CHAPTER 2
LITERATURE REVIEW

Literature on the behaviour of transmission line towers, related to this study, may be divided into four major categories. The first category relates to modern methods of mathematical analysis and design of power transmission line towers, the second one relates to the behaviour of an angle section as beam and beam-column, the third one deals with model analysis and testing of transmission line towers and the fourth one deals with prototype tower and segment tests.

The literature pertaining to the above four categories is presented and reviewed in the next few sections.

2.1 LITERATURE RELATED TO MODERN METHODS OF MATHEMATICAL ANALYSIS AND DESIGN


The behaviour of nonlinear space truss members were described by Smith [5]. The interaction between a space member and the adjoining structure was detailed by him and the "Chordal Snap Through" phenomenon was also described. He employed a nonlinear stepwise linearization analysis that did not require repeated updating of the structural stiffness matrix.

Kassimall [6] presented a numerical procedure for large deformation analysis of elastic-plastic frames. The procedure was based on an Eulerian formulation. Local member force deformation relationships were based on the beam-column approach and changes in member chord lengths due to axial strains and flexural bowing were taken into account.
An analysis program for the guyed towers that had been in vogue for the last few years was developed by King [7]. The program uses a large scale sparse matrix technique to solve a linear three dimensional truss problem with multiple load cases. The forces and displacements from the unit load cases are used as influence coefficients to solve for the nonlinear members. Among the special features of the program are a preprocessor to check input data against the prototype and plot views of the structure and a built-in recording device which allows the user to specify the data in an arbitrary order but still obtain an efficient solution.

The current version of SAP IV (Structural Analysis Program IV) for static and dynamic analysis of linear structural systems developed by Bathe et.al [8] was the result of several years of research and development experience. The program had proved to be a very flexible and efficient tool. Coded in FORTRAN IV, it operates without modifications on the CDC 6400, 6600 and 7600 Computers. The first version of program SAP was published in September, 1970. An improved static analysis program, namely SOLID SAP, or SAP II was presented in 1971. The program SAP III for static and dynamic analyses was released towards the end of 1972. In relation to SAP III the current version SAP IV is refined and has additional features like a new variable number nodes thick shell element, a three dimensional element and out of core direct integration for time history analysis.

The execution times of the Modified Stiffness Matrix Method and the Modified Load Vector Method were compared by Rossow et.al [9]. The Modified Load Vector method was found to be clearly superior. Much of the cost saving in using Modified Load Vector method was related to the simultaneous treatment of all loading cases and its rapid convergence especially in design analysis cycles.

Lo et.al [10] developed a TRANTOWER Program which had been extensively used by Sargent and Lundy to design and study transmission towers and space truss problems. The features of the Program are as follows:
1. Simplicity of input.

2. Tower generation capability - The program can generate symmetrical nodes and members. It is possible to define the complete tower by as few as one-fourth of the total nodes and members.

3. Convergence to the final design within a reasonable number of cycles regardless of the initial member sizes - This relieves the user from the burden of making estimates of member sizes.

4. Treatment of 'tension only' members - Conventionally, 'tension only' members are removed from the frame if there is any compressive force in them. This is contrary to the actual structural behaviour because these members do have some load carrying capacity even for compressive forces. The program allows these members to carry loads up to their capacity in compression and distributes the rest of the load to other members.

5. Simplicity of design - A number of tower configurations can be designed for optimization of the tower weight by simply changing the co-ordinate of the key nodes of the tower.

Analytical investigation for computing static and dynamic longitudinal loads on the tower was carried out by Mozer [11]. A computer program (BRODI 1) was developed to calculate the steady state longitudinal loads and their displacements.

Siddiqui and Fleming [12] presented a general procedure for computing the dynamic response of transmission line systems when one or more wires suddenly break. A computer program for performing a time history analysis of a nonlinear system had been developed. The program can compute the wire tensions, the arm loads and the ground line moments at the base of the support structures. A comparison was made between the computed results and experimental results of a small scale laboratory model.

A Computer Program DESCAL was developed by Peyrot and Dagher [13] to evaluate the reliability of transmission line designed according to any criterion and in particular to the NESC or LRFD criteria. Results
of reliability study conducted over a wide variety of components and structure types indicated that LRFD produces more consistent designs.

Bhattacharya and Lakshapati [14] also had enumerated the advantages of computer based analysis and design procedures.

An optimization program for the analysis and design of transmission line towers was developed by Selvanathan and Janardan [15]. Selvanathan [16] presented also a procedure for selecting the economical angle section with connection details for different types of transmission line tower members. Design of members without any repeated computations of compression and tension capacities were illustrated with sample designs. Design tables were prepared which help a designer.

Ahmad, Pande and Prem Krishna [17] studied analytically and experimentally the along-wind and across-wind response of self-supporting reinforced concrete and latticed steel towers. The analytical approach utilized existing methods of analysis with certain modifications, while experimental study was carried out on aeroelastic models placed in uniform as well as shear flow fields generated in a medium size close-circuit wind tunnel. Results obtained were compared with measured prototype data and the comparison was found to be satisfactory. Experimental results did further indicate that the towers can be modelled adequately with respect to their mass and stiffness distribution without adding discrete masses. However for prediction of along-wind response, these models cannot be relied upon unless they are tested in a boundary layer flow with adequate turbulence intensities.

TAPS [18] (Tower Analysis Program System) was developed by the Structural Engineering Research Centre, Madras, India. It is a versatile finite element software package for nonlinear static, dynamic and buckling analysis of guyed towers. It is written in FORTRAN and currently runs on PRIME 750 and IBM-PC Compatible configurations. It takes into account effects due to large deflections and rotations. The solution is obtained by using non-linear finite element iterative technique. TAPS has versatile
built-in load specification statements. It accepts wind load as a uniformly
distributed load or as pressure acting directly on the guys and mast. Typical
output includes an echo of input data, nodal displacements, member resisting
forces at nodes, axial forces and guy tensions.

2.2 **LITERATURE RELATED TO ANGLE SECTIONS AS BEAM AND BEAM - COLUMN**

2.2.1 Theoretical investigations

2.2.1.1 Flexural Buckling

The theory of elastic buckling of eccentrically loaded columns
was first formulated by Euler [19]. Ostenfeld [20], Karman [21],
Ros [22] and Young [23] were the earliest investigators who studied the
problem of eccentrically loaded columns. In his two papers, Jezek [24] presented
an approximate and simple-to-use method for estimating the flexural buckling
load of eccentrically loaded columns. Bleich [25] too had developed simple
algebraic formulae for the design of eccentrically loaded columns which
fail by flexural buckling about the weak axis. Differential equations of
equilibrium for the general case of biaxial eccentricities had been solved
by Vlasov [26], Thurliman [27], and Pekoz and Winter [28], using different
solution procedures. A simple method to obtain the exact interaction
relationship of general sections composed of rectangular elements that meet
each other at right angles was presented by Chen and Atsuta [29]. The
method is applicable to equal angles, unequal angles and built-up angles
and is extremely powerful and efficient for computer application.

2.2.1.2 Torsional Flexural Buckling

Wagner [30] was the first to investigate the problem of torsional
buckling of thin-walled open sections. However, in his theory, Wagner
assumed arbitrarily that the centre of rotation coincided with the shear
centre, which, in general would not be the case. The exact solution for
torsional flexural buckling of angle sections was first presented by
Ostenfeld [31]. Bleich and Bleich [32], Kappus [33], Goodier [34] and
Timoshenko [35] were some of the early investigators who studied the
problem of torsional flexural buckling. More accurate expressions for the torsional constant of hot-rolled angle sections were developed by El Darwish and Johnston [36]. An equivalent radius of gyration for torsional flexural buckling would simplify the computation of torsional flexural buckling load. Hence, to aid designers, equivalent slenderness ratios of angle sections rolled according to Czechoslovak standards were tabulated by Mrazik and Sadovsky [37] for both equal leg and unequal leg angles under simply supported end conditions.

2.2.1.3 Local Plate Buckling

While there is no distinction between torsional-flexural buckling and local plate buckling for single equal-leg angle members, for unequal-leg angles and built-up angles, torsional flexural buckling is distinctly different from local plate buckling. Bijlaard [38] studied the problem of local buckling of angles both in the elastic and inelastic region and derived design formulae for steel angles. The inelastic buckling of a cruciform column was investigated by Hutchinson and Budiansky [39] by a combination of analytical and numerical methods.

2.2.1.4 Interaction Between Torsional-Flexural Buckling and Plate Buckling

Goldberg, Bogdanoff and Glauz [40] developed a method for determining critical loads for thin walled members taking into account the effect of deformation of the cross section in its own plane. Studnicka [41] used the theory of folded plates to treat the problem of stability of a member cross section. In the parametric studies for cruciform and angle sections, the results of the exact solution were compared with the solution obtained for rigid cross section and with the values derived from the local buckling of the walls of the cross section. It was concluded that the deformation of the cross section was negligible in the case of long members, whereas, for short members it had significant influence especially for cross sections with large width to thickness ratios. Haaijer, Carskaddorm and Arubb [42] performed a finite element analysis of an eccentrically loaded single angle column connected by one leg. Only elastic behaviour was considered and hence the results were applicable to
relatively slender members. A combination of finite element and finite segment approaches was used by Hu and Lu [43] to determine the complete load-deflection relationship of single angle struts with or without end restraints subject to eccentric loads.

It should be mentioned that pure torsional buckling can occur when a doubly symmetric section, such as a cruciform section is subjected to a concentric axial load. For all other sections or other types of load or both, the mode of failure is that of coupled torsional flexural buckling.

2.2.2 Experimental Investigations

2.2.2.1 Studies Prior to 1965

The earliest tests on plain and flanged aluminium alloy angles were carried out by Wagner and Pretschner [44] to confirm Wagner's theoretical results about torsional buckling of open thin-walled sections. Tests on thin steel angle struts were conducted by Thomas [45]. Kollbrunner [46] had conducted tests on more than 500 steel and aluminium equal leg angles of various cross-sectional dimensions and lengths to study the behaviour of torsional buckling. Mackey and Williamson [47] had reported the results of tests on two steel lattice girders. Marshall, Nelson and Smith [48] had carried out tests on aluminium alloy equal-leg, unequal-leg and bulb-angles. The large difference in the load carrying capacities of multiple bolt and single bolt connections as allowed in the design rules was not observed. Contrary to the then existing design rules, unequal-leg angles connected by the short leg were found to be stronger than those connected by the long leg.

2.2.2.2 Studies From 1965-1975

In 1965 an experimental investigation was carried out by Wakabayashi and Nonaka [49] on the buckling strengths of $3 \frac{1}{2} \times 3 \frac{1}{2} \times 9/32$ inch (90 x 90 x 7mm) structural steel angles; various eccentricities and slenderness ratios were included. Test results agreed well with theoretical predictions. Fifty seven mild steel equal-leg angles of $3 \frac{1}{2} \times 3 \frac{1}{2} \times 9/32$ inch (90 x 90 x 7 mm) size were tested by Yokoo,
Wakabayashi and Nonaka [50] under both concentric and eccentric axial loading. The tests on concentrically loaded angles showed that more torsional deformation occurred at the middle region than at the end regions, confirming that the buckling strength of equal-leg angles was not significantly affected by the boundary conditions. Angle specimens loaded eccentrically with respect to the asymmetric axis i.e., for bending in the symmetric plane, showed predominantly flexural deformations while the angle specimens loaded eccentrically with respect to the symmetric axis showed considerable torsional deformation even at early stages of loading. The decrease in capacity due to eccentricity of load about the symmetric axis was found to be appreciable for long angles that mainly failed by bending about the weak axis. On the otherhand the load carrying capacity of eccentrically loaded short angles was considerably decreased as compared to concentrically loaded angles since failure was caused by local buckling or torsional deformations.

A total of 721 single angle bolt connections were tested by Kennedy and Sinclair [51] using 16mm diameter bolts. Failure in bearing occurred at a nominal bearing stress equal to approximately 4.5 times the yield stress. Bearing stress equal to 2.25 times the yield stress had produced insignificant hole elongation. Empirical formulae were developed for the ultimate load of bolted connections for both end and edge type failures.

Single angle columns were investigated theoretically and experimentally by Trahair, Usami and Galambos [52] as end restrained columns subjected to biaxial eccentric loading. Both equal-leg single angles and angles of size 75 x 50 x 6mm were included in the test program. These sizes are representative of those used as web members of standard long span steel joists. The ends of the test specimens were welded to structural T-sections to simulate the chords of steel joists and the load was applied through these T -blocks. It was found that this method of loading had an effect on the load carrying capacity of the angles. An analytical investigation was also carried out assuming that the column was made of an elastic perfectly plastic material. Since the out of plane stiffness of the web of the end T-block was small, an elastic plastic
rotational spring was assumed for the out of plane end restraint, whereas the inplane end restraint was simulated by an elastic rotational spring. Theoretical results were derived from the solution of the differential equations of equilibrium, including the effects of residual stresses and initial deflections. The correlation between the theoretical and test results were found to be reasonably good.

Usami and Fukumoto [53] had studied the behaviour of bracing members of steel bridges by testing 100 x 100 x 10mm and 130 x 130 x 9mm angles. Ends of the columns were rivetted or bolted to the webs of structural end T-blocks, simulating the gusset plates. The load was applied through knife edge end fixtures to the centre of the webs of the end T-blocks. Test results had shown good agreement with the theory developed by Trahair, Usami and Galambos. It was found that the effect of residual stresses was not significant, confirming the earlier finding of Ishida [54]; it was also revealed that the cross sectional dimensions had no influence on the maximum strength curves. It was, therefore suggested that the theoretical curves for 100 x 100 x 10mm angle could be utilized with good accuracy to predict the strength of columns of other cross sectional dimensions. It was also recommended that the maximum load of an angle bracing member connected through one leg can be taken as 58% of the load of a corresponding concentrically loaded member.

Tests were carried out by Kennedy and Murthy [55] on 72 single and double angle struts with hinged and fixed end conditions and subjected to concentric axial loading. The slenderness ratios were so chosen as to assure that all test specimens failed in the inelastic range. The results from the experimental investigation provided verification of the established theoretical solutions for inelastic flexural, torsional flexural and plate buckling. Based on the test results a procedure was suggested to determine a realistic estimate of the permissible buckling stress for a given angle strut.
The effective lengths of leg members in lattice columns were found from tests by Kloppel and Ramm [56]. The behaviour of laterally unsupported angles were investigated by Thomas, Leigh and Lay [57]. A total of 15 tests were conducted on laterally unsupported 3 x 3 x 3/16 inch (76 x 76 x 4.8 mm), 2 1/8 x 2 x 1/4 inch (64 x 51 x 6.4 mm) and 3 1/8 x 2 1/8 x 3/16 inch (89 x 64 x 4.8 mm) angles with length to thickness ratio ranging from 400 to 1600. The loading was a uniform moment over the entire span applied about an axis parallel to the angle leg. It was found that the angle of twist causes a reduction in the maximum stress produced. The angle of twist had a significant influence on the maximum deflection also. Angle sections used in practice as beams were seen to be governed by stress or deflection limitations rather than by buckling.

2.2.2.3 Studies From 1975-1985

The working Group 08 of the Study Committee 22 of the International Conference on Large High Voltage Electric Systems had conducted 153 buckling tests on equal-leg and unequal-leg angles to simulate the cross diagonals of latticed electrical transmission line towers [58]. The initial buckling stress for slenderness ratio between 120 and 250 was found to be higher than the Euler buckling stress, the ratio between the two stresses increasing moderately with increase in the slenderness ratio.

The influence of connections of the web members on their load-carrying capacity was experimentally investigated by Lorin and Cuille [59]. Tests were conducted on 3 1/8 x 3 1/8 x 11/32 inch (89 x 89 x 8.7mm) single angles and 2 1/8 x 2 1/8 x 1/4 inch (64 x 64 x 6.4mm) double angles connected back to back. It was found that increase in the yield strength of the gusset plate material did not increase the load-carrying capacity of the web member; however, doubling the thickness of the gusset plate increased the buckling load by approximately 40%. If the cross diagonal members were made continuous at their intersection their capacity was found to be 40% more than the capacity of cross diagonal members that were discontinuous at their intersection.
Twenty seven tests were carried out by Short [60] on 2 3/4 x 2 3/4 x 1/4 inch (70 x 70 x 6.4 mm) and 3 1/4 x 2 1/4 x 1/4 inch (83 x 57 x 6.4 mm) double angles connected back to back to determine their buckling loads about the asymmetric and symmetric axes. For those angles that failed by buckling about the asymmetric axis, the failure loads were found to be below those predicted by a column curve for single angle. An empirical relation that takes into account the spacing between the stitch bolts was developed for computing the effective slenderness ratio for buckling about the symmetric axis. The effect of varying the gap between the double angles connected back to back was also studied by Short [61]. It was observed from the tests that, although the buckling capacity increases with an increase in the gap width between the angles, the increase was not as great as would have been expected for a completely composite member. Tests were also carried out by Short [62] on single angles to study their buckling about their stronger and weaker axes. Weak axis buckling loads were found to agree well with column curves recommended by European Recommendations for Steel Construction [63]. However angles prevented from failing about the weak axis were observed to fail about the stronger axis at considerably smaller loads than those obtained from the recommended column curves. Reasonably good agreement was found between theory and experiment if the member was considered as a beam column with a bending stress equal to 70% of the axial stress. This point needs to be borne in mind while designing latticed transmission line towers [64].

To study the problem of interconnection, a series of tests was devised by Temple and Schepers [65] with the number of interconnectors varying from zero to five. From this study, it was found that when only one interconnector was used, the type of interconnector had an insignificant effect on the buckling load, it was also found that the interconnection requirements of North American Codes were adequate if the buckling load was calculated as twice the buckling load of one angle buckling about the weak axis with an effective length factor of 0.6.
Some specifications deem that positive eccentricities of load are more unfavourable than negative eccentricities (Eccentricities measured from the centroid towards the direction of the shear centre is negative). However, from an analytical study Chen [66] had concluded that the lateral torsional buckling load of double angle sections connected with long legs back to back under positive eccentricity of load was greater than the failure load under negative eccentricity. Tests were carried out on nine double-angle sections to confirm the analytical work.

Angle specimens were tested by Jain, Goel and Hanson [67] under static and slow dynamic loading conditions. It was concluded that the effective slenderness ratio of a member was the most influential parameter governing its hysteresis behaviour and that the total energy dissipation through hysteretic cycles was independent of the direction of loading. Tests were also conducted by Wakabayashi, Nakamura and Yoshida [68] on the elastic-plastic behaviour of angle-braced frames under repeated lateral loading.

Experiments were carried out on 120 x 120 x 6 mm angles by Massonnet and Plummier at the Universite de Liege, Belgium to ascertain the effectiveness of reinforcing the leg members of transmission line towers that are too susceptible to torsional buckling.

Kennedy and Murthy [69] had conducted theoretical and experimental studies to investigate the buckling of angle columns and beams. Theoretical analyses of flexural, torsional-flexural buckling followed by experimental investigations on single and built up angle sections were conducted by them.

Woolcock and Kitipornchal [70] had proposed a design method for single angle web compression members in trusses. The method was based on the experimental observation that the predominant mode of deformation is perpendicular to the plane of the connected leg. The design method takes into account the buckling and eccentricity.
Theoretical studies on inelastic buckling of single angles was carried out by Kitipornchai and Lee [71]. The inelastic buckling loads were calculated based on tangent modulus concept, assuming an idealized residual stress distribution.

Shan and Peyrot [72] had developed a finite element method to model angle members. Both material and geometric non-linearities had been included in the study. The method had been applied to determine the ultimate strength of angle members and subassemblies of angle members.

2.3 LITERATURE RELATED TO MODEL ANALYSIS AND TESTING OF TRANSMISSION LINE TOWERS

There is scant literature available related to model analysis and testing of transmission line towers. Only in the last decade has there been some research done in this area.

Babb [73] had discussed the need for, and various aspects of model study. Though various materials such as Aluminium, Perspex, Acrylic resin, Glass, Plaster and Mortars can be used for modelling structures, the best is to use the same material of which the full scale structure is made of.

Gardiner and Gomm [74] in their paper had discussed about the necessity of model tower testing and had recommended that model tests be performed if a large number of structures of the same or similar design were to be constructed.

Gopalan and Mohanram [75] had explained the principles of testing, the scales to be adopted, fabrication, assembly and erection of model towers. They had also explained the advantages and limitations of model tower testing and had given the salient features of model tower testing station at Central Power Research Institute, Bangalore, India.
Model tower tests were carried out in France [76] for the development of type designs. Such tests are especially useful for assessing the static and dynamic behaviour of the tower. The natural frequency of the tower can be obtained from such model tests. These can lead to rationalisation of dynamic load factors.

2.4 LITERATURE RELATED TO PROTOTYPE TESTS AND SEGMENT TESTS

A series of tests were carried out in 1964 by Carpena at the SAE Test Station, Lecco, Italy on open lipped (60°) and unslipped channels, used as posts for triangular towers. The results of these tests are shown in Fig. 2.1.

At the Cornell University, under a research project on cold-formed sections, a number of tests on axially loaded compression members were carried out by Fang [77]. Warping at the end sections of these members were prevented. The results were published in the Bulletin of Cornell University [78]. Fig 2.2 shows the results of the experiments done by Fang.

In 1977, ENEL (Ente Nazionale Per L' Energia Electrica - Italian National Power Board) [79] investigated the possibility of building an experimental line by adopting triangular rigid towers with cold-formed W-shaped post angles. The actual behaviour of such shapes had been investigated at the University of Rome. In these tests, warping and rotation of the ends were prevented. The results were in close resemblance to the behaviour of pin ended columns as shown in Fig. 2.3.

A short series of tests were carried out by Zavelani in 1979 at the Technical University of Milan, Italy on both rolled angles and coupled "open" channels. Fig. 2.4 shows the results of the experiments done by Zavelani.

Casarico et al [80] in 1981 investigated the possibility of adopting cold-formed sections for tower design. The tests had been conducted at the SAE Research Center. Two experimental prototypes of a 500 K.V. tower
FIG. 2.1. Results of Tests by Carpena, 1964

FIG. 2.2. Results of Tests by Fang, 1966

FIG. 2.3. Results of Tests by Caullillo, 1976
FIG. 2.4 Results of Tests by Zavelani, 1979

FIG. 2.5 Results of Tests by Casarico, Catenacci, and Faggiolo, 1981

FIG. 2.6 Results of Tests by Wilholte, Zandonini, and Zavelani, 1984
were built and tested under different load conditions. The results had confirmed the reliability of the design criteria adopted on the basis of AISI Specifications. For main loading case the collapse was observed to have reached with an actual load factor 14% higher than expected. The configuration of the tower and the results are shown in Fig. 2.5.

Wilhoite, Zandonini and Zavelani [81] had done extensive study on the behaviour of different shapes subjected to concentric and eccentric loads. The test set up and the results are shown in Fig. 2.6.

According to Zavelani and Faggiano [82], the use of cold-formed members in electrical transmission line lattice towers would allow higher flexibility; this favourably affects the overall cost. The AISI "Specifications for the Design of Cold-Formed Steel Structural Members" can be adopted as the basic guide to the design. Design formulae provided directly differ from AISI formulae in that the safety factor is made equal to 1.

Tests were conducted on the segment of a tower to study the eccentricities in the connections by Gaylord and Wilhoite [83]. The eccentricity in angles without lips and with bolts in both legs was observed to be small if the bolts were on the centre line of the legs.

The ORGRES (Organisation and Rationalisation of Electric Power Stations and Networks), Moscow [84] conducted tests on prototype towers not only for static loads but also for dynamic loads corresponding to broken-wire conditions. A 5 km long line with nine suspension supports and two anchor supports was set up for conducting dynamic tests on towers. Special electrical dynamometers and strain measuring devices had been installed for accurate measurements of strains.

Norville, Mehta and Vann [85] had conducted tests on six segments of a delta tower. Analysis of the data included plots of time histories, power spectra with determination of mean values, standard deviation and peak values of each record. Based on these studies mean and standard deviation for loading for probabilistic design were determined.
Susendran, Gopalan and Rao [86] had reviewed the experience gained during 150 prototype tower tests, keeping in view the existing IEC and IS specifications and current practices. Logical improvements in testing had been suggested so that strict quality control could be realised. They had suggested that:

1. Illogical loads should not be simulated. Possible complications in testing should be reduced and the wastage of material and time brought down. Simulation of self weight of the tower is one such and can be done away with.

2. The recommendations of IS regarding location of load cells at the pull-off points should be scrupulously followed.

3. Application of wind load equivalent on the cross arm tip is unjustified and results in increased weight of the tower.

4. It is highly desirable to have a parity between IEC and IS standard regarding waiting period.

5. Such of those practices which result in impact loads on tower should be checked.

Gopalan and Lakshminarayana [87] in their paper had concluded, after conducting tests on towers with members having slenderness ratios varying from 100 to 400, that the load carried by the redundants varied from 2 to 3.5% when mild steel angles were used.