APPROXIMATE LATERAL LOAD ANALYSIS OF TALL BUILDINGS- A COMPARATIVE STUDY

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ABSTRACT

For preliminary design of multistorey buildings, information regarding stress resultants due to lateral load are required even before arriving at member dimensions. Several alternatives have to be examined for arriving at member dimensions. Portal method and Cantilever method are commonly used for carrying out analysis as they do not require cross sectional dimensions. This paper discusses various other methods for approximate lateral load analysis of tall buildings. A 2D frame subjected to lateral load is chosen for the analysis. The results are then compared with exact solutions and the best alternative methods brought out. It is found that the methods discussed hereunder overcome the disadvantages of Portal method and Cantilever method. It is also highlighted that the solutions resulting from approximate methods are not realistic for those frames whose member dimensions are arbitrarily fixed without engineering judgement.

Keywords: Analysis, Approximate, Lateral load, Portal, Multistorey, Cantilever.

I. INTRODUCTION

Multistorey building design is an iterative procedure. The design is primarily governed by the lateral loads, viz., wind, earthquake and blast loads. For designing the columns, beams and beam-columns, to begin with knowledge regarding the stress resultants caused by these load is needed even before the cross sectional dimensions are known. Several alternatives have to be examined for evaluating best member dimensions. For arriving at the optimal member sizes, judicious choice of a method in preliminary analysis curtails the number of cycles facilitating easy reach of the final solution in one or two repetitions. Regular moment resisting framed building can be analysed as a plane frame building even though modern computers have the capability to perform three dimensional analyses. However, the restriction of computer use is that member properties (b, d, or I) and material properties i.e., Young’s modulus, Modulus of rigidity and Poisson’s ratio are necessary for use as input. Experienced analyst and architects will be in a position to predict member dimensions for the beams and columns. However, their estimation will be subjected to variation from time to time and may differ from person to person. In general, such empirical decisions may not be consistent. Hence, if a sound approximate method is used during early stages; personal errors will not creep in to the solution. To overcome these difficulties, preliminary analysis is adopted using the approximate methods. These approximate methods are based on some assumptions. For preliminary analysis of these frames subjected to lateral loads, approximate methods, i.e. portal and cantilever methods are used. The portal method is recommended for short frames and the cantilever method is advocated for tall frames. At present, there is no distinct guide line available to distinguish between tall and short frames. The analyst has to use his discretion to decide whether a given frame is tall or short.

II. OBJECTIVE

a) To analyze a multistorey frame subjected to lateral load by seven approximate methods.

b) To compare the results thus obtained and bring out the best method.

III. PROBLEM

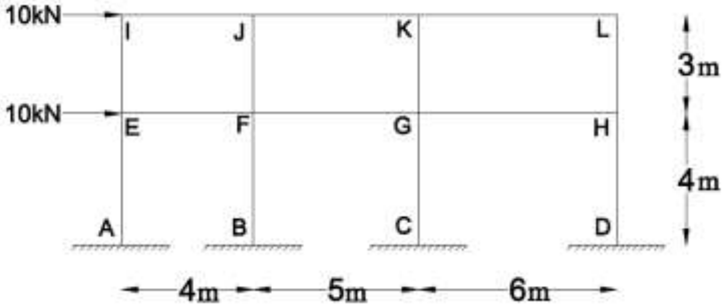

Figure 1: Frame Subjected To Lateral Load

IV. METHODOLOGY

4.1 Approximate Lateral Load Analysis by Load Index

A tentative assumption is made for the load distribution. The distribution of the load is shown in figure 2. The storey shear P is distributed between the rectangle and the parabola. For this purpose a parameter known as "load index" denoted as RXP is used. RXP means the rectangular portion carries X percent of total storey shear P. For example, R75P indicates, the rectangular section carries 75% of total storey shear P and 25% carries by the parabola. In the present study, three levels of load percentages are considered. They are R100P, R75P and R50P.

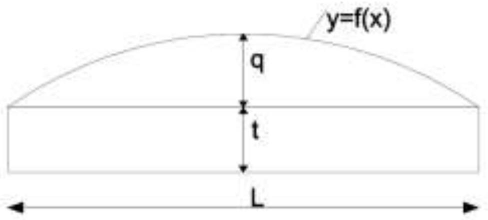

Figure 2: Distribution of Storey Shear

Procedure

a) Storey which consist of n bays is split into n aisles each carrying nodal load T_i.

b) Find the maximum ordinate for rectangle and parabola based on load index RXP

For rectangle

txL=X% of P

For parabola

$\frac{2}{3}qL=(100-X)\%$ of P

c) Calculate the nodal load for each aisle T_i.

$T_{1r}=\int_{li}^{li+1} f(x)\,dx$

$T_{1q}=\int_{li}^{li+1} f(x)\,dx$

$T_1=T_{1r}+T_{1q}$

d) Calculate the column shear

$V_1=0.5T_1$

$V_2=0.5(T_1+T_2)$

$V_3=0.5(T_2+T_3)$

$V_4=0.5T_3$

e) Calculate the column terminal moment by multiplying column shear with lever arm.

f) Compute beam end moment by moment equilibrium i.e, sum of column moment at a joint is equal to the sum of beam end moment at the same joint.

Table 1: Comparison of Results of Load Index Method and Exact Analysis for Column Moments

Member (columns)	R100P kNm	R75P kNm	R50P kNm	Exact Analysis	Error% R100P	Error% R75P	Error% R50P
AE	5.33	4.88	4.42	6.48	17.75	11.75	31.79
EA	5.33	4.88	4.42	5.53	3.62	7.20	20.07
BF	12.0	12.24	12.48	13.19	9.02	6.25	5.38
FB	12.0	12.24	12.48	11.52	4.17	6.49	8.33
CG	14.67	15.12	15.58	16.17	9.28	9.96	3.65
GC	14.67	15.12	15.58	13.75	6.69	24.88	13.30
DH	8.0	7.76	7.52	10.33	22.56	18.49	27.20
HD	8.0	7.76	7.52	9.52	15.97	39.69	21.00
EI	2.03	1.83	1.66	1.31	54.96	24.07	26.72
IE	2.03	1.83	1.66	2.41	15.77	32.28	31.12
FJ	4.5	4.59	4.68	3.47	29.68	13.72	34.87
JF	4.5	4.59	4.68	5.32	15.41	69.76	12.03
GK	5.51	5.67	5.84	3.34	64.97	10.14	74.85
KG	5.51	5.67	5.84	6.31	12.67	21.56	7.45
HL	3.0	2.91	2.82	3.71	19.14	24.02	23.99
LH	3.0	2.91	2.82	3.83	21.67	11.75	26.37

R100P	R75P	R50P
Mean=15.63	Mean=14.52	Mean=16.29
SD= 7.31	SD= 7.22	SD=8.53

Table 2: Comparison of Results of Load Index Method and Exact Analysis for Beam Moments

Member (beams)	R100P kNm	R75P kNm	R50P kNm	Error % R100P	Error % R75P	Error% R50P
EF	7.36	6.71	6.08	7.60	1.90	11.11
FE	7.36	6.71	6.08	12.2	2.29	7.31
FG	9.14	10.12	11.08	8.42	20.05	31.43
GF	9.14	10.12	11.08	3.28	14.35	25.19
GH	11.03	10.67	10.34	6.06	2.6	0.57
HG	11.03	10.67	10.34	3.08	0.28	3.36
IJ	2.03	1.83	1.66	15.77	24.07	31.12
JI	2.03	1.83	1.66	14.71	23.11	30.25
JK	2.47	2.76	3.02	15.7	5.80	3.07
KJ	2.47	2.76	3.02	13.64	3.5	5.59
KL	3.03	2.91	2.82	15.36	18.72	21.22
LK	3.03	2.91	2.82	29.21	32.1	34.11

R100P	R75P	R50P
Mean=10.53	Mean=10.6	Mean=11.97
SD= 4.75	SD= 9.02	SD=10.22

4.2 Split Frame Method for Short Frames

Procedure:

a) The frame is split into n number of single bay frames each carrying nodal load R_i.

b) Find the areas of column in proportion to the tributary length and combined areas of two column of each split frame.

c) Compute the nodal load by

$$R_i=\frac{R}{A}Q_i$$

d) Compute the column shear of split frame. Since the hinge occur in the middle of beam, shear in column of any storey in a split frame will be same.

e) Calculate column terminal moment and beam terminal moment of all split frames.

f) To get back to the original structure, all the split frames are added.

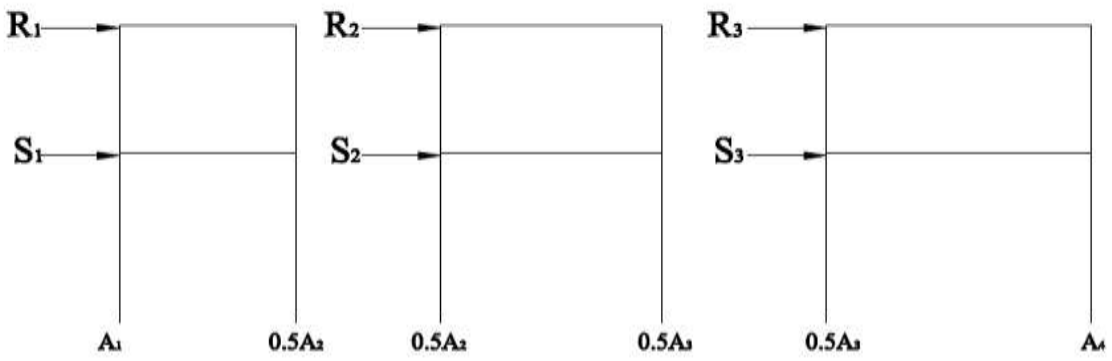

Fig 3: Split Frame with Nodal Load

Table 3: Comparison of Results of Split Frame Method for Short Frames and Exact Analysis

Members Beam	Split Frame Method (kNm)	Exact Analysis (kNm)	Error %
EF	7.77	6.84	13.59649
FE	7.77	6.56	18.44512
FG	9.185	8.43	8.956109
GF	9.185	8.85	3.785311
GH	10.56	10.4	1.538462
HG	10.56	10.7	1.308411
IJ	2.13	2.41	11.61826
JI	2.13	2.38	10.5042
JK	2.505	2.93	14.50512
KJ	2.505	2.86	12.41259
KL	2.88	3.58	19.55307
LK	2.88	4.28	32.71028

Members columns	Split frame method(kNm)	Exact Analysis (kNm)	Error %
AE	5.64	6.48	12.962
EA	5.64	5.53	1.98915
BF	12.32	13.19	6.595906
FB	12.32	11.52	6.944444
CG	14.36	16.17	11.19357
GC	14.36	13.75	4.436364
DH	7.68	10.33	25.65344
HD	7.68	9.52	19.32773
EI	2.13	1.31	62.59542
IE	2.13	2.41	11.61826
FJ	4.64	3.47	33.71758
JF	4.64	5.32	12.78195
GK	5.39	3.34	61.37725
KG	5.39	6.31	14.58003
HL	2.88	3.71	22.37197
LH	2.88	3.83	24.80418

Columns	Beams
Mean=11.5	Mean=10.6
SD= 7.62	SD= 5.94

4.3 Split Frame Method for Tall Frames

Procedure:

a) The frame is split into n number of single bay frames each carrying nodal load R_i.

b) Compute the nodal load R_i based on strength and displacement concept.

Displacement Concept:

$$\frac{R_1}{l_1^2}=\frac{R_2}{l_2^2}=\frac{R_3}{l_3^2}=\frac{P}{l_1^2+l_2^2+l_3^2}$$

Strength Concept:

$$\frac{R_1}{l_1}=\frac{R_2}{l_2}=\frac{R_3}{l_3}=\frac{P}{l_1+l_2+l_3}$$

c) The final nodal load is taken as the average of the two values.

d) Compute the column shear of split frame. Since the hinge occur in the middle of beam, shear in column of any storey in a split frame will be same.

e) Calculate column terminal moment and beam terminal moment of all split frames.

f) To get back to the original structure, all the split frames are added.

Table 4: Comparison of Results of Split Frame Method for Tall Frames and Exact Analysis

Members Columns	Split frame method(kNm)	Exact Analysis (kNm)	Error %
AE	4.74	6.48	26.85185
EA	4.74	5.53	14.28571
BF	11.32	13.19	14.17741
FB	11.32	11.52	1.736111
CG	15.24	16.17	5.751391
GC	15.24	13.75	10.83636
DH	8.66	10.33	16.16651
HD	8.66	9.52	9.033613
EI	1.78	1.31	35.87786
IE	1.78	2.41	26.14108
FJ	4.25	3.47	22.47839
JF	4.25	5.32	20.11278
GK	5.72	3.34	71.25749
KG	5.72	6.31	9.350238
HL	3.25	3.71	12.39892
LH	3.25	3.83	15.1436

Members beam	Split Frame Method (kNm)	Exact Analysis (kNm)	Error %
EF	6.52	6.84	4.678363
FE	6.52	6.56	0.609756
FG	9.05	8.43	7.354686
GF	9.05	8.85	2.259887
GH	11.91	10.4	14.51923
HG	11.91	10.7	11.30841
IJ	1.78	2.41	26.14108
JI	1.78	2.38	25.21008
JK	2.47	2.93	15.69966
KJ	2.47	2.86	13.63636
KL	3.25	3.58	9.217877
LK	3.25	4.28	24.06542

Columns	Beams
Mean=13.45	Mean=12.89
SD= 7.16	SD= 8.37

4.4 Variable Beam Shear Method

Procedure

a) Beam shear is proportioned according to bay length in terms of x.

$l_1x : l_2x : l_3x$

b) Column shear is then written by joint equilibrium condition in terms of x.

Column shear= $\frac{Sum\ of\ beam\ moment\ at\ a\ joint}{Lever\ arm}$

c) The unknown value x is found by storey condition of the storey.

Sum of column shear = Storey shear.

d) Compute column shear and beam shear and column moment is then obtained by multiplying column shear with the lever arm.

Table 5: Comparison of Results of Variable Beam Shear Method and Exact Analysis

Members beam	Variable Beam Shear(kNm)	Exact Analysis (kNm)	Error %
EF	5.7136	6.84	16.46784
FE	5.7136	6.56	12.90244
FG	8.9275	8.43	5.901542
GF	8.9275	8.85	0.875706
GH	12.8556	10.4	23.61154
HG	12.8556	10.7	20.14579
IJ	1.58	2.41	34.43983
JI	1.58	2.38	33.61345
JK	2.435	2.93	16.8942
KJ	2.435	2.86	14.86014
KL	3.5064	3.58	2.055866
LK	3.5064	4.28	18.07477

Columns	Beams
Mean=11.45	Mean=13.18
SD= 7.5	SD= 7.33

Members columns	Variable Beam Shear(kNm)	Exact Analysis (kNm)	Error %
AE	4.15	6.48	35.95679
EA	4.15	5.53	24.95479
BF	10.65	13.19	19.25701
FB	10.65	11.52	7.552083
CG	15.84	16.17	2.040816
GC	15.84	13.75	15.2
DH	9.35	10.33	9.486931
HD	9.35	9.52	1.785714
EI	1.56	1.31	19.08397
IE	1.56	2.41	35.26971
FJ	3.99	3.47	14.98559
JF	3.99	5.32	25
GK	5.94	3.34	77.84431
KG	5.94	6.31	5.863708
HL	3.51	3.71	5.390836
LH	3.51	3.83	8.355091

4.5 Stationary Beam Shear Method

This method is suitable for short frame whose height-width ratio is less than five. Since the frame is short, panel distortion occurs due to shearing action. Hence the bending action is very small and axial deformation in the interior columns will be negligible. Therefore it is assumed that axial forces in the interior columns are zero. This is the key assumption which facilitates the analysis to be performed in a simple manner.

Assumptions:

a) Hinges occur in the middle of all the beams.

b) In the top most storey, hinges occur in the columns at 0.55h from top where h is the height of the storey.

c) In the bottom most storey, hinges occur in the columns at 0.55h from bottom where h is the height of the storey.

d) Axial forces in the interior columns are zero. From this assumption it is stated that in any horizontal plane passing through the hinges of the columns, the overturning moment produced by lateral load is resisted by couple produced by the axial forces in the two outer exterior columns. Because of this shear in all beams in a storey is same.

Procedure:

a) Compute moment in each storey due to storey shear and find the axial force in outer column of each storey by

Axial force in outer column = $\frac{\text{Moment in each storey}}{\text{Total bay length}}$

b) Compute the beam shear of each storey.

c) Calculate the beam terminal moment by multiplying beam shear with lever arm.

d) Compute column shear by moment equilibrium at a joint

Column shear=$\frac{\text{Sum of beam moment at a joint}}{\text{Lever arm}}$

e) Calculate column terminal moment by multiplying column shear with lever arm.

Table 6: Comparison of Results of Stationary Beam Shear Method and Exact Analysis

Members beam	Stationary Beam Shear(kNm)	Exact Analysis (kNm)	Error %
EF	6.6	6.84	3.508772
FE	6.6	6.56	0.609756
FG	8.25	8.43	2.135231
GF	8.25	8.85	6.779661
GH	9.9	10.4	4.807692
HG	9.9	10.7	7.476636
IJ	2.2	2.41	8.713693
JI	2.2	2.38	7.563025
JK	2.75	2.93	6.143345
KJ	2.75	2.86	3.846154
KL	3.3	3.58	7.821229
LK	3.3	4.28	22.8972

Columns	Beams
Mean=11.5	Mean=10.6
SD= 7.62	SD= 5.94

Member columns	Stationary Beam Shear (kNm)	Exact Analysis (kNm)	Error %
AE	5.872	6.48	9.38271
EA	4.8	5.53	13.2007
BF	13.2	13.19	0.07581
FB	10.8	11.52	6.25
CG	16.13	16.17	0.24737
GC	13.19	13.75	4.07272
DH	8.8	10.33	14.8112
HD	7.2	9.52	24.3697
EI	2.195	1.31	67.5572
IE	1.795	2.41	25.5186
FJ	4.05	3.47	16.7147
JF	4.95	5.32	6.95488
GK	4.954	3.34	48.3233
KG	6.05	6.31	4.12044
HL	2.7	3.71	27.2237
LH	3.3	3.83	13.8381

4.6 Distribution of Shear to Column for Short Multistorey Frames

Procedure:

a) Split the given frame into n number of single bay frames each carrying nodal load R_i.

b) Calculate the proportion based on Theorem 1 and Theorem 2

Theorem 1:

$$\frac{R_1}{(h+l_1)} : \frac{R_2}{(h+l_2)} : \frac{R_3}{(h+l_3)}$$

Theorem 2:

$$\frac{R_1}{l_1} \quad \frac{R_2}{l_2} \quad \frac{R_3}{l_3}$$

c) The storey shear is then distributed as nodal load for each split frame by taking the average of the above two proportions.

d) Knowing the value of R_i, shear in each column in any floor is found by just halving the shear force at the level of column hinges.

e) Compute column terminal moment by multiplying column shear with lever arm.

f) Beam end moment is calculated by applying moment equilibrium at a joint i.e, sum of column moment at a joint is equal to sum of beam moment at same joint.

Table 7: Comparison of Results of Distribution of Shear Method and Exact Analysis

Members Beam	Distribution of shear to column(kNm)	Exact Analysis (kNm)	Error %
EF	7.65	6.84	11.84211
FE	7.65	6.56	16.61585
FG	9.16	8.43	8.659549
GF	9.16	8.85	3.502825
GH	10.73	10.4	3.173077
HG	10.73	10.7	0.280374
IJ	2.09	2.41	13.27801
JI	2.09	2.38	12.18487
JK	2.5	2.93	14.67577
KJ	2.5	2.86	12.58741
KL	2.93	3.58	18.15642
LK	2.93	4.28	31.54206

Members columns	Distribution of shear to column(kNm)	Exact Analysis (kNm)	Error %
AE	5.56	6.48	14.19753
EA	5.56	5.53	0.542495
BF	12.22	13.19	7.354056
FB	12.22	11.52	6.076389
CG	14.47	16.17	10.5133
GC	14.47	13.75	5.236364
DH	7.8	10.33	24.49177
HD	7.8	9.52	18.06723
EI	2.09	1.31	59.54198
IE	2.09	2.41	13.27801
FJ	4.59	3.47	32.27666
JF	4.59	5.32	13.7218
GK	5.43	3.34	62.57485
KG	5.43	6.31	13.94612
HL	2.93	3.71	21.02426
LH	2.93	3.83	23.49869

Columns	Beams
Mean=13.26	Mean=10.45
SD= 6.99	SD= 5.32

4.7 Factor Method

The factor method is more accurate than either the portal method or the, cantilever method. The portal method and cantilever method depend on assumed location of hinges and column shears whereas the factor method is based on assumptions regarding the elastic action of the structure. For the application of Factor method, the relative stiffness (k = I/l), for each beam and column should be known or assumed, where, I is the moment of inertia of cross section and l is the length of the member

Procedure:

a) The girder factor g, is determined for each joint from the following expression.

$$g=\frac{\Sigma k_c}{\Sigma k}$$

where, Σk_c - Sum of relative stiffness of the column members meeting at that joint.

Σk - Sum of relative stiffness of all the members meeting at that joint.

Each value of girder factor is written at the near end of the girder meeting at the joint.

b) The column factor c, is found for each joint from the following expression

c = 1-g

Each value of column factor c is written at the near end of each column meeting at the joint. The column factor for the column fixed at the base is one. At each end of every member, there will be factors from step (a) or step (b). To these factors, half the values of those at the other end of the same member are added.

c) The sum obtained as per step (b) is multiplied by the relative stiffness of the respective members. This product is termed as column moment factor C, for the columns and the girder moment factor G, for girders.

d) Calculation of column end moments for a typical member ij - The column moment factors [C values] give approximate relative values of column end moments. The sum of column end moments is equal to horizontal shear of the storey multiplied by storey height. Column end moments are evaluated by using the following equation,

Mij = Cij A

where, Mij - moment at end i of the ij column

Cij - column moment factor at end i of column ij

A - storey constant given by

$$A=\frac{Horizontal\ Shear \times Height\ of\ storey}{Sum\ of\ column\ end\ moment\ factor\ of\ the\ storey}$$

e) Calculation of beam end moments - The girder moment factors [G values] give the approximate relative beam end moments. The sum of beam end moments at a joint is equal to the sum of column end moments at that joint. Beam end moments can be worked out by using following equation,

Mij= GijB

where, Mij - moment at end i of the ij beam

Gij- girder moment factor at end i of beam ij

B - joint constant given by

$$B=\frac{Sum\ of\ column\ moments\ at\ that\ joint}{Sum\ of\ the\ girder\ end\ moment\ factors\ of\ that\ joint}$$

Table 8: Comparison of Results of Factor Method and Exact Analysis

Members Beam	Factor Method (kNm)	Exact Analysis (kNm)	Error %
EF	6.914	6.84	1.081871
FE	6.371	6.56	2.881098
FG	8.07	8.43	4.270463
GF	8.628	8.85	2.508475
GH	9.36	10.4	10
HG	9.978	10.7	6.747664
IJ	2.197	2.41	8.838174
JI	2.057	2.38	13.57143
JK	2.613	2.93	10.81911
KJ	2.686	2.86	6.083916
KL	2.85	3.58	20.39106
LK	3.095	4.28	27.68692

Columns	Beams
Mean=13.23	Mean=7.93
SD= 7.36	SD= 5.16

Members columns	Factor Method (kNm)	Exact Analysis (kNm)	Error %
AE	6.17	6.48	4.783951
EA	4.867	5.53	11.98915
BF	12.504	13.19	5.20091
FB	10.06	11.52	12.67361
CG	15.567	16.17	3.729128
GC	15.805	13.75	14.94545
DH	10.904	10.33	5.556631
HD	7.118	9.52	25.23109
EI	2.047	1.31	56.25954
IE	2.197	2.41	8.838174
FJ	4.388	3.47	26.45533
JF	4.67	5.32	12.21805
GK	5.185	3.34	55.23952
KG	5.55	6.31	12.04437
HL	2.86	3.71	22.91105
LH	3.095	3.83	19.1906

4.8 K Values Method

Computer solutions are based on member cross sectional dimensions. The principal use of this method is to furnish answers to check the computer solution. Secondly any two storeys can be analyzed independent of the other storeys. This is a significant advantage of this method. The K-values method is based on relative I/l values. In case of frame the shear carried by each column is directly proportional to its K value, when beam are assumed to be infinitely rigid. Assumptions regarding hinge formations are same as that of in stationary beam shear method.

Procedure

a) 25% of storey shear is distributed among beams in proportion to their K values. Each value in the bay is then equally divided between 2 columns in the bay.

b) Remaining 75% of storey shear is distributed among columns in proportion to their K values.

c) Column shear is then computed by adding the above two contribution in each column.

d) Column terminal moment is obtained by multiplying column shear with lever arm.

e) Beam moments are obtained by moment equilibrium at a joint, i.e sum of column terminal moment at a joint should be equal to the sum of beam moment at the same joint.

Table 9: Comparison of Results of K Values Method and Exact Analysis

Members Beam	K Values Method (kNm)	Exact Analysis (kNm)	Error %
EF	6.746	6.84	1.374269
FE	5.33	6.56	18.75
FG	8.54	8.43	1.304864
GF	7.995	8.85	9.661017
GH	9.99	10.4	3.942308
HG	10.8795	10.7	1.67757
IJ	2.245	2.41	6.846473
JI	2.044	2.38	14.11765
JK	2.575	2.93	12.11604
KJ	2.664	2.86	6.853147
KL	3.33	3.58	6.98324
LK	3.62	4.28	15.42056

Columns	Beams
Mean=9.01	Mean=8.25
SD= 4.98	SD= 5.59

Members columns	K Values Method (kNm)	Exact Analysis (kNm)	Error %
AE	5.997	6.48	7.453704
EA	4.906	5.53	11.28391
BF	12.342	13.19	6.429113
FB	10.098	11.52	12.34375
CG	15.99	16.17	1.113173
GC	13.08	13.75	4.872727
DH	9.677	10.33	6.321394
HD	7.9176	9.52	16.83193
EI	1.84	1.31	40.45802
IE	2.245	2.41	6.846473
FJ	3.78	3.47	8.933718
JF	4.62	5.32	13.15789
GK	4.91	3.34	47.00599
KG	5.995	6.31	4.992076
HL	2.962	3.71	20.16173

V. DISCUSSIONS

The different approximate methods are compared with the exact analysis done by slope deflection method. For a linearly elastic structure, exact analysis is one which satisfies both equilibrium and compatibility conditions. On other hand, approximate methods used in lateral analysis fulfill only equilibrium requirement. If the assumptions used in the approximate analysis regarding hinge and shear force or axial force coincide with that of the hinge positions and hinge conditions of the exact method then both results will be alike. Theoretically there can be innumerable frames with different member cross sectional dimensions but with same structure dimensions and loading. The exact solution of each frame will be different even though the structure dimensions and loading are same. On the other hand the approximate solution will give only one solution which will closely match with only one of the exact solution. In general multistorey building design is an iterative process. Several trials are needed before arriving at the final dimensions of the members for a given structure dimensions and prescribed loading. A practical frame is arrived after satisfying serviceability and strength criteria. It is found that for such a practical frame the approximate method yield reasonable solution. Therefore the method will fail if applied to a frame in which the member dimensions are fixed in an arbitrary manner.

The simplified portal method is based on the assumption that axial forces in the interior columns are zero. The flaw of this method is that it predicts same magnitude of beam terminal moment in all the bays of storey which is contrary to the expectations. Thus moderate magnitude of axial force is produced in inner columns which is contrary to the assumptions. This two flaws are rectified in the methods described for short frames. In load index method, the results of load index R100P is almost close with the results obtained in improved portal method. Also the results of load index R50P is almost close with the results obtained by cantilever method.

VI. CONCLUSIONS

a) The different load index from load index method have been implemented for analyzing the following type of frame.
b) R100P- Short frames.
c) R75P- Medium rise frames.
d) R50P- Tall frames.

e) The results obtained by split frame method for short frames and split frame method for tall frames are in harmony with the solution of improved portal method and cantilever method respectively. This method does not involve much computational effort. For tall building it is better to deal with shear force as done in split frame method than axial forces in cantilever method which is prone to mistakes.

f) Variable beam shear method and stationary beam shear method can be used as a supplement method to overcome the disadvantage of simplified portal method.

g) K values method can be used for checking the solution obtained by computer analysis.

h) Approximate method solutions will fail for those frames whose member dimensions are arbitrarily fixed.

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