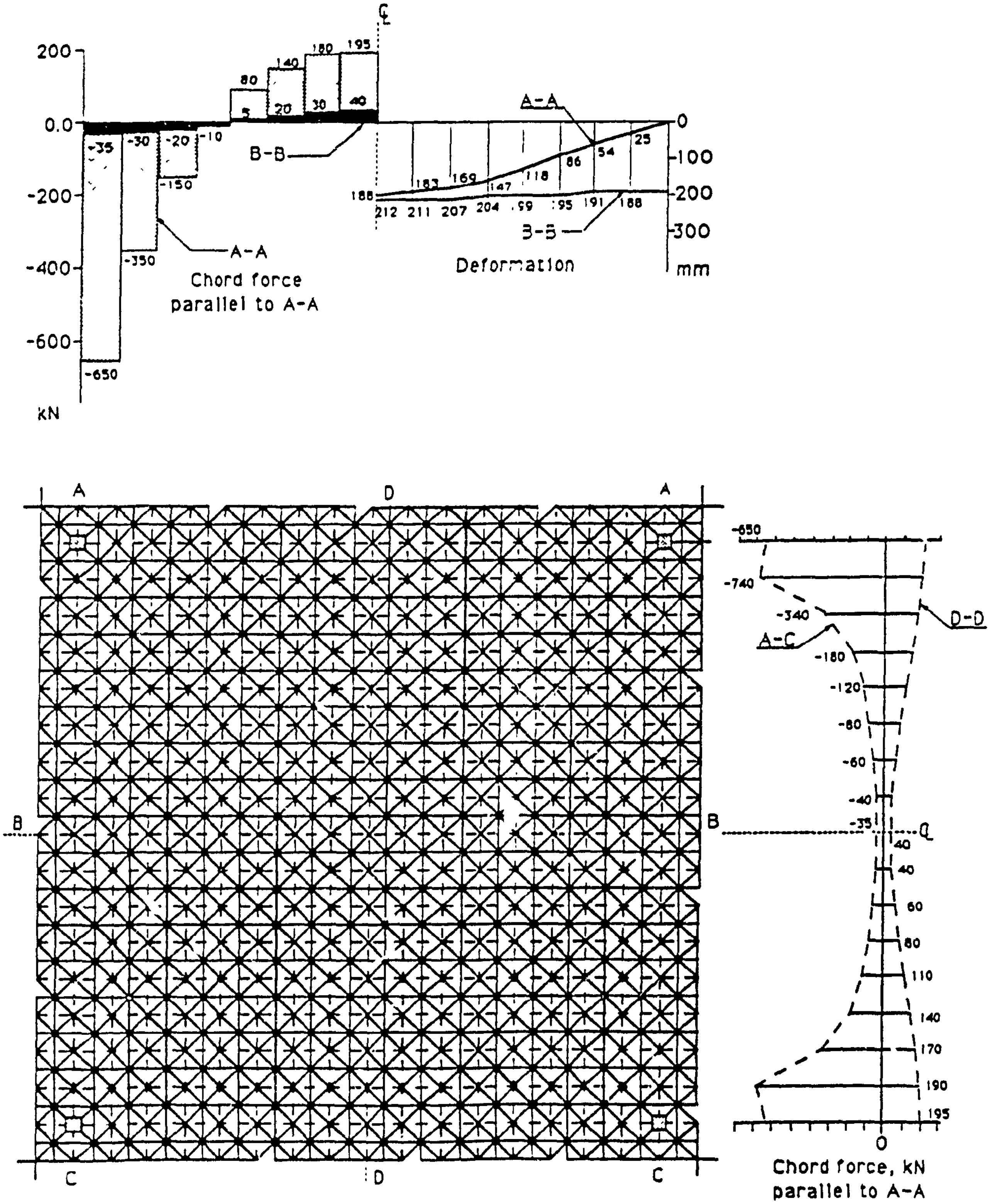


Fig. 2.20, Truss Type 4, bottom chord forces
and joint displacements

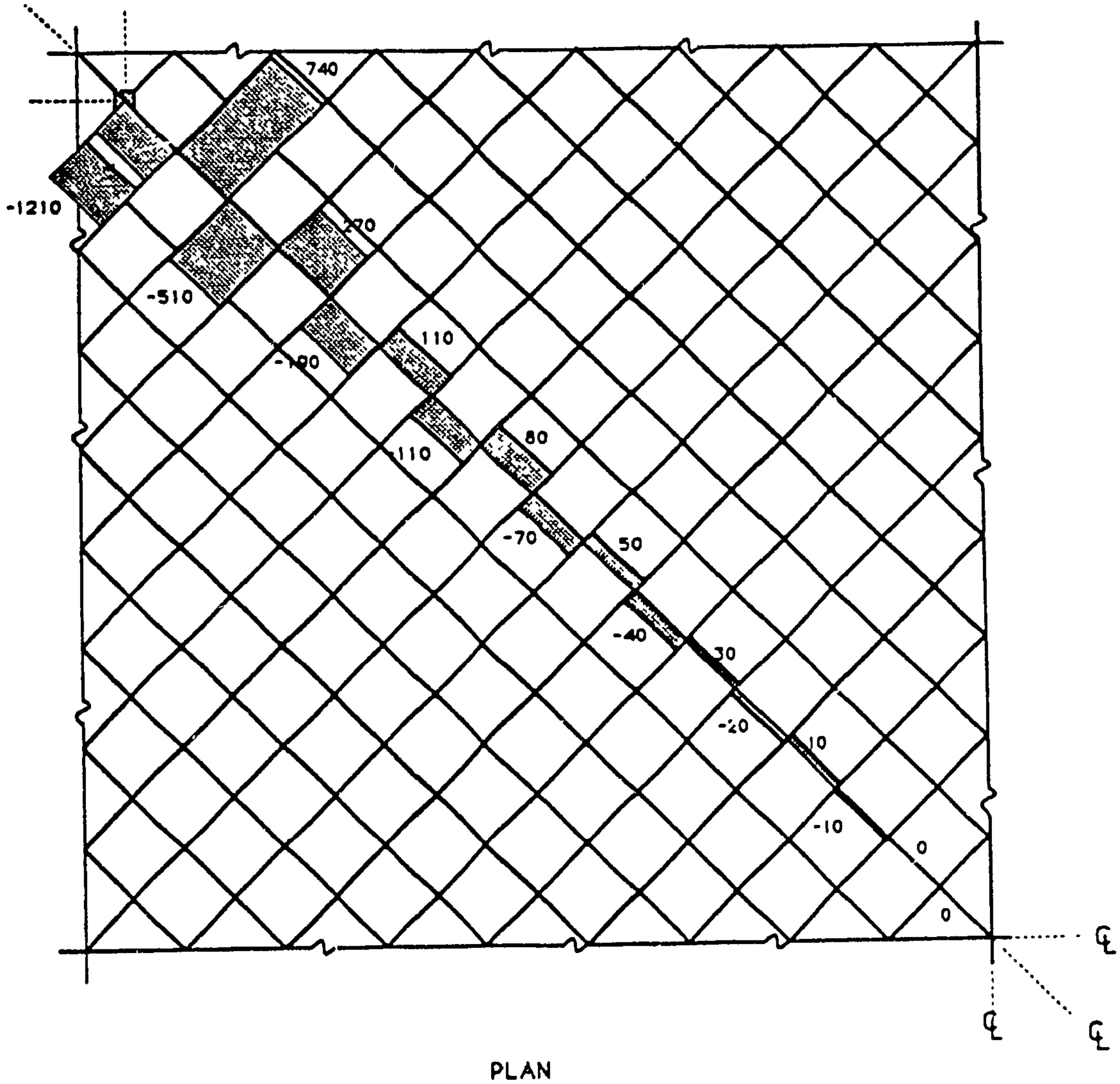


Fig. 2.21, Truss Type 4, force distribution in diagonal members

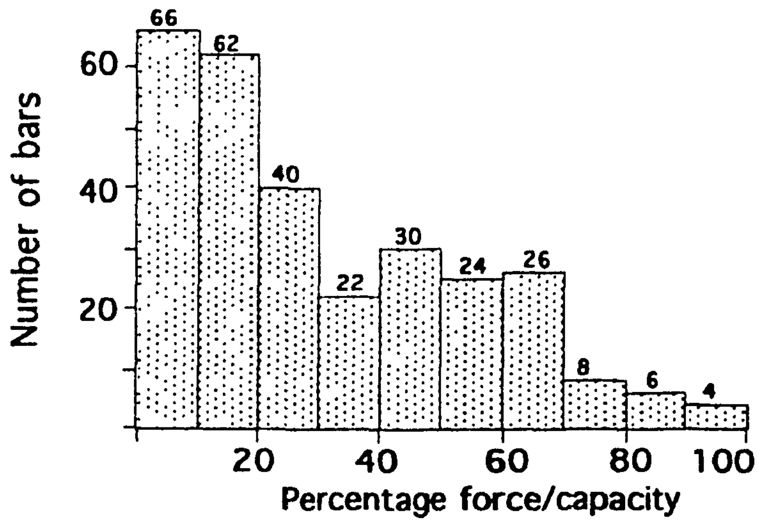


Fig. 2.22, Truss Type 4, force distribution
in chord members

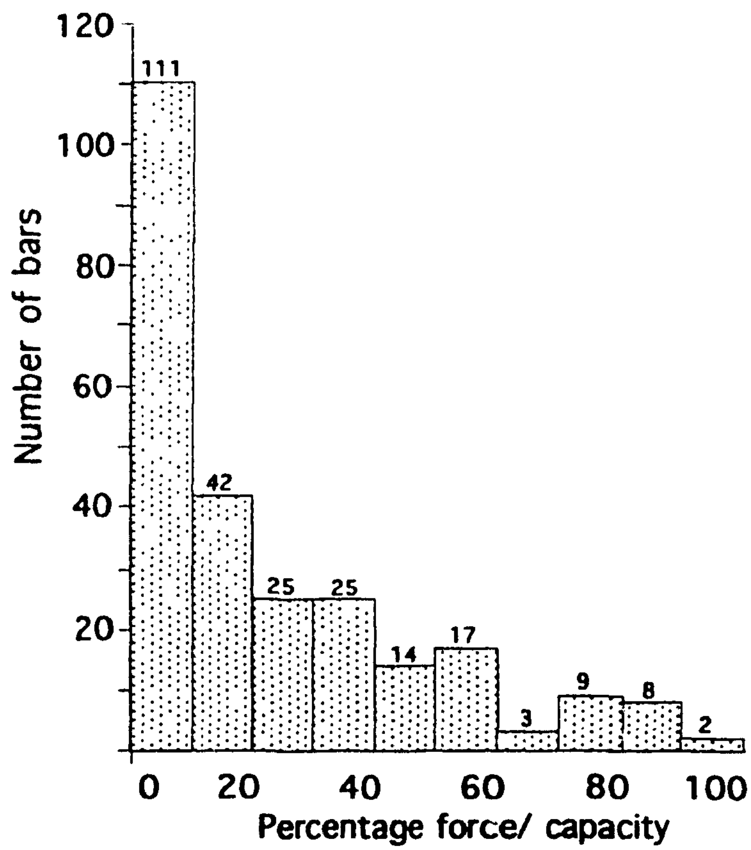


Fig. 2.23, Truss Type 4, force distribution
in diagonal members

2.6 OBSERVATIONS

The results obtained from the study of four types of space truss are tabulated in the Table 2.2:

Table 2.2, Load/weight, deflection/span and utilized capacity of the trusses

Truss	Load (kPa)	Weight (kPa)	Load/weight	Deflection/span (service load)	% Capacities
Type 1	1.25	0.42	2.98	1/190	18
Type 2	1.30	0.42	3.10	1/780	10
Type 3	0.94	0.162	5.78	1/158	30
Type 4	1.19	0.152	7.83	1/427	27

All four types of truss were targeted to support an ultimate nominal load of 1.2 kPa. This was met, reasonably, for all types but Type 3. Comparing the capacity of Type 2 and Type 1 it can be seen that boundary conditions did not affect the capacity as it might have been expected. The capacity of Type 1 was limited by failure in compression in top chords at the midspan, and that of Type 2 by failure in compression of bottom chords at the support.

The weights of the Type 1 and Type 2 are the same. The weights of Types 3 and 4 differ because of different member size distributions. For a truss having all members the same small size, S, the weight would be 0.141 kPa with a capacity of 0.262 kPa for the simply supported and 0.272 kPa for the continuous truss. The weight of Type 3 is only 15% above this value but the capacity is increased by 259%, while Type 4 is only 8% heavier with a 338% increase in capacity.

The vertical deflection/span at the service load for the continuous trusses Type 2 and Type 4 are not likely to be a governing factor, but for the simply supported trusses Type 1 and Type 3 it may govern the design. An interesting observation for the truss Type 1, is that the centre-line assumes a reverse curvature, causing the deflection at the middle of the column-line to be higher than that at the midspan. For the other three types, the midspan deflection is the maximum.

CHAPTER 3

METHODS FOR IMPROVING TRUSS BEHAVIOUR

3.1 NONLINEAR RESPONSE

3.1.1 Force-limiting devices (FLD) in compression chords

A number of force-limiting devices have been devised to impose on the truss members an elastic-plastic characteristic. Shmidt and Hanaor, (1979), introduced a metal skinning feature (Fig. 3.1).

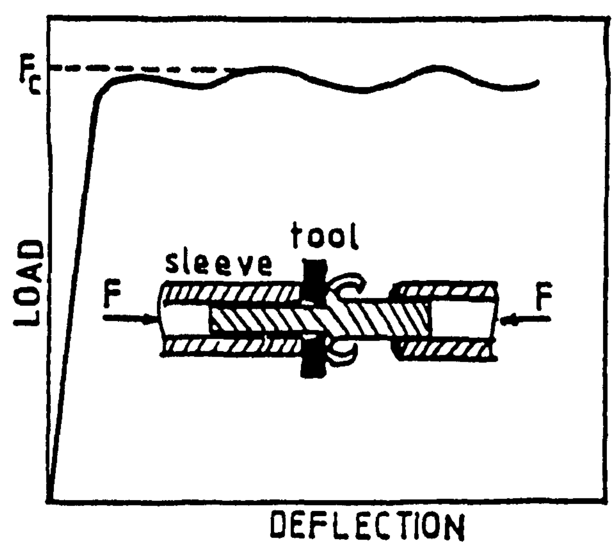


Fig. 3.1 Compressiu.ı force limiting device based on metal skinning principle

Friction based devices, (Fig. 3.2) similar to the ones used to control seismic response of structures, [30] can also be utilized to provide a quasi plastic behaviour.

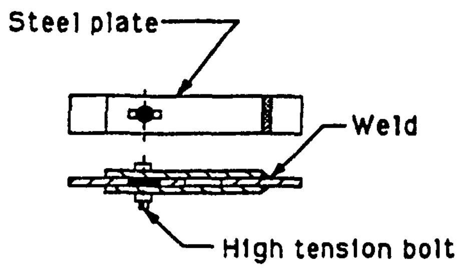


Fig. 3.2, Force-limiting device based on friction

Analysis of the four typical trusses using force-limiting devices, on compression members, with behaviour modelled as in Fig. 3.1, (elastic-fully-plastic), and similar behaviour assumed for tension members, gave the results shown in Table 3.1. This represents the capacity of a fully plastic system, with unlimited yielding, and is the highest value for the load capacity that can be expected to be achieved theoretically.

Table 3.1, Capacity and load/weight ratio using FLD

	Capacity (kPa)		Load/weight	
	Original	FLD	Original	FLD
Type 1	1.25	1.78	2.98	4.24
Type 2	1.30	2.72	3.10	6.50
Type 3	0.94	1.04	5.78	6.42
Type 4	1.19	1.53	7.83	10.1

Incorporation of the force limiting devices on the truss members requires special detailing and hence imposes an extra cost for manufacturing and supervision, which has not been shown to justify their implementation. In the case of load reversal, it is difficult to use the metal skinning device, but the friction joint is reversible.

3.1.2 Over-design of compression members in order to take advantage of nonlinear response of tension members

Compression members may be over-designed to ensure that the tension members yield first, Parke, 1986 [46]. To provide ductility, attention must be paid to the details as not all connections are such that the tension bars would yield across the gross section over the length of the member, and rupture across the net section often governs. The following factors should be established:

1- The amount of over-design.

2- Which compression members should be over-designed

3- The amount of ductility required from the tension members.

Only the truss Type 1, simply supported on columns at four bottom corners, (Fig. 2.1), was analyzed to demonstrate this method. Use was made of compression members listed in Table 3.4, whose behaviour was assumed to be linear elastic to failure. These compression members were modeled by the line OC in Fig. 3.3. The tension members, size L in the Table 2.1, were modeled by lines OAB in Fig. 3.3, where the available plastic strain was chosen to be three times the elastic strain.

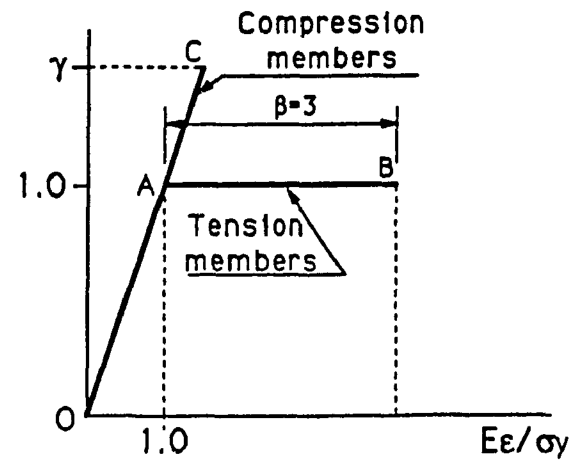


Fig. 3.3, Model for compression and tension members

First, all the members were assigned size L, and a linear elastic analysis was carried out. Those compression members which carried more than 70% of their capacity were replaced by size LG1, 50% stronger than L.

The nonlinear analysis showed that as the applied load increased, the bottom chords successively yielded in tension, starting at the centre of column-line, until the load in the top chords, at the centre of column-line reached their capacity .The failure in compression of the first LG1 was assumed to limit the capacity of the truss.

The sequence of failure is shown in the Table 3.2. For the node numbering see Fig. 2.12. At this point the load carrying capacity was 1.69 kPa giving a load/weight ratio of 3.94 which shows an increase of 32% with respect to the truss Type 1. Fig. 3.4 shows the 50% overdesigned compression members and also shows the yielded tension members.

By changing those members which are over the 70% of capacity of L to LG1, 90% of the over-designed members sustained forces higher than the capacity of size L.

The deflection due to the ultimate load, at the centre of column-line was the same as the deflection at the midspan of the truss, with a value of 575 mm.

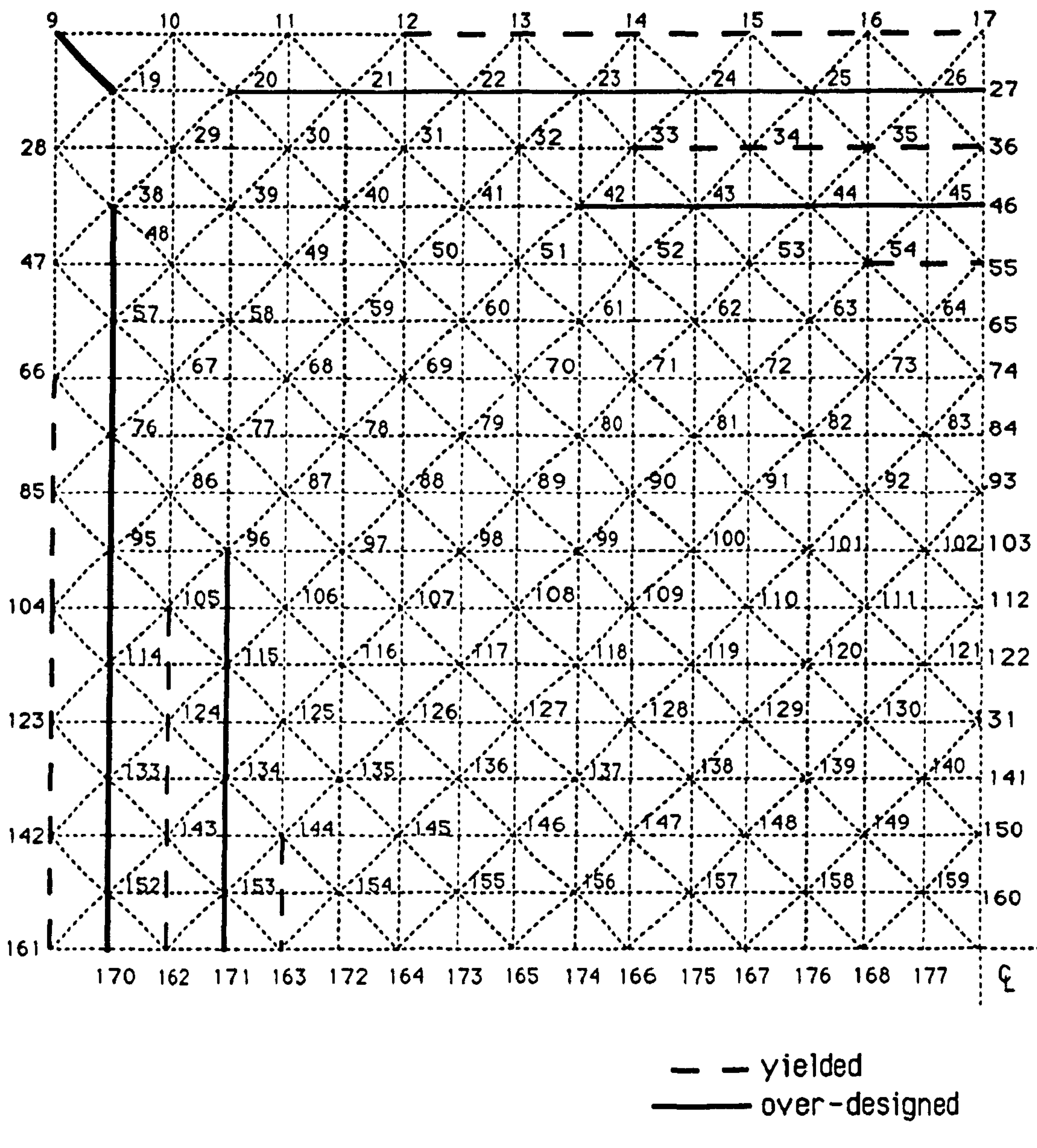


Fig. 3.4, Truss Type 1, yield pattern
(over-design 50%)

Table 3.2, Sequence of yielding truss Type 1, 50% overdesign

Cycles	1	2	3	4	5
Load (kPa)	1.39	1.45	1.53	1.57	1.62
Yielded members	15-16 16-17 123-142 142-161	13-14 14-15 85-104 104-123	35-36 143-162	34-35 124-143	12-13 33-34 66-85 105-124
Δ17, (mm)	455	480	515	537	575
Δ169, (mm)	465	489	522	542	575

Δ17, Δ169 = Deflection of joints 17 and 169 respectively.

The procedure was repeated for over-design factors of 100% , 120% and 150%, using member sizes given in Table 3.4. The 120% over-design gave a balanced case in the sense that the first compression member reached its capacity when a tension member reached the point B on the diagram of Fig. 3.3. The results are shown in the Table 3.3. The load/weight ratio corresponding to 120% over-design is 4.75 which shows an increase of 60% with respect to the truss Type 1.

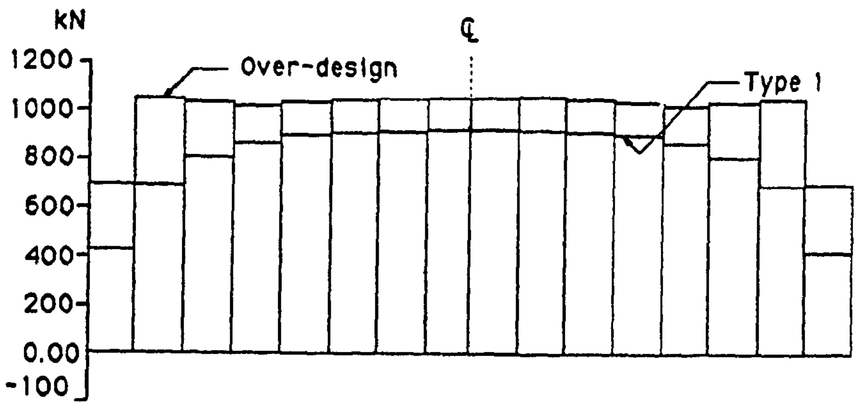
Table 3.3, Truss Type 1, Influence of over-design of compression chords

Over-design %	Load (kPa)	Weight (kPa)	Load/weight ratio	Deflection (mm)
0	1.25	0.42	2.98	470
50	1.69	0.429	3.94	575
100	1.99	0.436	4.56	908
120	2.08	0.438	4.75	1070
150	2.08	0.442	4.70	1028

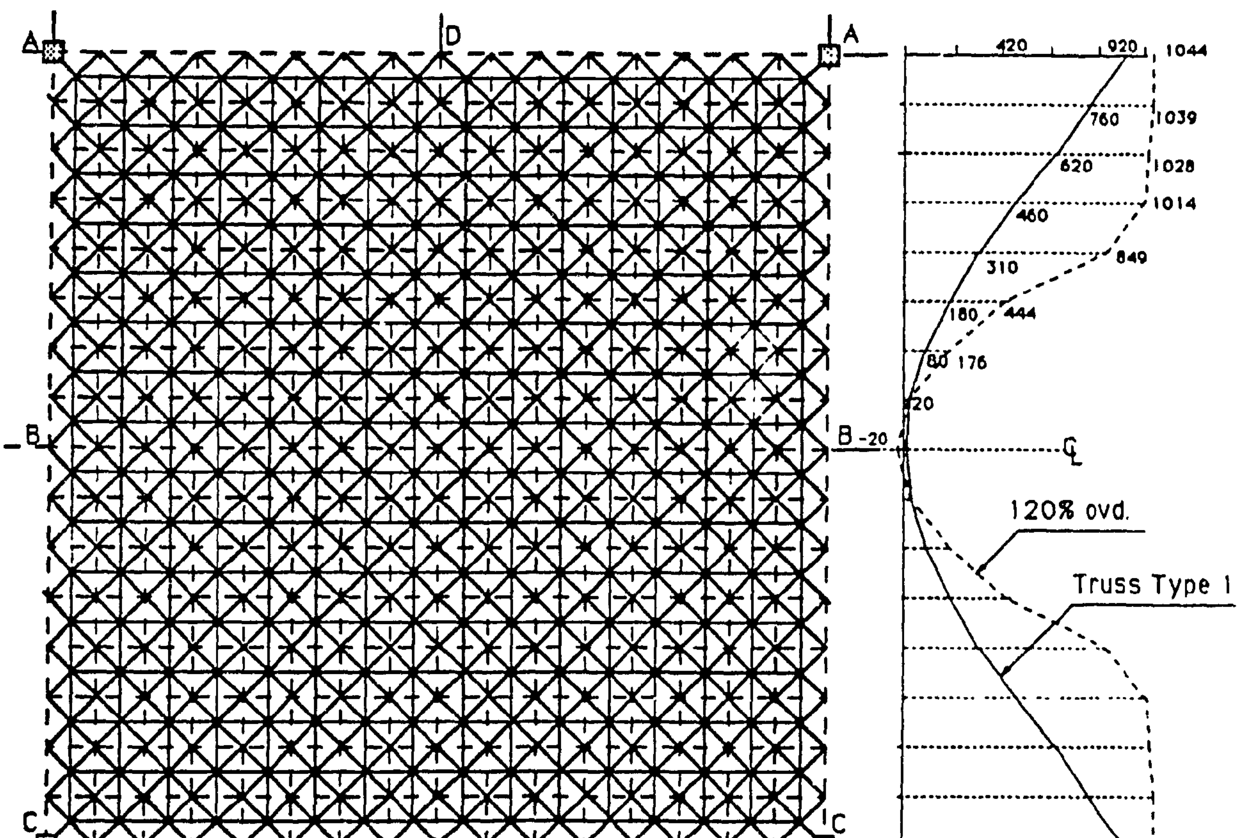
Table 3.4, Member sizes for over-designed compression chords

Over-design %	Size	Area (mm^2)	Capacity (kN)
0	L	5830	1050
50	LG1	8750	1575
100	LG2	11600	2100
120	LG3	12830	2310
150	LG4	14575	2625

Fig. 3.5 shows the force distribution in the bottom chords of the truss with over-designed top chords, (120%), and also the force distribution in the bottom chords of the original truss Type 1 as a



Chord forces, along A-A
chords parallel to A-A



Chord force, along D-D,
parallel to A-A, kN

Fig. 3.5, Truss Type 1, bottom chord forces
(top chord 120% over-designed)

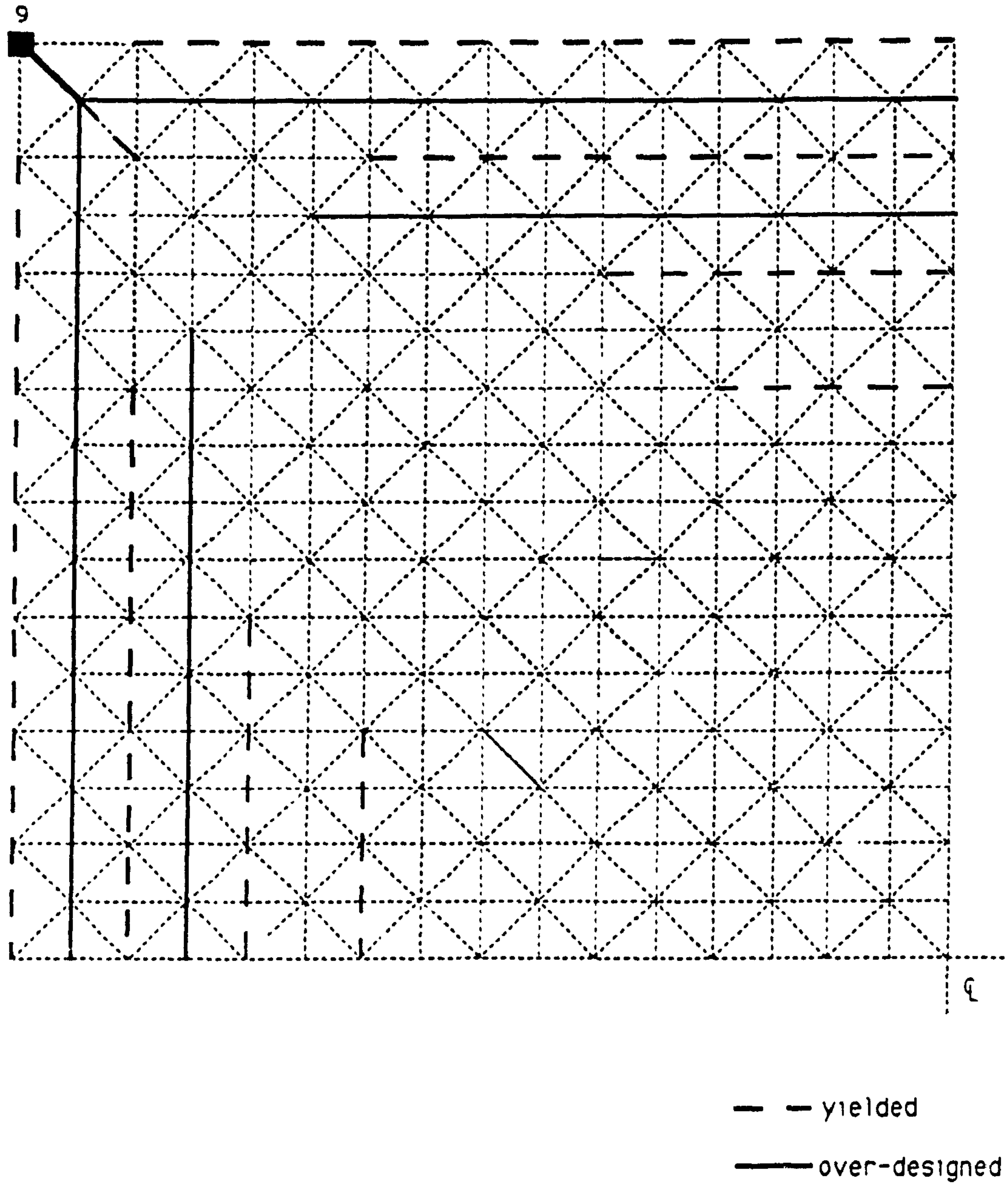


Fig. 3.6, Truss Type 1,yield pattern
(over-designed 120%)

reference.

It is seen that, by over-designing selected compression members, together with nonlinear behaviour of tension members, redistribution of forces among the less loaded members can be achieved. For the truss Type 1 with the chosen factors, (over-design of compression members (120%) and the plastic strain of tension members (β=3), the load/weight ratio was increased by 60% with only 4% extra cost for material.

Practically any of the proprietary systems can accommodate the over-design of compression members, but special details should be worked out to allow for the overall yielding of the tension members to provide the required nonlinearity. However the amount of nonlinearity, (β=3), is not very high and can be easily achieved with certain space truss systems.

3.1.3 Nonlinear response of diagonal members

Eccentrically loaded compression members (angles, channels, T's, etc.) can provide nonlinearity in the elastic range. Used as diagonal members they improve chord force distribution while remaining elastic, as has been demonstrated by Marsh and Fard [33]. This procedure was not examined as it parallels that of diagonal removal discussed later, in that it is a means of controlling the distribution of shear forces in the truss.

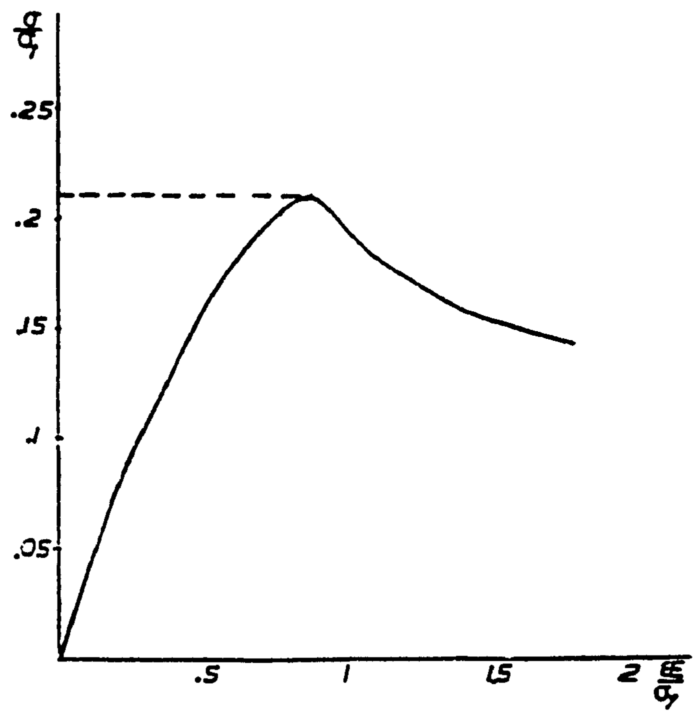


Fig. 3.7 Nonlinear behaviour in elastic range of eccentrically loaded T sections [33]

3.2 FORCE REDISTRIBUTION

In order to redistribute the forces in chords while still in the elastic range two methods are of particular value:

1) Removing selected diagonals to route the flow of forces towards the less loaded members (Marsh, 1986). This is discussed in detail in chapter 6.

2) Increase in load capacity can be achieved by imposing a lack of fit (LOF) of selected members in a controlled manner. Such prestressing utilizes the compressive capacity of nominally tensile members and can be used to compensate for the deleterious effect of random lack of fit. The algorithm for selection of the members is based on the flexibility method of structural analysis and the amount of lack of fit is determined by the Simplex method of linear programming (Hanaor and Levy 1986). The lack of fit will be imposed on N_X bars, where N_X =b+R-3j, b= the number of bars, R= the number of reactions, and j= the number of joints.

For the truss Type 1, previously analyzed, b=2048, j=545, R=12 and N_X=425. This number of members, which constitutes 21% of the total, would require individual amounts of LOF.

Fabricating members with the appropriate lack of fit would make assembly difficult, while only a few systems, such as MERO, permit adjustment after assembly, but, even then, marking these members and carrying out adjustments would require time in the shop and at site, which could cancel the savings gained by the reduction in weight of the structure. Moreover, in case of load reversal the prestressing would weaken the structure. No analysis was made using this procedure.

CHAPTER 4

INFLUENCE OF NONLINEAR MEMBER BEHAVIOUR ON THE LOAD CAPACITY OF SPACE TRUSSES

4.1 INTRODUCTION

None of the behaviour modification techniques described has been adopted by designers as established methods for improving the behaviour of the space truss, partly because they are not very straight forward and also because they have not yet been adequately demonstrated in practice. In the following sections the amount of influence of nonlinear member behaviour on the ovrall load capacity of space trusses is demonstrated.

In order to take advantage of nonlinear member behaviour, it is necessary to establish the range of nonlinearity required to bring about a useful redistribution of the forces in the chord members. That this amount of nonlinearity is attainable at a reasonable cost must also be demonstrated.

4.2 NONLINEAR MEMBER BEHAVIOUR MODEL

In Fig. 4.1 a typical load/change-of-length curve for a member is shown, in which σ is the mean stress and $\varepsilon=\delta/L$ is the effective axial strain. The segment OA represents the linear elastic range up to a stress of $\alpha\sigma_c$ and the segment AB represents the nonlinear range with a plastic strain of $\beta\varepsilon_e$, where σ_c is the limiting stress and $\varepsilon_e= \sigma_c/E$.

To simplify the analysis, the curves are replaced by a piecewise linear relationship. To model a particular behaviour, values are assigned to the parameters α and β, Fig. 4.1.

In order to study the influence of the point of deviation from elastic behaviour, α, the value of plastic strain, β, was chosen to be constant at 1 and α was assigned values between 0.1, and 0.95, Fig. 4.2.

The influence of β was then studied by taking α=0.7 and varying β between 1 and 10, which represent, respectively, linear elastic and almost plastic member behaviour, Fig. 4.3. The four types of truss previously analyzed with the members, S, M, and L from the Table 1.1, were reanalyzed. All sizes were assigned the same values of α and β in a given truss.

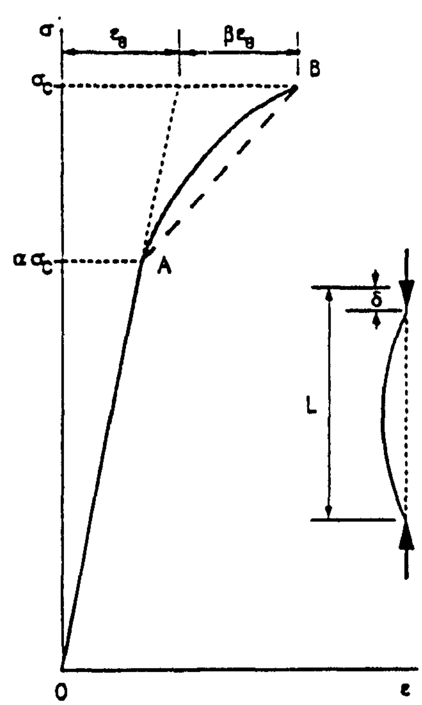


Fig. 4.1, Nonlinear behaviour model

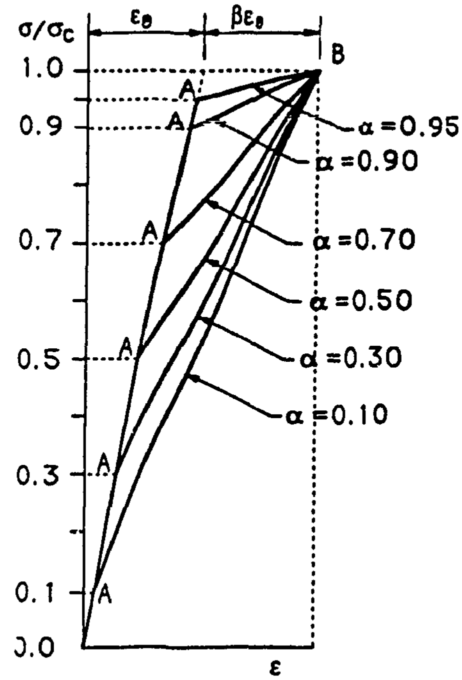


Fig. 4.2, Model for member behaviour, β=1, with different values of α

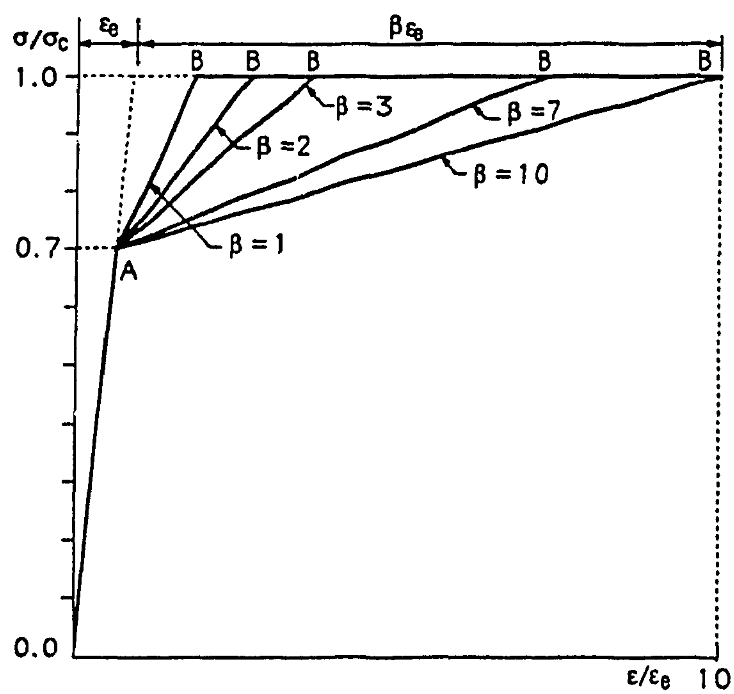


Fig. 4.3, Model for member behaviour, α =0.7, with different values of β

Truss Types 1 and 2 have uniform member sizes. Truss Types 3 and 4 use three different member sizes, Types 1 and 3 are simply supported, Types 3 and 4 are continuous.

The results of the analyses for variation in α, with $\beta=1$, are tabulated in Table 4.1.

Table 4.1 , Influence of α on the load/weight of a truss, ($\beta=1.0$)

α	0.0	0.1	0.3	0.5	0.7	0.9	0.95
Type 1 Load (kPa)	1.25	1.30	1.37	1.45	1.52	1.59	1.60
Load/weight ratio	2.98	3.10	3.25	3.45	3.60	3.80	3.80
Type 2 Load (kPa)	1.30	1.32	1.42	1.45	1.45	1.45	1.45
Load/weight ratio	3.10	3.15	3.40	3.45	3.45	3.45	3.45
Type 3 Load (kPa)	0.93	0.93	0.94	0.97	1.00	1.03	1.03
Load/weight ratio	5.78	5.78	5.82	5.98	6.17	6.36	6.36
Type 4 Load (kPa)	1.19	1.19	1.19	1.20	1.21	1.24	1.25
Load/weight ratio	7.83	7.83	7.83	7.88	7.96	8.17	8.23

Fig. 4.4 shows the influence of α on the load/weight ratio of each truss. It is seen that the value increases with α, but the influence is not great.

The four types of truss were analyzed with $\alpha=0.7$ and β varying from 1 to 10. The results are given in Table 4.2 and Fig.4.5.

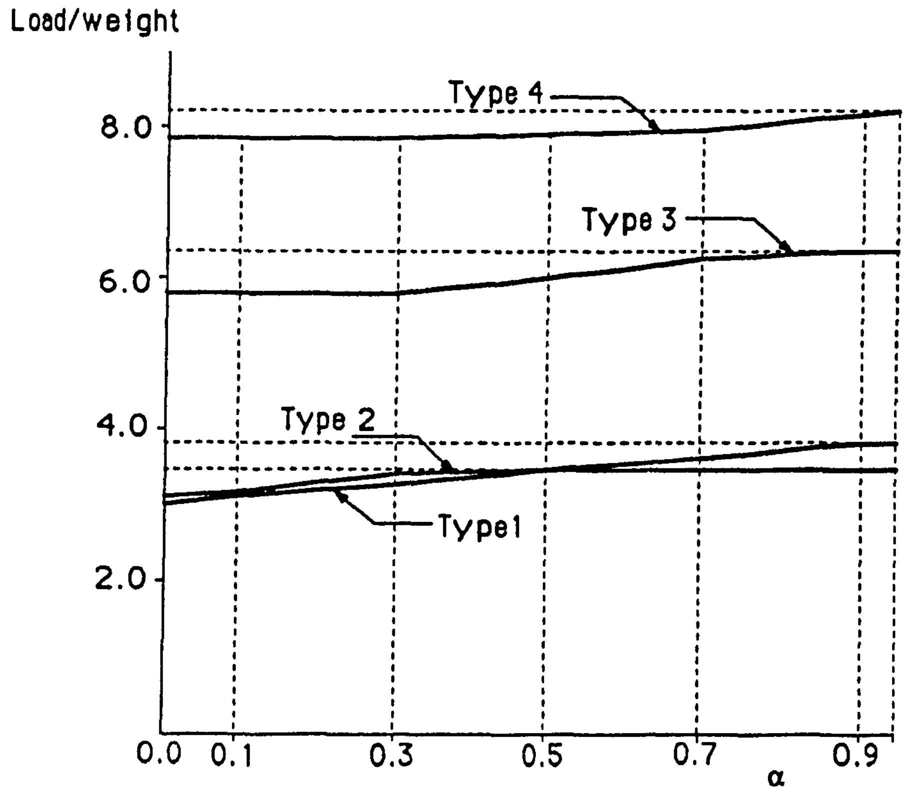


Fig. 4.4, Influence of α on load/weight of truss, $\beta=1$

Table 4.2, Effect of β on the load/weight for a truss, $\alpha=0.7$

β	0.0	1.0	2.0	3.0	7.0	10.0
Type 1 Ultimate load (kPa)	1.25	1.52	1.58	1.60	-	1.63
Load/weight	2.98	3.60	3.80	3.80	-	3.90
Type 2 Ultimate load (kPa)	1.30	1.45	1.57	1.72	2.16	2.31
Load/weight	3.10	3.45	3.74	4.10	5.14	5.50
Type 3 Ultimate load (kPa)	0.93	1.00	1.01	1.02	-	1.04
Load/weight	5.78	6.17	6.23	6.30	-	6.42
Type 4 Ultimate load (kPa)	1.19	1.21	1.23	1.24	1.27	1.28
Load/weight	7.83	7.96	8.09	8.16	8.36	8.42

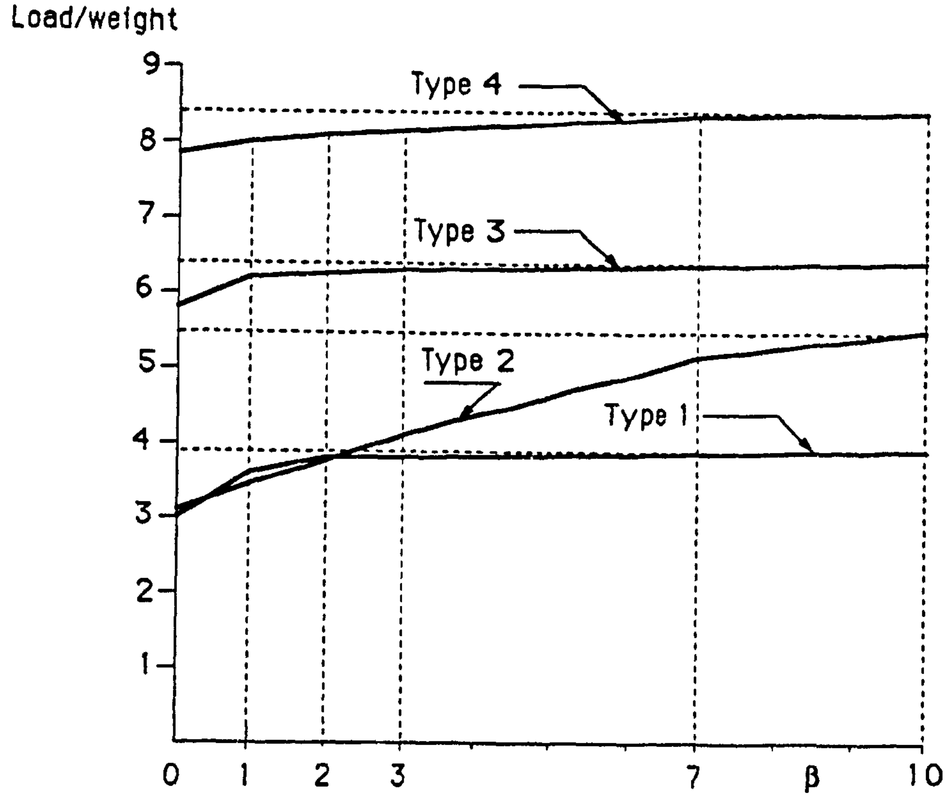


Fig. 4.5 Influence of β on the load/weight for a truss, α=0.7

It is seen from Fig. 4.5 that for simply supported trusses, Types 1 and 3, after a plastic strain of β=3 the curve becomes flat and a further increase in β does not influence the load/weight ratio substantially, this is because the bending moment curve for such a truss is smooth and most of the members at this point have already taken their share of load with not much unutilized capacity in the members at the column line. However, for the continuous trusses, Types 2 and 4, the load/weight ratio increases beyond β=3 due to the fact that the moment diagram is sharply changing from a large negative value at the support to a positive value at the centre. By increasing β, more members accept higher loads and the capacity of the truss continues to increase.

It is also seen that for trusses Type 1 and 2 which have all members the same size, nonlinear behaviour increases the load/weight ratio considerably by using their large reserve of capacity, while the Types 3 and 4, which use three different sizes, do not show the same increase, Fig. 4.5.

4.3 CONCLUSION

Both the point of deviation from linear elastic behaviour, α and the amount of nonlinearity, β, are seen to influence the load/weight ratio of the truss, but the influence of β is more important. The effect of these parameters varies with the boundary conditions and the member size distribution.

For a plastic strain of $\beta=3$, which is readily available, the load/weight ratio of a truss composed of uniform members can be increased by up to 30% of the theoretical elastic value. In a typical truss with varied member sizes a 10% increase may be realized. If this amount of nonlinearity can be guaranteed the increase in capacity may not be great but, of more importance, the nonlinear response of the structure avoids the danger of premature failure that can occur when compression members are linear elastic up to collapse.

CHAPTER 5

PRACTICAL DEVELOPMENT OF NONLINEAR MEMBER BEHAVIOUR

5.1 INTRODUCTION

Space truss systems on the market are generally concerned with transferring forces between members and joint pieces, they do not often attempt to provide ductility in order that the transfer of load from the over-loaded elements to the under-loaded elements could be possible. The brittle buckling of compression members, especially when they fall in the transitional portion of buckling curve, usually limits the capacity. Attempts have been made to provide for a redistribution of forces, as explained in chapter 3.

None of the techniques of behaviour modification described in chapter 3, have been adopted by the designers in a full size space truss. The reason may be because they are not very simple to use and there are not enough data available showing their performance.

A part of this study is concerned with nonlinear member behaviour, which shows that strength and safety of space trusses can be increased if the members could sustain designated forces and deform axially, to allow the understressed members to take their share of force. In order to show the influence of nonlinear member behaviour, a full scale structure was tested, but before testing the truss a number of tests were carried out on different truss elements to study their individual characteristics.

The HEMTEC system has been chosen for this study. In this system the force limiting device is incorporated in the fabrication of the members, Fig. 5.1. Round tubular members are attached to spherical node pieces by means of bolts passing from the inside of the sphere to threaded anchor plates held inside the tubes by means of grooves on the tube wall. The grooves also serve as a force limiting device and their capacity can be engineered to be slightly less than the buckling capacity of the strut and to sustain the force as the member shortens. The elements of the truss are shown in Figs. 5.1 and 5.2.

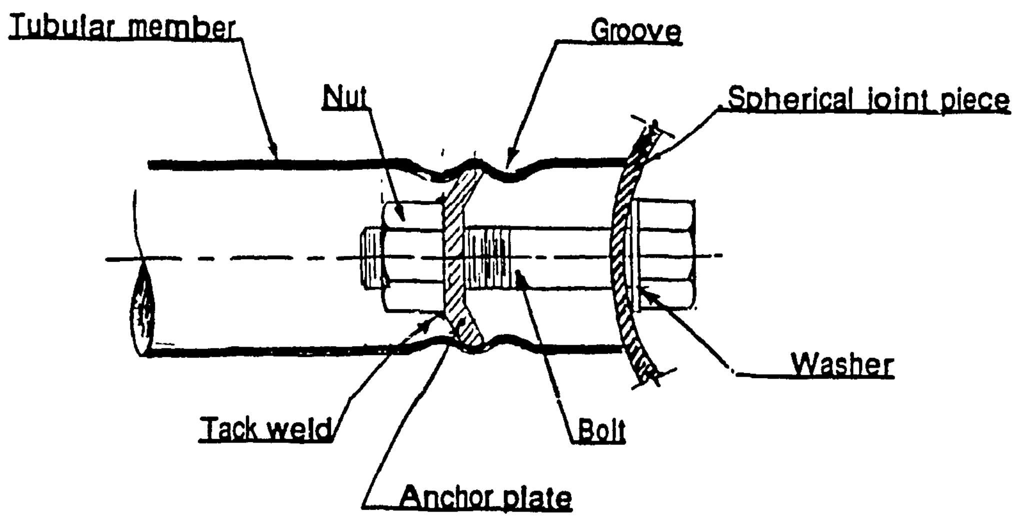
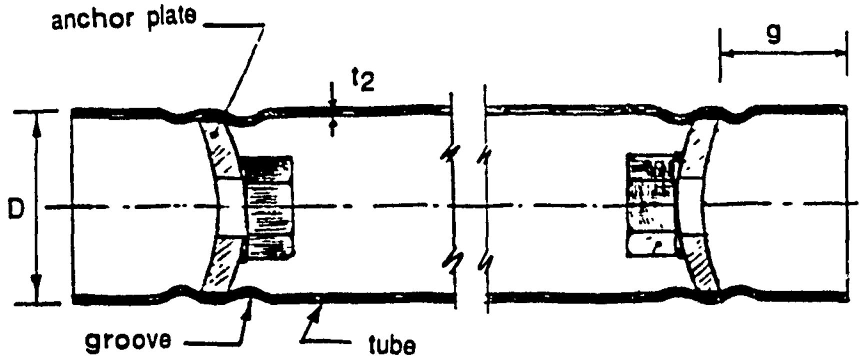
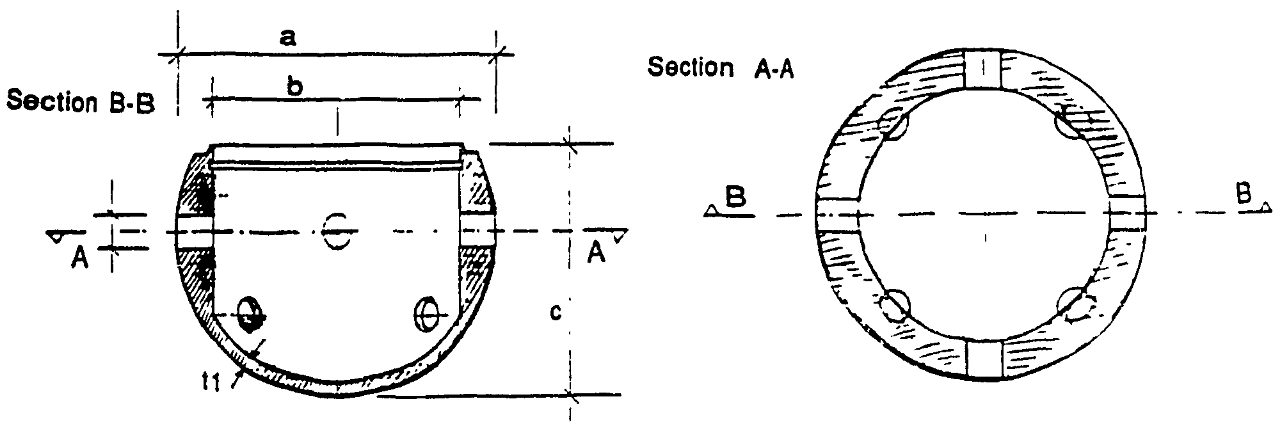


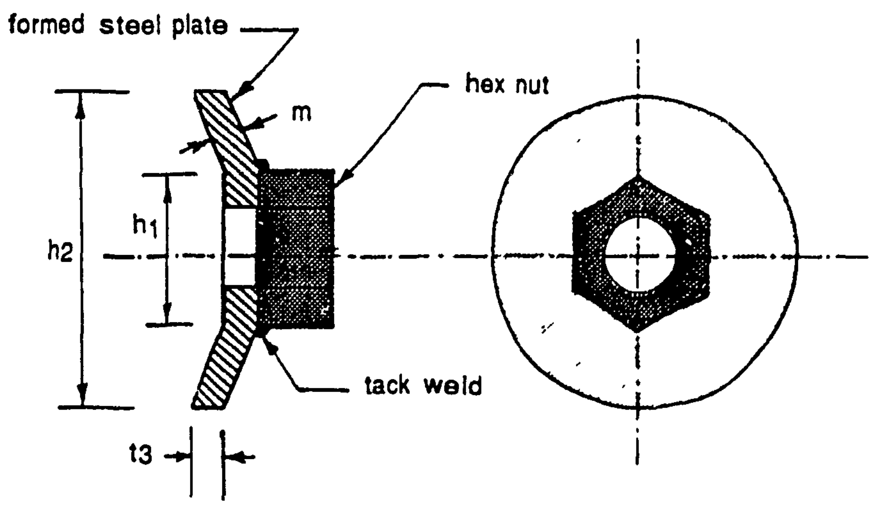
Fig. 5.1, HEMTEC space truss system



a) Member



b) Joint piece



c) Anchor plate

Fig. 5.2, Elements of HEMTEC space truss system

Specification of the truss components which have been tested are as follows:

- Spherical cast aluminum joint connectors, capable of receiving 8 bars, made of AA356 .

- Anchor plates of different forms and thicknesses, made of steel A36.

- Tubular members of diameter 38.1 mm (1.5 in) and 50.8 mm (2 in) made of AL 6063-T5 and AL 6061-T6 of different lengths, tested in compression and in tension.

5.2 TEST ON SPHERICAL CAST ALUMINUM JOINT PIECES

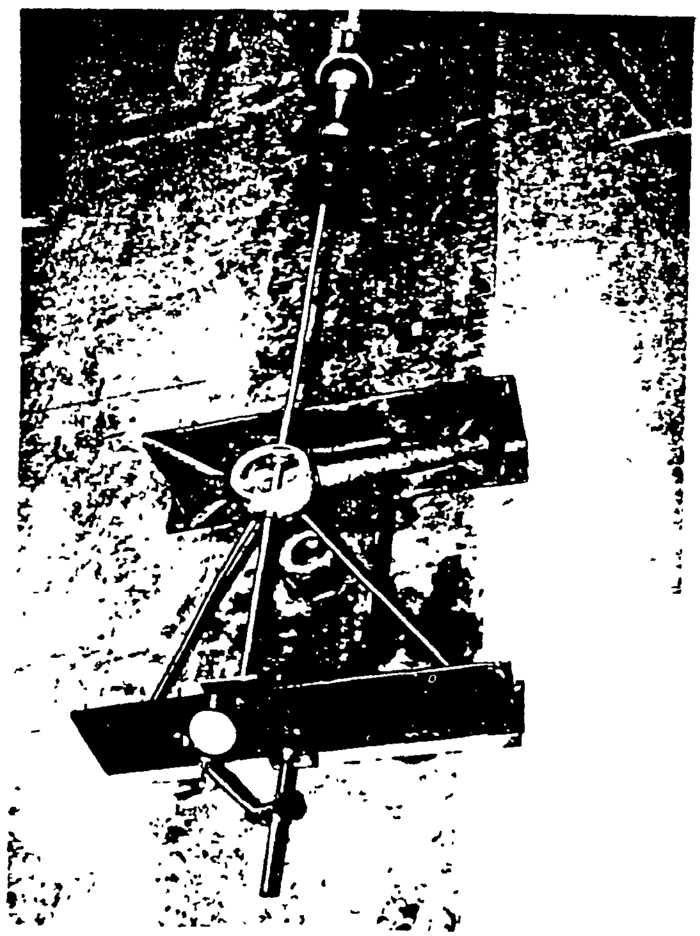


Fig. 5.3, Test set up

The spherical cast aluminum connectors were loaded in four different ways by means of chords and diagonals. Test assemblies and load-deformation graphs corresponding to each type of loading conditions are shown in Fig. 5.3 to 5.11. These tests were conducted not only to establish the strength of the casting, but also to learn if they exhibited any useful nonlinear behaviour.

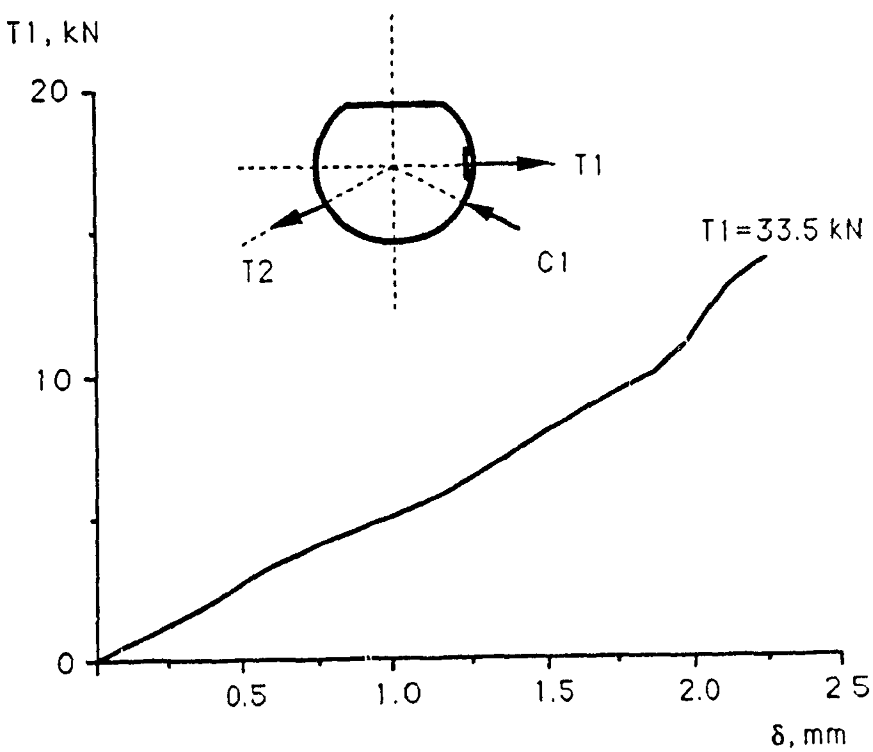


Fig. 5.4, Load-deformation of casting

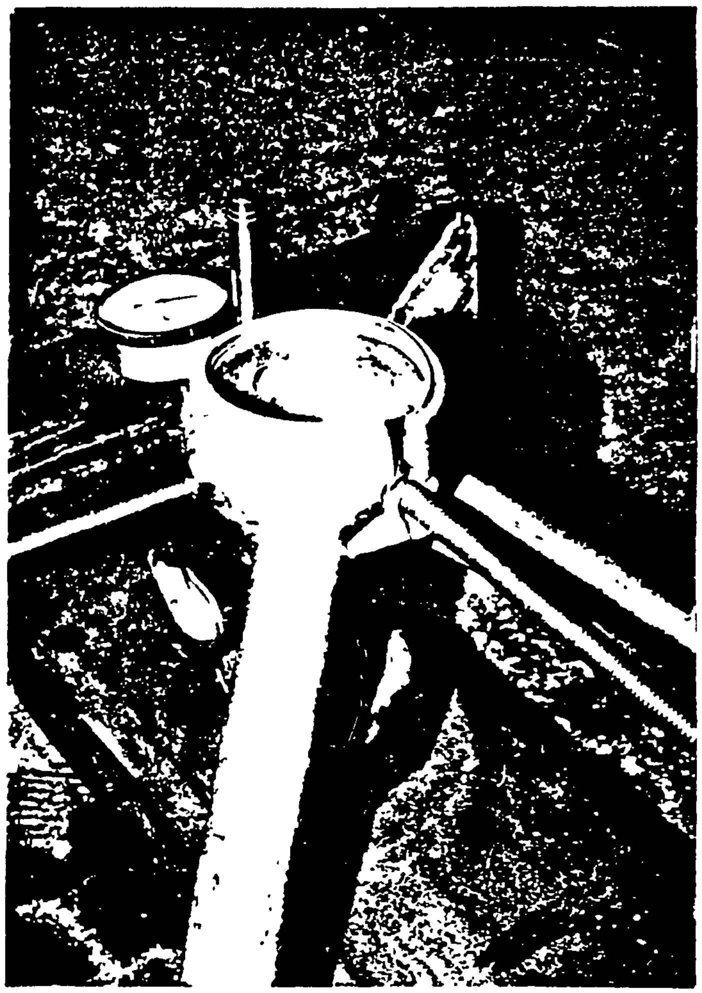


Fig. 5.5, Failure of the casting: tension chord

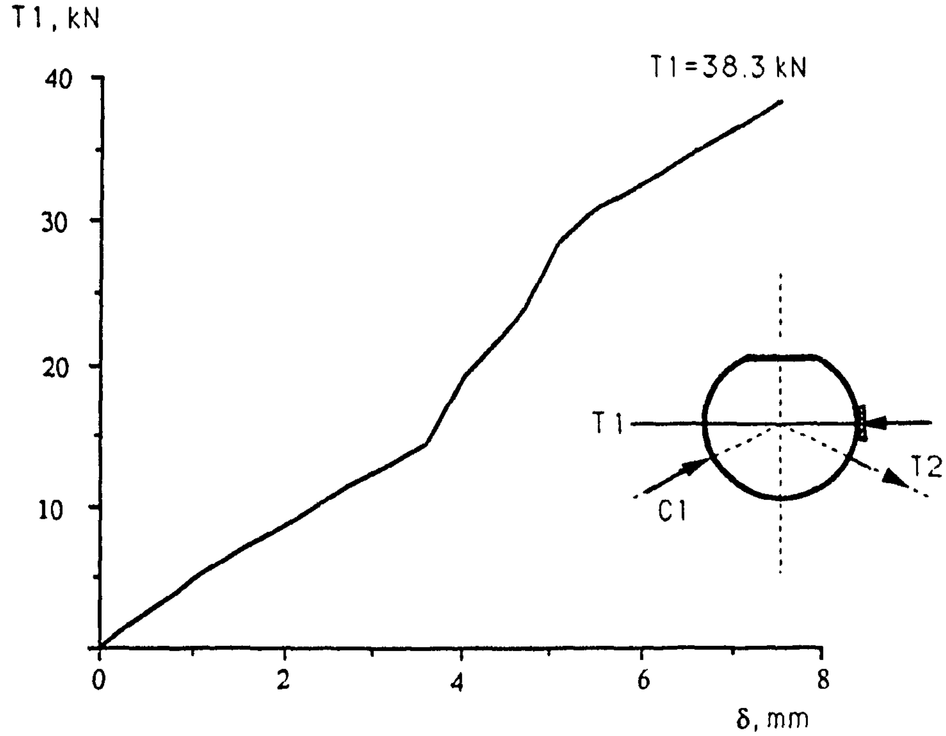


Fig. 5.6, Load-deformation of the casting

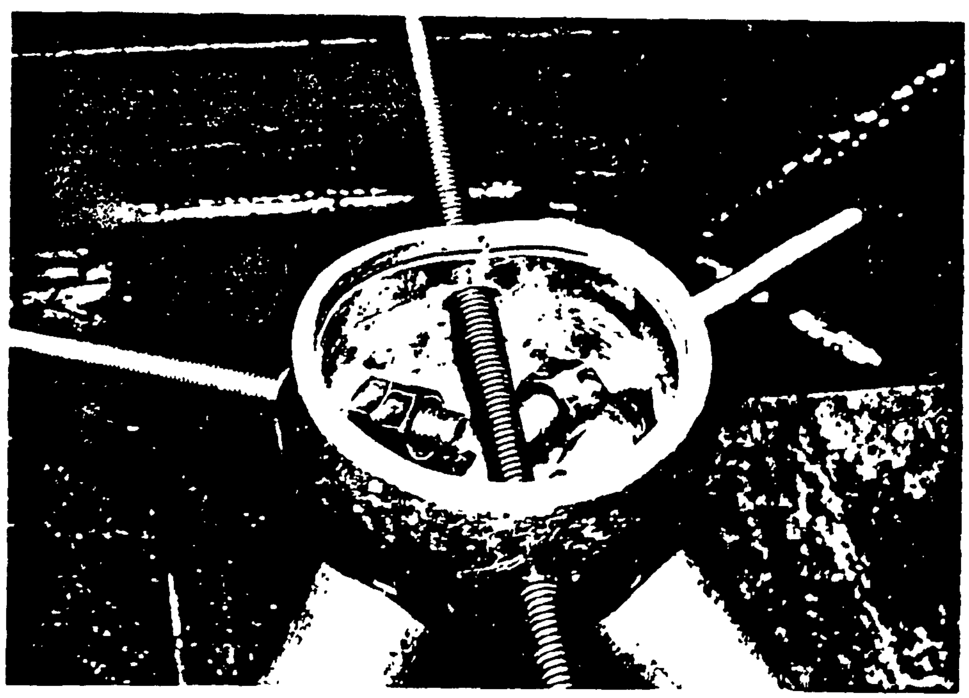


Fig. 5.7, Failure of the casting: tension chord using a through bolt

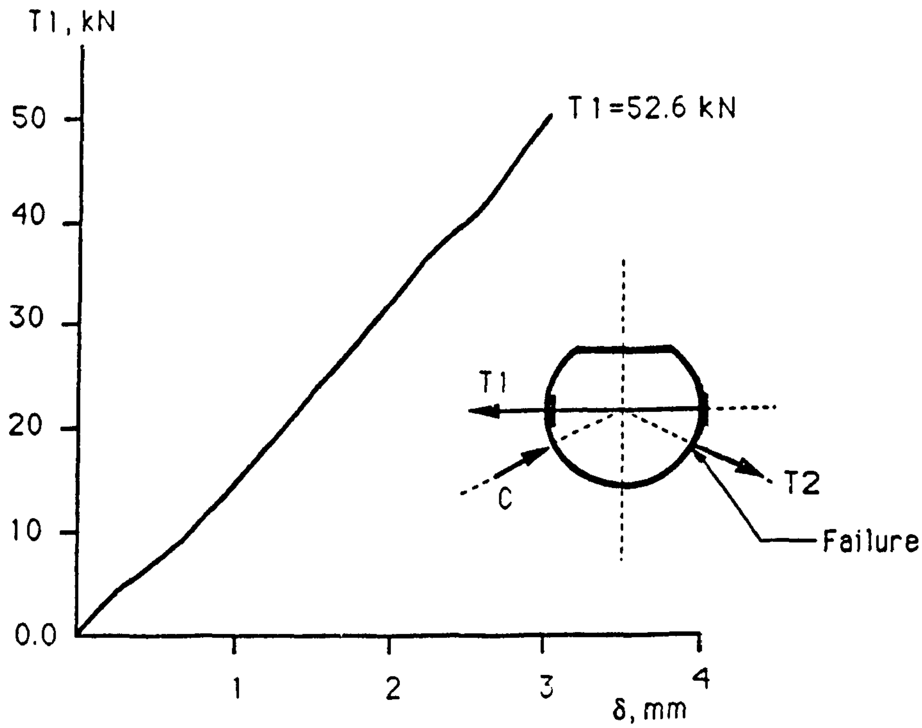


Fig. 5.8, Load-deformation of the casting

Fig. 5.9, Failure of the casting: tension diagonals

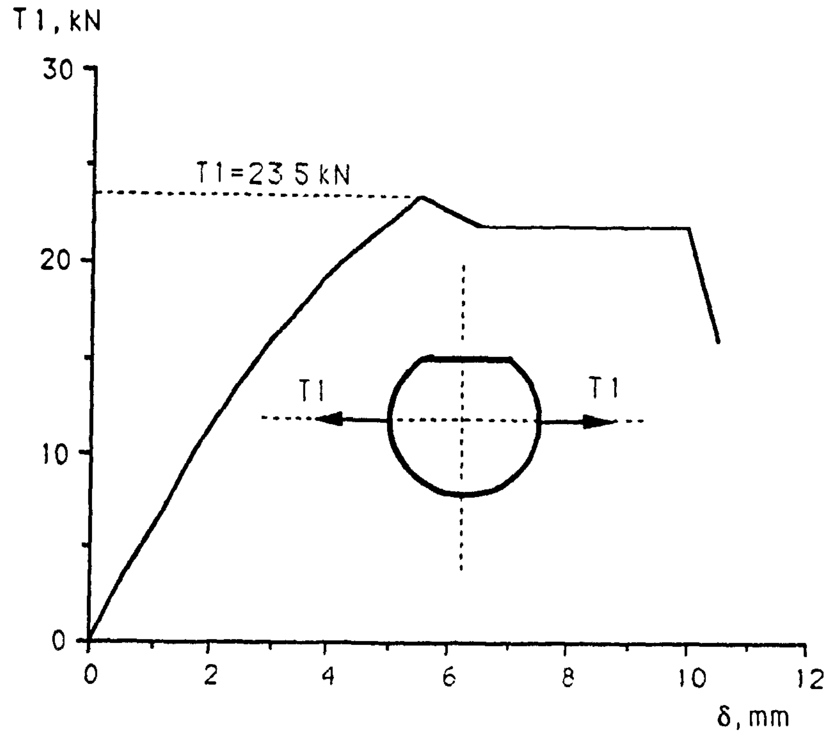


Fig. 5.10, Load-deformation of the casting

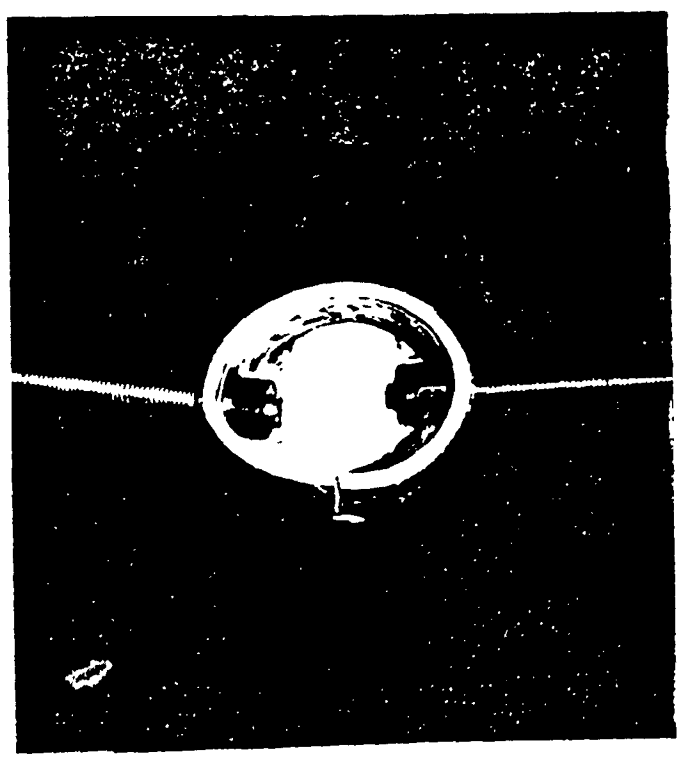


Fig. 5.11, Failure of the casting: direct tension

The load capacity of the spheres is established, and, further, it was observed that their behaviour is in general linearly elastic up to their load capacity, so they are not expected to make a noticeable contribution to the nonlinear behaviour of the assembly.

5.3 TEST ON ANCHOR PLATES

Flat circular anchor plates deform when loaded, and lose the grip on the grooves, unless they are very thick. The dished anchor plates, on the contrary, expand when deformed and fit more tightly in the grooves. Fig. 5.12 shows a test set up and Fig. 5.13 shows a typical load deformation graph.

The load capacity of the anchor plates tested were high enough to avoid any premature failure. Their behaviour, in the range of loading that they will be subject to, is linear elastic and their contribution to the nonlinear behaviour of the system will not be significant.

To provide the joint with a tension capacity close to the capacity of the tube, double anchor plates as shown in Fig. 5.14 were tested. Using this type of anchor plate a capacity 95% of the tension yield capacity of the tube was reached.

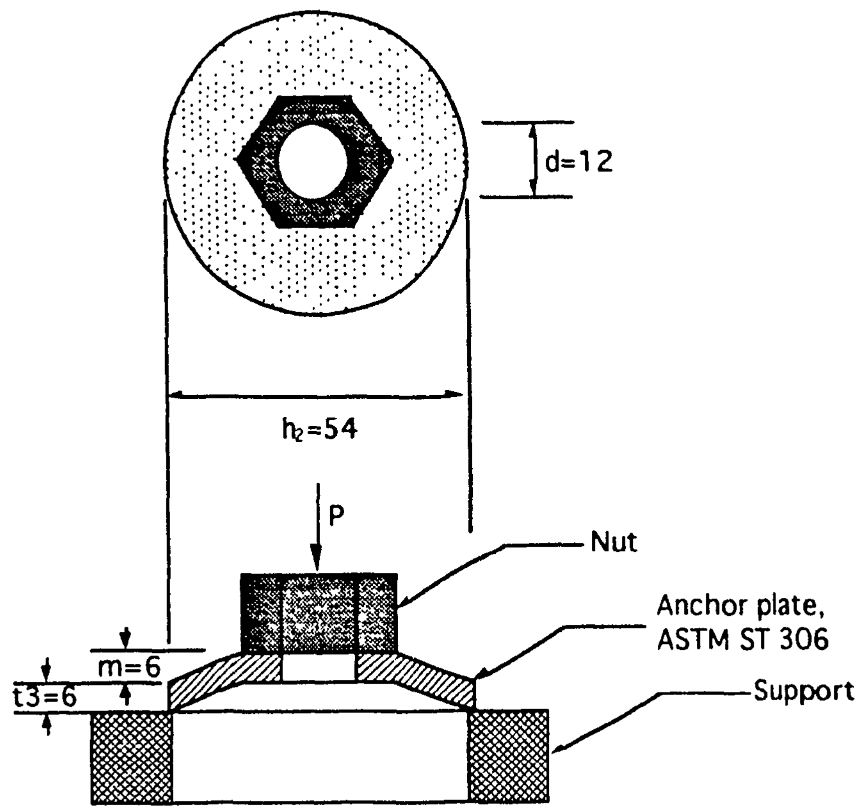


Fig. 5.12, Load bearing test of anchor plate in compression

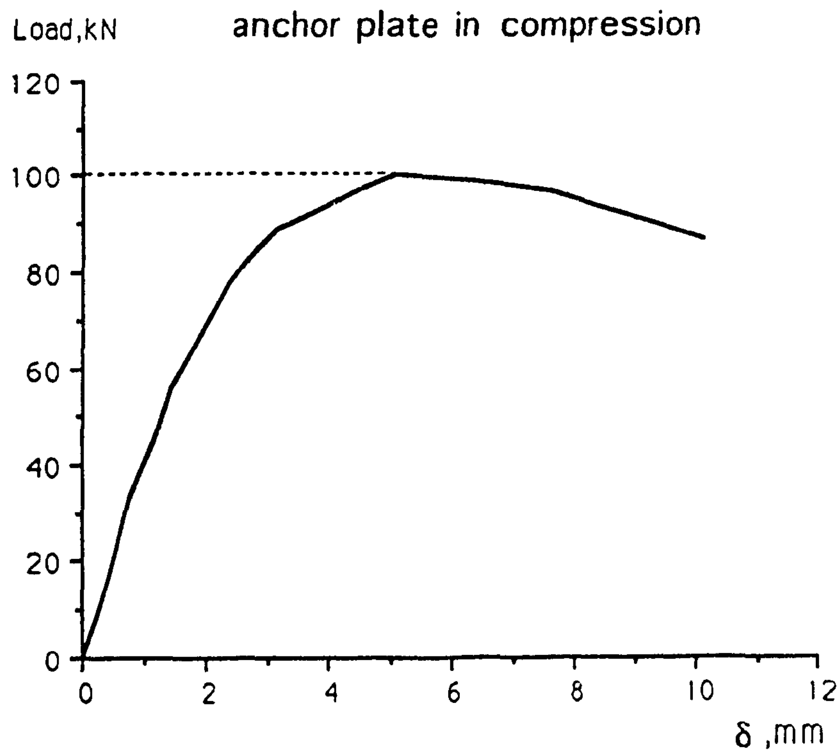


Fig. 5.13, Load-deformation for anchor plate

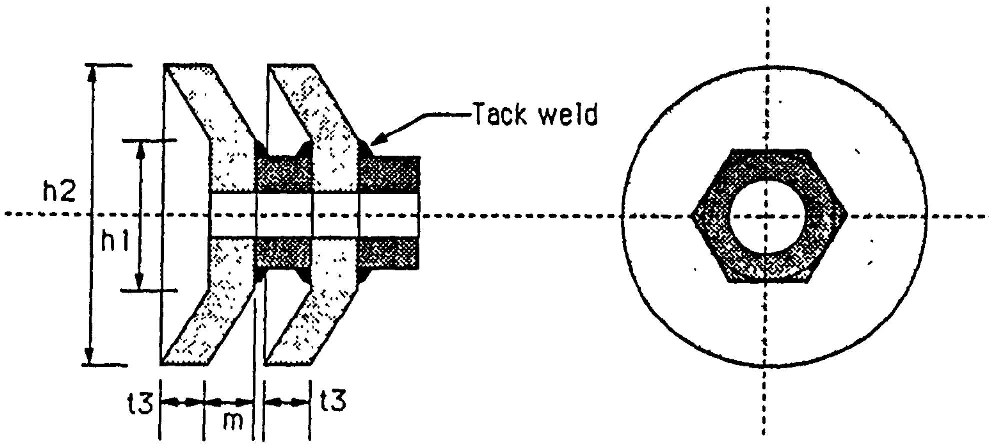


Fig. 5.14, Arrangement of Double Anchor Plate

5.4 CAPACITY OF GROOVED TUBES

The main feature of this system is to provide ductility in both compression and tension members. In order to avoid brittle buckling the compression capacity of the strut must be lower than its axial buckling load. The capacity of the grooved strut depends on the geometry of the grooves. Formulas for evaluation of the load capacity of axially loaded grooved tubes was derived and the theoretical values were compared with the results from tests. The tubes were aluminum alloys 6061-T6, 6063-T5 and 6063-T54. The diameters were 50 mm and 38 mm with 1.5 and 1.6 mm wall thickness.

5.4.1 Geometry of the grooves

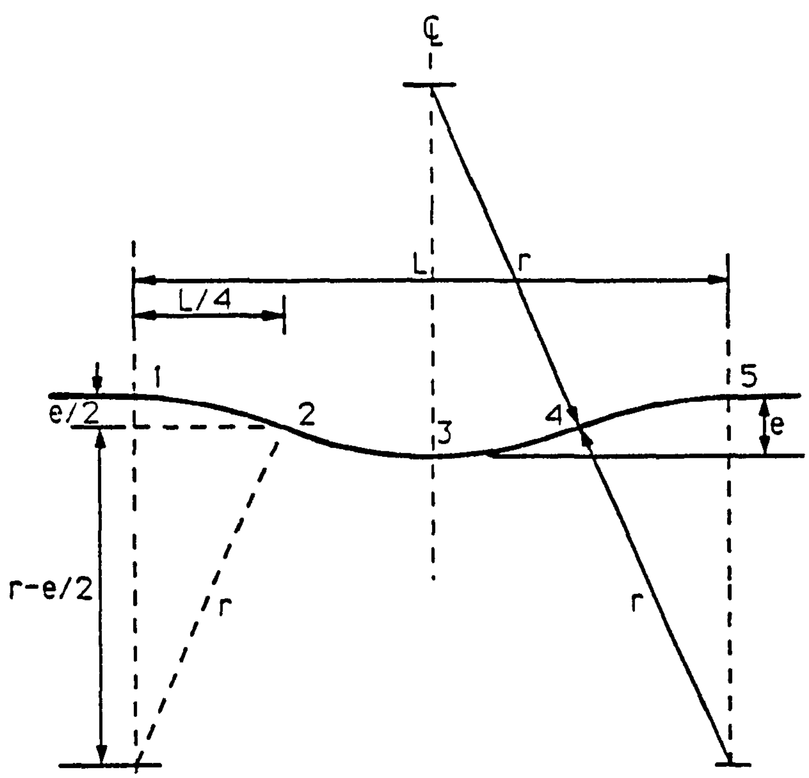


Fig. 5.15, Geometry of the grooves

The dimensions of the groove, (length; L, radius of curvature; r, and depth; e) are controlled as much by the governing yield condition as by the forming tool, and are calculated as follows:

5.4.1.1 Length of the groove; L

Consider an axisymmetric radial line force, w, applied to the tube, Fig. 5.16. The force required to form a groove would cause circumferential plastic hinges, due to longitudinal moment in the tube wall, which will occur where the moment is a maximum. This requires that the shear at the plastic hinge is zero thus the only forces acting on the strip are the radial resistance, q, of the tube walls, and the plastic moment m_0, [45]. The values are:

$$q=\sigma_y t/R$$

$$m_p=\sigma_y t^2/4$$

Equilibrium of a longitudinal strip AB of unit width, for moments about A, gives:

$$2m_p=qL^2/8$$

from which

$$L=4(m_p/q)^{1/2}$$

On replacing for m_0 and q gives:

$$L=4[(\sigma_y t^2/4)/(\sigma_y t/R)]^{1/2}$$

$$L = 2(Rt)^{1/2} \tag{1}$$

The ring load per unit length of perimeter:

$w = qL = 4(m_p q)^{1/2}$

or: $w = 4(m_p p_0/R)^{1/2}$ (2)

where $p_0 = \sigma_y t$

For the tubes used in the experiment, R=25.4 mm, t=1.6 mm giving L=12.7 mm and w=232 N/mm.

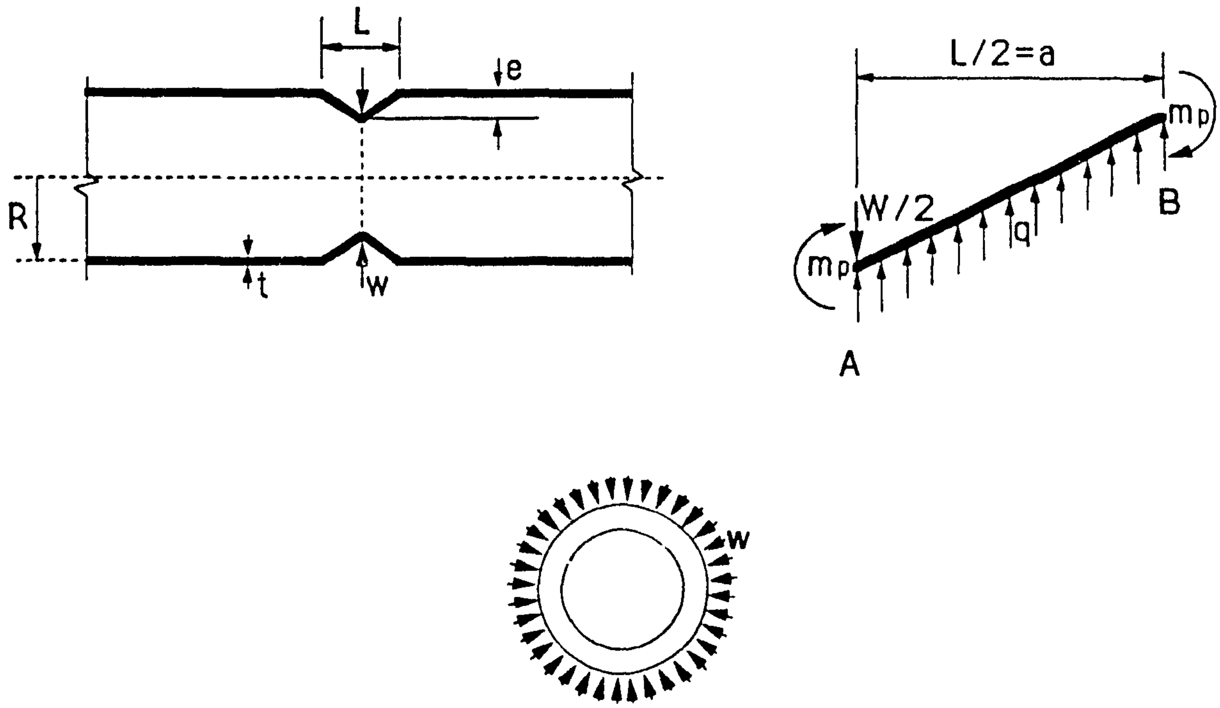


Fig. 5-16 Cylindrical shell under ring loading

5.4.1.2 Minimum radius of curvature of the groove; r

To avoid cracking the aluminum tube during the forming process, the strain, ε, due to forming of the groove should be less than 10%. This condition determines the minimum radius of curvature, r:

$$\varepsilon = t/2r = 0.10$$

$$r = 5t$$

for $t = 1.6$, $r = 8$ mm

5.4.1.3 Depth of the groove, e:

From Fig. 5-15: $r^2 = (L/4)^2 + (r-e/2)^2$

$$e = 2r - [(2r)^2 - (L/2)^2]^{1/2}$$

Using $L = 2(Rt)^{1/2}$, from (1) article 5.4.1.1, and $r = 5t$, from article 5.4.1.2, in above equation, that is for minimum value of L and minimum value of r, for $t = 1.6$ mm, and $R = 25.4$ gives:

$e = 1.3$ mm

<u>5.4.2 Collapse Mechanism</u>

5.2.2.1 Compression

For axially symmetric compression loading, a collapse mechanism as shown in Fig. 5-17 is assumed and plastic analysis is used to compute the axial load-carrying capacity of the grooves.

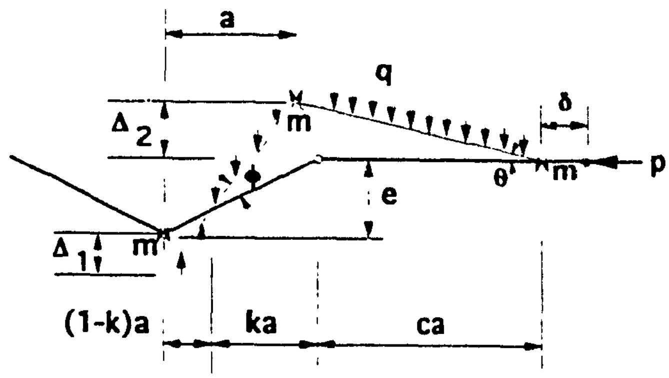


Fig. 5-17, Collapse mechanism

Axial force per unit length of perimeter; $p=\sigma_a t$ (1)

where σ_a is axial stress.

and $p_0=\sigma_y t$ (2)

When the hoop stress is equal to yield stress, then the radial resistance is:

$q=\sigma_y t/R$ (3)

Plastic moment of the hinge is:

$m_0=\sigma_y t^2/4$ (4)

Failure under combined action of axial force and bending moment occurs when $(m/m_0)+(p/p_0)^2=1$

Replacing from (1), (2) and (4) gives:

$m=(\sigma_y t^2/4)[1-(\sigma_a/\sigma_y)^2]=m_0\alpha$, where $\alpha=1-(\sigma_a/\sigma_y)^2$ (5)

Assuming a small rotation of ϕ for the yield mechanism, the longitudinal displacement is:

$\delta=e\phi$

and the radial deformations are:

$\Delta_1=(1-k)a\phi$, $\Delta_2=ka\phi ca\theta$

From the geometry of the groove:

$\theta/\phi=k/c$ (6)

The work done by external forces is $W=p\delta=pe\phi$

Strain energy, $U=2m(\phi+\theta)+(qa^2/2)[(1-k)^2\phi+k^2\phi+c^2\theta]$

Setting $W=U$

$$pe=2m(1+\theta/\phi)+(qa^2/2)[1-2k+2k^2+c^2\theta/\phi]$$

Using equation (6) and rearranging:

$$pe=2m+(qa^2/2)(1-2k+2k^2)+(2mk/c+qa^2ck/2) \quad (7)$$

Differentiating U with respect to c and setting it to zero:

$$dU/dc=-2mk/c^2+qa^2k=0$$

$$c=(1/a)(2m/q)^{1/2} \quad (8)$$

and equation (7) can be written as

$$pe=2m+qa^2(0.5-k+k^2)+2ak(mq)^{1/2} \quad (9)$$

Differentiating U with respect to k and setting it to zero:

$dU/dk=qa^2(-1+2k)+2a(mq)^{1/2}=0$

$$k=[1-(2/a)(m/q)^{1/2}]/2 \quad (10)$$

or, $k=(1-c)/2$

Using (9) and (10) in (8) :

$\sigma_a te=2m+(qa^2/2)[1-1+(2/a)(m/q)^{1/2}]+(2/4)[1-(2/a)(m/q)^{1/2}]^2+2(mq)^{1/2}(a)[1-(2/a)(m/q)^{1/2}]/2$

Which simplifies to:

$\sigma_a te=(m^{1/2}+aq^{1/2}/2)^2$ (11)

This can be expressed as:

$[1-(\sigma_a/\sigma_y)^2]^{1/2}-2(\sigma_a/\sigma_y)^{1/2}(e/t)^{1/2}+a/(Rt)^{1/2}=0$ (12)

This formula gives the ratio σ_a/σ_y in terms of R, t, a and e.

For example: $\sigma_y=290$, $t=1.6$, $R=25.4$, $a=6.35$, $e=2.5$

σ_a/σ_y=0.538.

Axial force P= $\pi dt\sigma_a$=40 kN

Note that $c=(1/a)(Rt)^{1/2}\alpha^{1/2}$ =0.85, k=(1-0.85)/2=0.75, and ka=0.5mm

This means that the grooves behave almost as shown in Fig. 5.18.

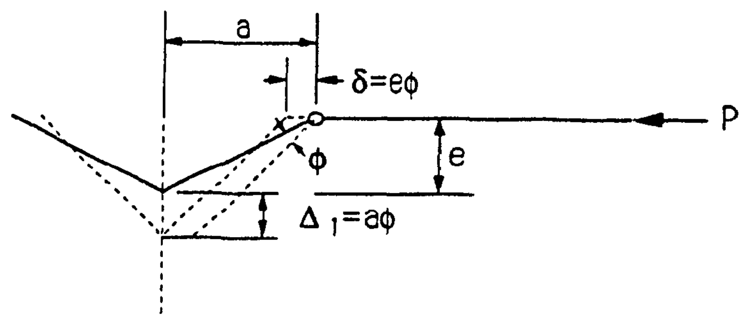


Fig. 5.18, Simplified collapse mechanism

For the simplified collapse mechanism:

$pe\phi=2m\phi+qa^2\phi/2$

$\sigma_a te=(\sigma_y t^2/4)[2\alpha+(q/m_p)a^2/2]$

$$\sigma_a/\sigma_y=(t/2e)(\alpha+a^2/Rt) \qquad (13)$$

and for values of the example:

σ_a/σ_y=0.543

which compares with σ_a/σ_y=0.538 obtained above.

Using the formula (12) the theoretical values of σ_a/σ_y were calculated for short tubes made of three aluminum alloys and two diameter sizes. The results are shown in Table 5.1. The values obtained for σ_a/σ_y are reasonably in agreement with the test results.

Table 5.1, Compression capacity of grooved tubes, test vs theory

Alloy	D (mm)	t (mm)	e (mm)	σ_y MPa	σ_a/σ_y test	σ_a/σ_y theory
6063-T54	50.6	1.6	2.27	236	0.48	0.55
6063-T54	50.6	1.6	2.00	236	0.52	0.60
6063-T54	38.0	1.6	2.33	236	0.50	0.54
6063-T54	38.0	1.6	2.29	236	0.41	0.54
6063-T54	38.0	1.6	1.53	236	0.75	0.71
6063-T54	38.0	1.6	1.06	236	0.74	0.84
6063-T54	38.0	1.6	0.70	236	0.84	0.94
6351-T6	50.8	1.6	0.55	297	0.84	0.98
6063-T5	38.0	1.5	2.50	175	0.54	0.51
6063-T5	38.0	1.5	2.50	175	0.56	0.51
6063-T5	38.0	1.5	2.50	175	0.55	0.51
6063-T5	38.0	1.5	2.40	175	0.59	0.53
6063-T5	38.0	1.5	2.40	175	0.59	0.53
6063-T5	38.0	1.5	2.40	175	0.59	0.53
6063-T5	38.0	1.5	2.30	175	0.57	0.55
6063-T5	38.0	1.5	2.30	175	0.59	0.55
6063-T5	38.0	1.5	1.80	175	0.60	0.64
6063-T5	38.0	1.5	1.80	175	0.58	0.64
6063-T5	38.0	1.5	1.50	175	0.72	0.71
6063-T5	38.0	1.5	1.50	175	0.72	0.71
6063-T5	38.0	1.5	1.50	175	0.75	0.71
6063-T5	38.0	1.5	1.30	175	0.73	0.77

5.4.2.2 Tension

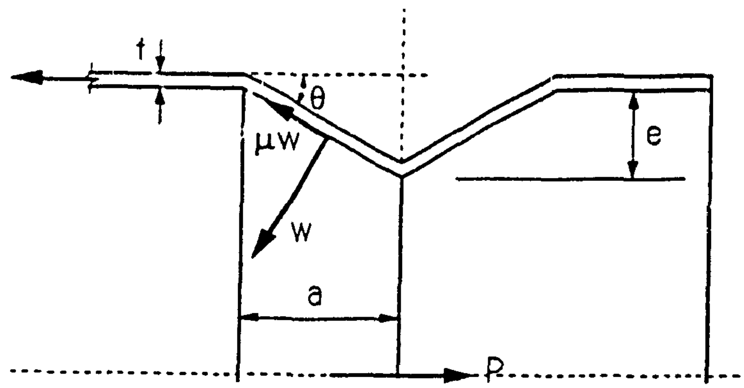


Fig. 5.19, Tension capacity of grooves

For a smooth cylinder using (2) from article 5.4.1.1:

$w=4(m_0p_0/R)^{1/2}+ap_0/R$

Replacing, $m_0=\sigma_yt^2/4$, $P_0=\sigma_yt$, gives:

$w=2\sigma_yt(t/R)^{1/2}+a\sigma_y(t/R)$

This force may be assumed to apply for the grooved condition.

With friction: μw

$P=2\pi R(w\sin\theta+\mu w\cos\theta)$

$\sigma_a 2\pi Rt=2\pi R\sigma_y t[2(t/R)^{1/2}+a/R](\sin\theta+\mu\cos\theta)$

$\sigma_a/\sigma_y=[2(t/R)^{1/2}+a/R](\sin\theta+\mu\cos\theta)$ (1)

For $e=2.5$, $a=6.35$, $R=25.4$, and $\mu=0.2$

$\sin\theta+\mu\cos\theta=0.56$

$\sigma_a/\sigma_y=[2(1.6/25.4)^{1/2}+6.35/25.4](0.56)=0.42$

This compares with $\sigma_a/\sigma_y=0.54$ obtained for tubes in compression.

5.5 TEST ON COMPRESSION MEMBERS

The grooves which hold the threaded anchor plates in position are made by pressing or turning. For pressing, a tool made as shown in Fig. 5.20, is positioned on the tube and then is squeezed by a vice. For turning, the blade of a pipe cutter was replaced by a roller, Fig. 5.21. The tube is fixed and the pipe cutter is rotated to make the grooves. Pressing is easier anc faster but turning provides uniform grooves with smoother appearance.

By varying the length, curvature and depth of the grooves the behaviour can be controlled, and the groove made to close before the strut buckles. The first set of tests was on short grooved tubes in order to study the effect of the geometry of the grooves on their behaviour, without the interference of overall buckling. The second set of tests was carried out on full size struts, with grooves proportioned to fail shortly before buckling of the member.

Three kinds of aluminum alloy, and two diameter sizes, 50.8 and 38.1 mm. were tested. A typical failure of a tube with no groove is shown in Fig. 5.22.

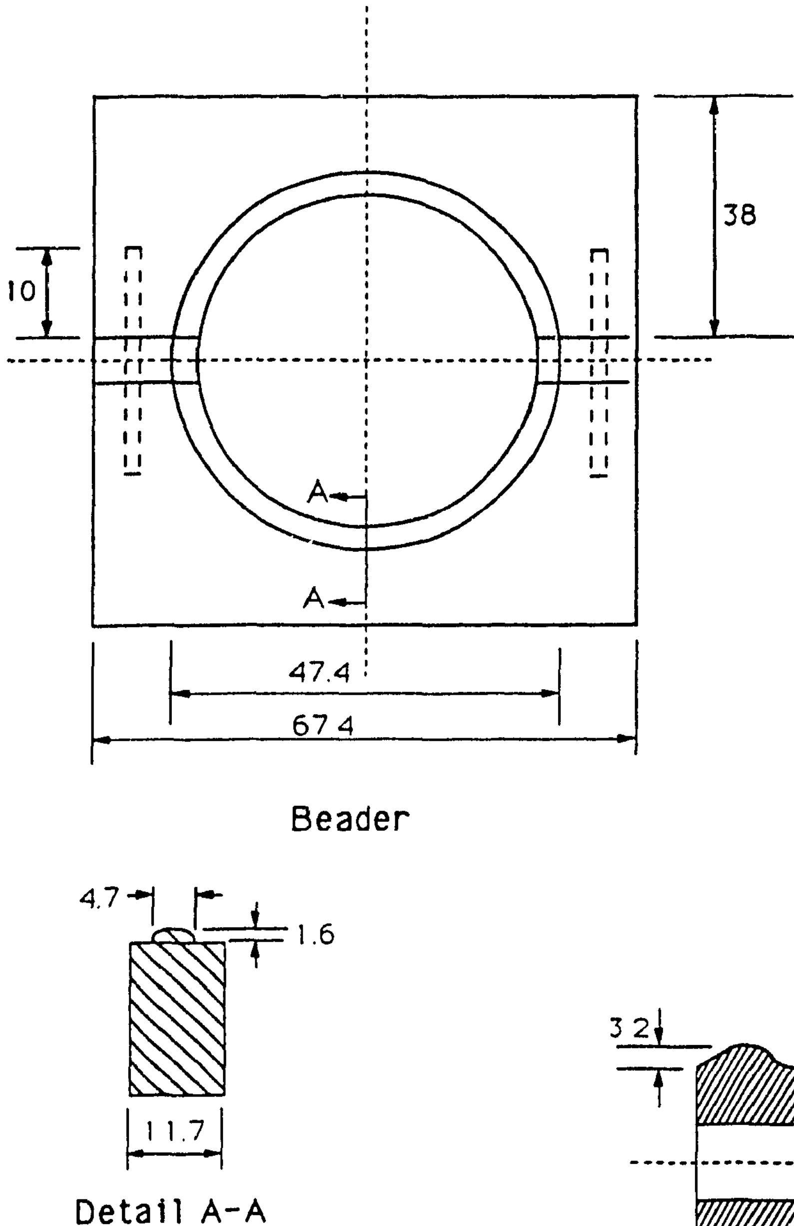


Fig. 5-20, Beading tool

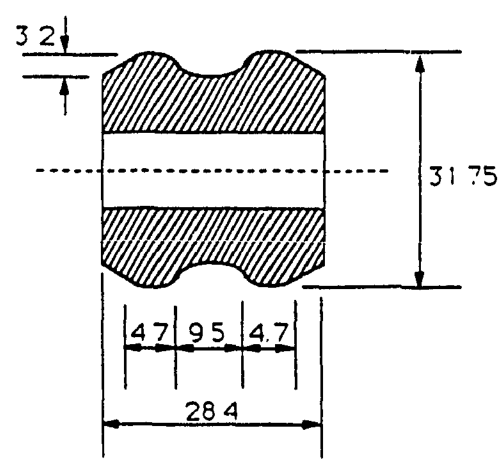
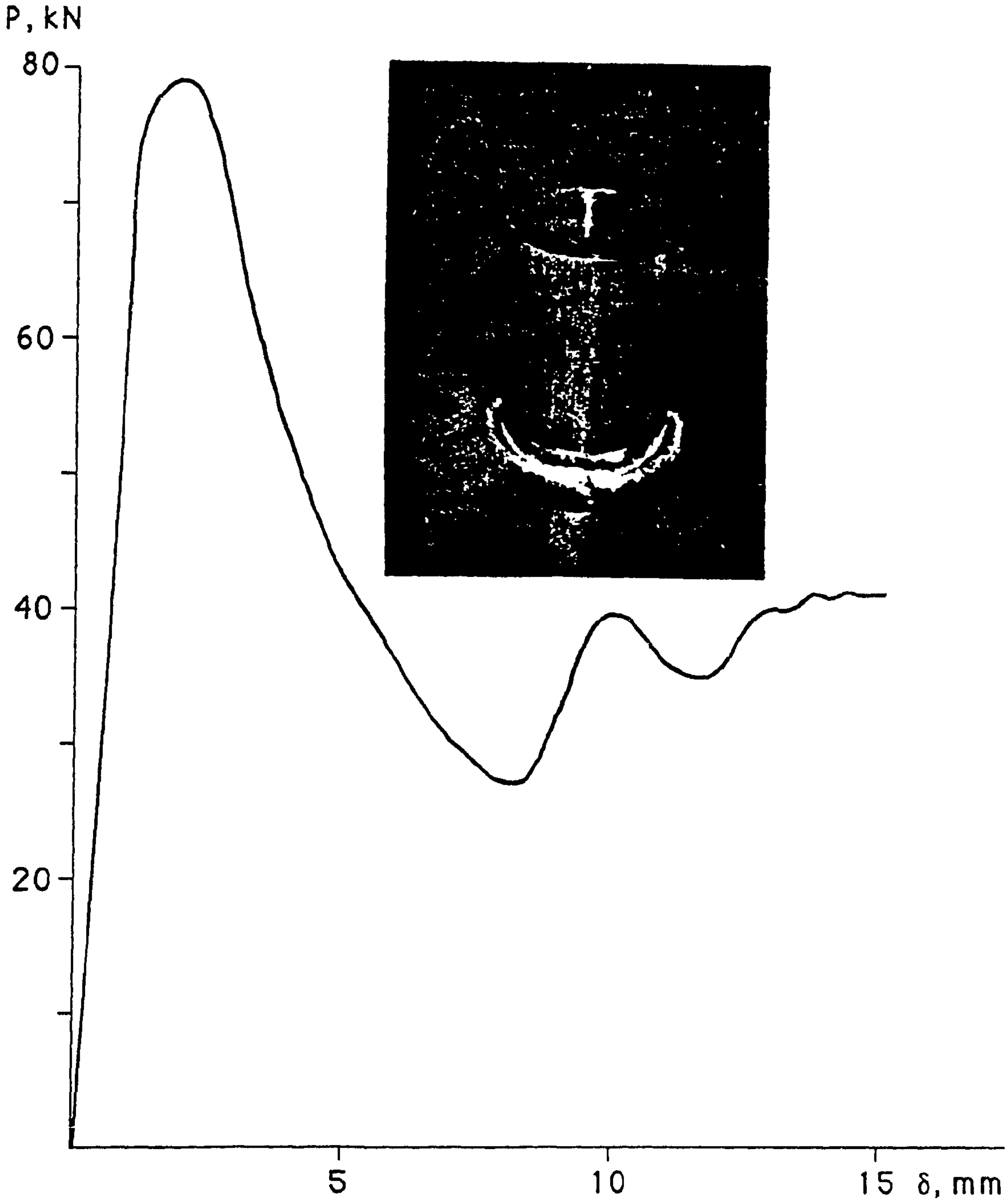


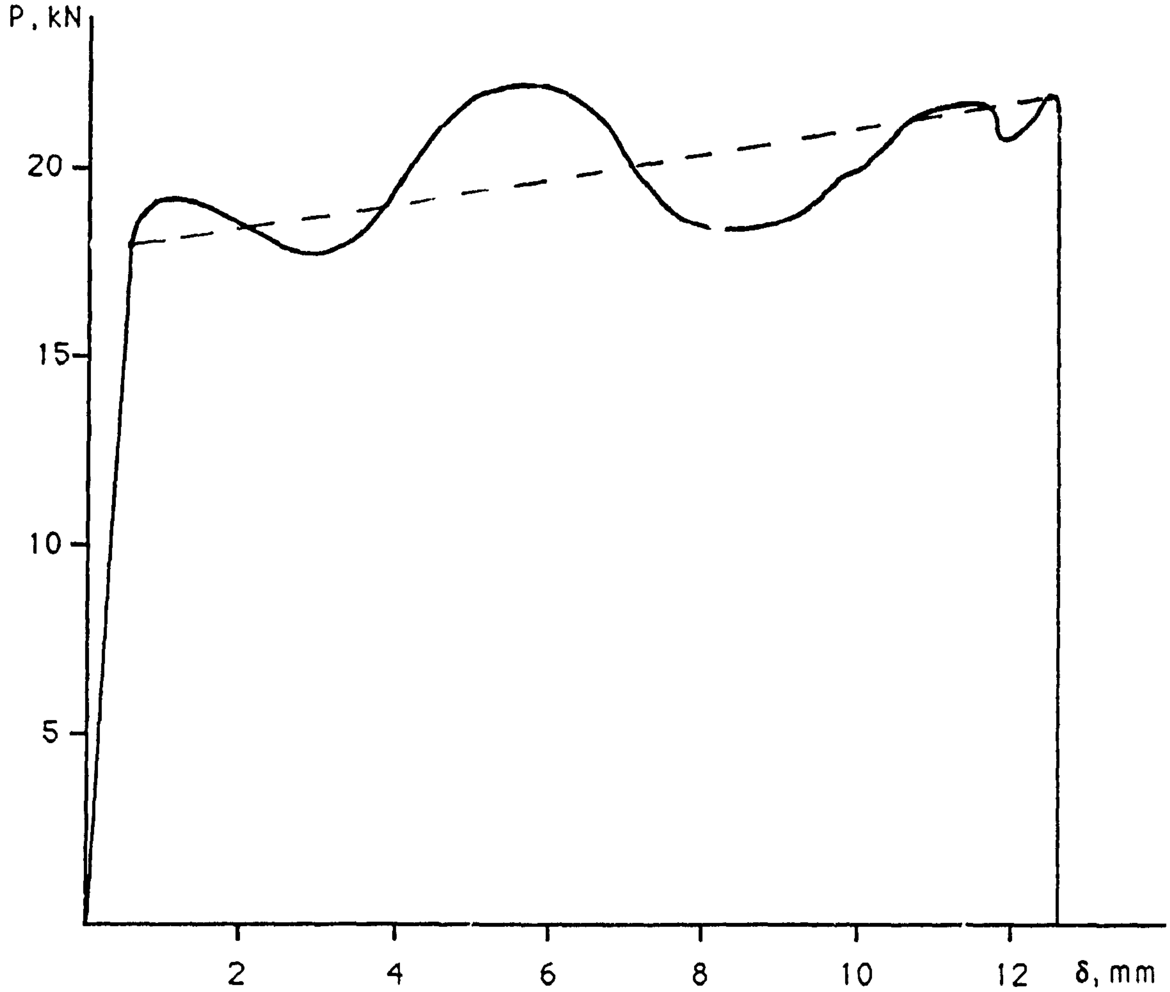
Fig. 5-21, Beading roller



Alloy	D	t	L	Pu(kN)
6061-T6	50.8	1.6	82.5	78.4

Fig. 5-22, Compression test on plain short tube

Under axial compression force, segments of the grooves close one after another, thus providing a quasi elasto-plastic plastic behaviour sh wn in Fig. 5.23. When the grooves yield, they take shapes that are schematically shown in Fig. 5.24.



Alloy	D	t	L	N	e	Pu(kN)
6063-T5	38.1	1.6	50.5	2	1.0	22.5

N= number of grooves, e= groove depth.

Fig. 5.23, Compression test on grooved short tube

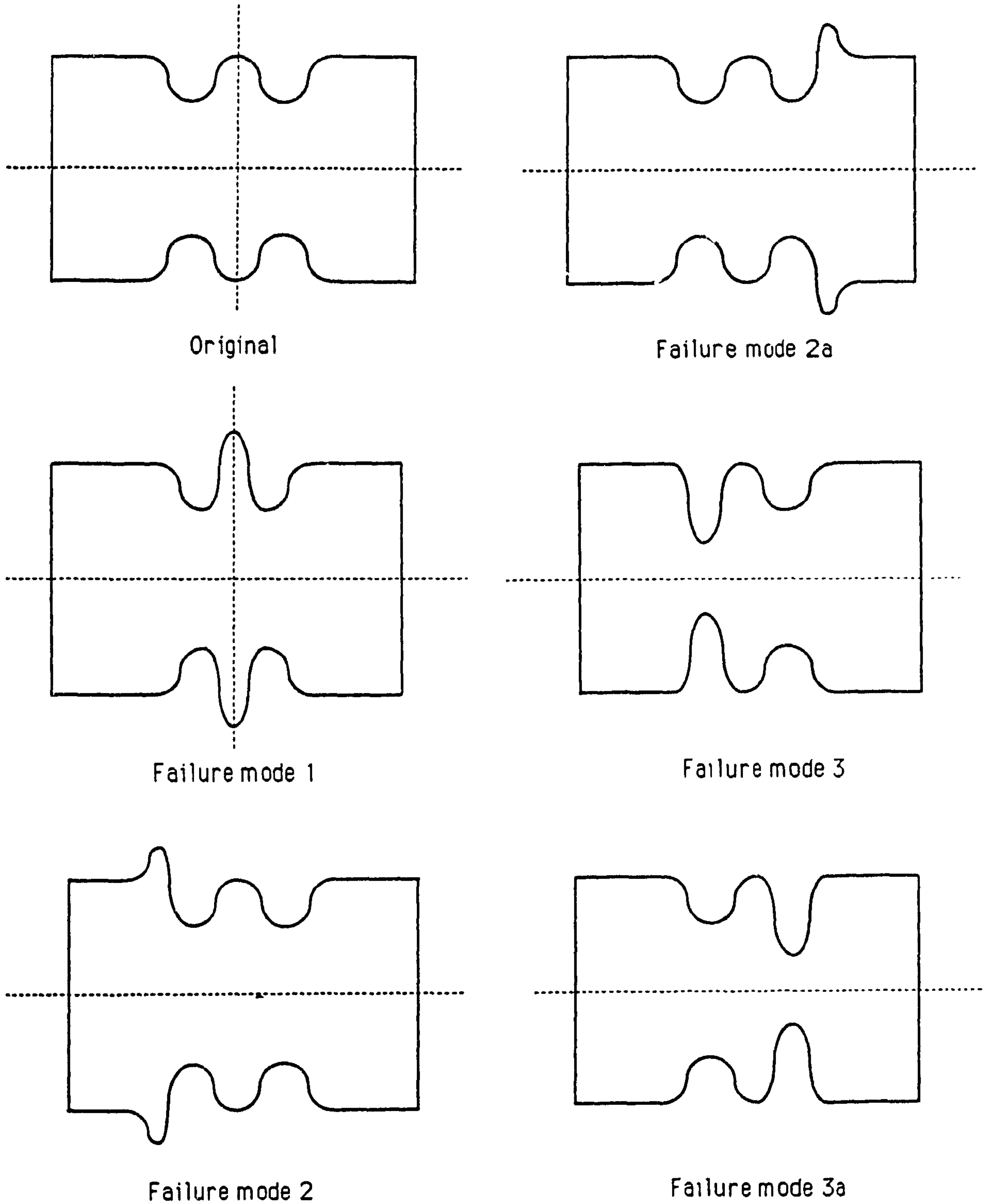


Fig. 5-24, Schematic failure mode of grooves observed in compression tests

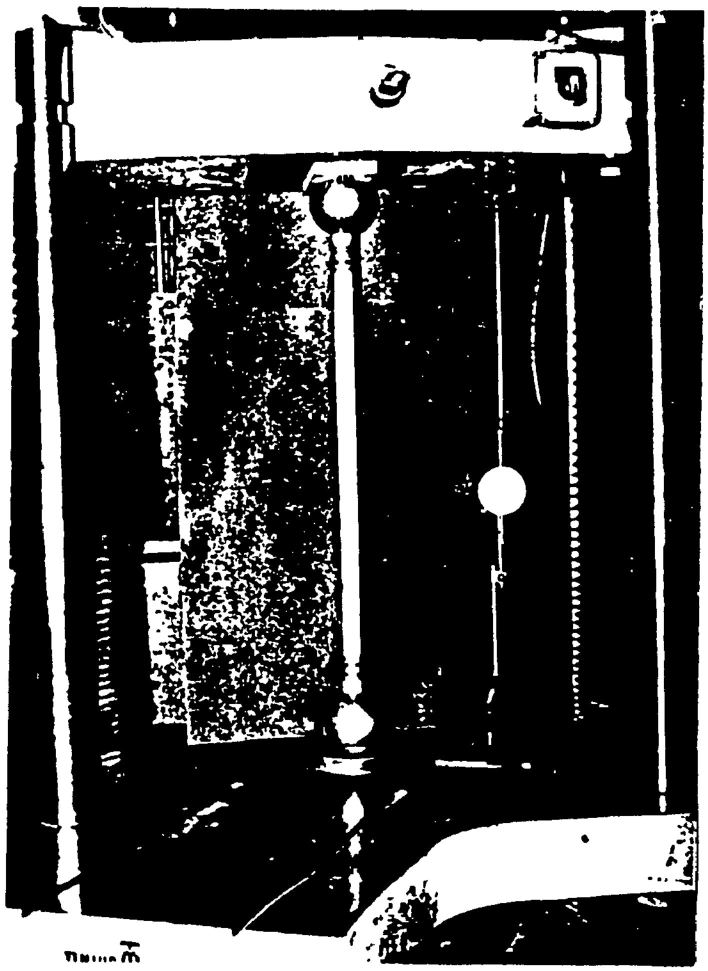
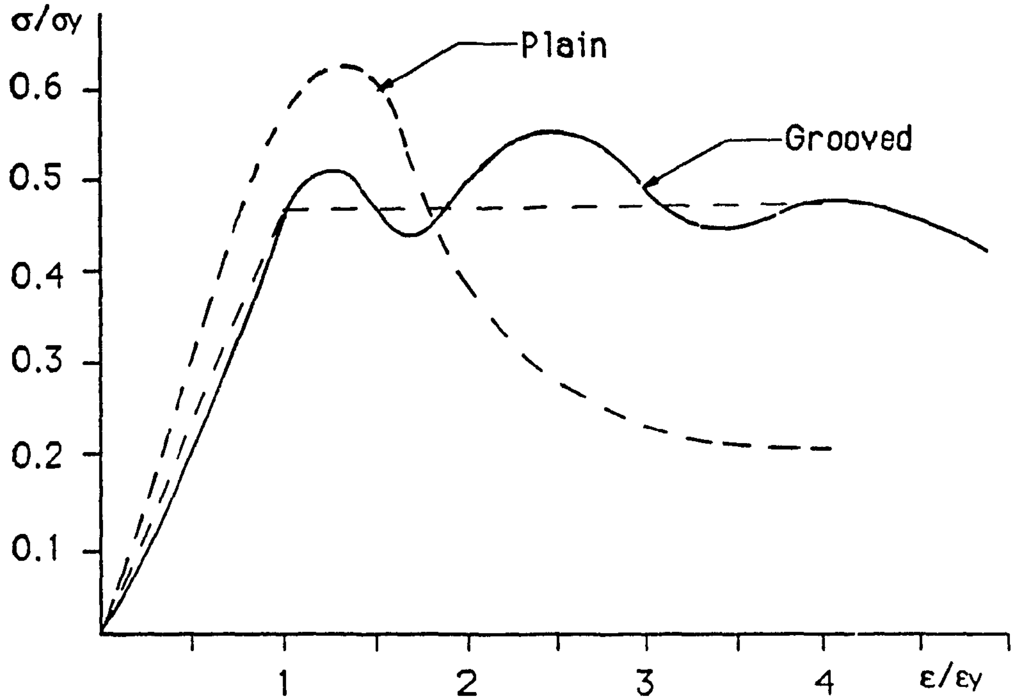


Fig. 5-25, Compression test on full size grooved member



σ= axial stress, σy= yield stress
ε = axial strain, εy= yield strain

Alloy	D	t	L	N	e	Pu(kN)
6063-T5	38.1	1.6	700	4	1.5	15.5

N= number of grooves, e= groove depth.

Fig. 5-26, Compression test on full size members, plain and grooved

5.6 TEST ON TENSION MEMBERS

For tension members the grooves serve two purposes: holding the anchor plate in place, and providing nonlinear behaviour. Grooves achieve both objectives, but as the members are usually mass produced and tension members are made the same way as the compression members, it is necessary to find a depth which satisfies the requirements of both. To this end a number of tests were performed.

The alloy of aluminum and the way the tubes are manufactured are more important in tension members than compression members. The seams, which occur in port hole extrusions, must be completely 'welded' to provide homogeneous material, otherwise they can fail prematurely as illustrated in Fig. 5.28 with the behaviour shown in Fig. 5.29.

With proper selection of the depth of the grooves, material alloy, and careful beading, a behaviour as shown in Fig. 5.30 was achieved. The "saw tooth" curve is attributed to the slip/stick motion of the anchor plate.

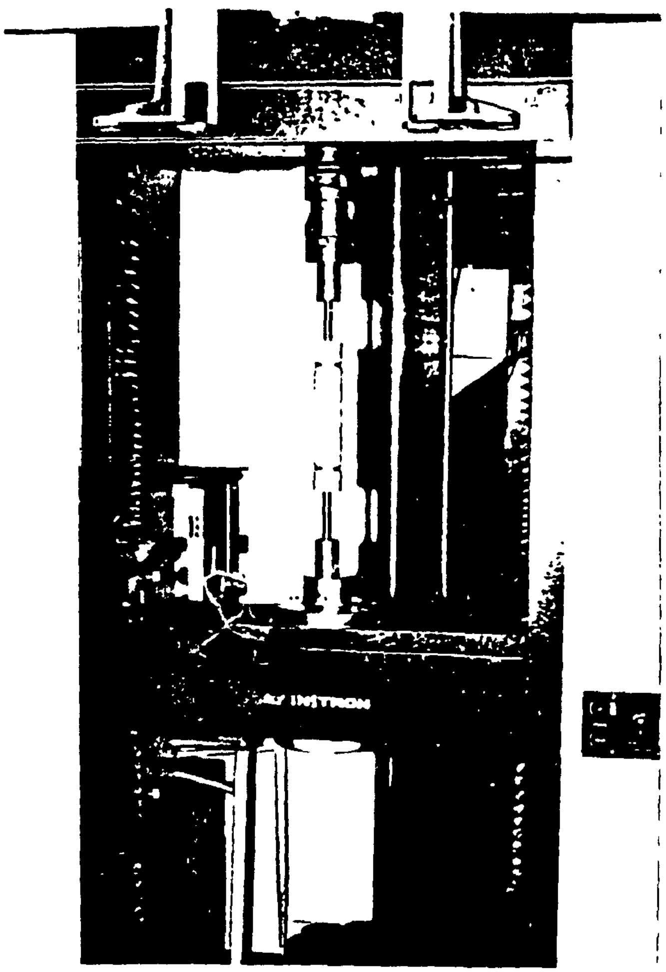


Fig. 5-27, Tension test set up

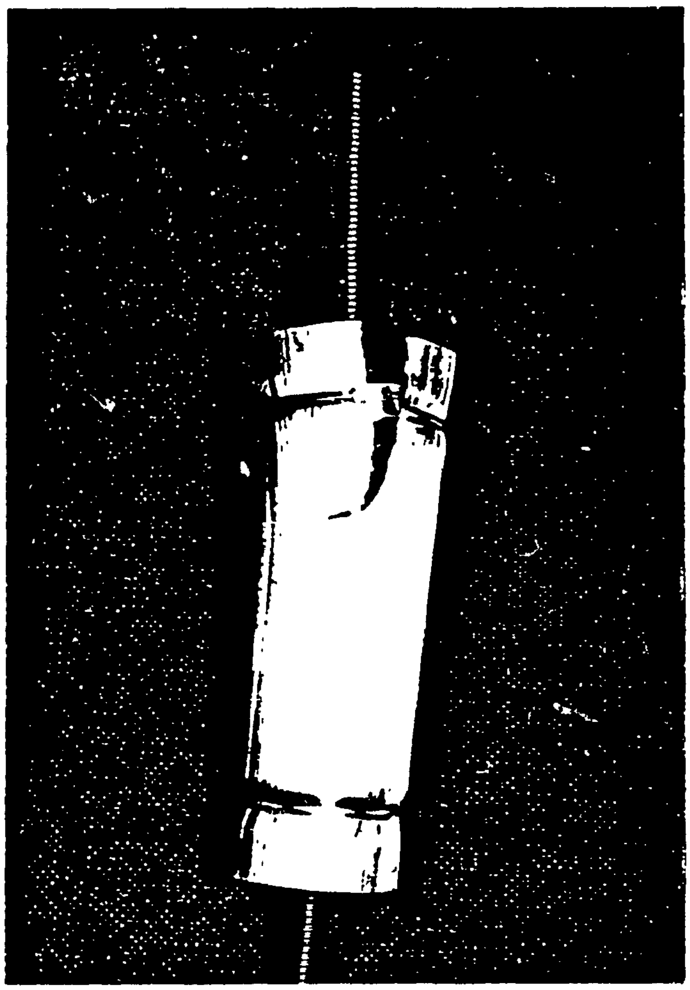
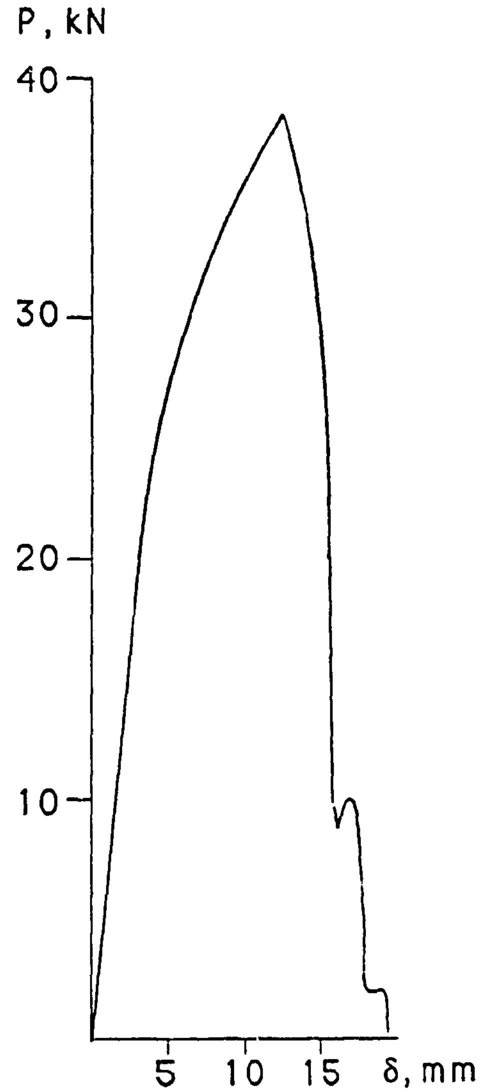


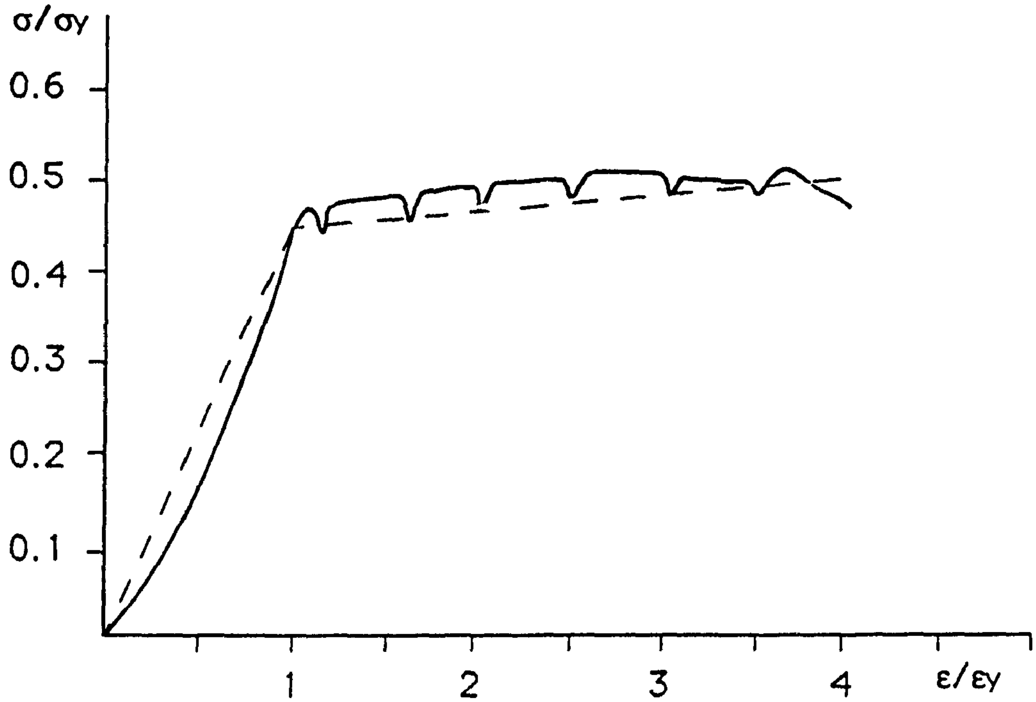
Fig. 5-28, Failure of a short tube with single grooves, and a simple anchor plate, in tension



Alloy	D	t	L	N	e	Pu(kN)
6061-T6	50.8	1.6	305.0	4	2.3	38.5

N= number of grooves, e= groove depth.

Fig. 5-29, Tension test on grooved short tube, which split at the weld



σ= axial stress, σ_y= yield stress
ε = axial strain, ε_y= yield strain

Alloy	D	t	L	N	e	Pu(kN)
6063-T5	38.1	1.6	700	4	1.5	16.5

N= number of grooves, e= groove depth.

Fig. 5-30, Tension test on full size members

5.7 TEST ON FULL SCALE TRUSS

5.7.1 Test arrangement:

A test was conducted on a space truss of the Hemtec type, as shown in Fig. 5.33. The purpose of the test was to check the influence of nonlinear member behaviour on the overall load carrying capacity. The truss was formed of square grids $bxbxb/\sqrt{2}$ deep, which gives uniform member lengths, in this case b was 800 mm and the depth was 566 mm. The truss was supported at four lower corners giving a span of 2400 mm. The supports were retained to prevent sliding in order to avoid tension failure of the bottom chords.

All members were circular tubes of aluminum 6063-T6 with diameter of 38.1 mm, (1.5 in), and thickness of 1.5 mm, (1/16 in). Each tube had two grooves of 1.5 mm depth at each end. The anchor plate was press formed from 5 mm thick mild steel plate. A 12.7 mm (1/2 in) standard steel bolt, a nut and a flat washer were used for the connection. The diameter of the spherical cast aluminum joint pieces was 100 mm.

The grooves of the members were so proportioned that they would yield before the member buckled, hence would prevent "brittle" failure. Based on previous compression tests on the typical struts, the initial yield load was 12.7 kN and the ultimate load was 14.2 kN. For computer analysis the member behaviour was modelled with α=0.9 and β=3. The tension members were assumed to have the same characteristics as the compression members. Theoretically, the truss was capable of carrying a total load of 51.6 kN at initial "yielding" of the central top chord and an ultimate load of 77.2 kN, if the nonlinear behaviour led to a redistribution of load between the three top chords. Based on the measured effective elastic modulus of the members (force/change of length from Fig. 4.26 and 4.30), Table 5.2 gives the theoretical deflection and sequence of "yielding" in the members.

Table 5.2, Theoretical sequence of yielding, and deflection

Cycle	1	2	3	4	5	6
P, kN	51.6	69.2	72.2	73	73.1	77.2
Δ_{18}, mm	7.2	11.0	11.6	12.6	12.8	27.2
Yielded	17-18	11-17	12-18	6-11	6-7	17-18
members	18-19	14-19	13-18	6-12	12-23	18-19
		17-22	18-23	8-13	7-8	
		19-25	18-24	8-14	13-24	
				22-28	28-29	
				23-28	29-30	
				24-30		
				25-30		

5.7.2 Test procedure

The load was applied by means of a central hydraulic jack and a spreader beam to the joints 17,18 and 19 in the ratio of 1:2:1. The applied load was measured by a load cell, the member forces were monitored by strain gauges and the vertical movements of the centres of members 23-24 and 12-13 were measured by dial gauges.

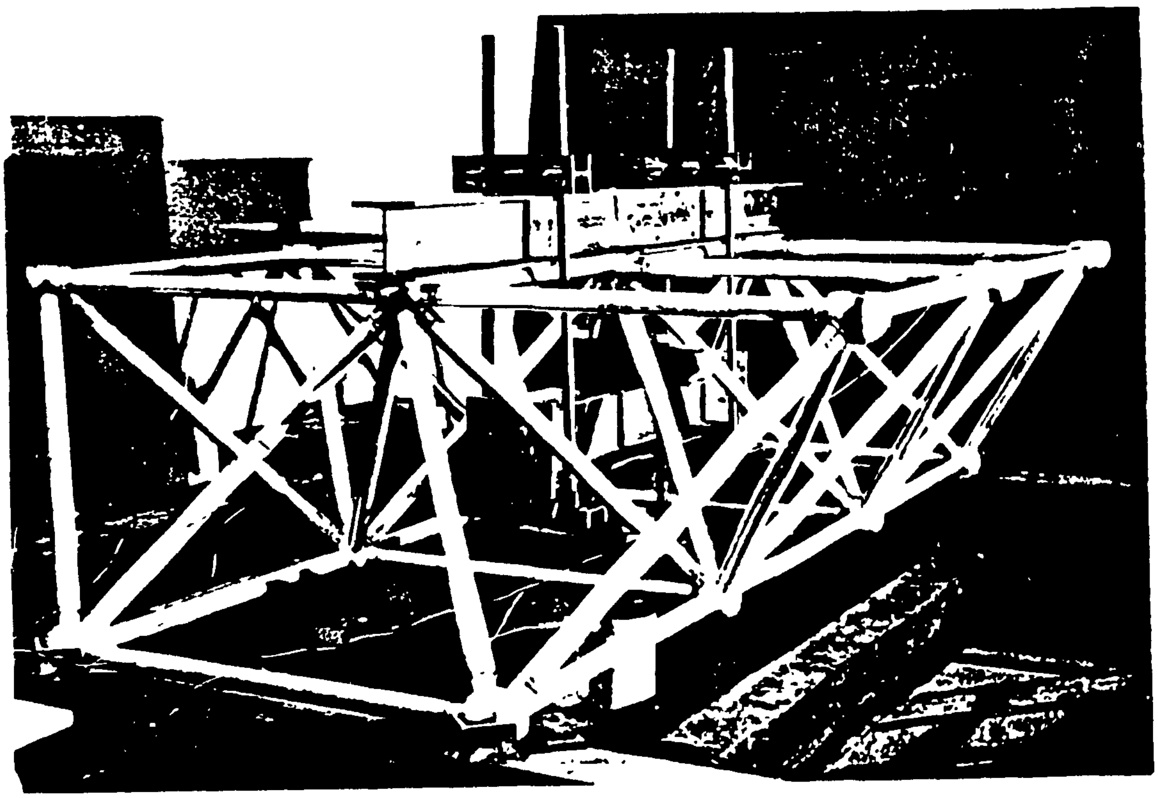


Fig. 5.31, Test set up

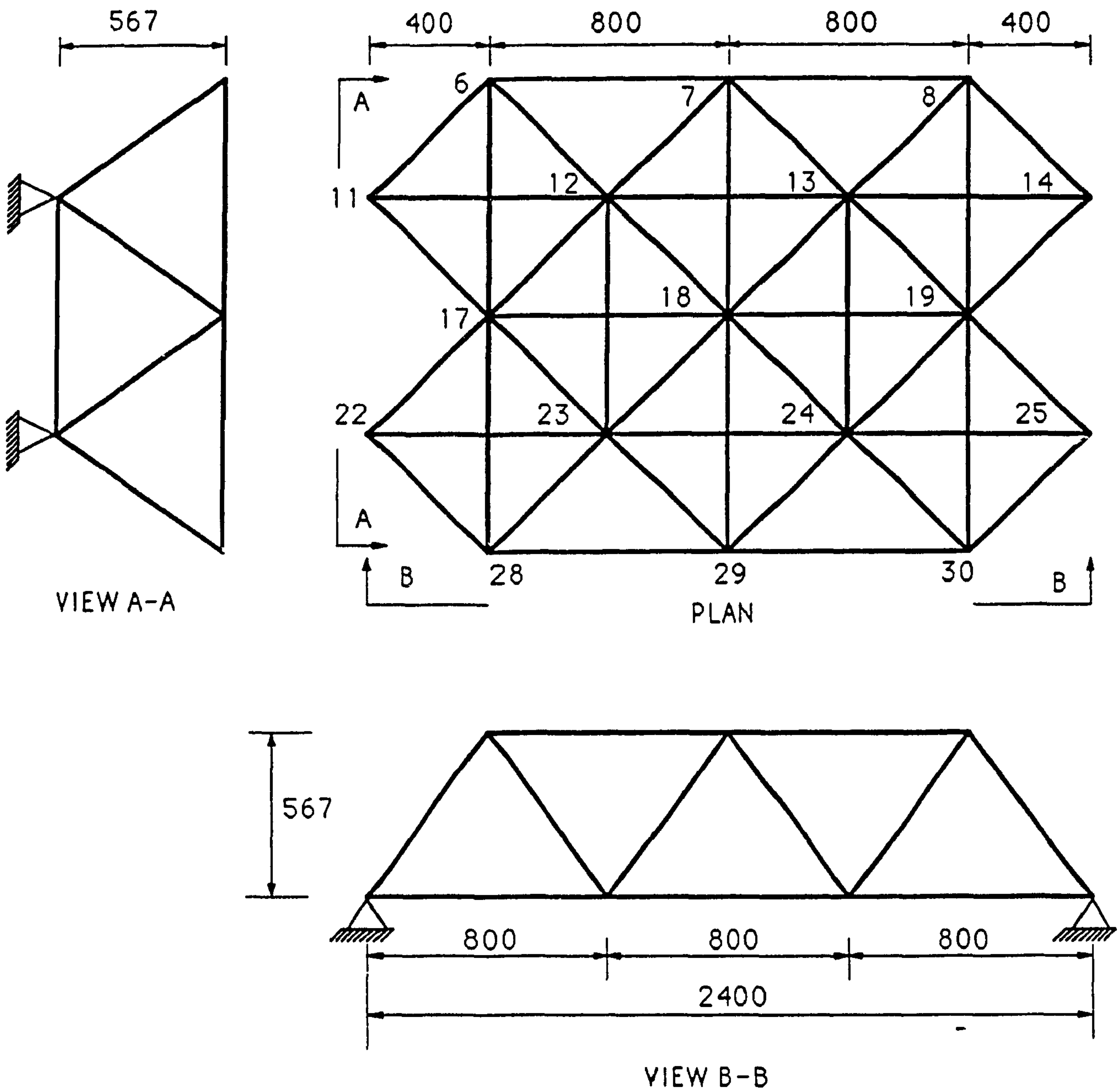


Fig. 5-32, Test truss geometry

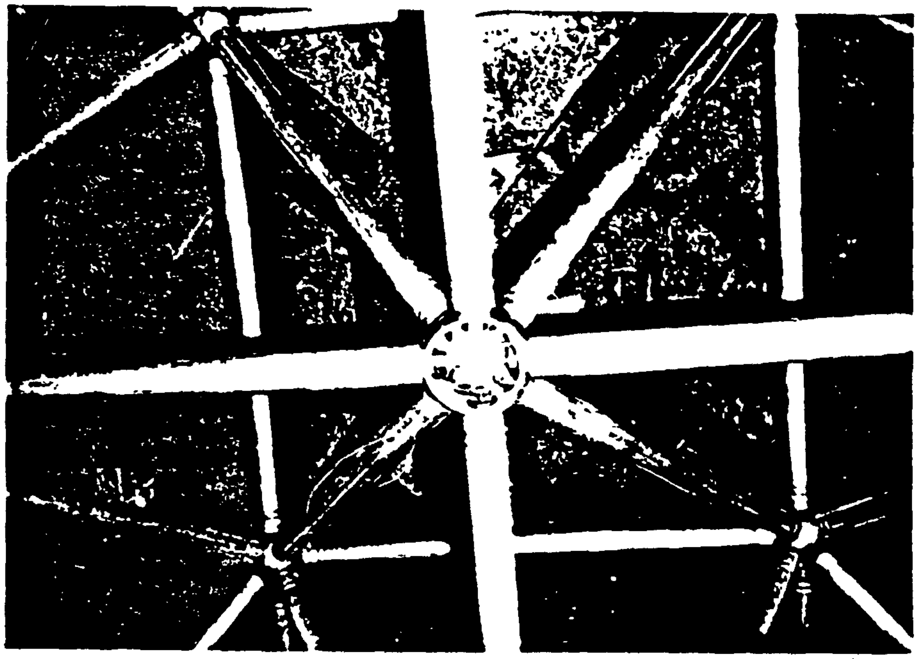


Fig. 5.33, HEMTEC joint

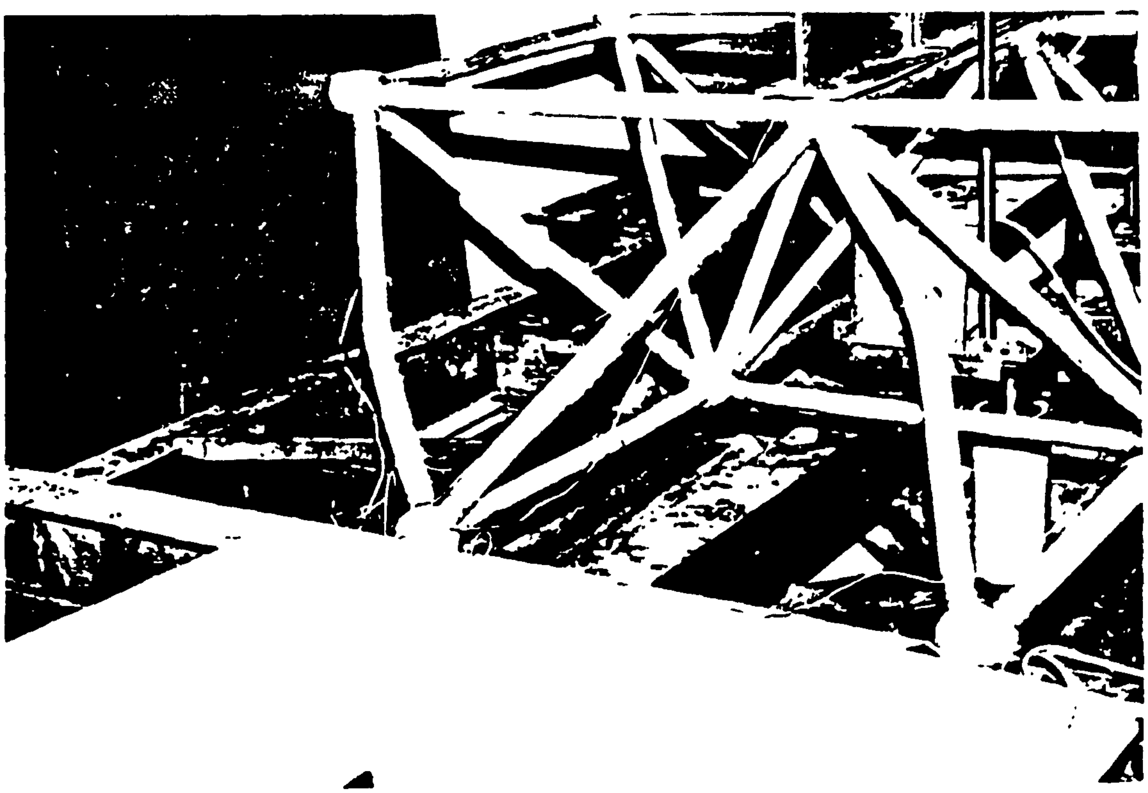


Fig. 5.34, Failed truss

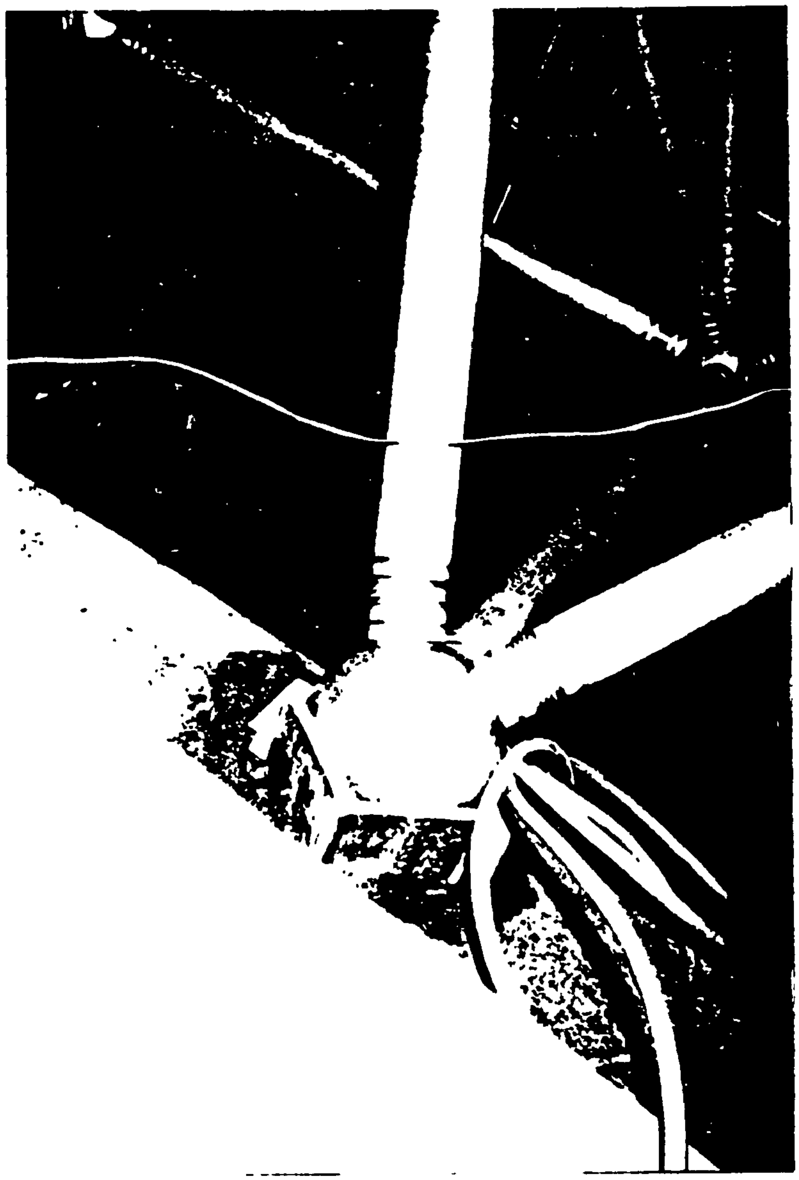


Fig. 5.35, Support detail

5.7.3 Test results and observations

Table 5.3 gives the forces in the bars of interest at various increments of loading:

Table 5.3, measured member forces and truss deflection

Load, kN	6-7	6-11	11-17	17-18	28-29	Δ_{18}
0	0	0	0	0	0	0.0
4.45	0.61	0.58	0.99	1.09	0.65	0.6
8.91	1.31	1.20	1.94	2.10	1.30	1.1
17.81	2.51	2.36	3.91	4.25	2.66	2.4
22.27	3.09	2.92	4.93	5.44	3.24	3.1
26.72	3.70	3.43	5.97	6.67	3.76	3.8
35.63	4.82	4.64	7.93	8.98	5.03	5.4
44.54	6.19	5.76	9.94	11.25	6.11	7.1
53.44	7.84	7.01	11.83	12.44	7.98	9.1
57.90	8.72	7.52	12.89	13.05	8.14	10.1
62.35	9.45	7.99	13.78	13.40	8.61	11.4
64.58	9.81	8.09	14.07	13.50	9.02	12.6
65.69	9.93	8.48	14.07	14.18	9.61	13.2

Fig. 5.37 shows the theoretical deflection of the centre of the truss, based on elastic behaviour, shown by dashed lines, versus the measured values and reflects the reduced rigidity caused by the grooves. The real behaviour of the truss is nonlinear from the beginning.

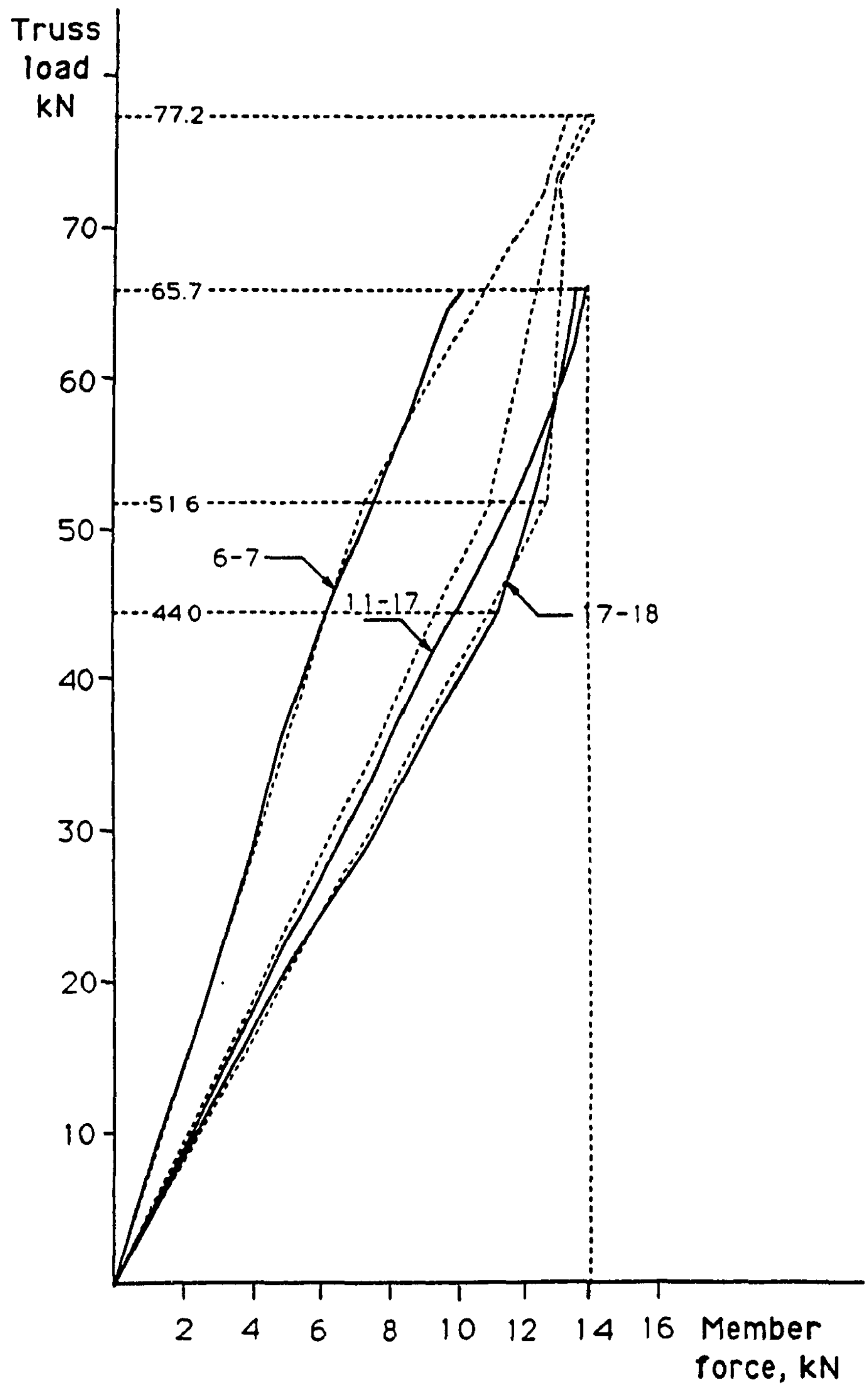


Fig. 5.36, Chord forces, theoretical vs test

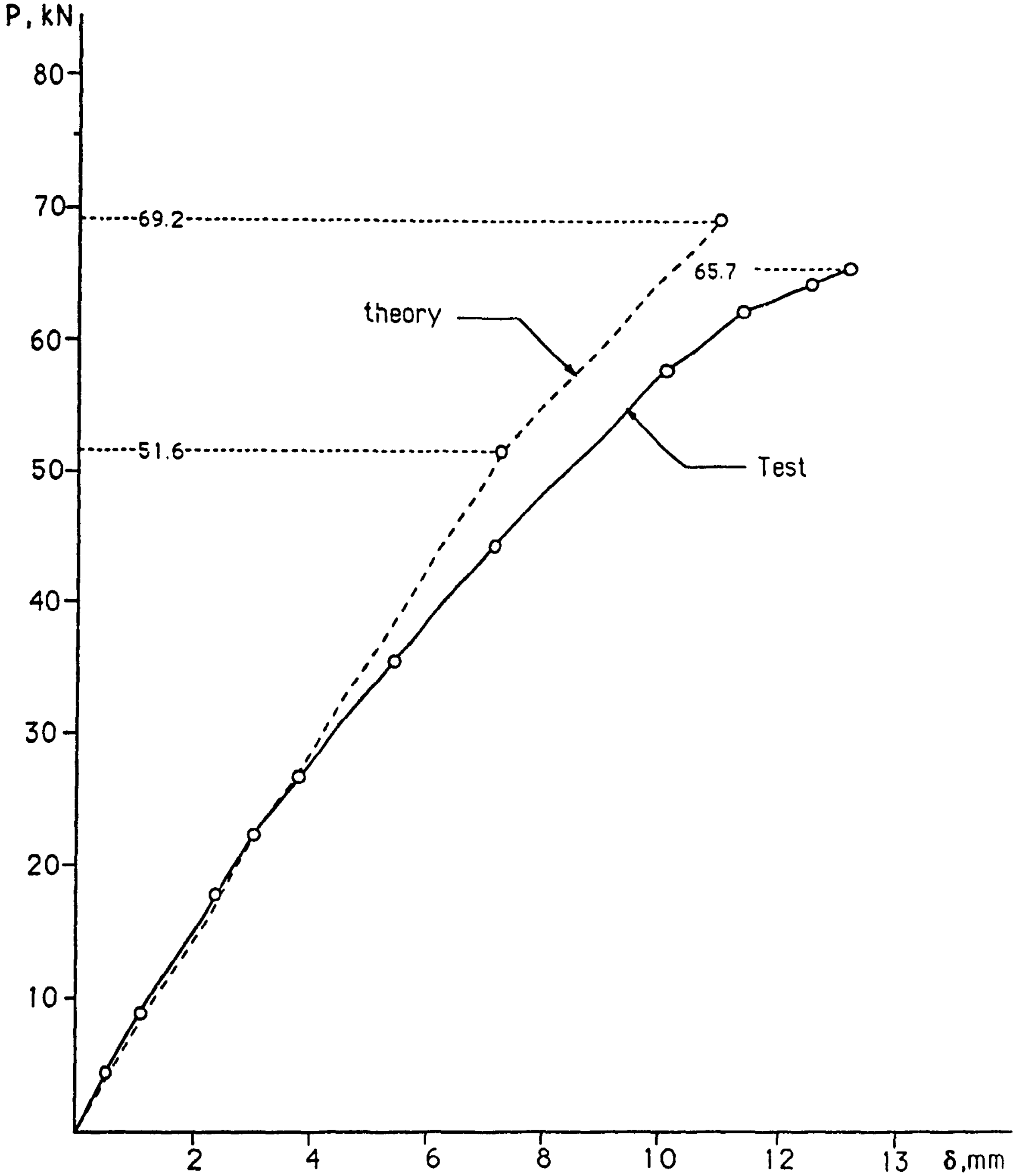


Fig. 5.37, Truss deflection,
theoretical vs test

The Fig. 5.36 shows the measured forces in the three chord members, 17-18, 6-7 and the chord 11-17 which are of special interest in this test, compared with the theoretical behaviour, shown by dashed lines.

The chord 17-18 became nonlinear at a total load of 44.0 kN, compared to a theoretical load of 51.6 The load it carried then increased more slowly, therefore the chord 6-7 should have picked up more load. As it did not do so, it appears that the load was transferred partially through the loading beam. At the applied load of 65.7 kN, the diagonal 17-22, buckled after yielding its grooves. This compares with the theoretical ultimate capacity of the truss of 77.2 kN. Failure was predicted to be controlled by member 17-18, but after redistribution due to nonlinear behaviour the diagonal 17-22 carried a higher load and controlled the capacity. As soon as the diagonal 17-22 failed, load was transferred to the diagonal 6-11 and lead to the collapse of the truss.

It is seen that nonlinear behaviour occurred, but the amount of it is lower than expected. After the test, the grooves in diagonal member 12-18, at the centre of the truss, were seen to have yielded. This member was not expected to yield, but this might have been due to an uneven force distribution caused by misfit of the members.

CHAPTER 6

INCREASING THE LOAD CAPACITY OF SPACE TRUSSES BY REMOVING SELECTED DIAGONALS

6.1 INTRODUCTION

Ideally a space truss is composed of bars each of which is suited to the load it carries. In practice, however, it is more convenient to use a limited number of bar sizes thus only a few of the members will be fully stressed when the maximum load capacity of the space truss is reached.

In order to effect a more equitable distribution of forces between chords of equal size, such that more chord members are loaded close to their capacity, selected diagonals are removed to control the path followed by the shear forces, and thereby control the forces entering the chords. In this study the effect of diagonal removal on space trusses is demonstrated and a method for selection of the diagonals whose removal gives optimum increase in the load carrying capacity of a space truss is described.

The force distribution between the bars of a space truss is determined by linear elastic analysis. Because of the "brittle" behaviour of compression members, in the economic range of slenderness, the overall load on a space truss is in general limited by the first member to reach its capacity, (for simplicity, in this introduced to the chords by the action of shear forces carried by the diagonal internal bracing bars. The paths that the shear forces take can be modified by removing some diagonals, and force can be directed away from the chords which are normally the most highly loaded, towards the understressed chords, thereby creating a more uniform distribution of forces between chords of equal size, and consequently increasing the overall load carrying capacity, [20].

To demonstrate the effect of diagonal removal, several different space trusses are studied. All of them show that removing appropriate diagonals increases the overall capacity of the system.

The pattern of diagonal removal and the amount of the increase in the capacity varies with factors such as:

Distribution of member sizes (whether all members are the same size or of different sizes);

Support conditions (continuous, simply supported, with columns at the corners only or supported along the boundaries);

Loading pattern (symmetrical or unsymmetrical);

Geometry of the truss (square, rectangular, etc.).

Because of the many possible combinations of the above factors, simple rules to give typical patterns for the diagonals to be removed, as an aid to designers, have not been found. It had been anticipated that the diagonals connected to the nodes adjacent to the highly loaded chord elements would be the ones to be removed, but this is not true. Even the yield line pattern of plastic failure of equivalent grillages provides no clues. An iterative selection technique has proved to be the only practical procedure.

6.2 PROCEDURE

The load capacity of the truss is the objective function and the presence or not of the diagonals is the design variable. The constraints are the member capacities. For practical applications the maximum deflection of the truss should also be considered as an active constraint.

"SPAN", structural analysis program, was modified to perform the routine of removing diagonals and recording the change in load carrying capacity effected, and then to use a steepest ascent method of optimization for the final selection of the diagonals to be removed. The flow chart is given in Fig. 6-1

Each of the diagonals is removed sequentially. In order not to disturb the symmetry, the diagonals, other than the ones located on the main diagonal line, are removed in pairs. When a set of diagonals is removed the corresponding capacity of the truss is recorded and the diagonals are then replaced. This is repeated for all the diagonals, and the results are sorted in descending order of the load carrying capacity of the truss. This procedure gives a list of diagonals whose removal increases the capacity of the truss.

Simultaneous removal of all these diagonals does not lead to the maximum increase in overall load carrying capacity of the truss, and it is necessary to find the optimum combination of diagonals whose simultaneous removal provides the maximum capacity.

First, the set whose removal alone gives the biggest increase in the capacity is removed permanently. The procedure of sequential diagonal removal is then repeated to find the next set whose removal leads to the highest additional increase in capacity of the truss; these members are then permanently removed. This is continued until further removal of diagonals does not increase the capacity.

To ensure that the maximum is global, the initial member to be removed is varied to see if any higher capacities can be realised. The analyses conducted show that the described procedure always gave the maximum capacity.

6.3 TYPICAL ANALYSIS

As space trusses are most useful when spanning in two directions between isolated columns, all the cases analysed are of this type. With all chord members the same size, the increase in capacity that can be effected is large. For more customary trusses, with a variety of chord sizes, proportioned to optimize the capacity of the initial truss, the increased capacity is not so great, but the benefit is still significant.

The first space truss analysed is a double-layer-offset-square-on-square space truss with eight modules of bxbxb/$\sqrt{2}$ deep. In this case b=1500 mm, the depth=1060 mm and all members are of length 1500 mm. The truss is simply supported on columns at four corners of the lower plane, giving a span of 12 m in each direction. Taking advantage of double symmetry, only a quarter of the truss is studied, Fig. 6-2.

All the members are aluminum tubes and are assigned the same size, except the diagonal member at the support which is a heavier section strong enough not to fail before the chords fail. The general member properties are:

A, cross-sectional area = 300 mm^2

Pt, tension capacity = 50.0 kN

Pc, compression capacity = 50.0 kN

For a uniformly distributed load the initial capacity of the truss, with all the members in place, is determined using a single cycle elastic analysis, assuming the first member to fail controls the capacity. This load capacity is P_0=1.0 kN/joint.

The effect of each diagonal removal on the overall capacity of the truss is shown in Table 6-1. The pair of diagonals whose removal leads to the maximum increase in capacity of the truss is 13-19, 33-39, giving P_1=1.01 kN/joint, so they are permanently removed . A second process of sequential removal of diagonals gives another list, similar to Table 6-1, from which the next pair to be removed is found, which is 8-13, 33-38 whose permanent removal leads to P_2=1.07 kN/joint. This procedure is repeated up to the eighth step where removal of the pair 19-25, 39-45 results in P_8=1.205 kN/joint, and further removal of diagonals does not increase the capacity, see Table 6-2. The location of removed diagonals are shown in Fig. 6.3, bottom chord force distribution in Fig. 6.13 and top chord force distribution in Fig. 6.14.

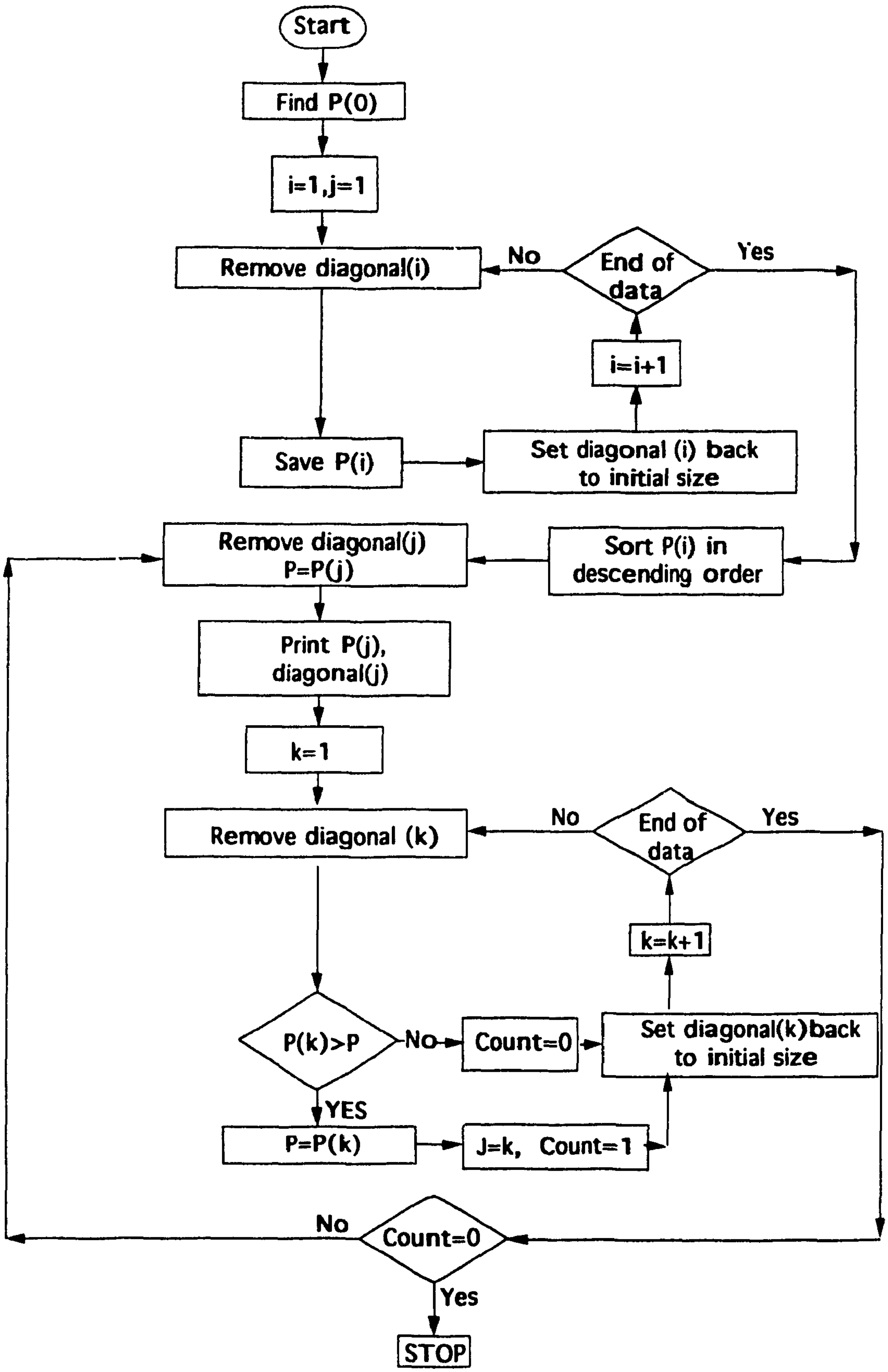


Fig. 6.1, Flow chart of diagonal removal

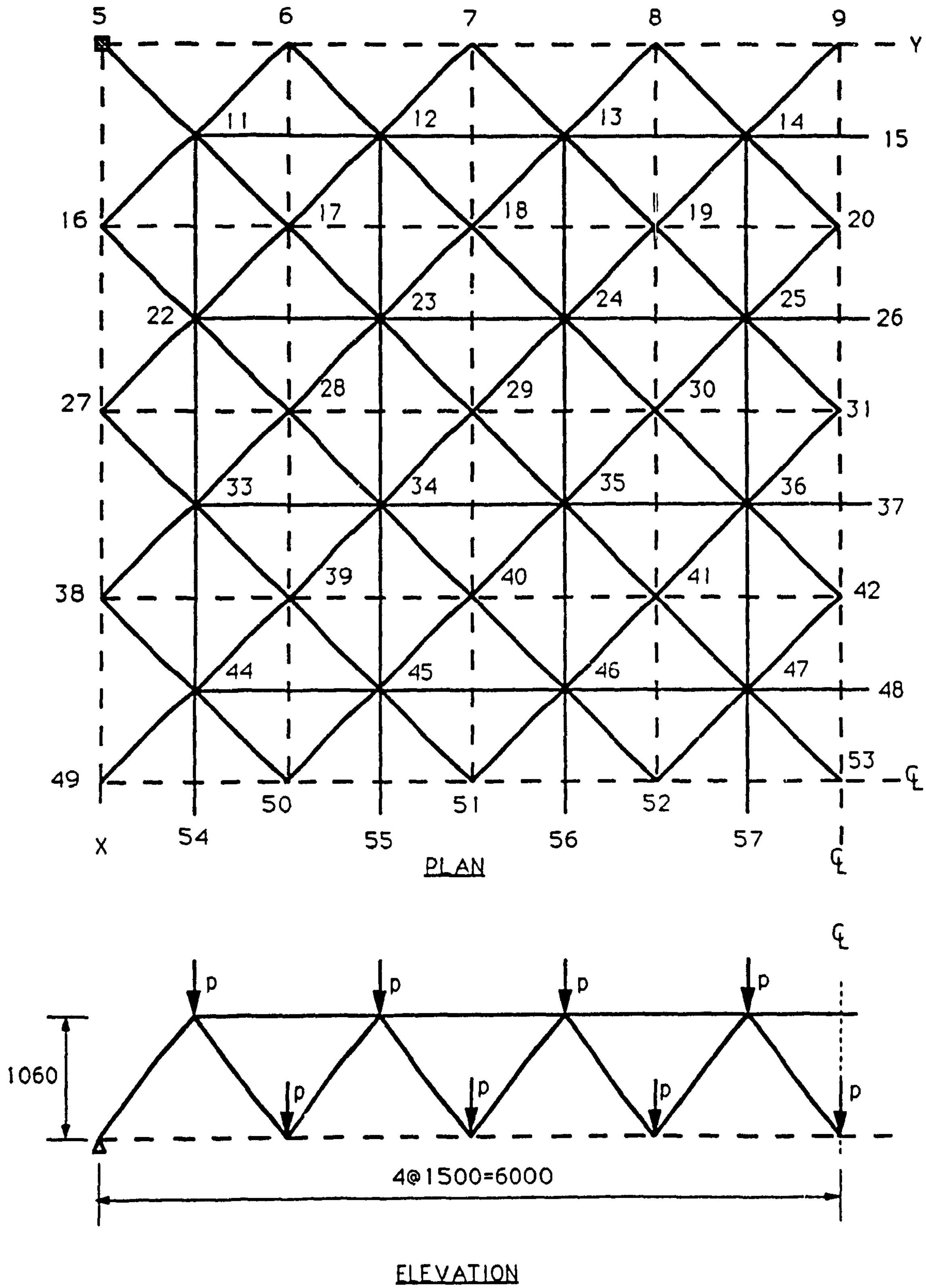


Fig. 6.2, Truss geometry

Table 6-1, Effect of individual diagonal removal

Diagonals removed		Load kN/joint
13-19,	33-39	1.0105
23-29		1.0085
12-18,	22-28	1.0080
14-19,	39-44	1.0050
13-18,	28-33	1.0040
30-36,	40-46	1.0030
8 -13 ,	33-38	1.0025
24-30,	34-40	1.0010
35-41		1.0005
41-47		1.0005
8 -14,	38-44	1.0005
------------------------------------		Limit of increase
31-36,	46-51	1.0000
36-41,	41-46	0.9995
42-47,	47-52	0.9990
25-30,	40-45	0.9985
36-42,	46-52	0.9980
24-29,	29-34	0.9970
25-31,	45-51	0.9970
30-35,	35-40	0.9970
14-20,	44-50	0.9945
29-35		0.9930
7 -13,	27-33	0.9865
7 -12,	22-27	0.9825
20-25,	45-50	0.9815
19-25,	39-45	0.9775
19-24,	34-39	0.9770
12-17,	17-22	0.9750
18-23,	23-28	0.9530
11-17		0.9510
18-24,	28-34	0.9310
17-23		0.9010
6 -12,	16-22	0.8760
6 -11,	11-16	0.8645

Table 6-2, Diagonals to be removed

steps	diagonals removed		P (N/joint)	Chords to fail	
0	none		1000	14-15,	44-54
1	13-19,	33-39	1010	14-15,	44-54
2	8 -13,	33-38	1070	14-15,	44-54
3	19-24,	34-39	1085	14-15,	44-54
4	24-30,	34-40	1145	13-14,	33-44
5	23-29		1150	13-14,	33-44
6	13-18,	28-33	1155	13-14,	33-44
7	12-18,	22-28	1200	7-8,	27-38
8	19-25,	39-45	1205	7-8,	27-38

The Table 6-2 shows that by removing 15 diagonal members from the quarter truss the overall load capacity of the truss increases from 1000 N/joint to 1205 N/joint, while the weight of the truss is reduced, thereby further increasing the load/weight ratio as shown in the Table 6-3. The truss with diagonals removed is shown in Fig. 6-3.

Table 6-3, Increase in load/weight by diagonal removal

	initial	modified
Load capacity, Pa	889	1071
Weight, Pa	42.2	37.4
Load/weight	21.0	28.6

Increase in load/weight ratio : (28.6-21.0)/21.0=36%

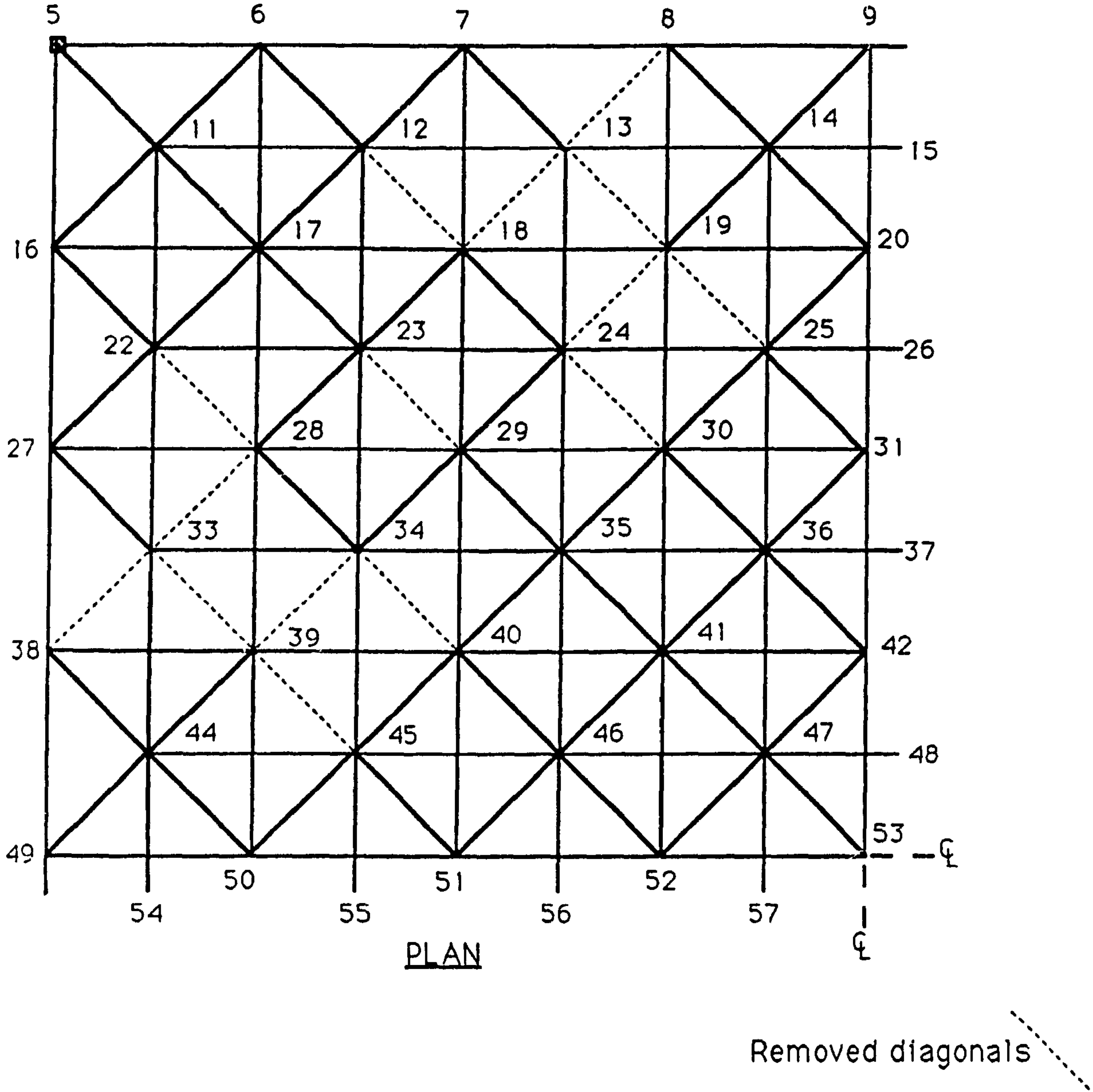


Fig. 6.3, Truss with removed diagonals

6.4 HOW DOES DIAGONAL REMOVAL INCREASE THE OVER ALL LOAD CARRYING CAPACITY OF A SPACE TRUSS?

In a space truss, diagonals carry the shear forces and transfer loads to the chord members. By removing selected diagonals the forces are directed away from the chords which are most highly loaded towards the understressed chords. This is explained by an example. The space truss analysed in section 6-3 is studied. Using one cycle of elastic analysis the chord forces are calculated in each step as diagonals are removed.

In the original truss the chords 14-15 and 44-54 are the first to reach their capacity. The load at each joint (P_O), at this stage is 1.0 kN. The forces in the chord 14-15 and the members adjacent to it are shown in Fig. 6-4, where it is seen that the chords in line 18-20 are loaded to only half their capacity. In step one, diagonals 13-19 and 33-39, which are shown by dashed lines, are removed. The critical chord remains the same, (14-15), but the force in some of the adjacent chords has increased, and, as a result, the capacity of the truss has increased to 1010 N, (P1). In step two the diagonals 8-13 and 33-38 are removed. The critical chord is still the same, but the chord 18-19 carries more load, and the capacity has increased to 1070 N, (P2). This procedure is repeated until the Fig. 6-12 is reached. By removing all the selected diagonals, the force distribution has changed, the critical members are different and the members on the line 18-20 are loaded close to their capacity. The influence of diagonal removal on the overall capacity of space truss and the pattern of force distribution after each step of diagonal removal are given in Figs. 6-5 to 6-12. Figs. 6-13 and 6-14 compare the force distribution in the original and the modified truss.

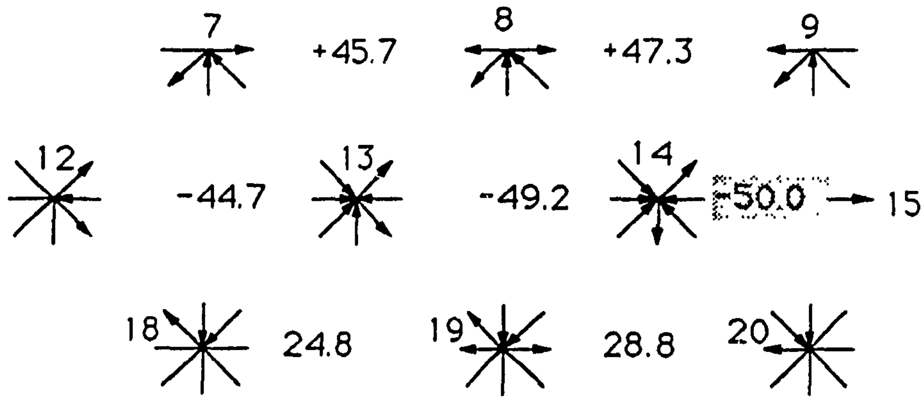


Fig. 6.4, Original truss
P0=1000 N

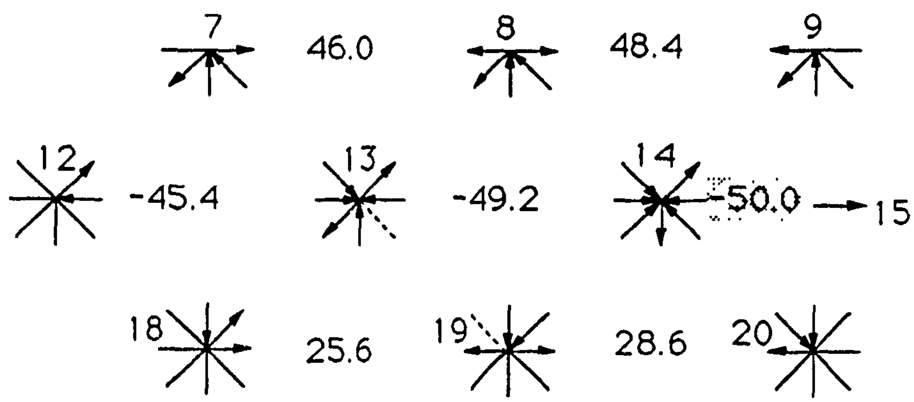


Fig. 6.5, Step 1
P1=1010 N

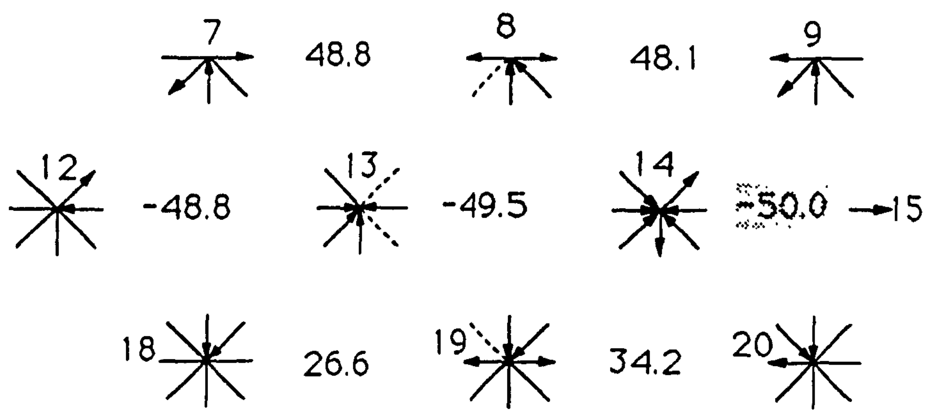


Fig. 6.6 , Step 2
P2=1070 N

removed diagonal

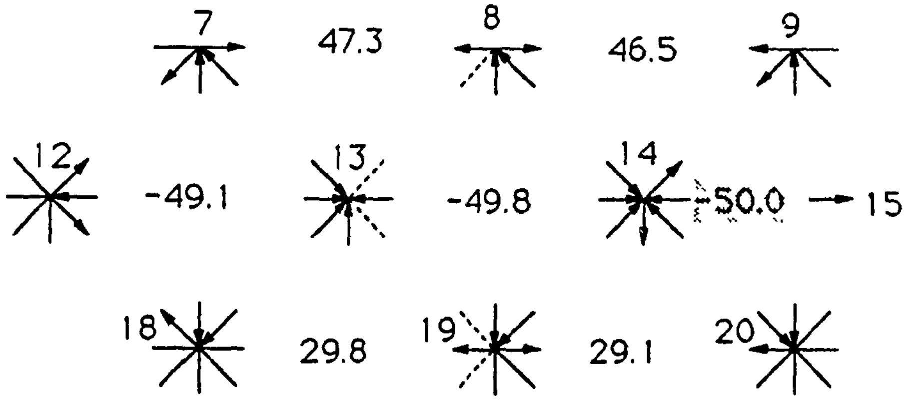


Fig. 6.7, Step 3
P3=1085 N

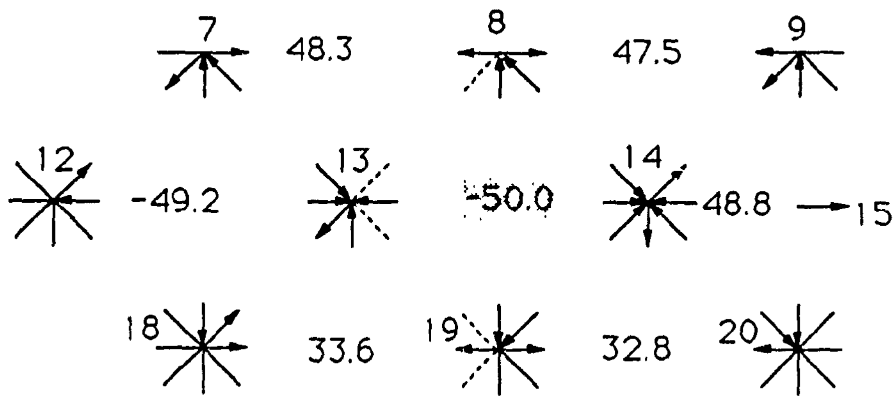


Fig. 6.8, Step 4
P4=1145 N

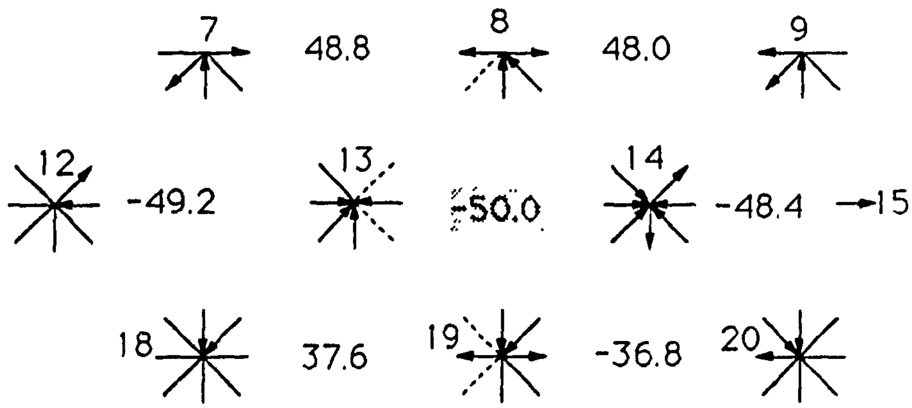


Fig. 6.9, Step 5
P5=1150 N

removed diagonal

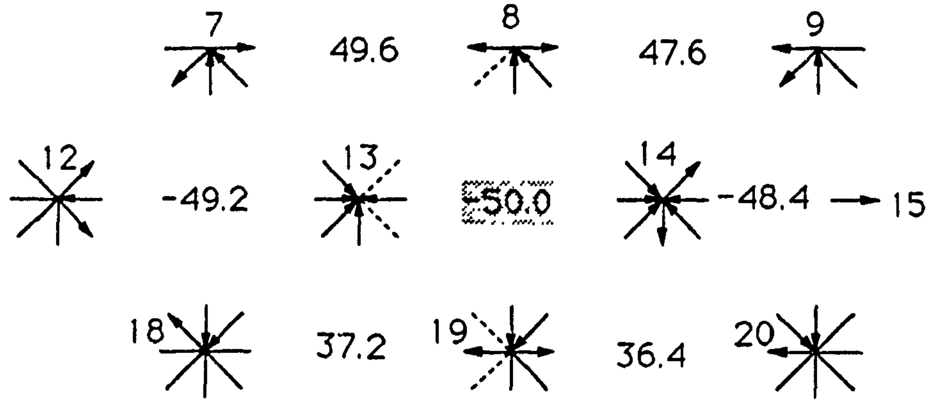


Fig. 6.10, Step 6
P6= 1155 N

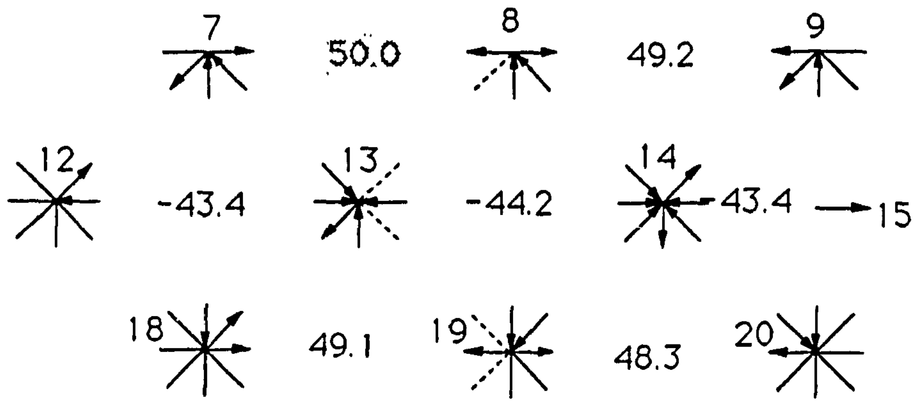


Fig. 6.11, Step 7
P7=1200 N

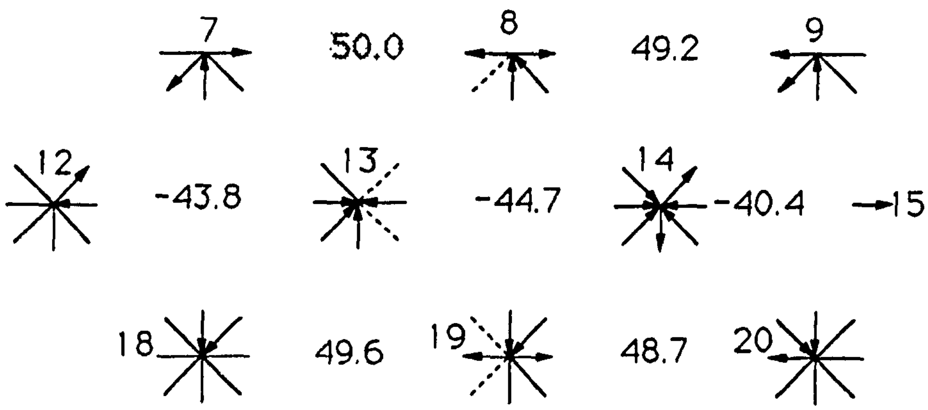
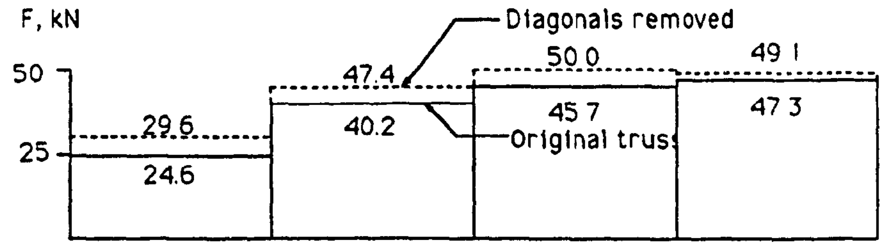


Fig. 6.12, Step 8
P8=1205 N



Chord forces along 5-9

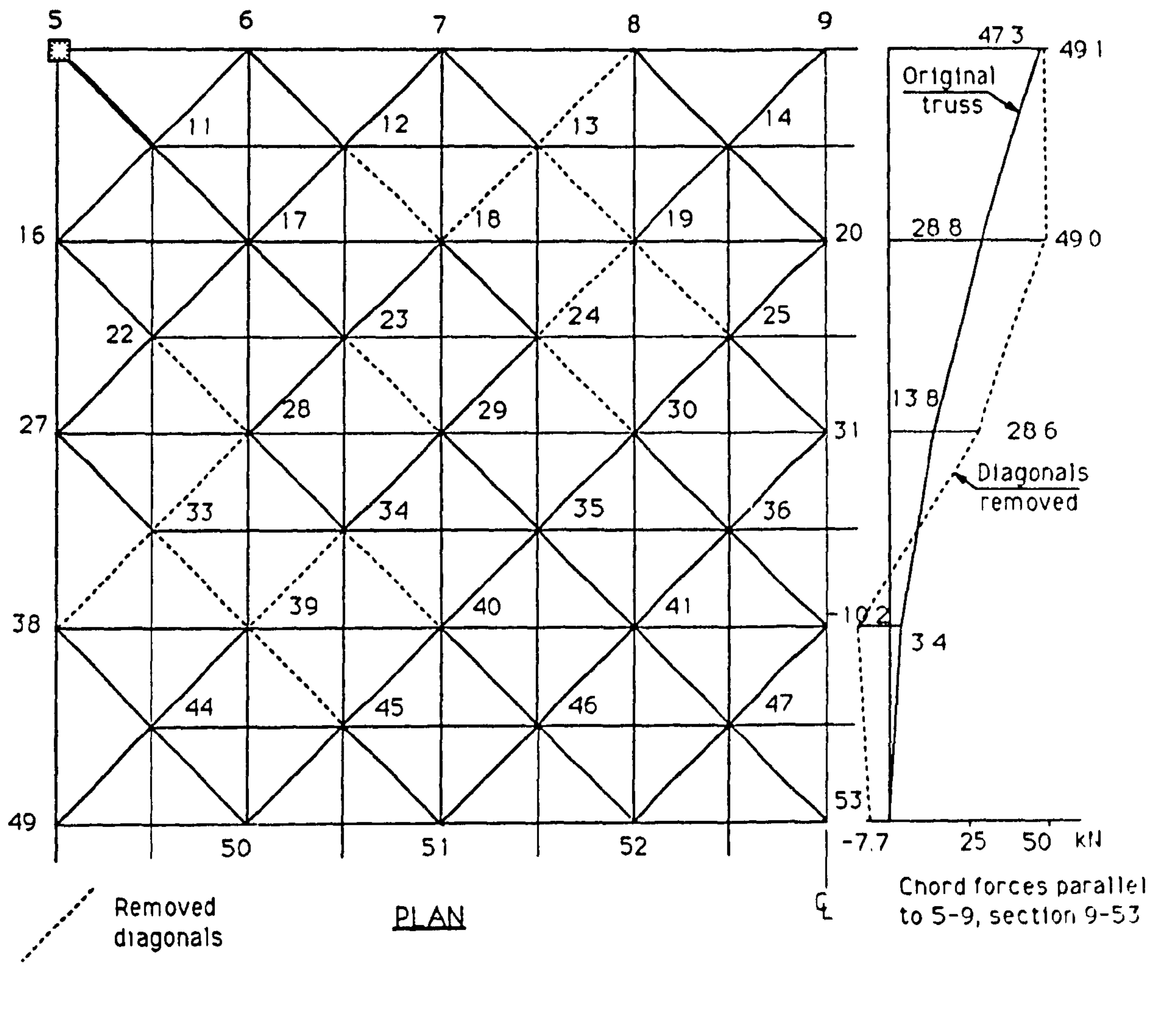


Fig. 6.13, Bottom chord force distribution

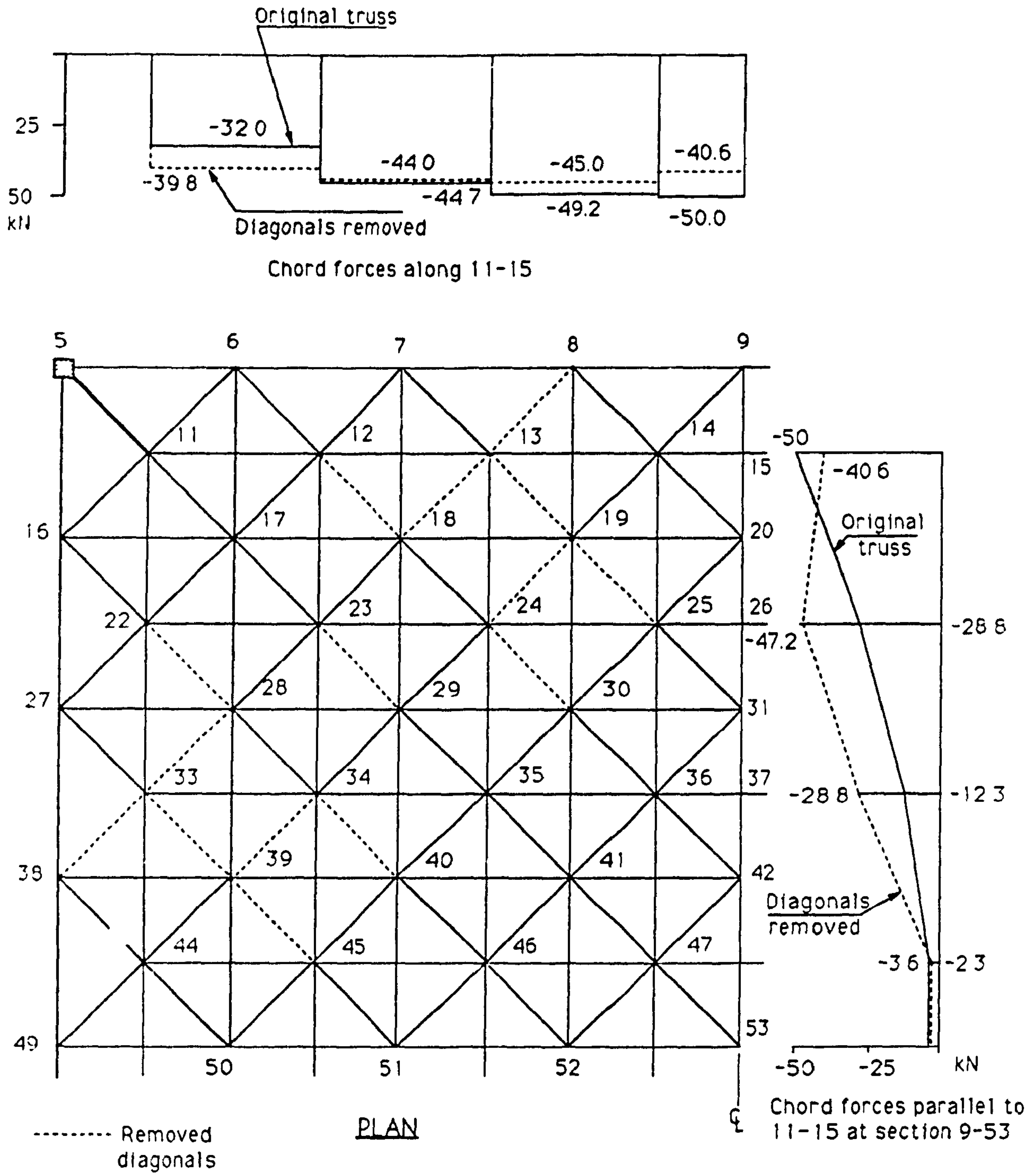


Fig. 6.14, Top chord force distribution

6.5 DEFLECTION OF SPACE TRUSS AFTER DIAGONAL REMOVAL

For the original truss under uniformly distributed load of 1.0 kN/joint, the deflection of the column-line, (5-9) and centre-line, (49-53) are shown in Fig. 6-15 and Fig. 6-16. The maximum deflection, which occurs at the centre of the span, (joint 53) is 90 mm. For the modified truss, with the applied uniformly distributed load of 1.205 kN/joint, the deflected shape, shown by dashed lines, has a maximum value of 147 mm at the centre of column-line, (joint 9). This is an increase of 64% for a 36% increase in load/weight ratio, indicating that deflections need to be checked.

It is interesting to observe that, after removing the diagonals, the structure deflects to a saddle shape and the midpoint of the column-line deflects 11 mm more than the centre of the structure.

6.6 APPLICATION TO LARGE SPAN TRUSSES

It has been shown that the technique of diagonal removal can be applied as a practical method for the behaviour modification of space trusses. In order to study the effect of the span, boundary conditions and the influence of using different member sizes, the four typical large span space truss arrangements treated in chapter one were analysed.

The increase in capacity for each truss at each step of diagonal removal is given in Tables 6-4 to 6-7. The patterns of diagonal removal are shown in Figs. 6-17 to 6-20. Fig. 6-21 shows the influence of the boundary conditions and member size distribution on the location of the diagonals to be removed.

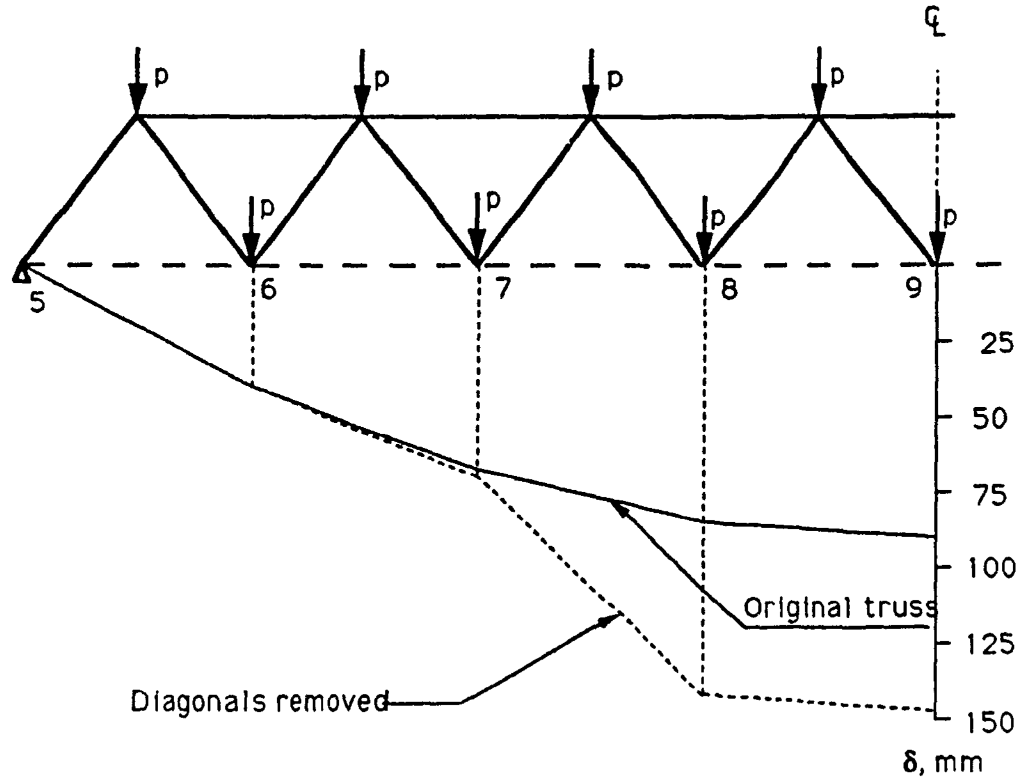


Fig. 6.15, Deflection of column-line

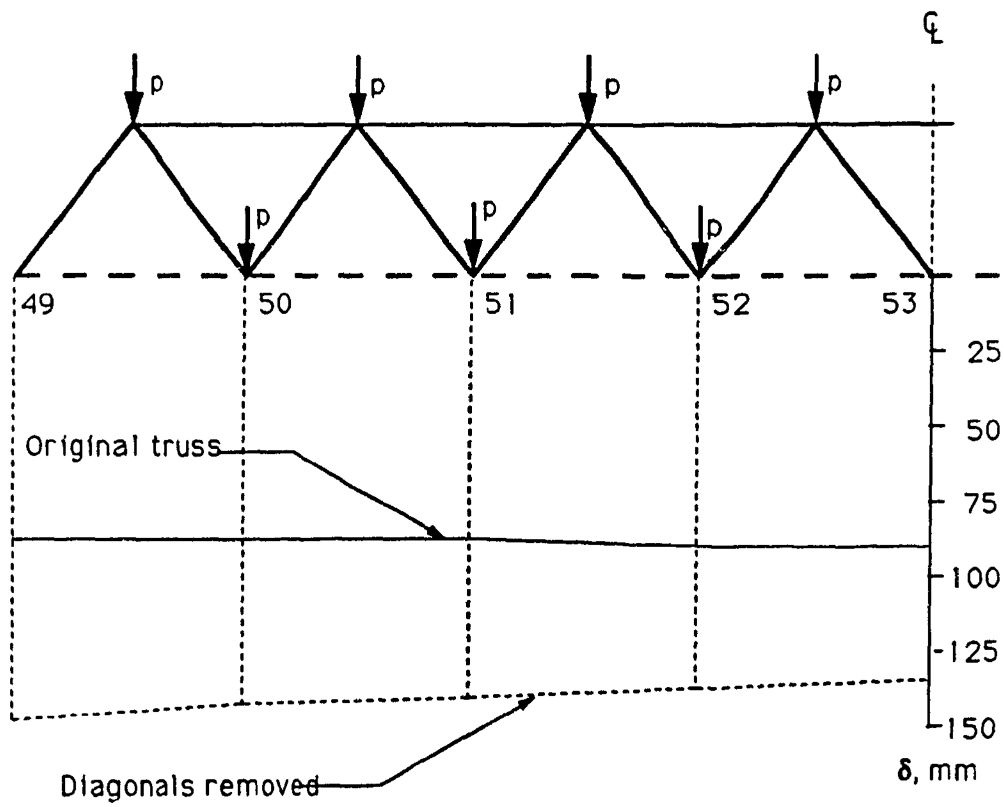


Fig. 6.16, Deflection of centre-line

Table 6-4 Truss Type 1 Diagonals removed

Steps	Diagonals	Load (kPa)	
Load/weight		Original truss	
1.250	2.98		
1	19-29	1.289	3.08
2	21-30, 48-57	1.308	3.14
3	11-20, 38-47	1.335	3.22
4	22-31, 67-76	1.360	3.29
5	12-22, 66-76	1.392	3.39
6	41-51, 77-87	1.399	3.42
7	14-23, 95-104	1.411	3.47
8	32-41, 77-86	1.450	3.58
9	22-32, 76-86	1.477	3.67
10	42-51, 87-96	1.507	3.75
11	24-33, 105-114	1.533	3.83
12	60-70, 78-88	1.538	3.86
13	52-61, 97-106	1.548	3.91
14	61-70, 88-97	1.557	3.95
15	71-80, 98-107	1.559	3.98
16	33-43, 105-115	1.560	4.00
17	39-49	1.568	4.03
18	55-64, 154-163	1.568	4.05
19	109-119	1.568	4.06
20	149-159	1.568	4.07

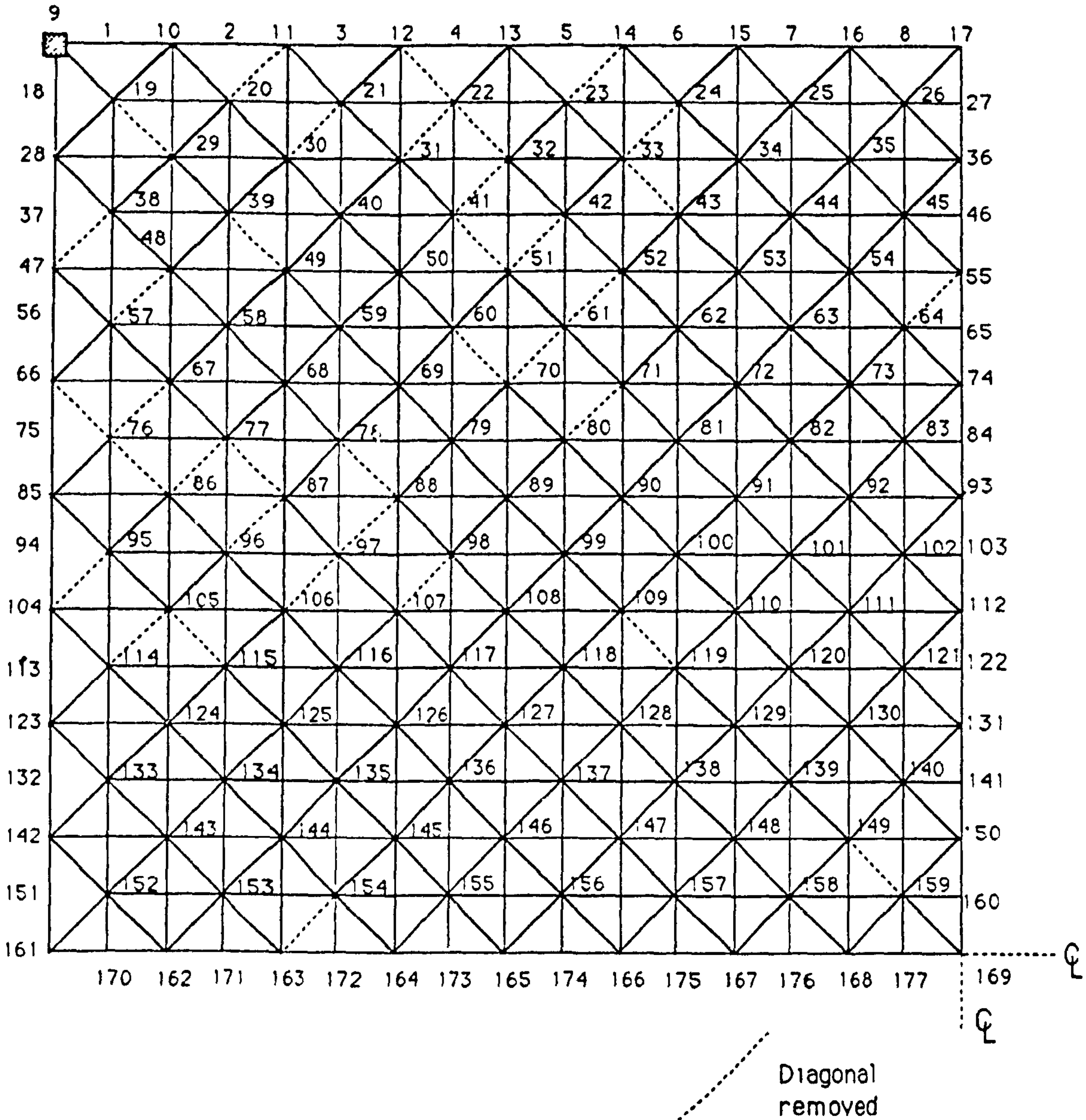


Fig. 6.17, Truss Type 1
with diagonals removed

Table 6-5 Truss Type 2 Diagonals removed

Steps	Diagonals	Load(kPa)	
Load/weight		Original	truss
1.300	3.10		
1	20-29, 29-38	1.369	3.27
2	21-31, 57-67	1.405	3.38
3	12-21, 57-66	1.525	3.68
4	22-31, 67-76	1.619	3.93
5	23-32, 86-95	1.647	4.02
6	22-32, 76-86	1.706	4.18
7	24-33, 105-114	1.724	4.25
8	23-33, 95-105	1.775	4.39
9	34-43, 115-124	1.783	4.44
10	34-44, 124-134	1.817	4.54
11	44-53, 125-134	1.818	4.56
12	35-45, 143-153	1.819	4.58
13	35-44, 134-143	1.827	4.63
14	59-69	1.828	4.64
15	50-59, 59-68	1.832	4.67
16	40-50, 58-68	1.835	4.71
17	41-51, 77-87	1.836	4.73
18	51-60, 78-87	1.837	4.76
19	61-70, 88-97	1.837	4.78
20	52-61, 97-106	1.837	4.81

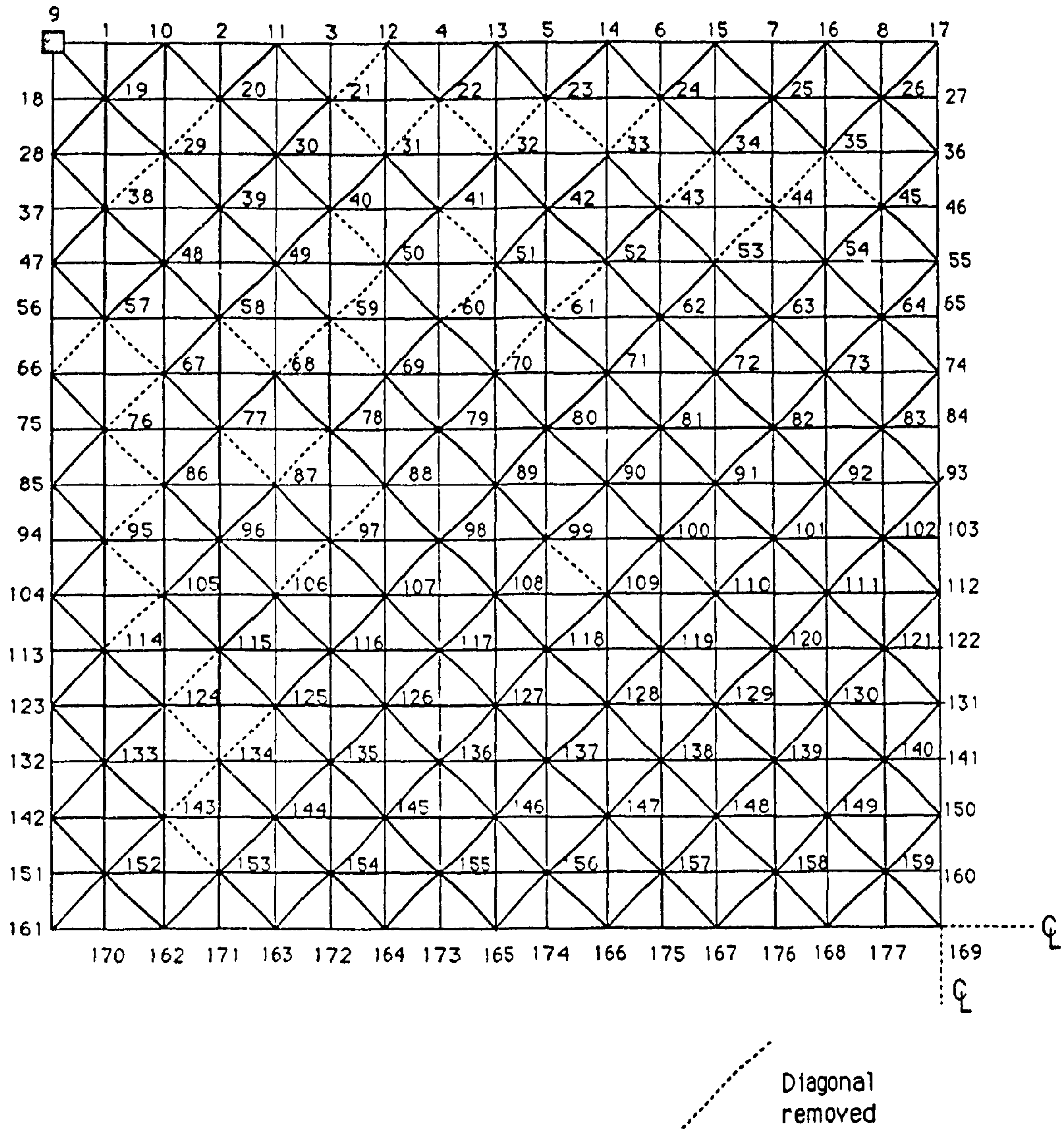


Fig. 6.18, Truss Type 2
with diagonals removed

Table 6-6 Truss Type 3 Diagonals removed

Steps	Diagonals	Load (kPa)	L/W
Original truss		0.936	5.78
1	21-31, 57-67	0.950	5.89
2	14-23, 95-104	0.964	6.00
3	22-32, 76-86	0.979	6.12
4	15-24, 114-123	0.988	6.20
5	26-36, 152-162	0.995	6.27
6	35-45, 143-153	0.999	6.32
7	39-49	1.000	6.34
8	25-35, 133-143	1.005	6.40
9	42-51, 87-96	1.006	6.43
10	43-52, 106-115	1.008	6.47
11	45-54, 144-153	1.010	6.51
12	26-35, 143-152	1.014	6.56
13	83-93, 155-165	1.018	6.62
14	72-82, 126-136	1.020	6.66
15	54-64, 144-154	1.023	6.71
16	79-89	1.026	6.74
17	54-63, 135-144	1.029	6.79
18	102-112, 156-166	1.032	6.84
19	111-121, 147-157	1.033	6.88
20	70-79, 79-88	1.035	6.92
21	50-59, 59-68	1.036	6.96
22	111-120, 138-147	1.038	7.00
23	92-101, 137-146	1.039	7.04
24	121-130, 148-157	1.041	7.08
25	91-100, 118-127	1.042	7.12

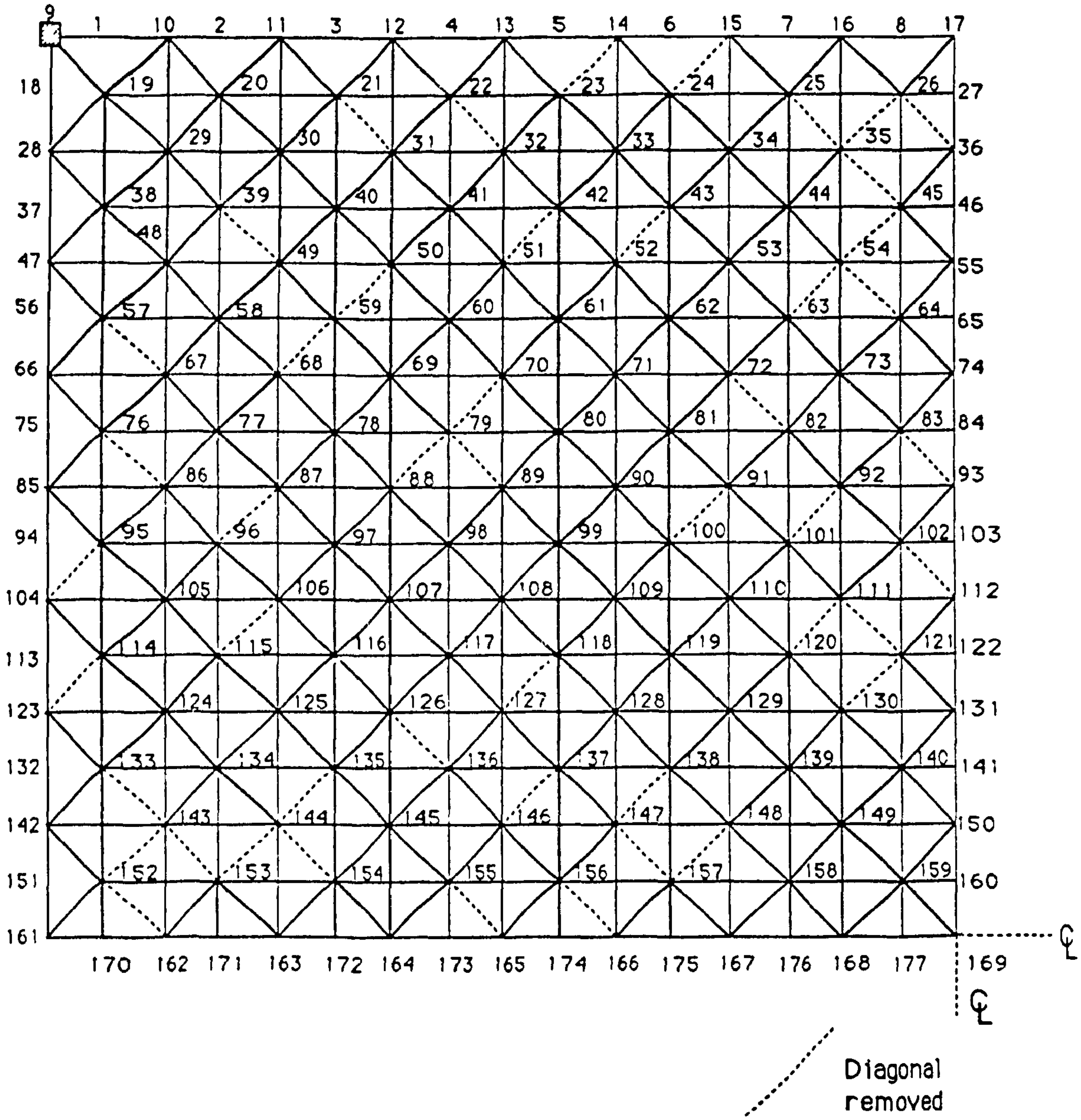


Fig. 6.19, Truss Type 3
with diagonals removed

Table 6-7 Truss Type 4 Diagonals removed

Steps	Diagonals	Load (kPa)	L/W
Original truss		1.190	7.83
1	20-30, 38-48	1.228	8.12
2	21-31, 57-67	1.266	8.40
3	20-29, 29-38	1.284	8.58
4	22-31, 67-76	1.286	8.64
5	24-34, 114-124	1.292	8.71
6	25-35, 133-143	1.294	8.77
7	44-54, 134-144	1.296	8.82
8	24-33, 105-114	1.297	8.86
9	62-71, 107-116	1.298	8.91
10	63-73, 135-145	1.299	8.96
11	36-45, 153-162	1.300	9.00
12	35-45, 143-153	1.302	9.06
13	16-25, 133-142	1.305	9.12
14	44-53, 125-134	1.306	9.17
15	64-74, 154-164	1.306	9.21
16	100-109, 109-118	1.307	9.26

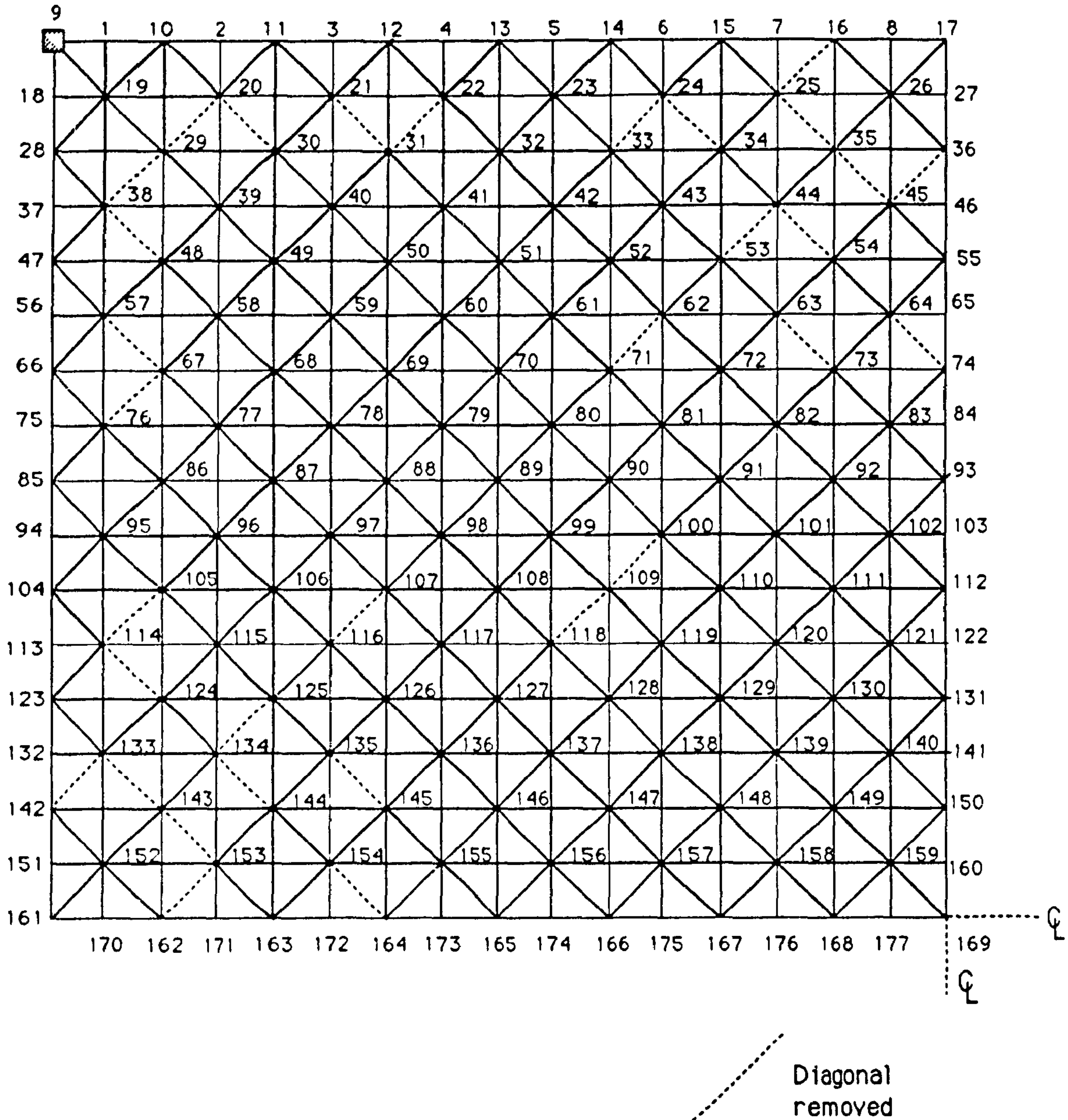


Fig. 6.20, Truss Type 4
with diagonals removed

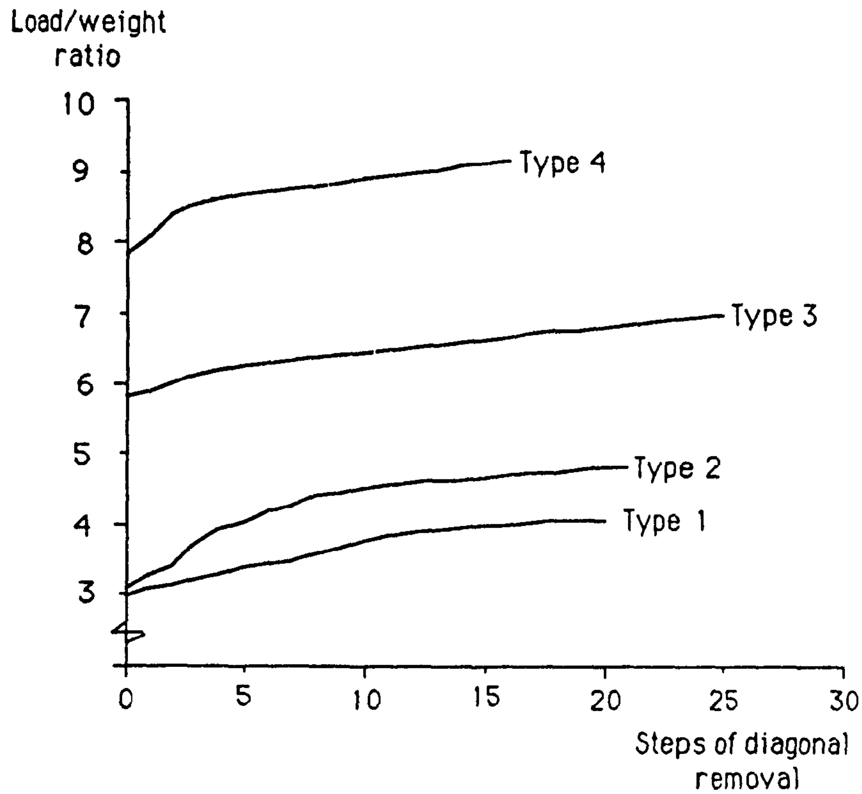


Fig. 6.21, Influence of diagonal removal on trusses with different boundary conditions and member sizes

Table 6-8, Percentage increase in load/weight ratio effected by removing diagonals

Truss type	Status	P*(kPa)	L/W	increase in P
1. Uniform members simply supported	original	1.25	2.98	-
	modified	1.57	4.09	26%
	E.P.**	1.78	4.24	42%
2. Uniform members continuous	original	1.30	3.10	-
	modified	1.84	4.85	41%
	E.P.**	2.72	6.50	109%
3. Three member sizes simply supported.	original	0.93	5.78	-
	modified	1.02	7.12	10 %
	E.P.**	1.04	6.42	12 %
4. Three member sizes continuous	original	1.19	7.83	-
	modified.	1.30	9.26	10%
	E.P.**	1.53	10.1	28%

* Truss load capacity

** Elastoplastic

The values of P, after diagonal removal, are compared with those for the fully plastic behaviour in Table 1.3, which, theoretically, are the maximum values achievable. Truss Type 3 reached 98% of the theoretical value. The trusses Types 1, 4 and 2, reached 88%, 85% and 68% of the theoretical values respectively. For the truss Type 3, taking in account the reduction in weight due to the diagonal removal, the load/weight ratio exceeds that for the fully plastic case.

6.7 COMBINING TWO TECHNIQUES OF NONLINEAR MEMBER BEHAVIOUR AND DIAGONAL REMOVAL

The idea is to check whether or not a truss , that has been enhanced by removal of selected diagonals, can be further improved to carry more load by providing nonlinear behaviour for the members.

The same four types of large span space trusses were studied. First, using the technique described in section 6-2, selected diagonals were permanently removed from each truss. Then a nonlinear member behaviour was attributed to the members. The nonlinear behaviour was modeled by a piecewise linear load/ change-of-length curve of parameters $\alpha=0.9$ and $\beta=3$.

The analyses showed that combining the two techniques does not increase the load carrying capacity of the trusses studied, as shown in Table 6-9, except in the case of truss Type 4 for which an increase of 7% was achieved, bringing the load/weigh ratio up to 98% of the maximum given in Table 1.3.

Although the techniques of diagonal removal and using members with nonlinear behaviour both lead to nearly the same load carrying capacity, the following are some differences between the two techniques which may help in making the choice of a means of behaviour modification.

- Diagonal removal increases the load carrying capacity of the truss in a single step in the elastic range, so there will be no permanent deformation.

Table 6-9, Load/weight ratio, combining the two techniques of behaviour modification

Truss type	Status	Load/Joint (kPa)	Load/weight
	Original	1.25	2.98
1	Diag. remd	1.57	4.09
	Nonlinear	1.57	4.09
	Original	1.30	3.09
2	Diag. remd	1.84	4.85
	Nonlinear	1.84	4.85
	Original	0.93	5.78
3	Diag. remd	1.02	7.12
	Nonlinear	1.02	7.12
	Original	1.19	7.83
4	Diag. remd	1.30	9.26
	Nonlinear	1.39	9.90

- Diagonal removal decreases the weight of the structure while increasing its load carrying capacity. Reduction in weight , in certain cases, might be considered as a positive point, but in some cases where the appearance of the structure is important instead of removal of the diagonals they may be deactivated by simply providing a loose connection while leaving the original member in place.

- Diagonal removal does not prevent the problems which arise due to brittle buckling of struts and misfit.

- Nonlinear behaviour not only increases the load carrying capacity of the truss, it also diminishes the risk of brittle buckling of struts and can eliminate deleterious effects of misfit.

- Nonlinear behaviour requires attention to the detail of the joint connection, as nonlinear strain must be provided in the members in order to effect the required redistribution of forces.

6-8 CONCLUSION

It has been shown that:

- The load/weight ratio of space truss can be increased by selective diagonal removal, and a method to select the set of diagonals whose simultaneous removal gives the optimum increase in load/weight ratio of the truss has been described.
- For similar support conditions, the pattern of diagonal removal differs for different distribution of chord sizes, and when all members are the same size the pattern of diagonal removal is different for different boundary conditions.
- No simple rules for predicting the optimum pattern of diagonal removal have been found.
- Nonlinear behaviour of chord members does not, in general, further improve the load carrying capacity of a truss which has been modified using the technique of diagonal removal, however it decreases the risk of brittle failure in compression members.

CHAPTER 7

SPACE TRUSS STRUCTURAL SYSTEMS

In general, space trusses can be classified as proprietary systems and non-proprietary systems:

Patents for proprietary systems are held by many commercial firms throughout the world specialising in space trusses. The main aim is to arrive at a standardised system of construction, made of elements that can be mass produced to the required precision and that can be erected using mechanical joints.

Non-proprietary systems are custom made for each particular project. In this case truss elements and joint connectors are designed to suit the contractor and are often site welded.

7.1 PROPRIETARY SYSTEMS

Some of the proprietary systems are described below:

7.1.1) A-DECK, developed in England. The basic unit is a 2 m long lattice beam. The units are assembled alternating at right angle to give a grid of lattice beams, at 1 m centres, spanning in two directions.

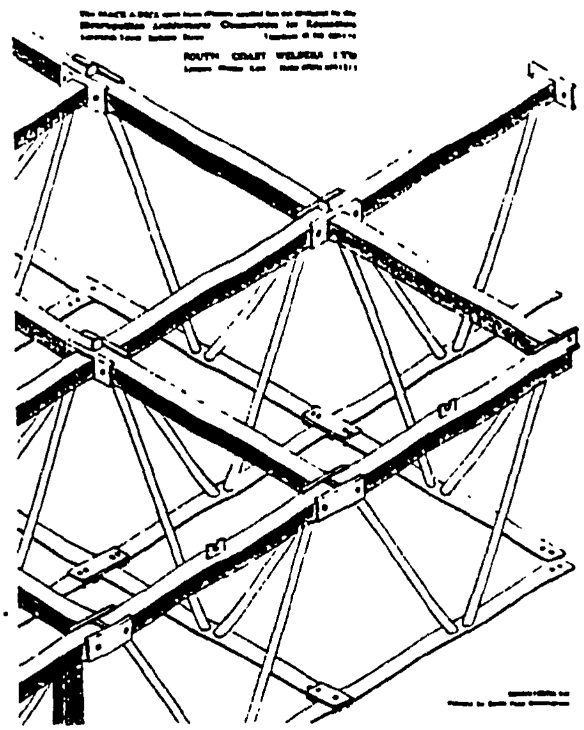


Fig. 7.1, A-DECK System

7.1.2 **HEMTEC**, developed in Canada. Round tubular members attached to spherical node piece by means of bolts passing from inside the sphere to threaded plugs held inside the tubes by means of swaging the tube wall.

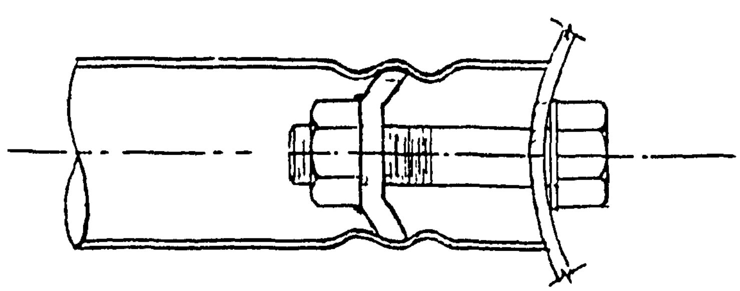


Fig. 7.2, HEMTEC System

7.1.3 **M-DEC**, developed in Canada. Continuous splayed channel, chord members with no separate node pieces. The web members bolt directly to the chords.

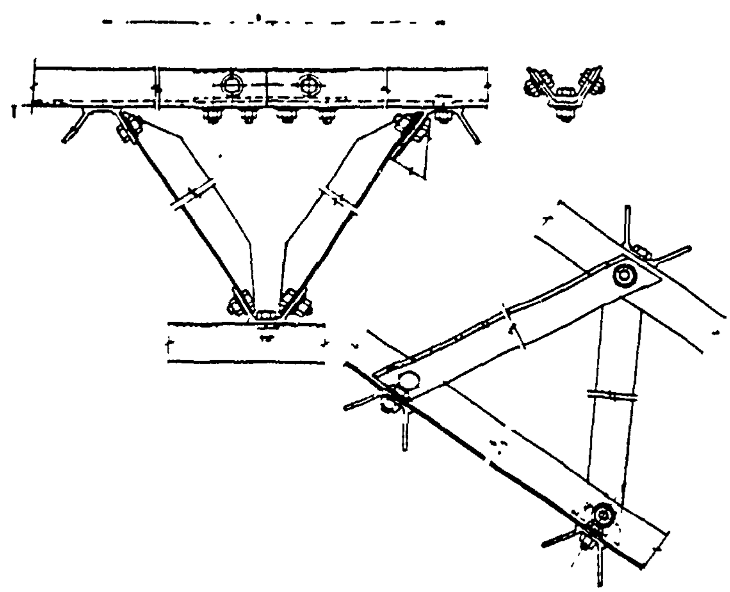
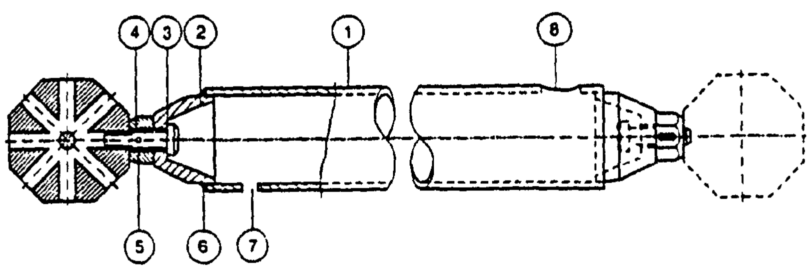


Fig. 7.3, M-DEC System

7.1.4 **MERO**, developed in Germany. The system consists of forged solid spherical shaped joint pieces, with machined facets, tapped to receive bolts. The round tubular members have tapered solid cast steel pieces welded to their ends which accommodate the high-strength connection bolts. The elements are fixed to the joint by the bolts, which are turned by a transverse pin and a tightening sleeve.



1. Circular hollow section (CHS)
2. End cone
3. Threaded bolt
4. Sleeve
5. Dowel pin
6. Welded seam
7. Drainage hole
8. Bolt insertion hole

Fig. 7.4, MERO System

7.1.5 **MODUSPAN**, developed in USA. The connector is press formed from mild steel plate into an 8-way cup with holes and shear lugs punched. The members are cold roll-formed lipped channels with holes punched in the web at each end. The members are used singly or in double configuration.

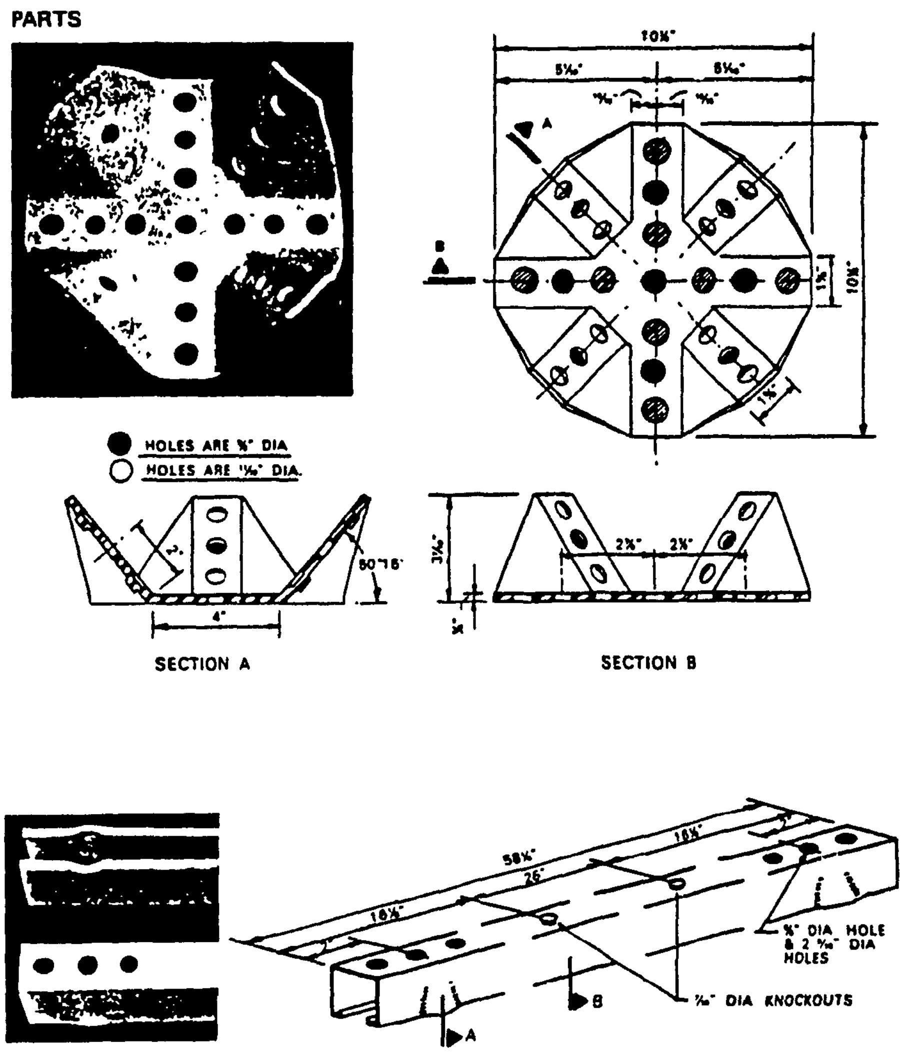


Fig. 7.5, MODUSPAN System

7.1.6 **NODUS**, developed in England. The basic joint consists of two castings. Square chord members have chord connectors, shop-welded to their ends, which fit into parallel grooves in the castings. When the chords are in place, one centre bolt completes the assembly of the joint. Round tubular diagonals have eye-connectors, shop-welded to their ends, which fit onto lugs on the casting and are held in position by steel pins.

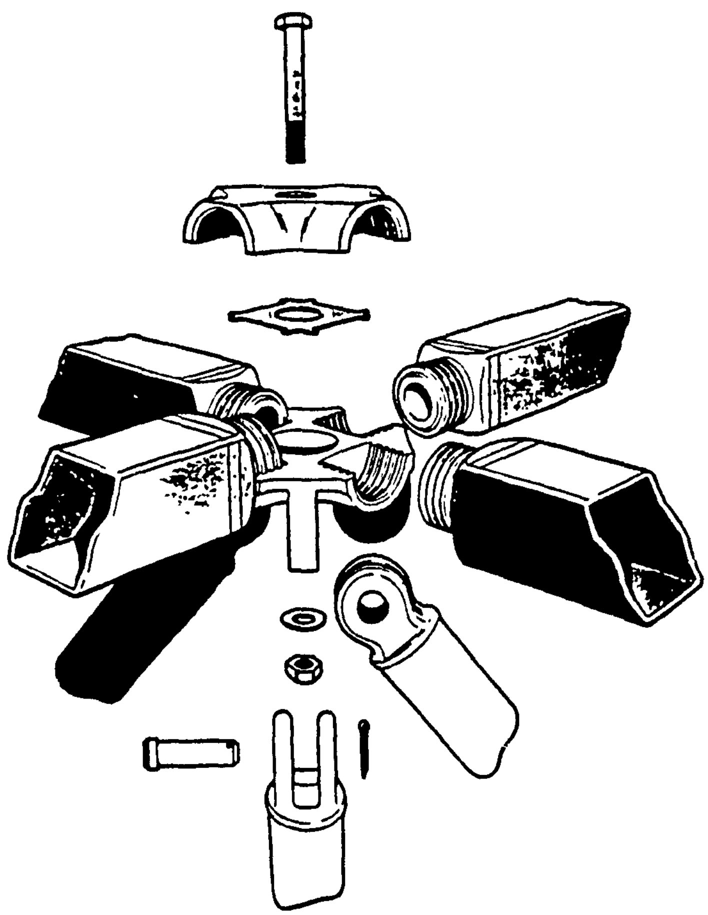


Fig. 7.6, NODUS System

7.1.7 **OCTATUBE**, developed in The Netherlands. The joint piece is an octagonal base plate with two welded ribs perpendicular to each other. The struts are pipes with flattened ends bolted to the joint piece.

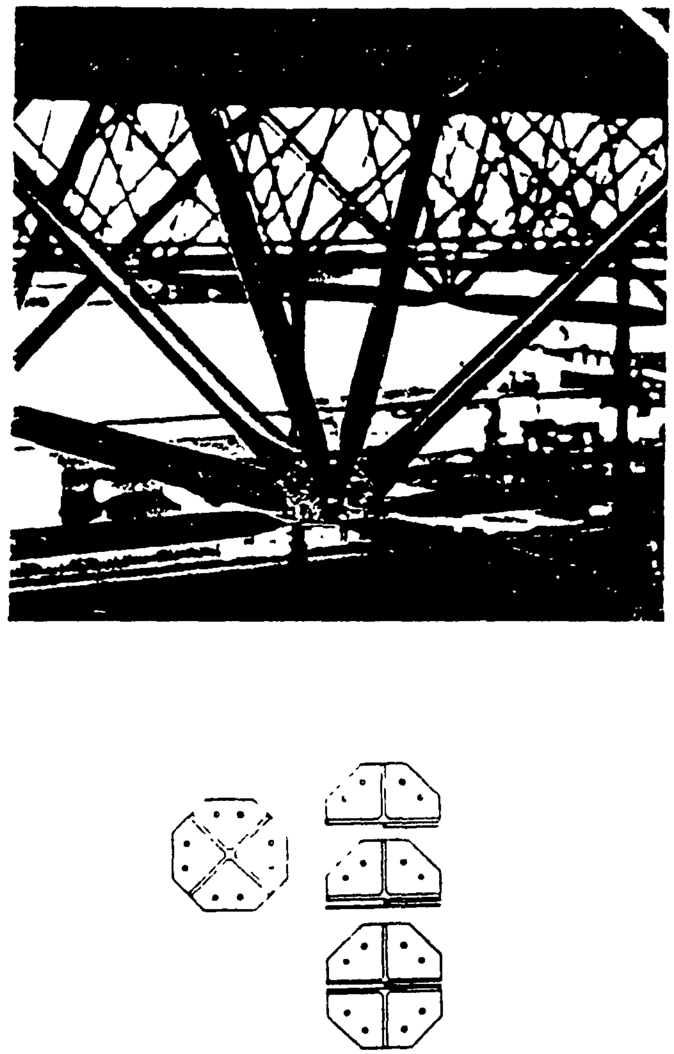


Fig. 7.7, OCTATUBE System

7.1.8 **PAVLIC**, developed in France. Uses small-sized circular steel rods. In most applications the diameter of the rods varies between 8 and 16 mm. The steel rods, which are bent at appropriate points, are interconnected by welding. Some intricate three dimensional patterns are formed.

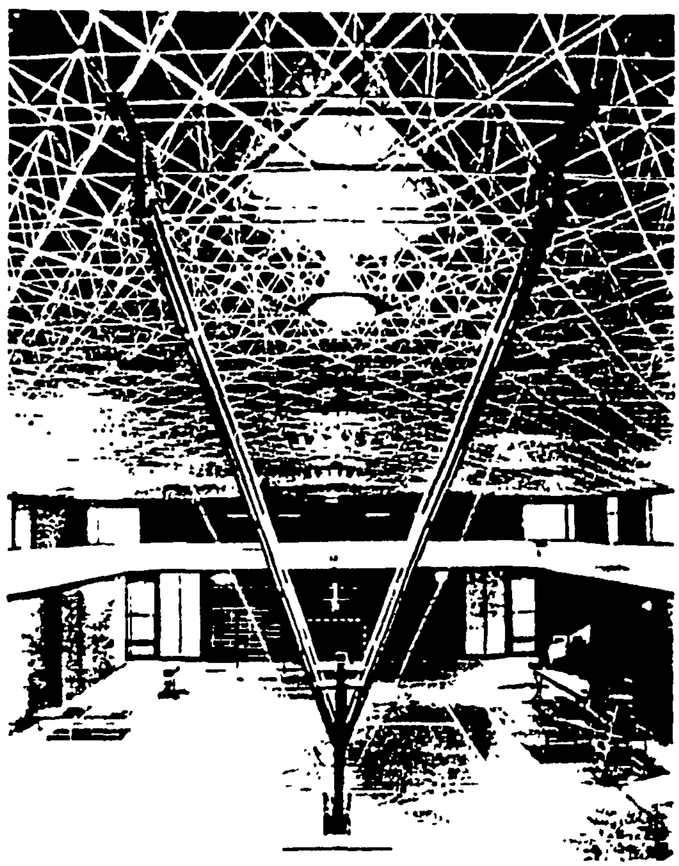
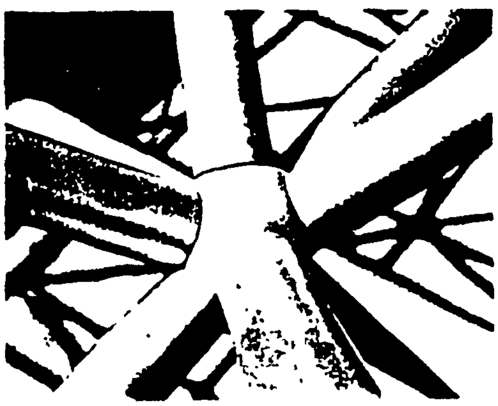
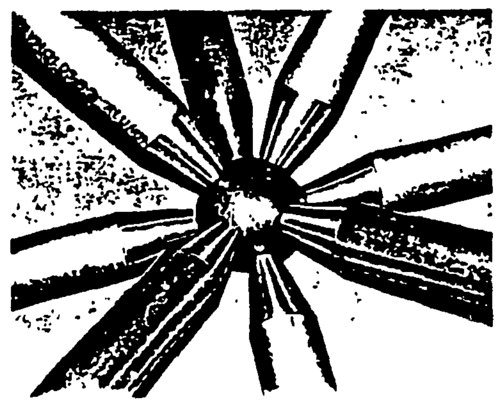


Fig. 7.8, PAVLIC System

7.1.9 **PG**, developed in The USA. Cast hollow spherical steel hubs receive tubular members with full length tension rods inside the members. A removable portion of the spherical hub provides access for placing the nuts on the ends of the steel rods.



TYPICAL PIPE SPACE FRAME JOINT



TYPICAL TAPERED PIPE JOINT

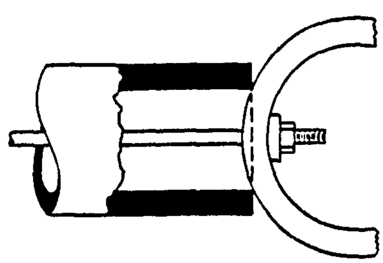


Fig. 7.9, PG System

7.1.10 **POWERSTRUT**, developed in the USA. Individual members are bolted to the Joint piece, machined from flat steel plate. The members are steel square tubing, with the web members bent as shown below:

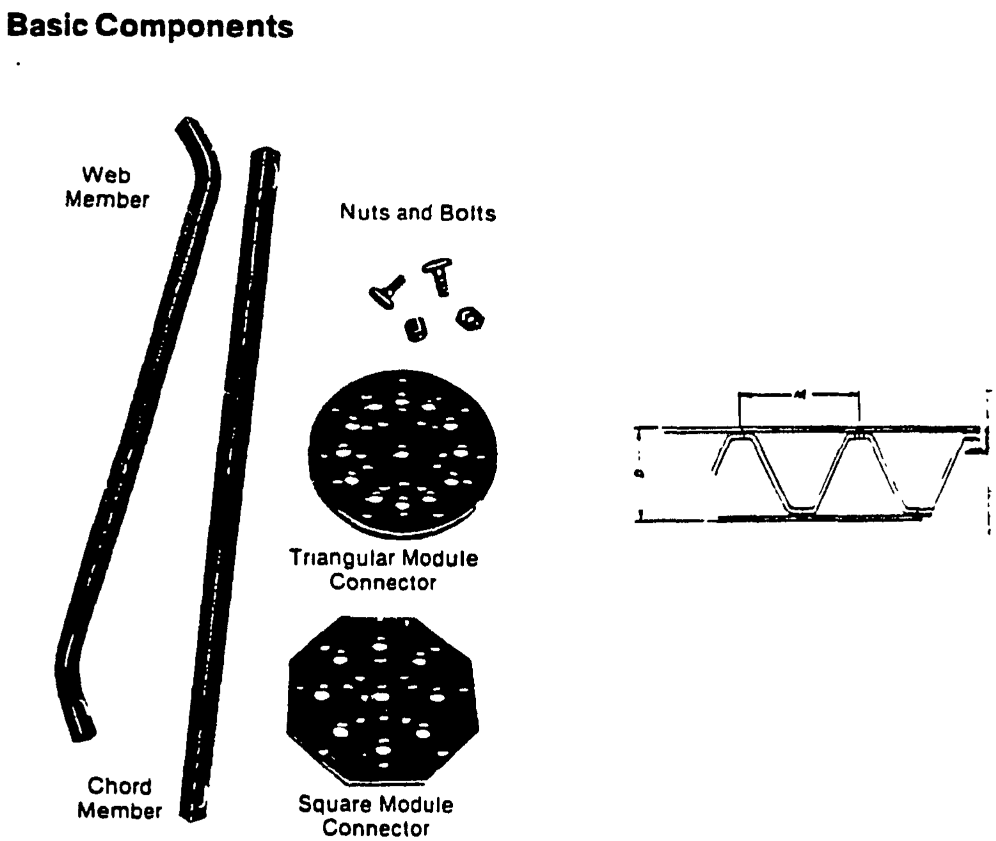


Fig. 7.10, POWERSTRUT System

7.1.11 **SDC** developed in France. A special type of connector allows the tubular members to be welded after adjustments are complete.

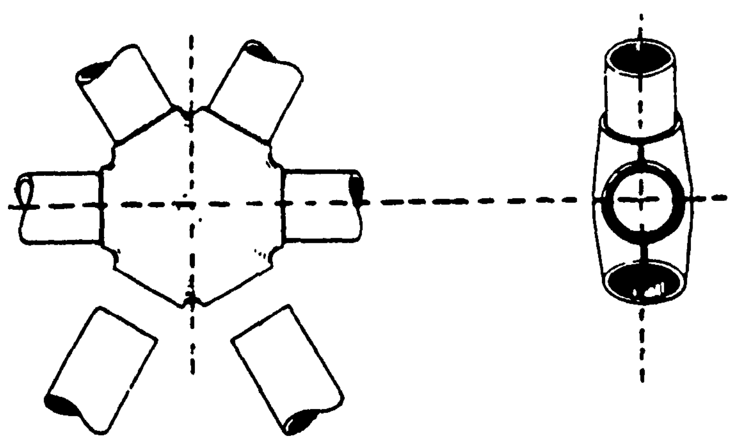


Fig. 7.11, SDC System

7.1.12 **SPACEDECK**, developed in England. Modular welded inverted pyramid assemblies as shown below. The squares of the upper layer are interconnected by bolts and nuts. The tubular sections of the lower layer are connected to the apexes of the pyramids by turnbuckles.

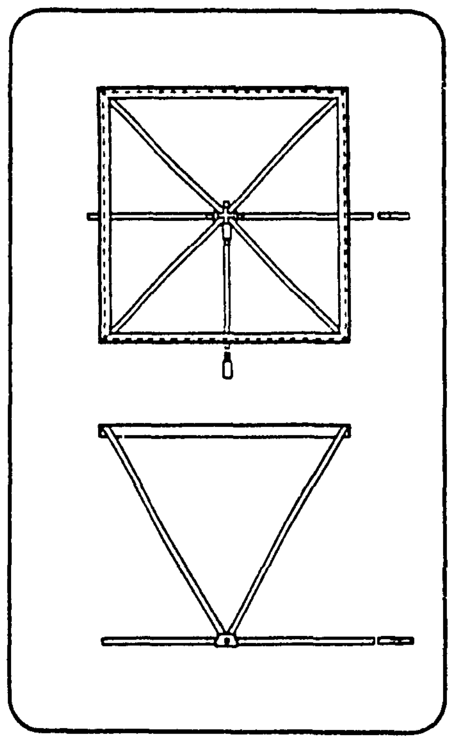


Fig. 7.12, SPACEDECK System

7.1.13 **SPHEROBAT**, developed in France. Hollow spherical forged or cast steel or aluminum nodes made in two unequal pieces. The struts have tapered end pieces, threaded internally, to receive the connection bolts which pass from the inside of the spherical node. The two halves of the node are fastened together by means of a through bolt.

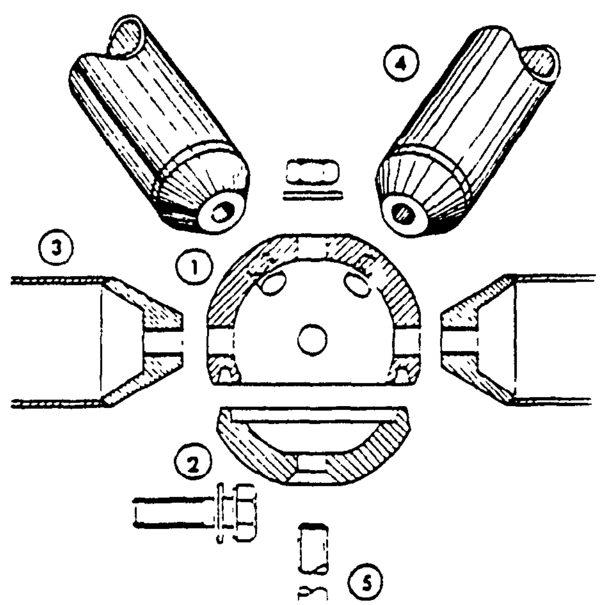


Fig. 7.13, SPHEROBAT System

7.1.14 **TRIODETIC**, developed in Canada. Grooved, extruded aluminum hubs, accept the members. The tubular members have flattened ends with transverse ribs, coined into the surface, which fit in the hub grooves and are secured by a washer held in place by means of a bolt through the hub.

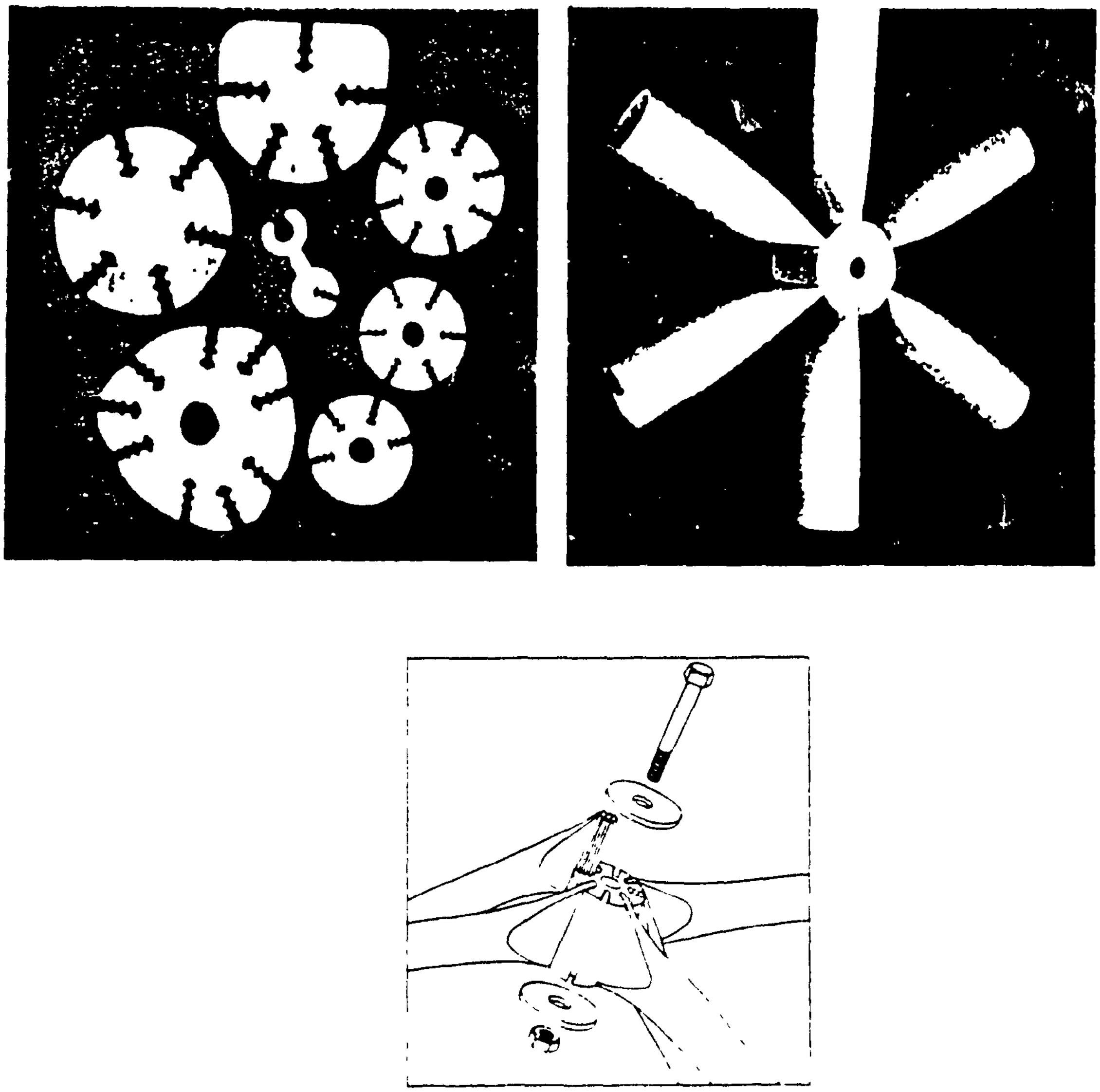


Fig. 7.14, TRIODETIC System

7.1.15 **TUBAL**, developed in The Netherlands. Spherical hollow joint connector, which is made in two unequal pieces, receives tubular members, and after securing the struts, the two pieces are connected to each other. The bolts are inserted from the sphere into the prop, which is welded to the inside of the tube.

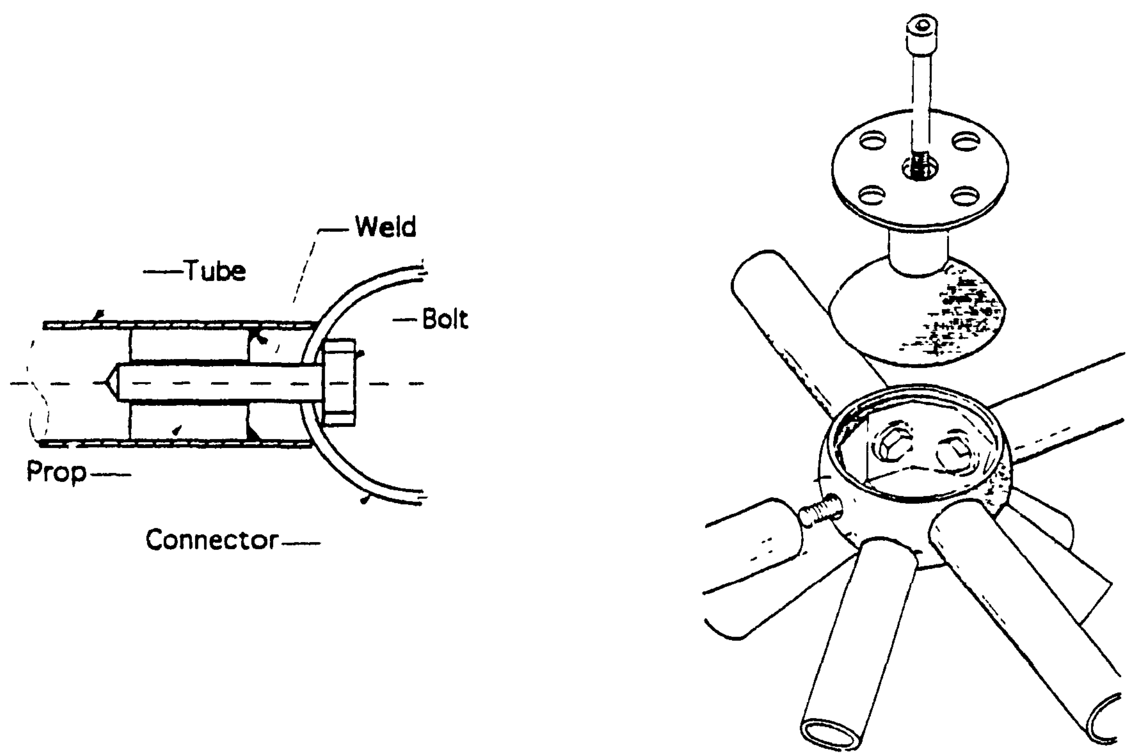


Fig. 7.15, TUBAL System

7.1.16 **UNIBAT**, developed in France. There are no separate joint components in this system. Prefabricated steel inverted pyramid modules bolted directly together in the field, using high-tensile steel bolts. The top chords are set on a diagonal grid relative to the bottom chords.

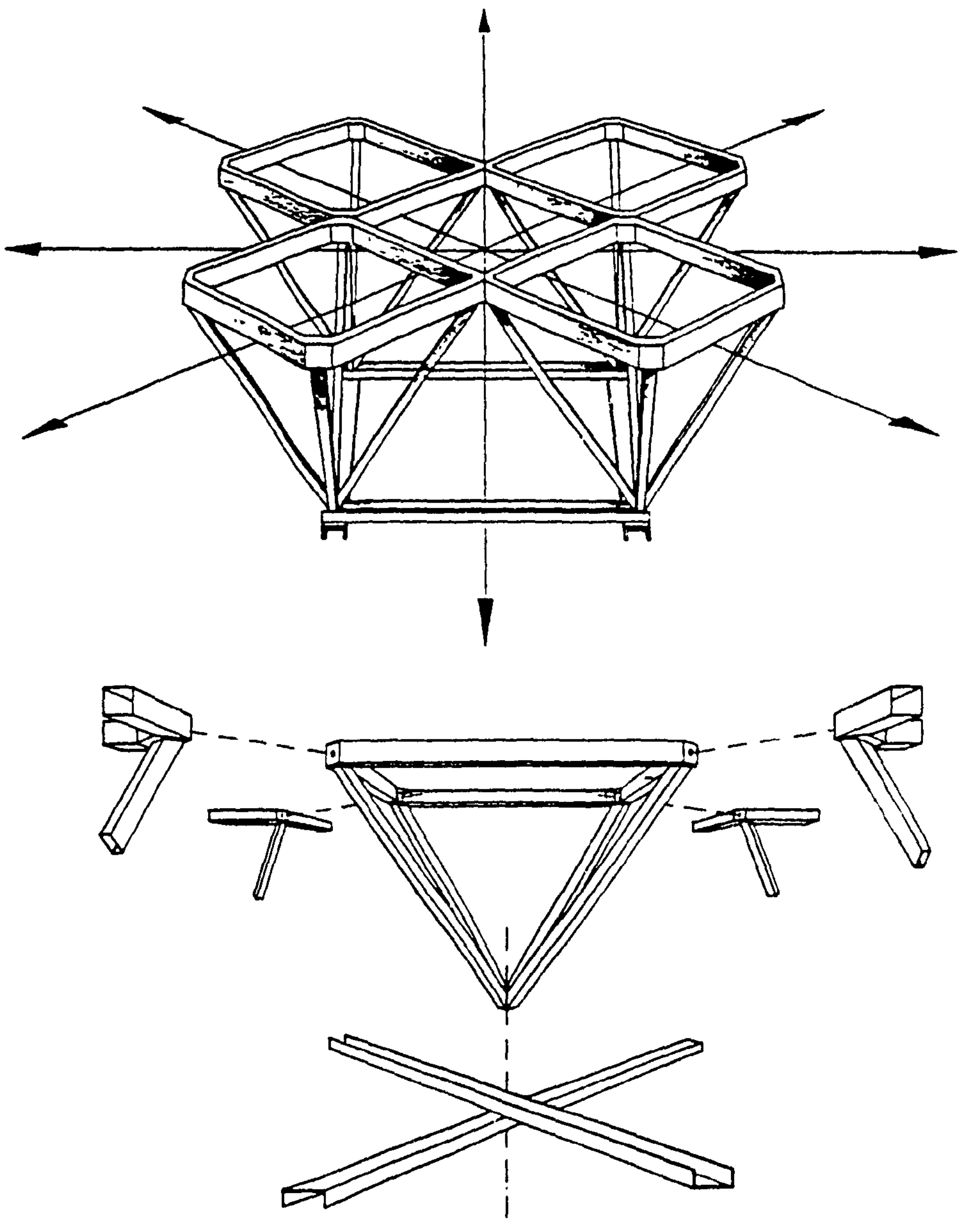


Fig. 7.16, UNIBAT System

7.2 SUITABLE BEHAVIOUR MODIFICATION TECHNIQUES

Certain behaviour modification techniques are best suited to some of the existing space truss systems, while, others may be practically impossible to implement or economically unacceptable. Table 7.1 shows the proprietary systems, described in the section 7.1, that are suitable to the techniques described in chapter 3 and chapter 4.

Table 7.1, Suitable techniques

	FLD	OD	NL	PR	DR	NLD
A-DECK						
HEMTEC	●		●		●	
M-DEC			●		●	●
MERO				●	●	
MODUSPAN					●	●
NODUS					●	
OCTATUBE					●	
PAVLIC						
PG		●			●	
POWERSTRUT					●	●
SDC					●	
SPACEDECK		●				
SPHEROBAT					●	
TRIODETIC					●	
TUBAL					●	
UNIBAT		●				

FLD = Force limiting device
OD = Over-design of compression members
NL = Nonlinear member behaviour
PR = Prestressing
DR = Diagonal removal
NLD= Nonlinear diagonals

CHAPTER 8

CONCLUSION

8.1 SUMMARY OF INFLUENCE OF THE BEHAVIOUR MODIFICATION TECHNIQUES

It can be observed from the Table 8.1 that the increase in capacity that can be effected by the techniques studied, as is to be expected, is the greatest in trusses with uniform member sizes.

- Of interest here is that by the removal of diagonals the capacity can be increased to match that achieved by nonlinear behaviour, but well short of "elastic-fully plastic" condition.

- For trusses with different member sizes, the increase in capacity is not so dramatic, and both techniques have comparable influence.

- By combining the two techniques there is little or no increase in the capacity, but as the ultimate loads are close to those for fully plastic condition, there is evidently little reserve capacity available to be exploited.

- Removing diagonals, or "disengaging" them if their presence is needed for appearance, leads to an optimum arrangement that finally requires only a single cycle of elastic analysis to determine the ultimate load. Up to the ultimate load the truss will behave in a linear elastic manner, and at the ultimate load it will behave in a brittle manner typical of lattice structures.

- If nonlinear member behaviour is introduced, an iterative analysis is required to establish the ultimate capacity but the ductile characteristics reduce the risk of brittle failure.

Table 8.1, Capacity and Load/Weight ratio

Truss	Status	Load, kPa	Load/Weight
Type 1	Original	1.25	2.98
	Diag. rem'd	1.57	4.09
	Nonlinear	1.60	3.80
	Combination	1.57	4.09
	Elastoplastic	1.78	4.24
Type 2	Original	1.30	3.09
	Diag. rem'd	1.84	4.85
	Nonlinear	1.72	4.10
	Combination	1.84	4.85
	Elastoplastic	2.72	6.50
Type 3	Original	0.93	5.78
	Diag. rem'd	1.02	7.12
	Nonlinear	1.02	6.30
	Combination	1.02	7.12
	Elastoplastic	1.04	6.42
Type 4	Original	1.19	7.83
	Diag. rem'd	1.30	9.26
	Nonlinear	1.24	8.16
	Combination	1.39	9.90
	Elastoplastic	1.53	10.1

8.2 FURTHER STUDIES

Space trusses, being highly redundant are very sensitive to lack of fit, (members being longer or shorter than size required by geometry). With members numbering in the thousands, which is typical for a large space truss the likelihood of such shortcomings are high. This is one reason why analytical predictions often exceed the test results. For example, in a joint where four diagonals are under a vertical load, the assumption is that they share the force equally, but if one of them is longer than the others it will take more than its share of the load. If this leads to overloading of the member, and it fails in brittle manner it may cause a chain reaction of failures leading to the collapse of the structure. This is the reason why most of the researchers recommend that there should be a nonlinear behaviour in the truss members to allow for redistribution of forces. The amount of nonlinearity required to increase the capacity of an ideal truss was the subject of the present study which showed that increase can be achieved in the load carrying capacity.

The next question is whether the designer should count on this increase in capacity and thereby reduce the member sizes in an attempt to save the cost of material. The answer to this question depends on how the imperfections and lack of fit affect the load carrying capacity of the truss. The amount of lack of fit or imperfection cannot be measured on site because the design precedes the construction, so it should be based on statistical analysis of the manufacturing process.

- Tests should be performed on truss elements to establish the design models and the reliability of their performance.

- Tests should be performed on full size trusses to examine the validity of the computer analyses, which used the design models derived in section 1, to predict the behaviour of the truss. These tests should be large enough to represent the truss behaviour statistically.

- Diagonal removal was shown to be a means of channelling load from overloaded members towards the less loaded ones and it was shown that the load/weight ratio increased considerably. So far no tests or practical application of this idea has been reported. Whether or not diagonal removal adversely influences the stability of the joints should be established by tests.

- Selection of material, topology, module size, number of member sizes, ratio of different sizes, roof covering, and above all the structural systems, which are so diversified, require in depth study and documentation.

- Economy of design also needs a thorough research in order to optimize overall cost consisting of the main structural elements, joint pieces, roofing and facing from the point of view of material, production, supervision and assembling.

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APPENDIX 'A', LOAD SHORTENING RELASHIONSHIP FOR AXIALLY LOADED MEMBER

The shortening of a strut under axially applied load for the elastic behaviour has two components:

1. Shortening due to axial strain;

$\delta_1=PL/AE$

where: P= applied load

L= length of the strut

A= area

E= modulus of elasticity

2. Shortening due to the bow;

using $y=\Delta\sin(\pi x/L)$

where Δ= central deflection

$y'=(\pi/L)\Delta\cos(\pi x/L)$

gives $\delta_2=(\pi^2/4L)\Delta^2$

$\delta=\delta_1+\delta_2=(\sigma L/E) + (\pi\Delta/2L)^2 \; (L)$

Letting $\sigma=P/A$ and $\sigma_e = P_e/A$ where $P_e = (\pi^2 EA)/(L/r)^2$

effective axial strain; $\varepsilon = \delta/L = (\sigma/E) + (\pi\Delta/2L)^2$ (1)

At the first yield

$(P/A) + (P\Delta c/I) = \sigma_y$ where $I = Ar^2$

gives $\Delta = [(\sigma_y / \sigma) - 1)](r^2/c)$

for the purpose of illustration let $r = c$

then: $\varepsilon(E/\sigma y) = (\sigma / \sigma_y) + (\sigma_e / 4\sigma_y)(\sigma_y/\sigma - 1)^2$

The relationship between σ / σ_y and $\varepsilon(E/\sigma_y)$ for various values of σ_e/σ_y is given in Fig. 1.3.

Appendix B

```
****** PROGRAM NUM

****** This program numbers the joints of rectangular double layer
****** space truss, number of bays in X and Y directions are the
****** only data to be entered.

      DIMENSION K(50,50)
      OPEN(5,FILE='OUTPUT',STATUS='NEW')
      WRITE(*,*)'ENTER  NUMBER OF  BAYS IN X  AND Y  DIRECTIONS,'
      WRITE(*,*)'NUMBER OF BAYS IN Y DIRECTION NOT MORE THAN TEN'
      READ(*,*)M,N
      WRITE(5,1)M,N
 1    FORMAT(1X,I2,' BY',I2,'  SPACE TRUSS')
      WRITE(5,*)' '
      WRITE(5,*)' PLAN VIEW'
      WRITE(5,*)' '
      WRITE(5,*)' '
      I=1
      J=2
      NUM=1
      K(I,J)=1
 10   J=J+2
      NUM=NUM+1
      K(I,J)=NUM
      IF(NUM.LT.N)GOTO 10
      J=-1
 20   J=J+2
      NUM=NUM+1
      K(I,J)=NUM
      IF(NUM.LT.2*N+1)GOTO 20
      GOTO(501,502,503,504,505,506,507,508,509,510), N
 501  CONTINUE
 502  WRITE(5,512)  (K(I,J),J=1,2*N+1)
 512  FORMAT(2X,5(I1,'---'),'Y')
      GOTO 31
 503  WRITE(5,513)  (K(I,J),J=1,2*N+1)
 513  FORMAT(2X,7(I1,'---'),'Y')
      GOTO 31
 504  WRITE(5,514)  (K(I,J),J=1,2*N+1)
 514  FORMAT(2X,9(I1,'---'),'Y')
      GOTO 31
 505  WRITE(5,515)  (K(I,J),J=1,2*N+1)
 515  FORMAT(1X,11(I2,'--'),'Y')
      GOTO 31
 506  WRITE(5,516)  (K(I,J),J=1,2*N+1)
 516  FORMAT(1X,13(I2,'--'),'Y')
      GOTO 31
 507  WRITE(5,517)  (K(I,J),J=1,2*N+1)
 517  FORMAT(1X,15(I2,'--'),'Y')
      GOTO 31
 508  WRITE(5,518)  (K(I,J),J=1,2*N+1)
```

```
518   FORMAT(1X,17(I2,'--'),'Y')
      GOTO 31
509   WRITE(5,519) (K(I,J),J=1,2*N+1)

519   FORMAT(1X,19(I2,'--'),'Y')
      GOTO 31
510   WRITE(5,520) (K(I,J),J=1,2*N+1)
520   FORMAT(1X,21(I2,'--'),'Y')
31    CONTINUE
      GOTO(201,202,203,204,205,206,207,208,209,210),N
201   CONTINUE
202   WRITE(5,212)
212   FORMAT('  ',2('| \ | / '),'|')
      GOTO 40
203   WRITE(5,213)
213   FORMAT('  ',3('| \ | / '),'|')
      GOTO 40
204   WRITE(5,214)
214   FORMAT('  ',4('| \ | / '),'|')
      GOTO 40
205   WRITE(5,215)
215   FORMAT('  ',5('| \ | / '),'|')
      GOTO 40
206   WRITE(5,216)
216   FORMAT('  ',6('| \ | / '),'|')
      GOTO 40
207   WRITE(5,217)
217   FORMAT('  ',7('| \ | / '),'|')
      GOTO 40
208   WRITE(5,218)
218   FORMAT('  ',8('| \ | / '),'|')
      GOTO 40
209   WRITE(5,219)
219   FORMAT('  ',9('| \ | / '),'|')
      GOTO 40
210   WRITE(5,220)
220   FORMAT('  ',10('| \ | / '),'|')
 40   I=I+1
      J=1
      NUM=NUM+1
      K(I,J)=NUM
      J=J+1
      NUM=NUM+1
      K(I,J)=NUM
50    J=J+2
      NUM=NUM+1
      K(I,J)=NUM
      IF(J.LT.2*N)GOTO 50
      J=J+1
      NUM=NUM+1
      K(I,J)=NUM
      GOTO(601,602,603,604,605,606,607,608,609,610),N
601   CONTINUE
```

```
602 WRITE(5,612) k(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
612  FORMAT(1X,I2,'--',1(I2,'---+--'),I2,'--',I2)
     GOTO 400


603 WRITE(5,613) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
613  FORMAT(1X,I2,'--',2(I2,'---+--'),I2,'--',I2)
     GOTO 400
604 WRITE(5,614) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
614  FORMAT(1X,I2,'--',3(I2,'---+--'),I2,'--',I2)
     GOTO 400
605 WRITE(5,615) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
615  FORMAT(1X,I2,'--',4(I2,'---+--'),I2,'--',I2)
     GOTO 400
606 WRITE(5,616) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
616  FORMAT(1X,I2,'--',5(I2,'---+--'),I2,'--',I2)
     GOTO 400
607 WRITE(5,617) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
617  FORMAT(I3,'-',6(I3,'---+-'),I3,'-',I3)
     GOTO 400
608 WRITE(5,618) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
618  FORMAT(I3,'-',7(I3,'---+-'),I3,'-',I3)
     GOTO 400
609 WRITE(5,619) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
619  FORMAT(I3,'-',8(I3,'---+-'),I3,'-',I3)
     GOTO 400
610 WRITE(5,620) K(I,1),(K(I,J),J=2,2*N-2,2),K(I,2*N),K(I,2*N+1)
620  FORMAT(I3,'-',9(I3,'---+-'),I3,'-',I3)
400  CONTINUE
     GOTO(401,402,403,404,405,406,407,408,409,410),N
401  CONTINUE
402  WRITE(5,412)
412  FORMAT('  ',2('| / | \ '),'|')
     GOTO 500
403  WRITE(5,413)
413  FORMAT('  ',3('| / | \ '),'|')
     GOTO 500
404  WRITE(5,414)
414  FORMAT('  ',4('| / | \ '),'|')
     GOTO 500
405  WRITE(5,415)
415  FORMAT('  ',5('| / | \ '),'|')
     GOTO 500
406  WRITE(5,416)
416  FORMAT('  ',6('| / | \ '),'|')
     GOTO 500
407  WRITE(5,417)
417  FORMAT('  ',7('| / | \ '),'|')
     GOTO 500
408  WRITE(5,418)
418  FORMAT('  ',8('| / | \ '),'|')
     GOTO 500
```

```
409   WRITE(5,419)
419   FORMAT('   ',9('| / | \ '),'|')
      GOTO 500
410   WRITE(5,420)
420   FORMAT('   ',10('| / | \ '),'|')
500   IF(I.EQ.2*M)GOTO 90
65    I=I+1
      J=1
      NUM=NUM+1
      K(I,J)=NUM
70    J=J+2
      NUM=NUM+1
      K(I,J)=NUM
      IF(J.LT.2*N)GOTO 70
      GOTO(701,702, 703,704,705,706,707,708,709,710) N
701   CONTINUE
702   WRITE(5,712)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
712   FORMAT(1X,2(I2,'---+--'),I2)
      GOTO 82
703   WRITE(5,713)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
713   FORMAT(1X,3(I2,'---+--'),I2)
      GOTO 82
704   WRITE(5,714)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
714   FORMAT(1X,4(I2,'---+--'),I2)
      GOTO 82
705   WRITE(5,715)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
715   FORMAT(1X,5(I2,'---+--'),I2)
      GOTO 82
706   WRITE(5,716)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
716   FORMAT(1X,6(I2,'---+--'),I2)
      GOTO 82
707   WRITE(5,717)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
717   FORMAT(7(I3,'---+-'),I3)
      GOTO 82
708   WRITE(5,718)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
718   FORMAT(8(I3,'---+-'),I3)
      GOTO 82
709   WRITE(5,719)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
719   FORMAT(9(I3,'---+-'),I3)
      GOTO 82
710   WRITE(5,720)(K(I,J),J=1,2*N-1,2),K(I,2*N+1)
720   FORMAT(10(I3,'---+-'),I3)
82    IF(I.EQ.2*M)GOTO 90
      GOTO 31
90    I=I+1
      J=-1
100   J=J+2
      NUM=NUM+1
      K(I,J)=NUM
      IF(J.LT.2*N)GOTO 100
      J=0
110   J=J+2
      NUM=NUM+1
```

```
      K(I,J)=NUM
      IF(J.LT.2*N)GOTO 110
      IF(NUM.GT.100)GOTO 2
      WRITE(5,120)(K(I,J),J=1,2*N+1)
      GOTO 129
 2    WRITE(5,121)(K(I,J),J=1,2*N+1)
 120  FORMAT(1X,21(I2,'--'))
 121  FORMAT(21(I3,'-'))
 129  WRITE(5,130)
 130  FORMAT('  ','|')
      WRITE(5,140)
 140  FORMAT('  ','X')
      END


*** PROGRAM DIAGEN
*** This program generates diagonals for removal
*** N is the number of bays of square truss, j contains the joints
*** of top layer, in the first column the joints of upper triangle
*** and in the second column their symmetric pairs. The joints
*** which are on the main diagonal are paired with a 0.

       Integer j(10,2), N , N1 , N2, k1, k2, k3, k4
       open ( unit=20, file='diarmv', status='new')
       data j/11,12,13,14,23,24,25,35,36,47,0,22,33,44,0,34,
      + 45,0,46,0/
       data N/4/
       N1 = N+1
       N2 = N+2
       DO 10 i=1,10
       j1=j(i,1)-N2
       j2=j(i,1)-N1
       j3=j(i,1)+N1
       j4=j(i,1)+N2
       k1=j(i,2)-N2
       k2=j(i,2)-N1
       k3=j(i,2)+N1
       k4=j(i,2)+N2
       if(j(i,2).eq.0) then
          write(20,20) j1,j(i,1)
       write(20,40)
          write(20,30) j2,j(i,1),j(i,1),j3
          write(20,40)
       write(20,20) j(i,1),j4
       write(20,40)
        else
       write(20,30) j1,j(i,1),k1,j(i,2)
       write(20,40)
       write(20,30) j2,j(i,1),j(i,2),k3
       write(20,40)
       write(20,30) j(i,1),j3,k2,j(i,2)
       write(20,40)
       write(20,30) j(i,1),j4,j(i,2),k4
```

```
      write(20,40)
      endif
 10   continue
 20   format (i3,',',i3)
 30   format (i3,',',i3,/,i3,',',i3)
 40   format ('#')
      end

**** SUBROUTINE TESTR
**** This subroutine runs "SPAN" and gives the truss load capacity
**** at each step after removal of each set of diagonals
echo "Output from SPAN05" > span05.output
exec 3<diarmv
exec 0<&3
until [ "$LINE" = "END" ]
do
echo "'old'" > span05.input
echo "'mem'" >> span05.input
echo "'o'" >> span05.input
read LINE
echo $LINE >> span05.input
echo "" >> span05.output
echo "****************************************" >> span05.output
echo "Member to be removed $LINE " >> span05.output
read LINE
if [ "$LINE" != "#" -a "$LINE" != "END" ]
then
  echo $LINE >> span05.input
  echo "                    $LINE " >> span05.output
  read LINE
fi
echo "-1" >> span05.input
echo "-1" >> span05.input
echo "'bkpt'" >> span05.input
echo "1" >> span05.input
echo "'stop'" >> span05.input
span05 < span05.input >> span05.output
done
echo "END" >> span05.output

*** Program EXT
*** This program extracts the required data from the output of
***"SPAN"
     program ext
     character*255 line
        character*40  result
     logical omit
        result = ' '
     open (unit=1,file='span05.output',status='old')
        omit = .TRUE.
     do while (.TRUE.)
       read (1,'(a)',end=32767) line
       if (line(1:20).eq.'Member to be removed') then
```

```
            result(11:11)=':'
         result(22:22)=':'
            result(40:40)=':'
            print *,result
         result =line(21:30)
            omit = .TRUE.
            read (1,'(a)',end=32767) line
            if (line(1:20).eq.'                    ') then
              result(12:22)= line(21:30)
           endif
            endif
       if (line(2:11) .eq. '*** MEMBER' .and. omit) then
            result(24:40)=line(45:60)
            omit=.false.
            endif
      end do
32767     continue
      close (unit=1)
      end
```