Project Report On

THE BUCKLING LOAD OF A STRUSS DETERMINED BY THE CONVERGENCE CRITERION OF THE MOMENT DISTRIBUTION METHOD

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology

In

Civil Engineering

Submitted By

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National Institute of Technology, Rourkela

Rourkela

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Under the Guidance of Prof.MANORANJAN BARIK

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<u>CERTIFICATE</u>

This is to certify that the thesis entitled "***THE BUCKLING LOAD OF A STRUSS DETERMINED BY THE CONVERGENCE CRITERION OF THE MOMENT DISTRIBUTION METHOD***" submitted by Sri Baruna Malik(Roll No.10501017) in partial fulfillment of the requirements for the award of Bachelor of Technology degree in Civil Engineerig at the National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

Date:

Prof. Manoranjan Barik
Dept. of Civil Engineering,
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ACKNOWLEDGEMENT

My heart pulsates with the thrill for tendering gratitude to those persons who helped me in completion of the project.

The most pleasant point of presenting a thesis is the opportunity to thank those who have contributed to it. Unfortunately, the list of expressions of thank no matter how extensive is always incomplete and inadequate. Indeed this page of acknowledgment shall never be able to touch the horizon of generosity of those who tendered their help to me.

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An assemblage of this nature could never have been attempted without reference to and inspiration from the works of others whose details are mentioned in reference section. I acknowledge my indebtedness to all of them.

Last but not least, my sincere thanks to all my friends who have patiently extended all sorts of help for accomplishing this undertaking.

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DATE:

PLACE:

ABSTRACT

The buckling load analysis of a truss and bars has been and continues to the subject of numerous researches, since it embraces a wide class of problem with immense importance in engineering science. When the buckling load of these trusses exceeds the permissible limit failure of the structure occurs. The ends of the bars in actual not ideal, frameworks are welded or riveted together rather than connected by pin joints. When a compression member of a framework buckles its ends are consequently not free to rotate but are restrained by the other members. Naturally this restrained is not absolutely rigid. There is some given in the system and the joints to which the compression member is attached rotate slightly and elastically because of moments exerted on them by the member that buckles. Basically it depends on stiffness of the member and carryover factor and moment of the members. Depending on the assumptions adopted, the type of analysis used, the kind of loading or excitation and the overall truss characteristics a variety of approaches have been reported in the literature and a great a number of both theoretical and experimental findings are related to load analysis. This paper describes a numerical method which is used to determine the most unstable post buckling mode capable of developing at the ultimate critical state of complex pin-jointed structures. In this method, the set of independent variables are the flexural shortenings of those members that are a subset of the critically loaded members in the lattice structure. This choice of independent variables greatly simplifies the analysis and promotes the evaluation of the most degrading mode under both equilibrium and collapse conditions. In this Technical Note, this method is used to evaluate these two modes for some rather simple structures. The deformed modes are plotted in each case and differences between the most degrading mode under equilibrium and collapse conditions are noted.

CONTENTS

1.1 INTRODUCTION

The buckling load analysis of a truss and bars has been and continues to the subject of numerous researches, since it embraces a wide class of problem with immense importance in engineering science. When the buckling load of these trusses exceeds the permissible limit failure of the structure occurs. The ends of the bars in actual not ideal, frameworks are welded or riveted together rather than connected by pin joints. When a compression member of a framework buckles its ends are consequently not free to rotate but are restrained by the other members. Naturally this restrained is not absolutely rigid. There is some given in the system and the joints to which the compression member is attached rotate slightly and elastically because of moments exerted on them by the member that buckles. Basically it depends on stiffness of the member and carryover factor and moment of the members.

Depending on the assumptions adopted, the type of analysis used, the kind of loading or excitation and the overall truss characteristics a variety of approaches have been reported in the literature and a great a number of both theoretical and experimental findings are related to load analysis. In reality the entire truss buckles as a singe unit .Whenever one highly compressed member deflects , continuity at the rigidly connected joints requires that all other members also deflect. Inspite of the complexity of this problem a simple procedure is available for it’s solution because of a peculiarity of the Hardy Cross moment distribution method. It converges only when the loads are smaller than the critical values and it diverges when the loads exceeds the critical values .The procedure suggested for evaluating the bulking load of the truss .

1.2 OBJECTIVE:

To find out the buckling load of a truss determined by the convergence criterion of the moment distribution method.

Chapter 2.0

LITERATURE SURVEY

2.1 Truss:-

An assemblage of structural members joined at their ends to form a stable structural assembly. If all members lie in one plane, the truss is called a planar truss or a plane truss. If the members are located in three dimensions, the truss is called a space truss.

A plane truss is used like a beam, particularly for bridge and roof construction. A plane truss can support only weight or loads contained in the same plane as that containing the truss. A space truss is used like a plate or slab, particularly for long span roofs where the plan shape is square or rectangular, and is most efficient when the aspect ratio (the ratio of the length and width) does not vary above 1.5. A space truss can support weight and loads in any direction.

Because a truss can be made deeper than a beam with solid web and yet not weigh more, it is more economical for long spans and heavy loads, even though it costs more to fabricate. *See also* Bridge; Roof construction.

The simplest truss is a triangle composed of three bars with ends pinned together. If small changes in the lengths of the bars are neglected, the relative positions of the joints do not change when loads are applied in the plane of the triangle at the apexes.

Multiple-span plane trusses (defined as statically indeterminate or redundant) and space trusses require very complex and tedious hand calculations. Modern high-speed digital computers and readily available computer programs greatly facilitate the structural analysis and design of these structures.

2.2 Buckling:-

In engineering, **buckling** is a failure mode characterized by a sudden failure of a structural member subjected to high compressive stresses, where the actual compressive stress at the point of failure is less than the ultimate compressive stresses that the material is capable of withstanding. This mode of failure is also described as failure due to elastic instability. Mathematical analysis of buckling makes use of an axial load eccentricity that introduces a moment, which does not form part of the primary forces to which the member is subjected.

The Euler formula for columns is

$$F = \frac{\pi^2 EI}{(Kl)^2}$$

where

F = maximum or critical force (vertical load on column),

E=modulus of elasticity,

I=area moment of inertia

l = unsupported length of column,

K = column effective length factor, whose value depends on the conditions of end support of the column, as follows.

For both ends pinned (hinged, free to rotate), $K = 1.0$.

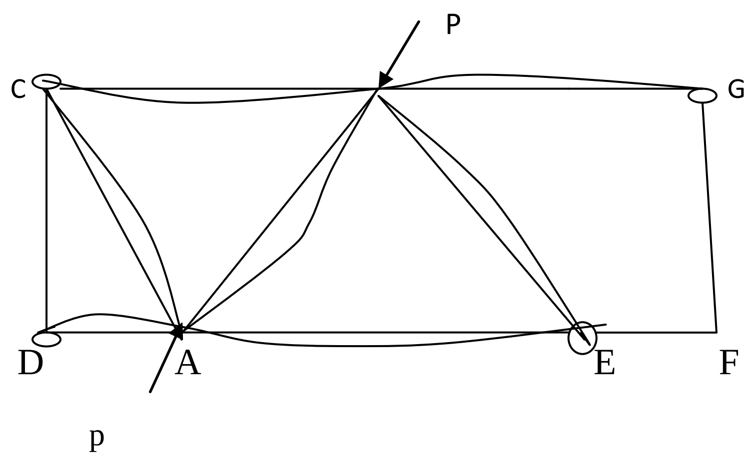
For both ends fixed, $K = 0.50$.

For one end fixed and the other end pinned, $K = 1.0/\sqrt{2.0}$.

For one end fixed and the other end free to move laterally, $K = 2.0$.

2.3 Bucking of column with elastic end fixation

The end Restraint:

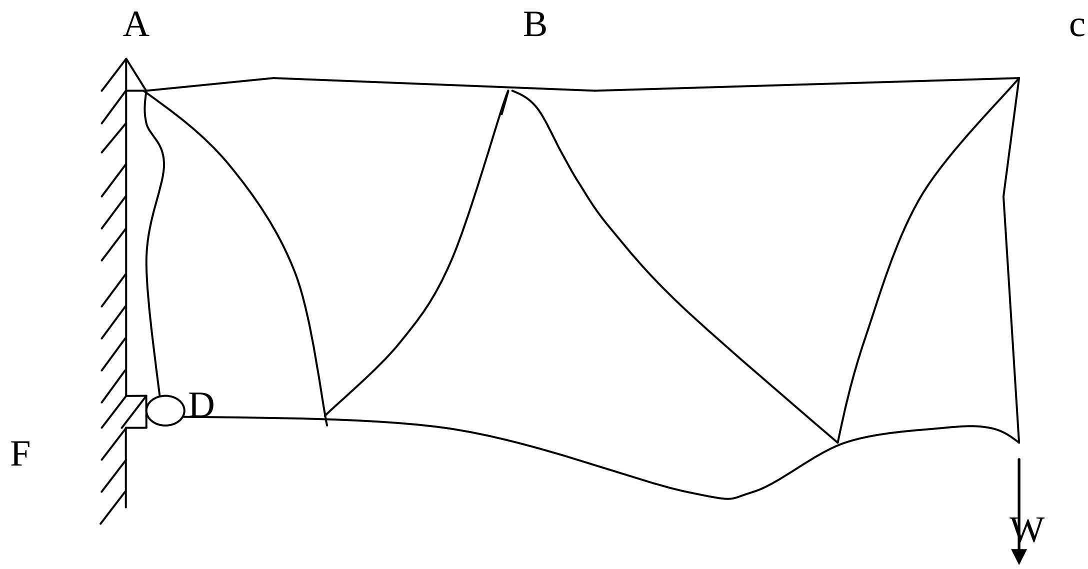


The above figure shows that a truss the elastic restrained provided by the riveted or welded connection can increase the buckling load of the compressed bar considerably above the Euler load. The elastic resistance of the joints to rotation by the value of C=∑ S was calculated under the simplifying assumptions and the multiplying factor of Euler load was presented in diagrams as a function of the relative stiffness factor "Ω". This factor was defined as the bending rigidity of the compression bar divided by C. where S= stiffness factor

S=4EI/L (for when the far end of the bar is fixed as Hardy Cross method)

S=3EI/L (for the far end of the bar is pinned)

The disadvantages of the approaches to the solution of the buckling problem is that imaginary pins must be inserted in the framework as shown in the above figure in order to get definite values for the stiffnesses of the supporting members and to make sure that no supporting member is used in conjunction with more than one principal compression member. The imaginary pins naturally introduce new degree of freedom of motion into the truss, and the resulting system has a lower buckling load than the actual one.



In reality the entire truss buckles as a single unit. Whenever one highly compressed member deflects, continuity at the rigidly connected joints requires that all the other members also deflect, as indicated in above figure.

Chapter 3.0

CONVERGENCE CRITERION OF MOMENT DISTRIBUTION METHOD

Inspite of the complexity of this problem a simple procedure is available for its solution because of its peculiarity of the Hardy cross moment distribution method. It converges only when the loads are smaller than the critical values, and it diverges when the load s exceed the critical values. The procedure suggested for evaluating the buckling load of the truss as a whole is as follows.

First a guess must be made regarding the magnitude of load W under which the truss buckles. Next the axial loads are calculated in the members of the truss by any of the routine methods based on the assumption of pin connections. The stability of the truss is then tested by assuming a moment M acting at any one of the joints; say at joint E in the figure 2.In the plane of the truss and tending to rotate joint E clockwise so as to make the truss assume the deflected shape shown.

3.1 The Hardy cross moment distribution method:

The Hardy cross moment distribution method is a procedure of step by step approximations, which can be used in place of the three moment equations for the calculation of the moment in continuous beams on several supports. It is particularly useful in the solution of rigid frame problems. When the structure is so highly so redundant that its analysis by classical methods is too cumbersome or even impossible for practical problems. The moment distribution method is not an approximate method; it is a method of successive approximations, the accuracy of which can be increased at will by increasing the no of steps under taken. In most problems accuracy sufficient for engineering purposes can be reached by carrying out a comparatively small number of operations. First step is to find out the moment at the joint, then that moment is distributed by the distribution factor. At that joint the unbalanced moment is multiplied by the carryover factor to the right side and left side of that joint. The process is continued for the minimum of the balancing moment $M=C\alpha$, $C=\sum S$. It is applied to a simple frame work. It permits the calculation of the internal moments acting at each of the joints of the framework. In the step by step approximation procedure of the moment distribution method. The effect of the end loads acting in the bars must be taken into account. This means that the stiffness coefficient and the carryover factor must be computed. Experience shown that the moment distribution process converges sufficiently rapidly; that is, after reasonably small number of cycles of the process the unbalanced moments at each joint are reduced to small enough quantities to be disregarded in engineering applications .However this statement is true only when the truss is in a stable state of equilibrium under the load W. When the equilibrium is unstable, the unbalanced moments increase in magnitude beyond all limits as the balancing process is continued. This peculiarity of the moment distribution method can be used to find out the buckling load of truss. One can assume a few different values for W and carry out the moment distribution with the same assumed moment M at joint E but with the different stiffness coefficients and carryover factors corresponding to each W. It is then easy to bracket the critical load of value W between closely spaced upper and lower limits the higher value resulting in divergent process and the lower value leading to convergence. However that in ordinary application in civil engineering the safety factors are so high that the effect of the end loads on the stiffness and the carryover factors can be disregarded when the moment distribution is carried out for the working loads. Under such circumstances the distribution factor is always less than unity or usually not greater than one-half and the carryover factor is one-half for a bar of uniform cross-section. Hence the moment carried over from one joint to the other is generally smaller than one quarter of the moment balance and the process always converges. On the other hand the calculation of the actual buckling load of the truss, compressive loads equal to or exceeding the Euler load of the pin ended column are encountered. With such high compressive loads the stiffness is greatly reduced and can even be negative while the carryover factor may attain values much greater than unity. It is quite natural than that the process may become divergent.

3.2 METHOD OF PROOF OF THE CONVERGENCE CRITERION

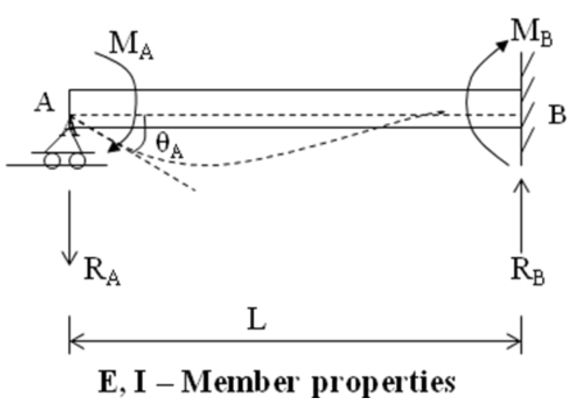
The proof of the convergence criterion can be brought in the following manner: it can be shown that in each cycle of moment distribution procedure, consisting of balancing, distribution, and carryover operations, the potential of the system decreases to the minimum value compatible with the requirements that the far ends of the bars are rigidly fixed. This means that if the analyst succeeds in eliminating all the unbalanced moments in straight forward application of the moment distribution process, he can sure that the system investigated is stable, when he is unsuccessful in his task the failure may be due to instability .In practice the lack of convergence is hardly ever the consequence of an unsuitable of moment distribution operations because it is very easy to eliminate the unbalances when the system is stable. Similarly the divergence of the procedure is readily detected when the system is unstable.

3.3 SOME BASIC DEFINITIONS

In order to understand the five steps mentioned in section 3.3, some words need to be defined and relevant derivations made.

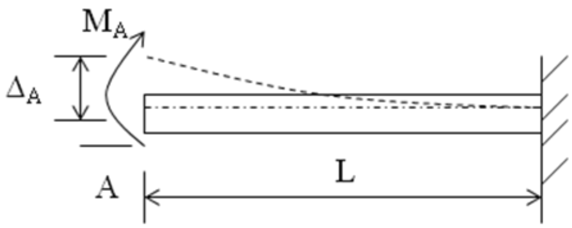
3.3.1 Stiffness and Carry-over Factors

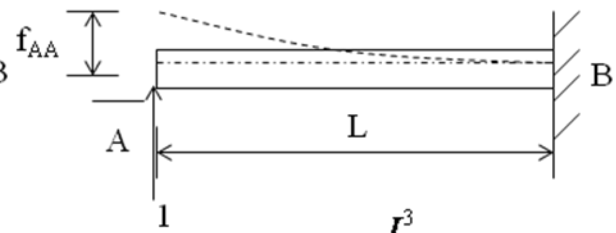
Stiffness = Resistance offered by member to a unit displacement or rotation at a point, for given support constraint conditions



A clockwise moment M_A is applied at A to produce a +ve bending in beam AB. Find θ_A and M_B.

Using method of consistent deformations





$$\Delta_A = +\frac{M_A L^2}{2EI} \qquad f_{AA} = \frac{L^3}{3EI}$$

Applying the principle of consistent deformation,

$$\Delta_A + R_A f_{AA} = 0 \rightarrow R_A = \frac{3M_A}{2L} \downarrow$$

$$\theta_A = \frac{M_A L}{EI} + \frac{R_A L^2}{2EI} = \frac{M_A L}{4EI} \qquad \therefore M_A = \frac{4EI}{L}\theta_A; \quad hence\ k_\theta = \frac{M_A}{\theta_A} = \frac{4EI}{L}$$

Stiffness factor = k_θ = 4EI/L

<u>Considering moment M_B,</u>

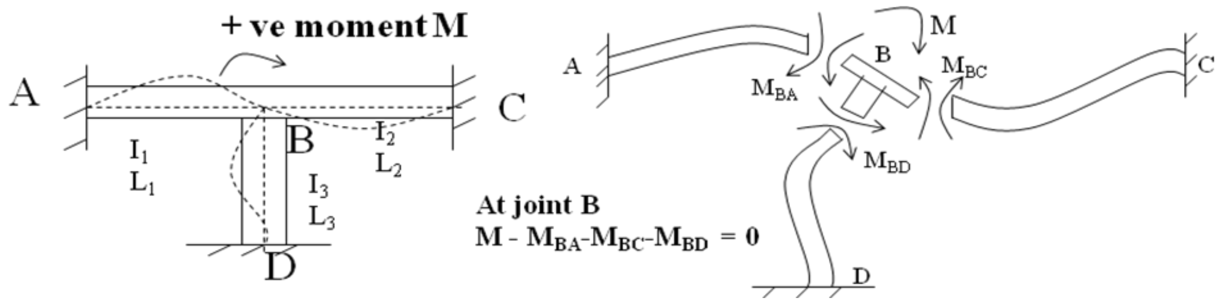
$M_B + M_A + R_A L = 0$

$\therefore M_B = M_A/2 = (1/2)M_A$

Carry - over Factor = 1/2

3.3.2 Distribution Factor

Distribution factor is the ratio according to which an externally applied unbalanced moment M at a joint is apportioned to the various members mating at the joint



i.e., $\mathbf{M = M_{BA} + M_{BC} + M_{BD}}$

$$= \left[\left(\frac{4E_1I_1}{L_1}\right)+\left(\frac{4E_2I_2}{L_2}\right)+\left(\frac{4E_3I_3}{L_3}\right)\right]\theta_B$$

$$= (K_{BA} + K_{BC} + K_{BD})\theta_B$$

$$\therefore \quad \theta_B = \frac{M}{(K_{BA} + K_{BC} + K_{BD})} = \frac{M}{\sum K}$$

$$M_{BA} = K_{BA}\theta_B = \left(\frac{K_{BA}}{\sum K}\right)M = (D.F)_{BA}M$$

Similarly

$$M_{BC} = \left(\frac{K_{BC}}{\sum K}\right)M = (D.F)_{BC}M$$

$$M_{BD} = \left(\frac{K_{BD}}{\sum K}\right)M = (D.F)_{BD}M$$

Chapter - 4

SOLUTION OF PROBLEMS -

Solved the previously given problem by the moment distribution method

MOMENT DISTRIBUTION METHOD -

4.1 Introduction And Basic Principles

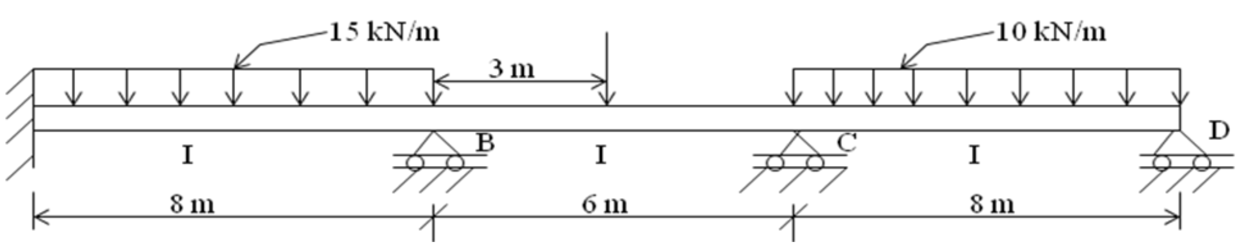
Introduction

(Method developed by Prof. Hardy Cross in 1932)

The method solves for the joint moments in continuous beams and rigid frames by successive approximation.

Statement of Basic Principles

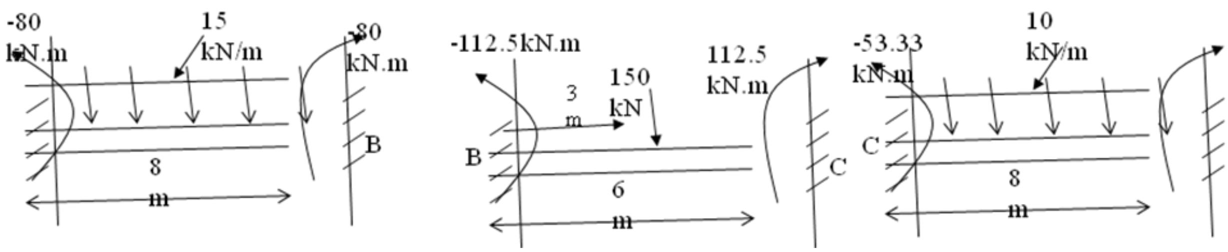
Consider the continuous beam ABCD, subjected to the given loads, as shown in Figure below. Assume that only rotation of joints occur at B, C and D, and that no support displacements occur at B, C and D. Due to the applied loads in spans AB, BC and CD, rotations occur at B, C and D.



4.2 steps

Step I

The joints B, C and D are locked in position before any load is applied on the beam ABCD; then given loads are applied on the beam. Since the joints of beam ABCD are locked in position, beams AB, BC and CD acts as individual and separate fixed beams, subjected to the applied loads; these loads develop fixed end moments.



In beam AB

Fixed end moment at A = $-wl^2/12$ = - (15)(8)(8)/12 = - 80 kN.m

Fixed end moment at B = $+wl^2/12$ = +(15)(8)(8)/12 = + 80 kN.m

In beam BC

Fixed end moment at B = $-(Pab^2)/l^2 = -(150)(3)(3)^2/6^2$

= -112.5 kN.m

Fixed end moment at C = $+(Pab^2)/l^2 = +(150)(3)(3)^2/6^2$

= + 112.5 kN.m

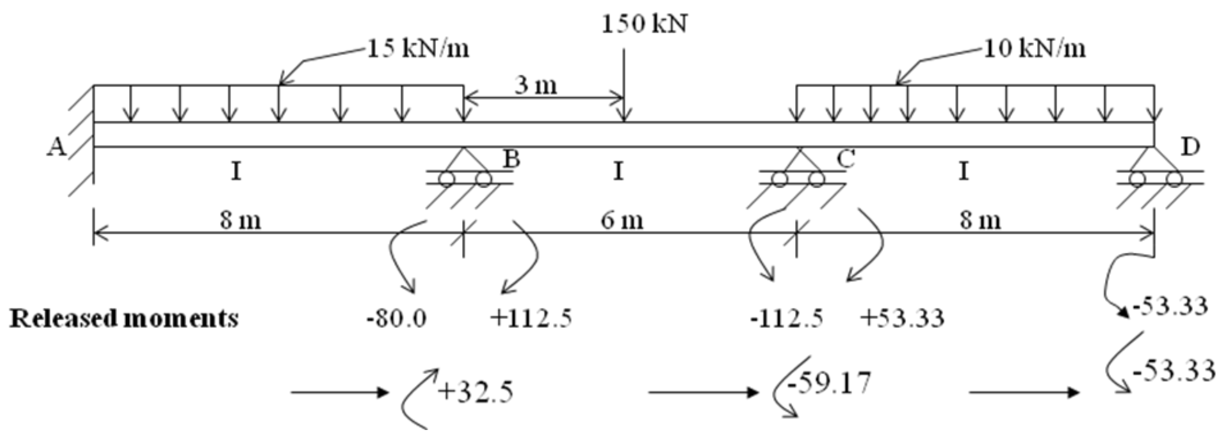
In beam AB

Fixed end moment at C = $-wl^2/12$ = - (10)(8)(8)/12 = - 53.33 kN.m

Fixed end moment at D = $+wl^2/12$ = +(10)(8)(8)/12 = + 53.33kN.m

Step II

Since the joints B, C and D were fixed artificially (to compute the the fixed-end moments), now the joints B, C and D are released and allowed to rotate. Due to the joint release, the joints rotate maintaining the continuous nature of the beam. Due to the joint release, the fixed end moments on either side of joints B, C and D act in the opposite direction now, and cause a net unbalanced moment to occur at the joint.



Step III

These unbalanced moments act at the joints and modify the joint moments at B, C and D, according to their relative stiffnesses at the respective joints. The joint moments are distributed to either side of the joint B, C or D, according to their relative stiffnesses. These distributed moments also modify the moments at the opposite side of the beam span, viz., at joint A in span AB, at joints B and C in span BC and at joints C and D in span CD. This modification is dependent on the carry-over factor (which is equal to 0.5 in this case); when this carry over is made, the joints on opposite side are assumed to be fixed.

Step IV

The carry-over moment becomes the unbalanced moment at the joints to which they are carried over. Steps 3 and 4 are repeated till the carry-over or distributed moment becomes small.

Step V

Sum up all the moments at each of the joint to obtain the joint moments.

4.3 Distribution Factors

$$DF_{AB} = \frac{K_{BA}}{K_{BA} + K_{wall}} = \frac{0.5EI}{0.5 + \infty\ (wall\ stiffness)} = 0.0$$

$$DF_{BA} = \frac{K_{BA}}{K_{BA} + K_{BC}} = \frac{0.5EI}{0.5EI + 0.667EI} = 0.4284$$

$$DF_{BC} = \frac{K_{BC}}{K_{BA} + K_{BC}} = \frac{0.667EI}{0.5EI + 0.667EI} = 0.5716$$

$$DF_{CB} = \frac{K_{CB}}{K_{CB} + K_{CD}} = \frac{0.667EI}{0.667EI + 0.500EI} = 0.5716$$

$$DF_{CD} = \frac{K_{CD}}{K_{CB} + K_{CD}} = \frac{0.500EI}{0.667EI + 0.500EI} = 0.4284$$

$$DF_{DC} = \frac{K_{DC}}{K_{DC}} = 1.00$$

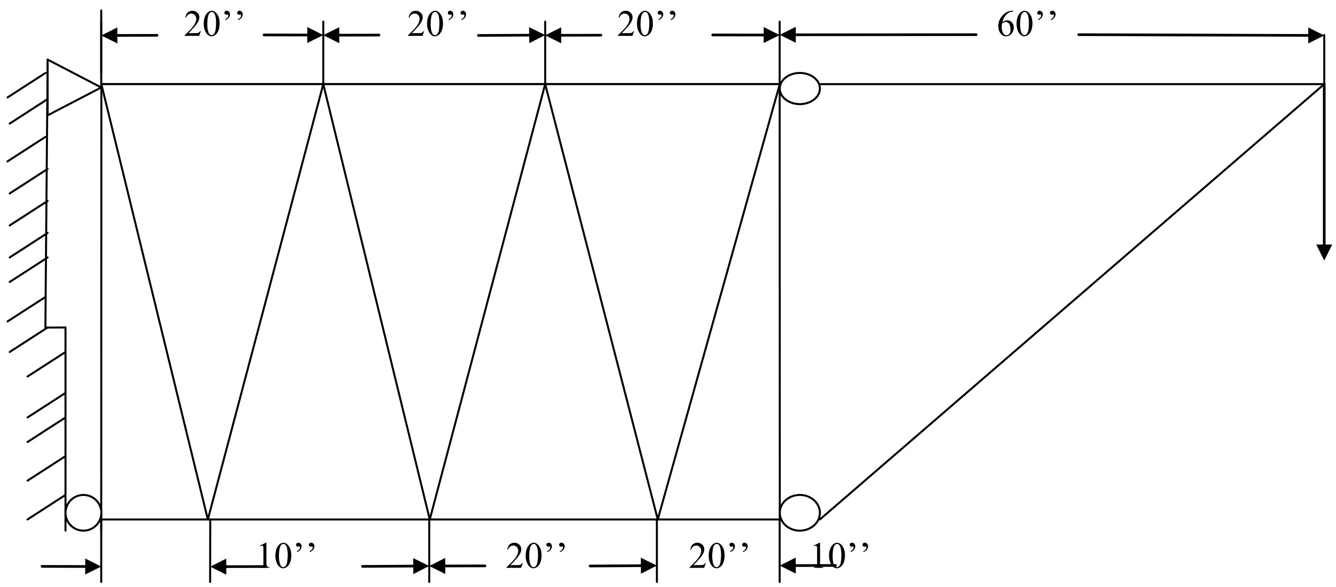
4.4 Moment Distribution Table

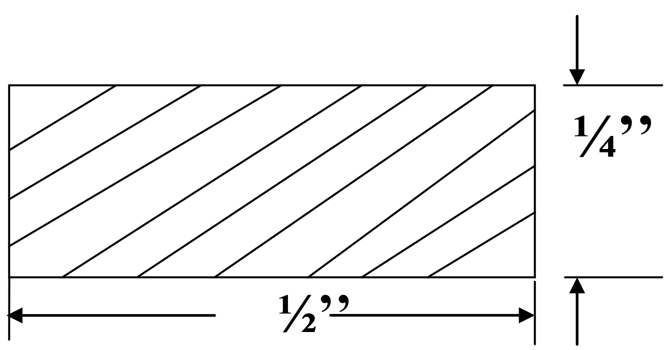
Joint		A	B		C		D
Member		AB	BA	BC	CB	CD	DC
Distribution Factors		0	0.4284	0.5716	0.64	0.36	1
Cycle 1	Computed end moments	-80	80	-112.5	112.5	-53.33	53.33
	Distribution		13.923	18.577	-37.87	-21.3	-53.33
Cycle 2	Carry-over moments	6.962		-18.93	9.289	-26.67	-10.65
	Distribution		8.111	10.823	11.122	6.256	10.65
Cycle 3	Carry-over moments	4.056		5.561	5.412	5.325	3.128
	Distribution		-2.382	-3.179	-6.872	-3.865	-3.128
Cycle 4	Carry-over moments	-1.191		-3.436	-1.59	-1.564	-1.933
	Distribution		1.472	1.964	2.019	1.135	1.933
Cycle 5	Carry-over moments	0.736		1.01	0.982	0.967	0.568
	Distribution		-0.433	-0.577	-1.247	-0.702	-0.568
	Summed up moments	-69.44	100.69	-100.7	93.748	-93.75	0

CHAPTER 5

5.1 PROBLEM ANALYSIS

This will be evident from the numerical examples which will is given below:





section of bars

Stiffness coefficients and carryover factors when W=100lb

bar	length-l in inch	load-p in lb	type of load+	K=√p/EI in 1/inch	KL	1/C++	SL/EI	C	20S/EI
AB	**20**	**550**	**T**	**0.2836**	**5.67**	**…**	**9.63**	**0.265**	**9.63**
BC	**20**	**450**	**T**	**0.2565**	**5.13**	**…**	**8.92**	**0.286**	**8.92**
CD	**20**	**350**	**T**	**0.2262**	**4.53**	**…**	**8.15**	**0.320**	**8.15**
LK	**10**	**600**	**C**	**0.2962**	**2.96**	**1.075**	**3.40**	**0.930**	**6.80**
KJ	**20**	**500**	**C**	**0.2704**	**5.41**	**-0.550**	**2.80**	**-1.818**	**-2.80**
JH	**20**	**400**	**C**	**0.2419**	**4.84**	**-0.120**	**-0.44**	**-8.33**	**-0.44**
HG	**10**	**300**	**C**	**0.2095**	**2.09**	**1.450**	**4.10**	**0.690**	**8.20**
AL	**20**	**000**	**…**	**000**	**000**	**…**	**5.078**	**0.5655**	**5.078**
AK,BJ,CH	**22.37**	**111.9**	**T**	**0.1279**	**2.86**	**…**	**6.38**	**0.426**	**5.70**
KB,JC,HD	**22.37**	**111.9**	**C**	**0.1279**	**2.86**	**1.130**	**3.48**	**0.885**	**3.11**
DJ	**20**	**100**	**T**	**0.1209**	**2.42**	**…**	**6.02**	**0.456**	**6.02**

Where

T=Tension

C=compression

C++=compression bars only

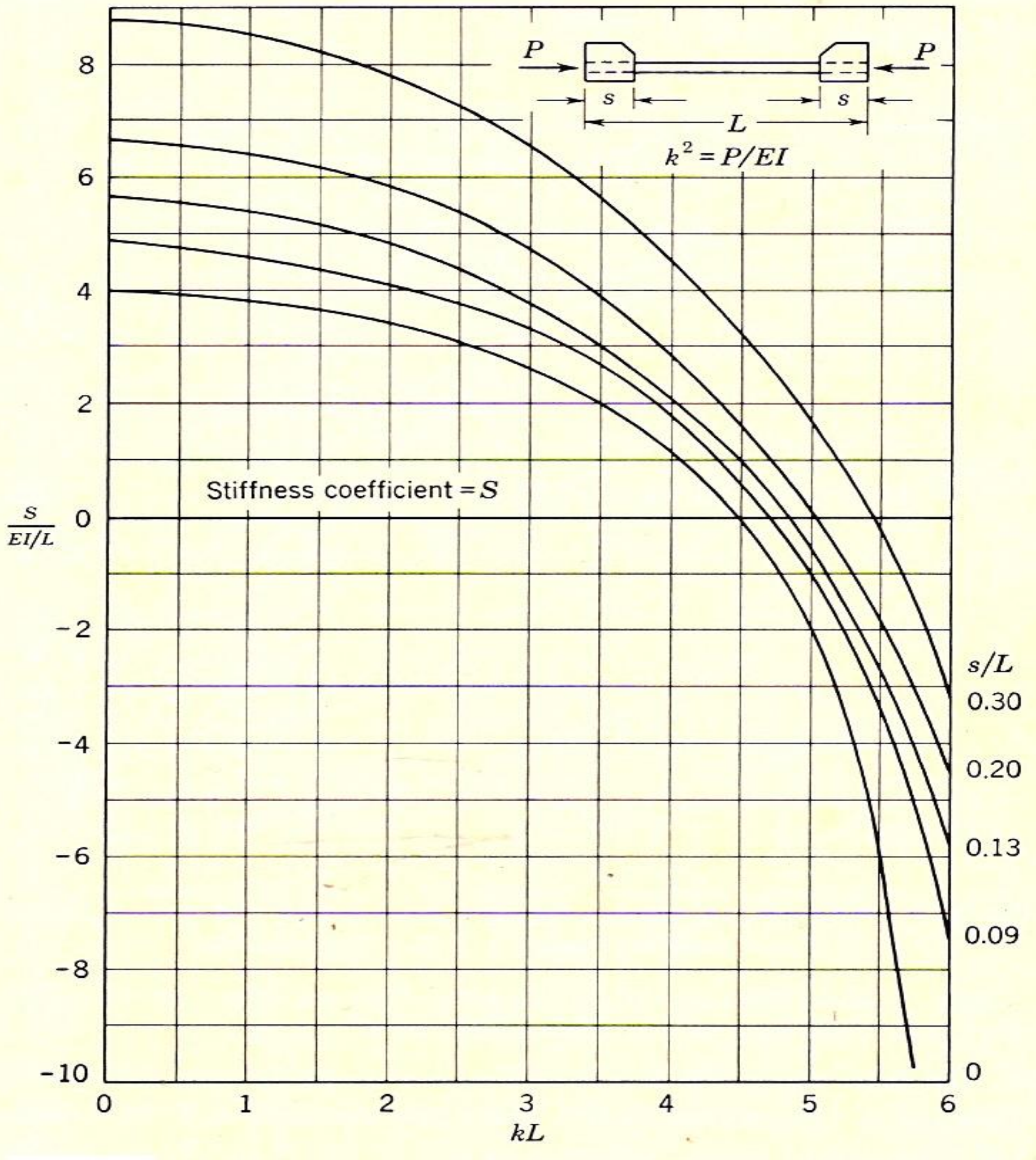
EI=6835.5lb.in2

Stiffness S= [EI/L*KL]*[(sinKL – KL cosKL)/(2 - 2cosKL – KL sinKL)]

C=(KL - sinkL)/(sinKL – KL coskL)

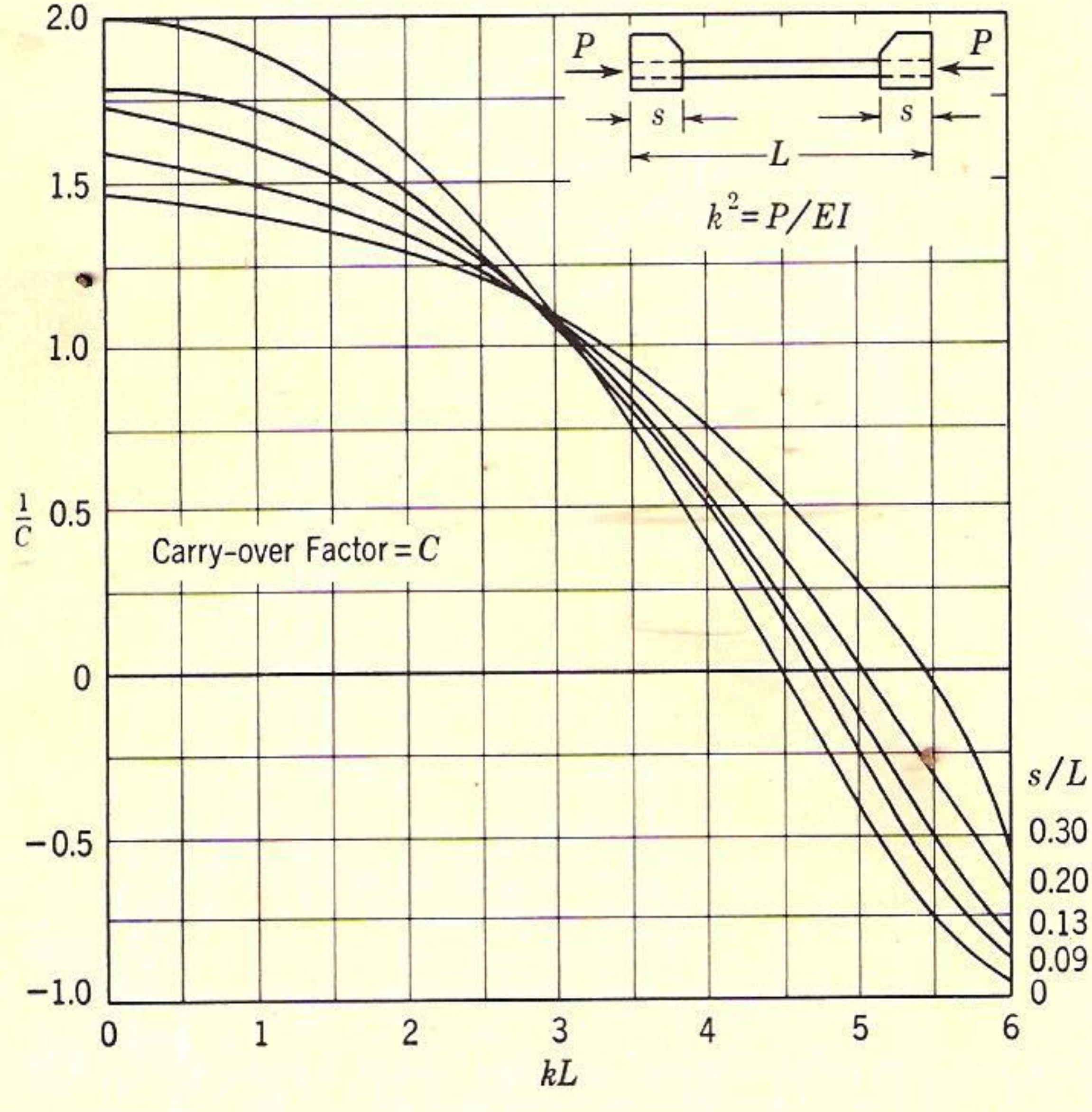
Stiffness coefficients and carryover factors when W=105lb

bar	length-l in inch	load-p in lb	type of load+	K=√p/EI in 1/inch	KL	1/C++	SL/EI	C	20S/EI
AB	20	577.5	T	0.2906	5.81	…	9.82	0.259	9.82
BC	20	472.5	T	0.2629	5.26	…	9.08	0.281	8.08
CD	20	367.5	T	0.2318	4.64	…	8.30	0.314	8.30
LK	10	630	C	0.3035	3.04	1.03	3.25	0.971	6.50
KJ	20	525	C	0.2771	5.54	-0.65	-3.65	-1.538	-3.65
JH	20	420	C	0.2478	4.96	-0.21	-0.85	-4.762	-0.85
HG	10	315	C	0.2146	000	1.42	4.00	0.704	8.00
AL	20	000	…	0000	000	…	5.078	0.5655	5.078
AK,BJ,CH	22.37	117.5	T	0.1311	2.93	…	6.44	0.420	5.76
KB,JC,HD	22.37	117.5	C	0.1311	2.93	1.09	3.40	0.917	3.04
DJ	20	105	T	0.1239	2.48	…	6.10	0.453	6.10



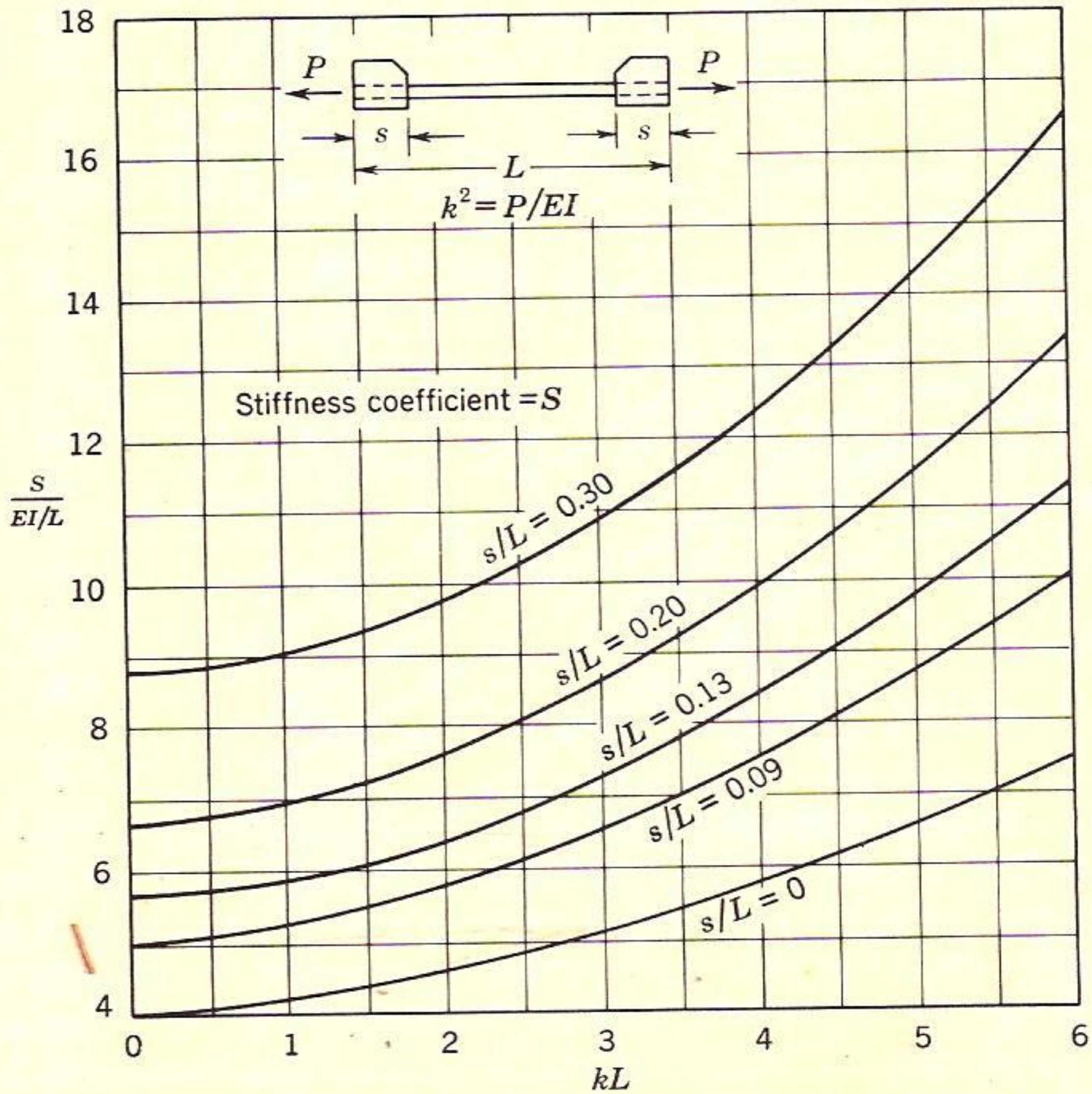
Stiffness coefficients for compression bars with gusset plates. From author's paper in *ASCE Transactions*.

Figure . 1



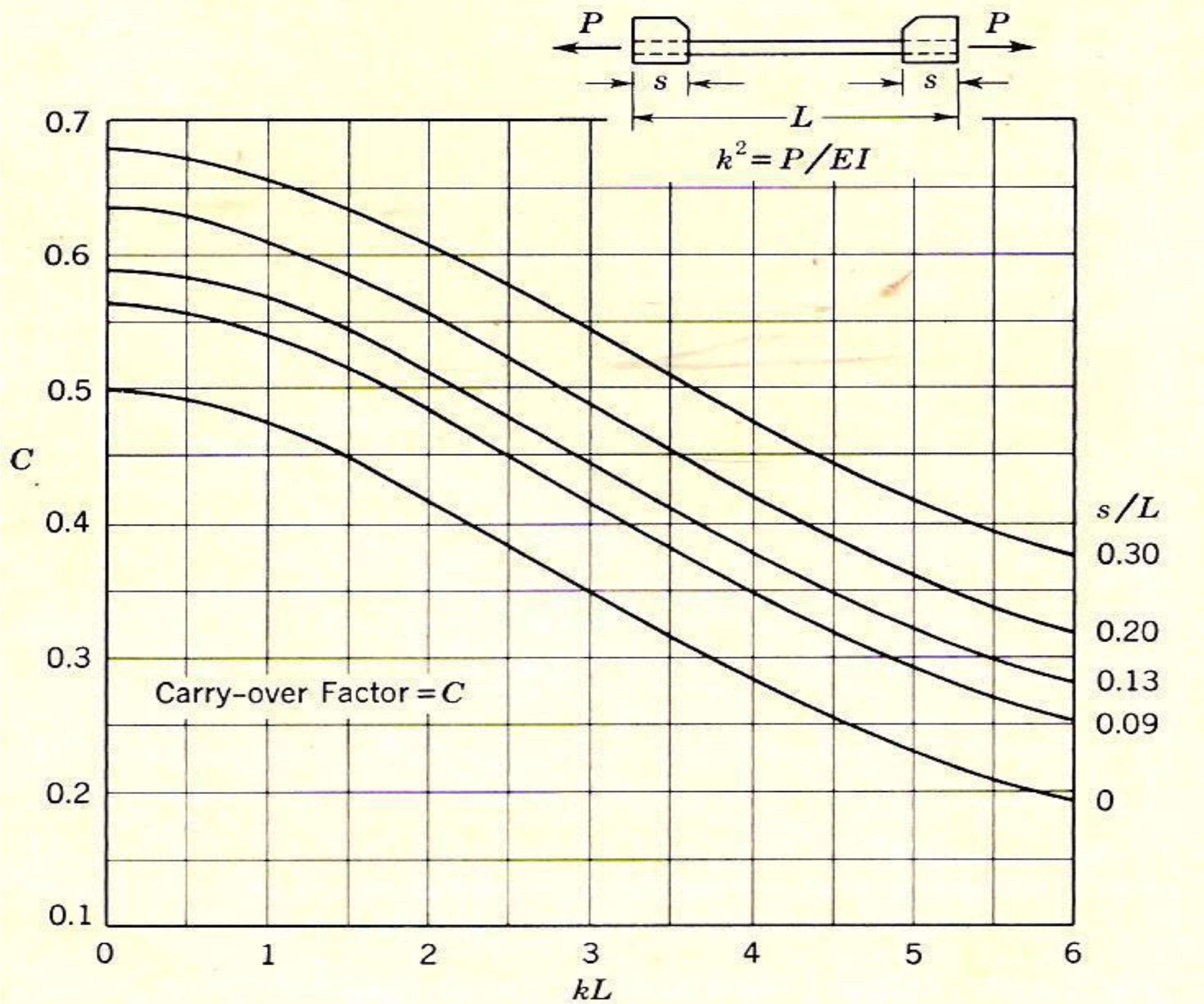
Carry-over factors for compression bars with gusset plates.
From author's paper in *ASCE Transactions*

Figure .2



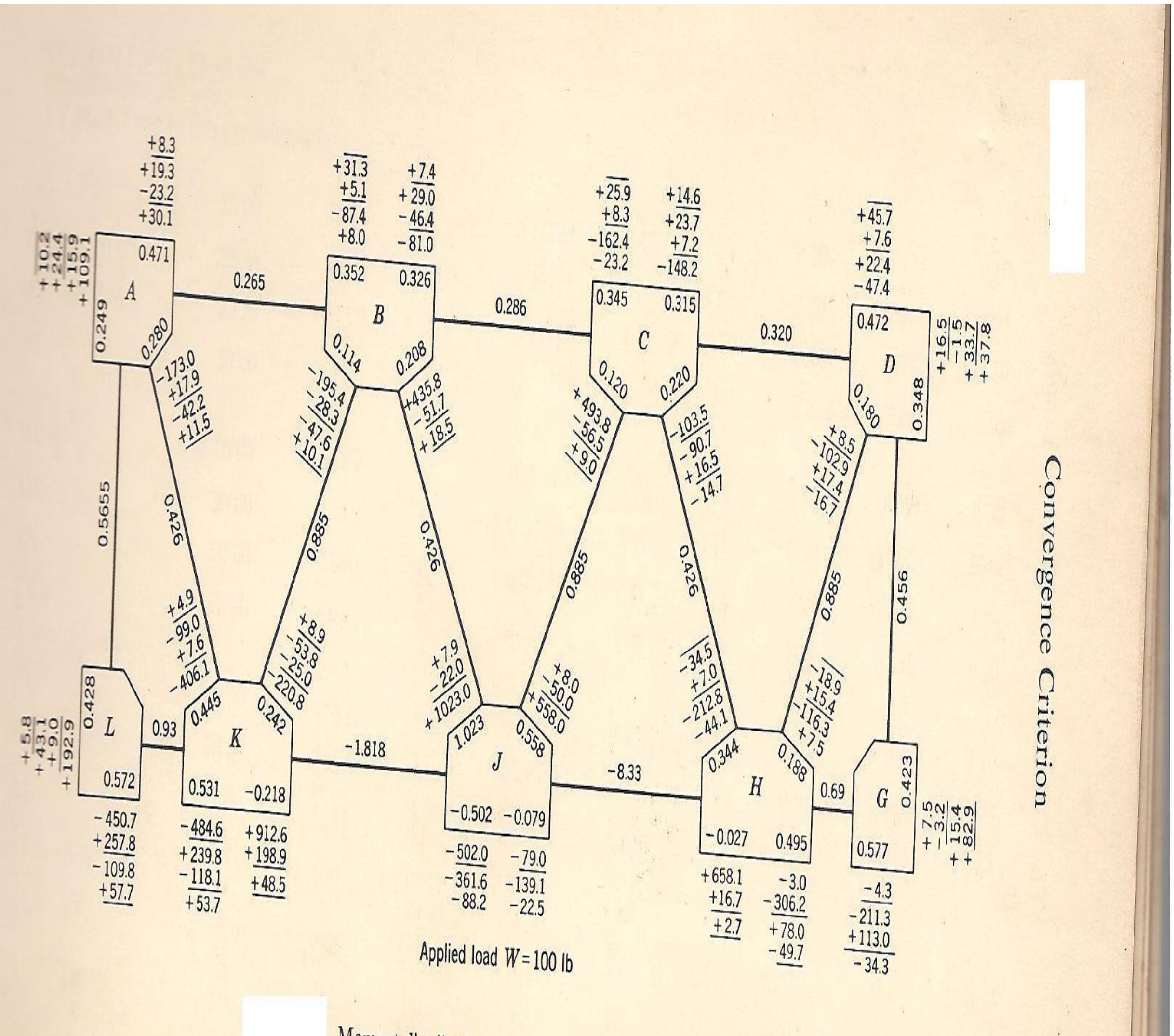
Stiffness coefficients for tension bars with gusset plates.
From author's paper in *ASCE Transactions*

Figure .3



Carry-over factors for tension bars with gusset plates. From author's paper in *ASCE Transactions*.

Figure . 4



Moment distribution converges. From author's paper in *Transactions ASCE.*

Figure . 5

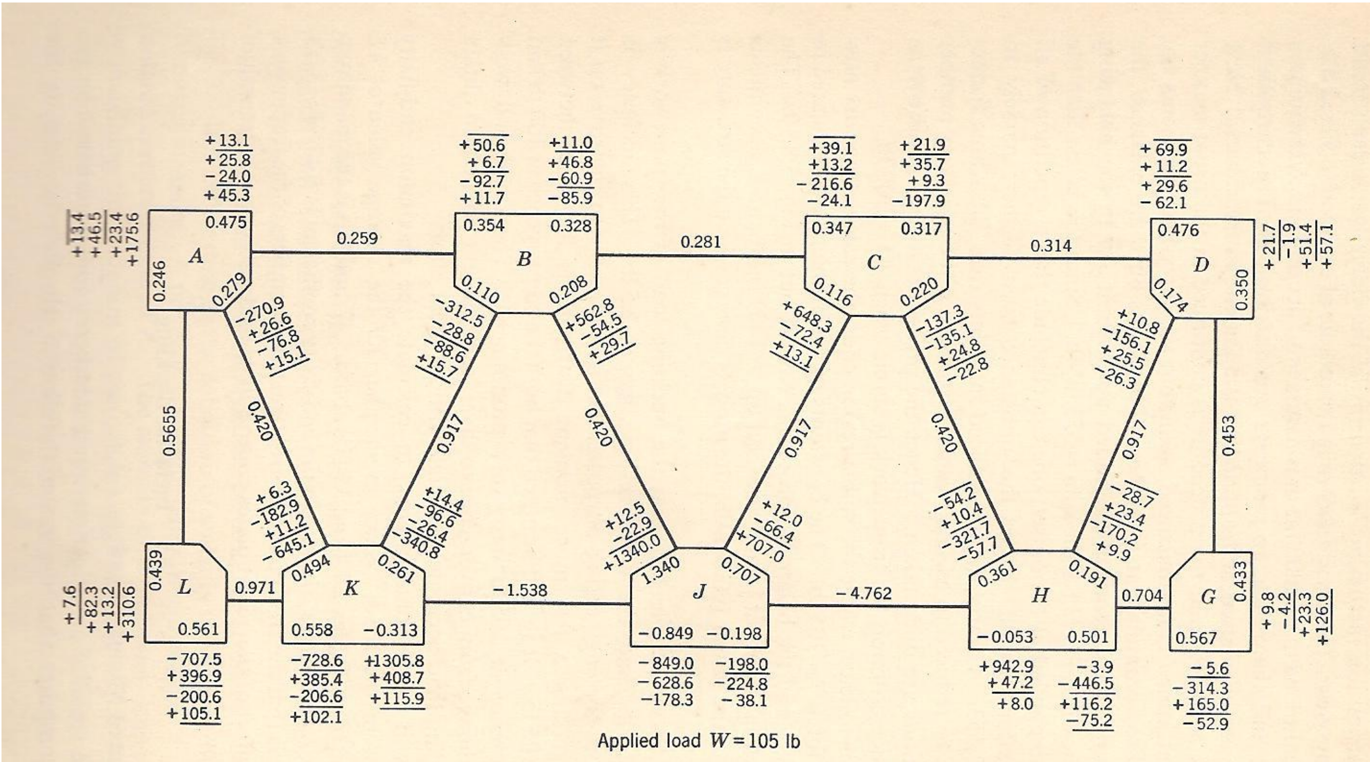


Figure .6

If the ratio of gusset plate length to total length of bar is 0.09 for each of the members of the framer, the value of (SL/EI) and C can be read from the above figures(1,2,3,4) .

(Moment distribution diverges .from authors in transactions ASCE)

CHAPTER – 6

6.1 PROGRAM TO DETERMINE THE STABILITY OF A STRUCTURE

```
#include<iostream.h>
#include<conio.h>
#include<math.h>
main()
{
    int i,j,k,l,m,n,f;
    float b[15],c[15],e[15],z[15],s[15],r[15],y[9],g;
    char a[15],d[15];
    clrscr();
    for(f=0;f<15;f++)
    {
     b[f]=c[f]=e[f]=z[f]=s[f]=r[f]=y[f]=g=0.0;
    }
    cout<<"enter name of the bars" ;
    for(i=0;i<15;i++)
    cin>>a[i];
    cout<<"enter length of each bar";
    for(j=0;j<15;j++)
    cin>>b[j];
```

```
    cout<<"enter load in each bar";
    for(k=0;k<15;k++)
    cin>>c[k];
    cout<<"enter type of load";
    for(m=0;m<15;m++)
    cin>>d[m];
    cout<<"enter the stiffness";
    for(n=0;n<15;n++)
    cin>>e[n];

  float x[15];
    for(int p=0;p<15;p++){
    x[p]=sqrt(c[p]/6835.5);}

    for(int q=0;q<15;q++)
    {
    r[q]=x[q]*b[q];
    z[q]=e[q]*b[q];
    s[q]=z[q]/6835.5;
    }

    cout<<"BAR"<<"\t"<<"LENGTH"<<"\t"<<"LOAD"<<"\t"<<"TYPE"<<"\t"
<<"STIFFNESS"<<"\t"<<"k"<<"\t"<<"kL"<<"\t"<<"SL/EI"<<endl;
```

```
    for(l=0;l<15;l++){
    cout<<a[l]<<"\t"<<b[l]<<"\t"<<c[l]<<"\t"<<d[l]<<"\t"<<e[l]<<"\t"<<x[l]<<
"\t"<<r[l]<<"\t"<<s[l]<<endl;

  }
  //calculation of the moments of the nodes
  cout<<"enter moments of the nodes";
  for(int w=0;w<9;w++)
  cin>>y[w];
  for (int o=0;o<9;o++)
  {
     g+=y[o];
  }
  if(g<=1000)
    cout<<"The truss is stable";
  else
    cout<<"The truss is unstable";

    getch();
    }
```

Chapter - 7

Results and Discussion:

Firstly it is determined the frame work is stable when the loads W=100lb.The forces P caused by W in the members of frame work are calculated and listed in table 1.The designation of the individual members is the frame work is found. The table also contains the values of $K=\sqrt{P/EI}$ and KL of each bar.

As a pin-ended column buckles when KL= π, and a rigidly fixed one at KL=2 π.A frame work is obviously stable when the value of KL= π is not exceeded in any of its compression members, obviously unstable if the KL>2 π, that is basically for statically determinate. Column 6 of the table-1 that the frame work has two compression members with KL values between the limits mentioned. Hence the stability of frame work must be determined by the convergence criterion of the moment distribution process. If the ratio of gusset plate length to the total length of the bar is 0.09 for each of the members of the framework, the values of S/ (EI/L) and C was calculated. They are listed in columns 8 and 9.The stiffness of each member multiplied by the constant factor 20/EI is given in column 10.It can be seen that the sum of the stiffness's is positive at each joint indicating that the subsystems are all stable. If this were not true a smaller load W would have to be assumed for the moment distribution processes since the entire structure would be unstable.

The distribution factors are written inside the polygons at each joint, and the carryover factors are shown on the middle of each bar. An external moment of 1000 in-lb is assume to act on joint J, the joint is balanced, and the balancing moments are distributed. The carryover moment are calculated and applied to the far ends of the bars. Next the joint K is balance and the procedure is continued in the usual manner except that joint J is not balanced again, but the carry back moments are permitted to accumulate at J .when all the joints but J is balanced, the total accumulated moments are added of at joint J. In the balancing process the moments shows that total moment is 553in-lb.Since this is less than 1000in-lb applied at the outset one can conclude that the structure is stable. So the process converges and by the convergence criterion, the framework is stable, when W=100lb.

The calculations are now repeated for W=105lb.The constants of the problems are calculated and balancing process is carried out. The carry back moments at joint J now add up to 1409in-lb and since this is larger than the initial 1000in-lb, the process diverges. So the structure is unstable. The buckling load of the framework is w=102.5lb and the error is less than +/- 2.5. It is noted that both the convergence at 100lb and the divergence at 105lb are very pronounced. The rapid and well defined change from convergence to divergence made it possible to obtain the buckling load more accurately. The maximum end fixity coefficient of KJ member is KL/ π2=3.02.

This shows that the rigid connection between the members of the framework and the effect of the gusset plates contribute considerably to the strength of the framework.

CHAPTER - 8

CONCLUSION

The end of bars in actual in actual not ideal frame works are welded or riveted together rather than connected by pin joints. When a compression member of frame work buckles , its ends are consequently not free to rotate , but are restrained by the other members . naturally the restraint absolutely rigid and the joints to which the compression member is attached rotates slightly and elastically because due to the moment exerted on them by the member that buckles.

When the imaginary pins are inserted at the joints into the truss, the resulting system has a lower buckling load than the actual one.in reality the entire truss buckles as a single unit .whenever one highly compressed member deflects , continuity at the rigidly connected joints requires that all the other members also deflect . inspite of the complexity of this problem , a simple procedure is available for its solution because of a peculiarity of Hardy Cross method: it converges only when the loads are smaller than the critical values, and it diverges when the loads exceed the critical values. Hence this procedure is suggested for evaluating the buckling load of the truss. It is a step by step moment distribution method .it is shown that in each cycle of moment distribution , consisting of balancing, distribution and carry over factor of the system decreases to the minimum value compatible with requirementthat the far ends of the bars be rigidly fixed. And this shows that the rigid connections between the members of the frame work and the effect of the gusset plates contribute considerably to the strength of the framework.

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