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A QUICK METHOD FOR ESTIMATING THE STOREY STIFFNESS OF A BUILDING BY USING SAP-2000

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ABSTRACT

This research the formulation of storey stiffness is done by conceptual simplification of converting actual stiffness matrix into tri diagonal matrix by converting actual frame into shear frame. This method is applicable for all type of buildings to calculate storey stiffness without applying corrections factors at boundary stories. A simple example is included to illustrate the ease with which the proposed expression is applied. The high efficiency and satisfactory precision of this method are ascertained by comparison with shear building for different response spectrum analysis and time history analysis.

Keywords : story stiffness, stiffness matrix, building, SAP-2000.

I. INTRODUCTION

Storey stiffness, a measure of lateral stiffness, is often used in preliminary seismic design. By definition, it is the ratio of storey force to storey drift and holds good only in case of shear building, i.e., with floor beams of infinite rigidity. In such cases, lateral stiffness matrix of a building takes a tri-diagonal form. However, in buildings with finite beam stiffness have fully populated stiffness matrix. In such a case, storey stiffness hardly exists, but preliminary seismic design requires it to be known in an approximate sense. Therefore, there is a need to redefine the storey stiffness which was also required for seismic design of building with metallic dampers.

In spite of the existence of many computer packages for the analysis and design of building systems, a quick estimation method for the response of building systems to lateral loads is still of great value. This is for two reasons. First, all computer programs need some initial values for the cross-sectional properties of the building structural elements for which the designer has to perform some approximate hand calculations. Obviously, if the initial guesses for the cross-sectional properties are very far from the real required values, the design optimization process will be too time-consuming. Second, there is usually no certainty of the correctness of the data entry or matching of the data entered by the user with reality, specially in the case of inexperienced engineers working with complicated software. In this case, a control or final checking tool is very useful or even necessary.

In most practical cases, the assumption of zero joint rotations introduces a substantial amount of error. Rubinstein and Hurty (1961) have indicated that neglecting the effect of joint rotations can lead to gross errors in computed dynamic properties. They demonstrated that the majority of this error can be eliminated with reasonable assumptions of joint behavior, such as equal rotations for exterior and interior joints in a floor of a frame and equal rotations for all joints in a given floor of a structure comprising multiple frames that are not identical. Goldberg (1972) successfully approximated the effect of joint flexibility by assuming an approximate average value for joint rotation at each floor of a multistory frame. He was using an iterative slope-deflection procedure to calculate drift. The works of these authors clearly demonstrate that if the stiffness of stories are modified to reflect girder flexibility in a realistic manner, the shear building becomes aaviable mathematical model for approximating the response of laterally loaded elastic frames.

The purpose of this Thesis is to present explicit, closed-form expressions for approximating the lateral stiffnesses of stories in elastic frames. The expressions presented in this thesis are limited to rectangular frames thatare fixed at the base and for which only flexural deformations are important.Several existing expressions are reviewed and compared. An alternate formulations presented that includes correction factors that enable the approximate stiffness expression to

(1) Simulate the effect of variation in adjacent story heights;
(2) more accurately represent the stiffnesses of boundary stories (first, second, and top); and
(3) approximate the stiffening effect of a fixed base in low-rise frames.

The approach taken herein achieves the same goal as static condensation of rotational degrees of freedom.However,this process is performed prior to formulation of equilibrium equations. Consequently, the softening effect of joint rotations on story stiffness is only approximated. A simple example is included to illustrate the ease with which the proposed expression is applied.

Apparent lateral stiffness of a story

The horizontal stiffness of the TADAS element K_a is a function of the lateral stiffness of the braces K_b and the device stiffness K_d(Fig1-2). If the ratio of the horizontal TADAS element stiffness K_a to the storey stiffness is defined as SR (Xia and Hanson 1992).

$$SR = \frac{K_a}{K_f} \tag{1}$$

$$K_a = \frac{K_b K_d}{K_b + K_d} \tag{2}$$

In the figure the elastic stiffness K_s of the frame with TADAS is given by

$$K_s = K_a + K_f \tag{3}$$

Where, K_f represents the story stiffness of the frame. In addition Δ_{y1} and Δ_{y2} represent the yield displacements of the TADAS device and the frame. R_{y1} and R_{y2} represent the total restoring forces developed in the system when Δ_{y1} and Δ_{y2} are reached. Let SHR_a is defined as the ratio of the TADAS element stiffness after yield to the initial element stiffness. Then the post yielding strength of the TADAS element is K_a x SHR_a. And also,

$$U = \frac{R_{y2}}{R_{y1}} \tag{4}$$

$$K_a = SRxK_f \tag{5}$$

$$R_{y1} = (K_a + K_f)\Delta_{y1} \tag{6}$$

$$R_{y2} = K_f\Delta_{y2} + K_a\Delta_{y1} + SHR_a K_a(\Delta_{y2} - \Delta_{y1}) \tag{7}$$

Therefore, from equations**Error! Reference source not found.**,**Error! Reference source not found.** ,**Error! Reference source not found.** and **Error! Reference source not found.**

$$\frac{\Delta_{y2}}{\Delta_{y1}} = 1 + \frac{1+SR}{1+SRxSHR_a}x(U-1) \tag{8}$$

From the equation **Error! Reference source not found.** it can be concluded that Δ_{y2} needs to be estimated accurately, which depends on the storey stiffness of the building to fix the design parameters of the TADAS element.

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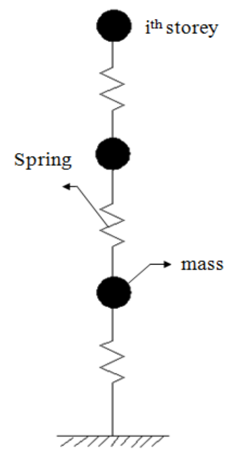


Fig. Shear Building

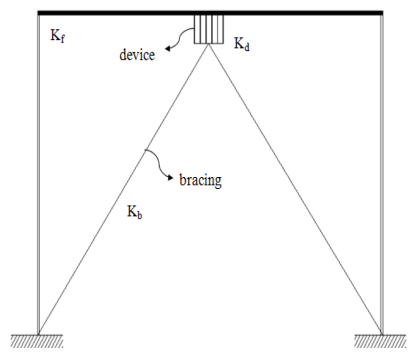
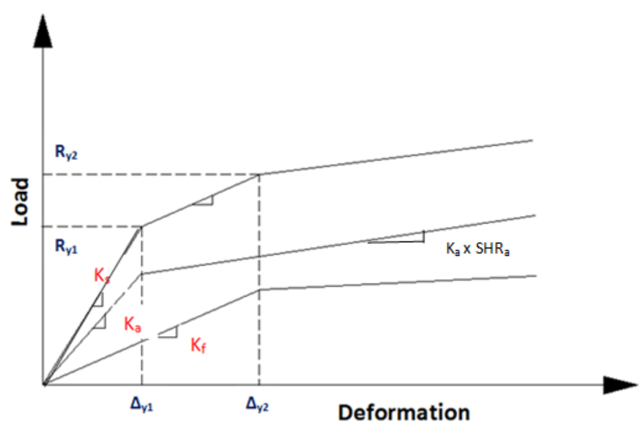


Fig. Building frame with TADAS device



Force Deformation relationship of the TADAS frame system (Modified from T.sai and Chen 1993)

The high efficiency and the satisfactory precision of this method are ascertained by comparison with shear building for different response spectrum analysis and time history analysis.

Benjamin (1959) In his text on indeterminate frame analysis, Benjamin (1959) outlines a method for estimating the stiffness of a story in a laterally loaded elastic frame. The slope deflection formulas are applied successfully to both ends of the four members bounding a typical panel. The effects of gravity loads are neglected, as well as axial deformations of the members. By appropriate manipulation, joint rotations are eliminated from the slope-deflection equations,yielding expressions for drift of the columns in the panel. Benjamin combines column drifts to obtain an average value for the story and indicates that this drift can be used to obtain story stiffness. With some rearrangement o***f*** terms, stiffness *Ks* can be expressed as

$$Ks= \frac{\left(\frac{24Vn}{H}\right)}{\left[\Sigma\left(\frac{\Sigma M_{ec}}{K_{ec}}\right)+\Sigma\left(\frac{\Sigma M_{ic}}{K_{ic}}\right)-\Sigma\left(\frac{\Sigma M_{ga}}{K_{ga}}\right)-\Sigma\left(\frac{\Sigma M_{ga}}{Kgb}\right)\right]}$$

when *n, H,* and 2M = number of panels, the story height, and the sum of the two member end moments, respectively, for a story shear force *V;*flexural stiffness *k* of a member = *EI/L,* and the subscripts *ec, ic, ga,* and *gb* = respectively, exterior columns, interior columns, girders in the floor above, and girders in the floor below.

Muto (1974) In his treatise on seismic analysis of buildings, Muto (1974) approaches the problem of approximating lateral stiffnesses of columns in elastic stories by applying the slope-deflection equations to members in a panel of an idealized regular frame, as did Benjamin (1959). Muto, however, assumes that the frame is an infinite array of members, and that all columns at a story resist shear forces of equal magnitude. He further assumes that both ends of all members undergo equal end rotations. Using the slope-deflection formulas, expressions for member end moments are obtained. Muto uses these expressions in moment equilibrium equations for a typical beamcolumnjoint, from which he extracts the following expression for stiffness *Kc*of the column

$$K_c=\left(\frac{12E_cI_c}{H^3}\right)\left(\frac{4K_g}{4K_c+K_g}\right)$$

To extend this equation to columns in real frames, Muto interprets the term *4kg* as the sum of the flexural stiffnesses of the two girders each framing into the joints at the top and the bottom of the column. Thus column stiffness can be rewritten as

$$K_c=\left(\frac{12E_cI_c}{H^3}\right)\left(\frac{\Sigma K_{ga}+\Sigma K_{gb}}{4K_c+\Sigma K_{ga}+\Sigma K_{gb}}\right)$$

where~*Zkga*and*1,kgb* = respectively, the sum of the flexural stiffnesses of the girders framing into the joint above and the joint below the column. Story stiffness *Ks* is obtained by summing the stiffnesses of all columns at a story.

Schultzet.al(1992) A series of nine-story, five-bay, elastic frames were analyzed to verify the concept of apparent lateral stiffness of a story. As indicated in Table, all stories above the first have the same height, Hs, and the first story is 33% taller. All bays have a span L equal to twice the nominal story height Hs. Moments of inertia for columns and girders are smaller at upper floors, as indicated in Table 1. This variation in stiffness is typical of actual building frames and introduces small or moderate irregularities in profile. Modulus of elasticity E is the same for all members of a frame. A relative stiffness parameter a is defined as the ratio of IJL to IJHS, where IgandIc, respectively, are the nominal values of girder and column moments of inertia. The parameter a is used as a global indication of the relative flexural stiffnesses of girders to columns; its inverse p indicates column stiffness relative to girder stiffness. For each of the frames analyzed, a or p was assigned a value between 1 and 10.

$$K_c=\left(\frac{12E_cI_c}{H}\right)\left[1-\left(\frac{K_c}{\Sigma K_a}\right)-\left(\frac{K_c}{\Sigma K_a}\right)\right]$$

where*kc*= the flexural stiffness of the column. The sums of the stiffnesses of all connecting members in the joints above and below the column are given by *l,ka*andE£fc, respectively. MahmoodHosineni (1999)

For regular moment frames based on fact (1), it is possible to substitute the main frame (a) with the equivalent frames,which are connected to each other by hinges. The values of *I*c and *I*g are given by

$$I_c = \frac{\left(\sum_{j=1}^{m} I_{cj}\right)}{(2m)}$$

$$I_{cj} = \frac{1}{2}\left(\sum_{j=1}^{m} I_{cij}\right)$$

in which *L* is an arbitrary value. It is obvious that the frames are equivalent to *m* of the single frame. Using the same idea for the *n*-story frame makes it possible to introduce the single-bay equivalent frame, and then by using the second aforementioned fact the frame can be substituted by the one, which consists of *n* sub-frames or frame modules connected to each other by hinges. The values of *Ici*and*Igi*in are given by

$$\frac{I_g}{L} = \frac{\left[\sum_{j=1}^{m}\left(\frac{I_{gi}}{l_j}\right)\right]}{m}$$

Where *L* is again an arbitrary value. It is noticeable that the stiffness matrix of the equivalent frame in a full matrix, while that of the is a 3-diagonal matrix, because any force applied in the *i*th floor, keeping other floors unmoved, is resisted by only the *i*th and (*i* . 1)th sub-frames. In fact each of the subis a simple frame module, like that, which has the lateral stiffness of (Image-e-Naiini,4 1997)

$$k_{fm} = \frac{12k_c}{h^2}\frac{k_c(k_d + k_u) + 6k_dk_u}{k_c^2 + 2k_c(k_d + k_u) + 3k_dk_u}$$

$$k_c = \frac{EI_c}{h} \quad k_d = \frac{EI_{gd}}{L} \quad k_u = \frac{EI_{gu}}{L}$$

in which *h, L, I*c, *I*gd and *I*gu are the dimensions and the cross-sectional properties of the frame module, respectively, and *E* is the modulus of elasticity of the frame material. In the case of irregular moment frames in which there are some offsets in the axes of beams or some of beams and columns are omitted, there is no problem in the regular parts. The main frame module of the simplified system for regular moment frames 250 M.

II. DATA ANALYSIS

A seven storey building for a commercial complex has plan dimensions as shown in Fig. The building is located in seismic zone IV on a site with medium soil. Design the building for seismic loads as per IS 1893 (Part 1): 2002.

General

1. The considered building is a seven storey 3 bay structure.
2. Secondary floor beams are so arranged that they act as simply supported beams and that maximum number of main beams get flanged beam effect.
3. The main beams rest centrally on columns to avoid local eccentricity.
4. For all structural elements, M25 grade concrete will be used. Sizes of all columns in upper floors are kept the same. However, for columns up to plinth, sizes are increased.
5. Preliminary sizes of structural components are assumed by experience.

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6. For analysis purpose, the beams are assumed to be rectangular so as to distribute slightly larger moment in columns. In practice a beam that fulfils requirement of flanged section in design, behaves in between a rectangular and a flanged section for moment distribution.
7. Seismic loads will be considered acting in the horizontal direction (along either of the two principal directions) and not along the vertical direction, since it is not considered to be significant.
8. All dimensions are in mm, unless specified otherwise.

Data of the Example:
The design data shall be as follows:

Live load	:	4 kN/m^2 at typical floor
	:	1.5 kN/m^2 on terrace
Floor finish	:	1 kN/m^2
Water proofing	:	2 kN/m^2
Terrace finish	:	1 kN/m^2
Location	:	Ahmedabad
Wind location	:	As per IS: 875-Not designed for wind load, Earthquake loads exceed the wind loads
Earthquake load	:	As per IS-1893 (part 1)-2002
Type of soil	:	Type II, Medium as per IS:1893
Storey height	:	3.5m
Floors	:	G.F + 6 upper floors.
Walls	:	150 mm thick brick wall.

Material Properties

Concrete
All components unless specified in design: M25 grade all
$E_c = 5000\sqrt{fck}$` N/mm$^2$ = $5000\sqrt{fck}$` MN/mm^2
= 25000 N/mm^2 = 25000 MN/mm^2

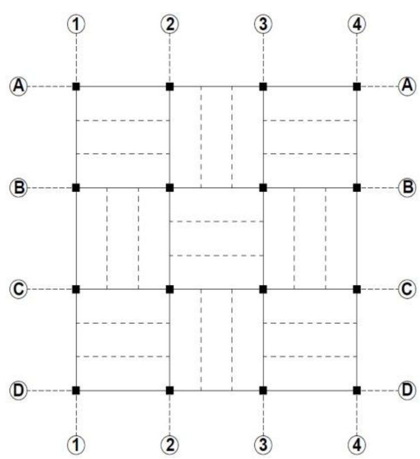

Figure : Typical floor plan

Steel
HYSD reinforcement of grade Fe 415 confirming to IS: 1786 is used throughout

Geometry of the Building
The general layout of the building is shown in above Figure .

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Storey numbers are given to the portion of the building between two successive grids of beams.For the example building between two successive grids of beams. For the example building, the storey numbers are defined as follows:

Portion of the building	**Storey no.**
Foundation top – First floor	1
First floor – Second floor	2
Second floor – Third floor	3
Third floor – Fourth floor	4
Fourth floor – Fifth floor	5
Fifth floor – Six floor	6
Seventh floor – Terrace	7

Column number

In the general plan of Figure 4.1, the columns from C_1 to C_{16} are numbered in a convenient way from left to right and from upper to the lower part of the plan. Column C_5 is known as C_5 from top of the footing to the terrace level. However, to differentiate the column lengths in different stories, the column lengths are known as 105, 205, 305, 405, 505, 605 and 705 [Refer to Figure]. The first digit indicates the storey number. Thus, column length 605 means column length in sixth storey for column numbered C_5. The columns may also be specified by using grid lines.

Floor beams (Secondary beams)

All floor beams that are capable of free rotation at supports are designed as FB in Figure.4. The reactions of the floor beams are calculated manually, which act as point loads on the main beams. Thus, the floor beams are not considered as the part of the space frame modeling.

Main beams number

Beams, which are passing through columns, are termed as main beams and these together with the columns from the space frame. The general layout of Figure 4.1 numbers the main beams B_1 to B_{12} in a convenient way from left to right and from upper to the lower part of the plan. Giving 90^0 clockwise rotations to the plan similarly marks the beams in the perpendicular direction. To floor-wise differentiate beams similar in plan (say beam B_5 connecting columns C_6 and C_7) in various floors, beams are numbered as 1005, 2005, 3005, and so on. The first digit indicates the storey top of the beam grid and the last three digits indicate the beam number as shown in general layout of Figure 4.1. Thus, beam 4007 is the beam located at the top of 4^{th}storey whose number is B_7 as per the general layout.

Gravity Load calculations

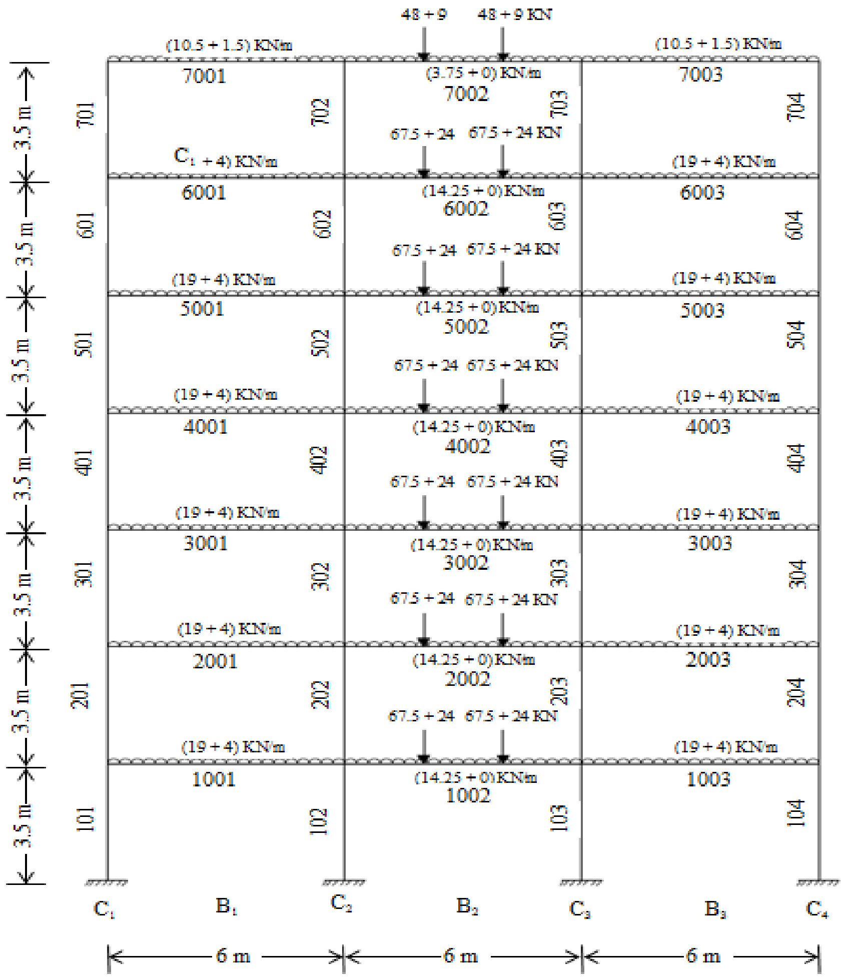

Figure: Gravity loads : Frame A-A

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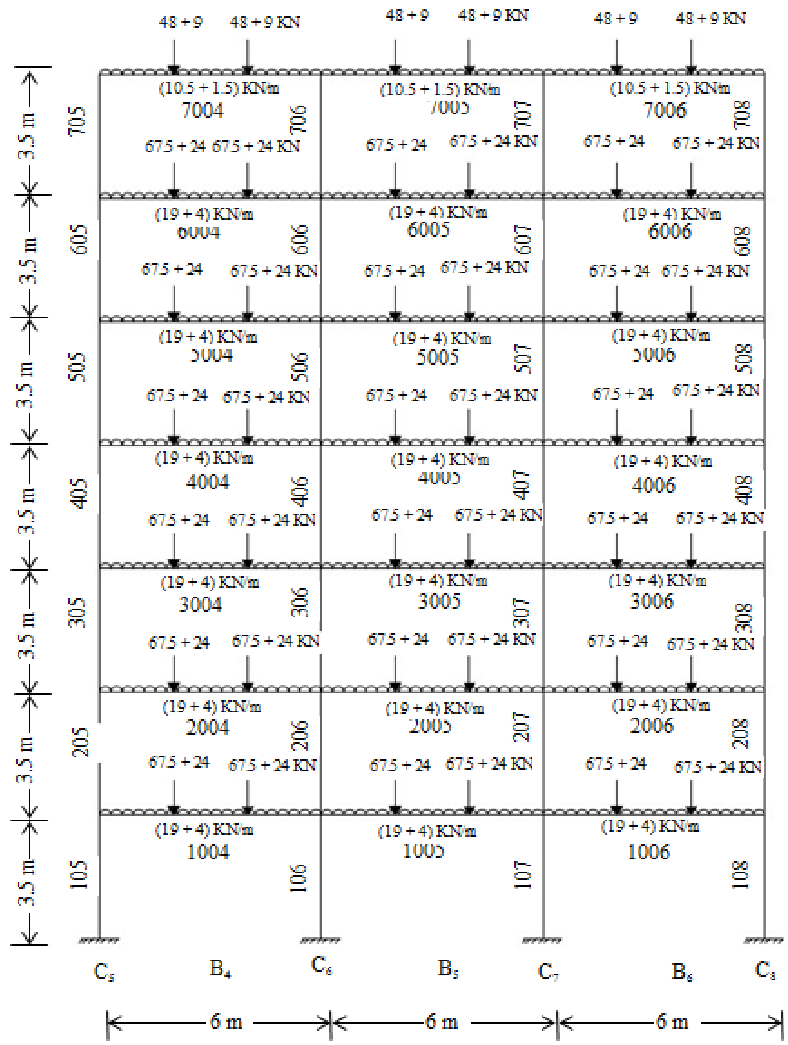

Figure : Gravity loads : Frame B-B

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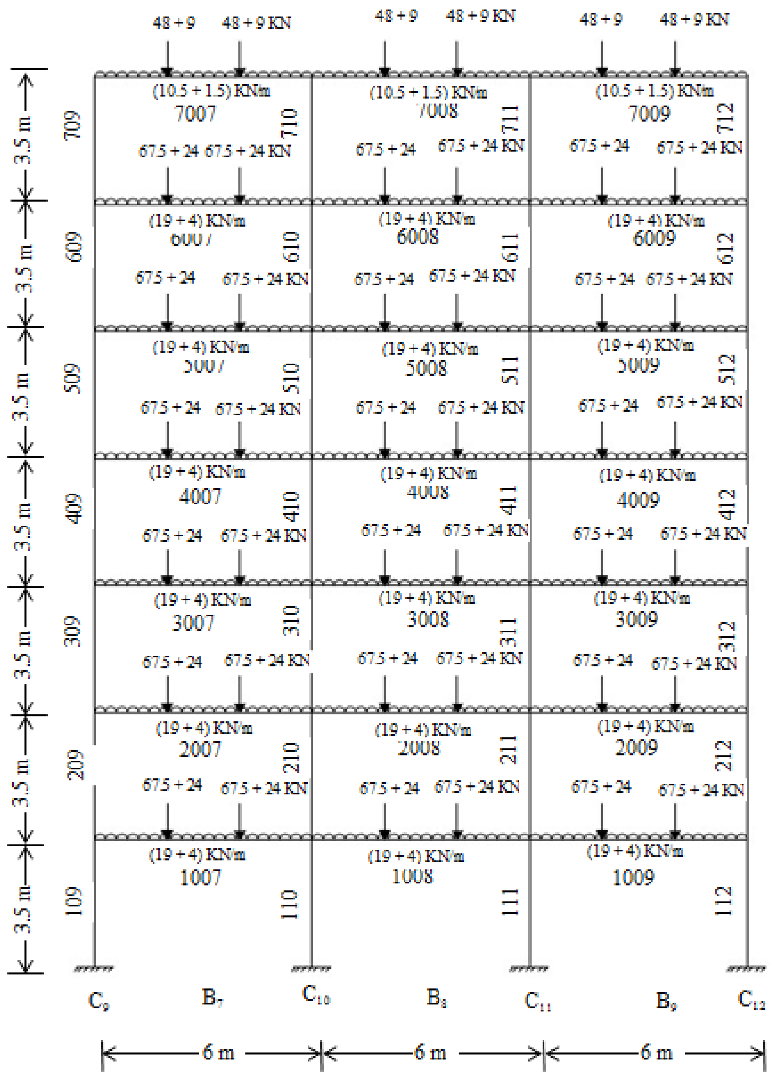

Figure : Gravity loads : Frame C-C

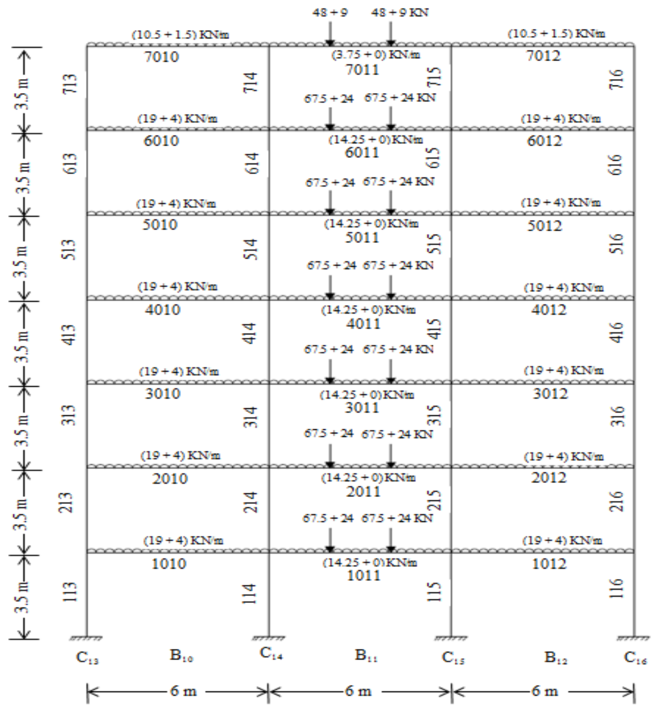

Figure: Gravity loads : Frame D-D

Table- Distribution of horizontal load at each floor level

TOREY NO	WEIGHT (kN)	Hight (m)	W H^2	Q_I	V_B
7	5840	24.5	3505460	772.27	772.27
6	8835	21	3896235	858.36	1630.62
5	8835	17.5	2705718.75	596.08	2226.70
4	8835	14	1731660	381.49	2608.19
3	8835	10.5	974058.75	214.59	2822.78
2	8835	7	432915	95.37	2918.16
1	8835	3.5	108228.75	23.84	2942.00
	58850		**13354276.25**	2942	

SAP 2000 model

In the current thesis SAP 2000 is used for the analysis of building. The same building which is described in the previous sections is created and analyzed for the considered dead and live loads. Some of the figures of SAP 2000 are presented below.

Material properties

This section provides material property information for materials used in the model.

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Material Properties - Basic Mechanical Properties

Material Properties - Basic Mechanical Properties

Table- Material properties given in SAP 2000

Material	UnitWeight N/mm3	UnitMass N-s2/mm4	E1 N/mm2	G12 N/mm2
HYSD415	0.0000E+00	0.0000E+00	200000.00	
M25	0.0000E+00	0.0000E+00	25000.00	10416.67

Section properties

Frame Section Properties

Table4.1 Frame section properties given in SAP2000

SectionName	Material	Shape	t3	t2	I23	Area
			Mm	mm	mm4	mm2
Beam	M25	Rectangular	500	300	0	150000
Column	M25	Rectangular	500	500	0	250000

Load combinations

This section provides load combination information.

Combination Definitions

Table- Design load combination in SAP2000

ComboName	ComboType	CaseName	ScaleFactor
1.5(DL+LL)	Linear Add	DL	1.500000
1.5(DL+LL)		LL	1.500000
1.2(DL+LL+EX)	Linear Add	DL	1.200000
1.2(DL+LL+EX)		LL	1.200000
1.2(DL+LL+EX)		EQ_X	1.200000
1.2(DL+LL+EY)	Linear Add	DL	1.200000
1.2(DL+LL+EY)		LL	1.200000
1.2(DL+LL+EY)		EQ_Y	1.200000
1.5(DL+EX)	Linear Add	DL	1.500000
1.5(DL+EX)		EQ_X	1.500000
1.5(DL+EQY)	Linear Add	DL	1.500000
1.5(DL+EQY)		EQ_Y	1.500000

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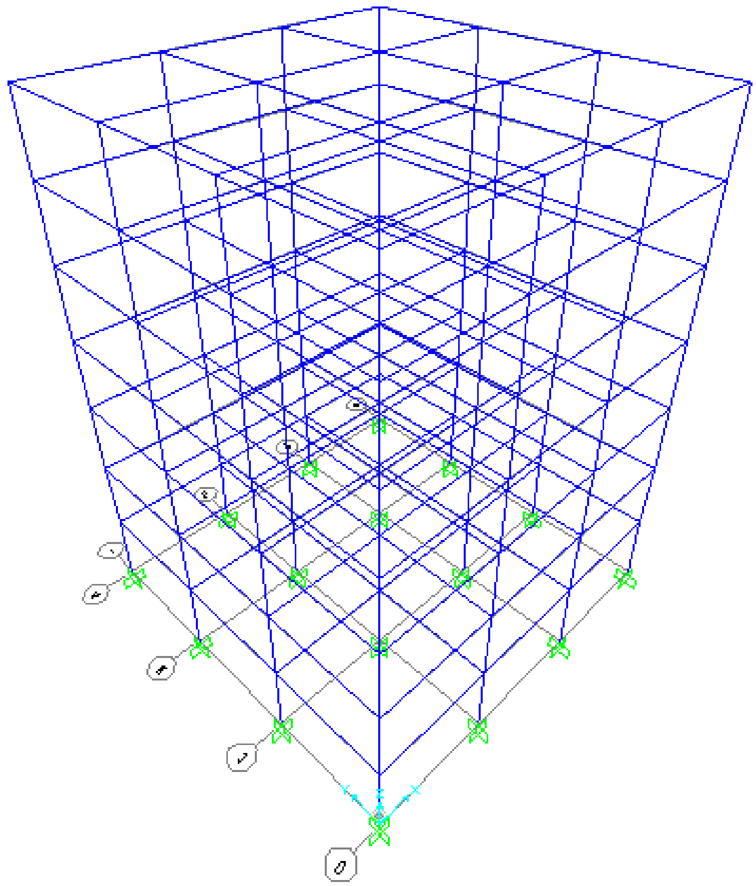

Figure: Building model in SAP 2000

III. RESULT AND DISCUSSION

Considered intensity is incrementally changed (Figure) and a series of dynamic analyses is carried out, as reflected by the name.

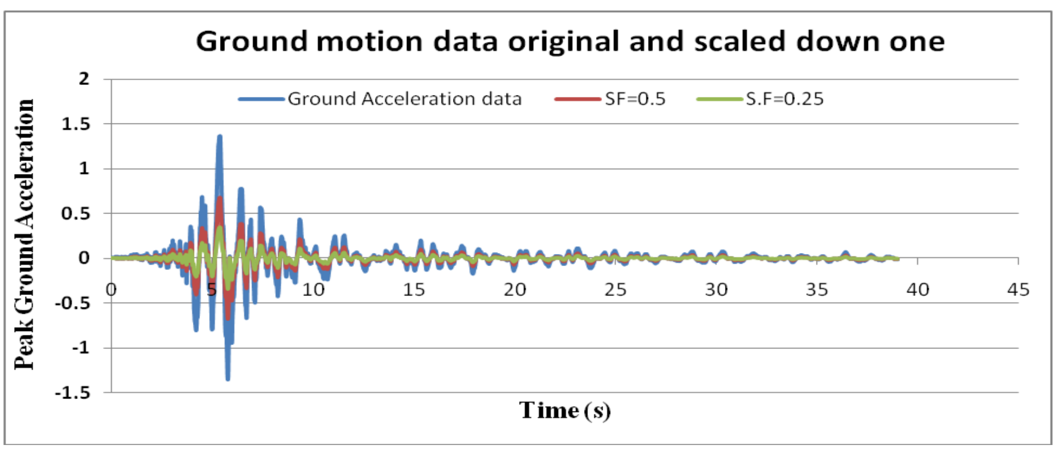

Figure: Ground motion data: original one and scaled down

The IDA is aimed to meet the following objectives:
1. Variation of response or demand parameters with increasing strength or intensity of ground motion.
2. Assessment of structural behaviour, if subjected to rare / more severe ground motion.
3. Better understanding of the structural response when the intensity of ground motion increases (e.g., peak deformation patterns across the height, onset of stiffness and strength degradation with patterns and magnitudes).
4. Estimating dynamic capacity of the global structural system.
5. Given a multi-record IDA, how stable (or variable) the structure is from one ground motion record to another.

Three sets of ground motion, as presented in Section 3.5, are considered for IDA. First Peak Ground Acceleration (PGA) of recorded EW and NS directions are noted and whichever is greater component. The process is applied to a series of chosen intensity levels. Time history analysis, is chosen for scaling in IDA. Suitable multiplication factor (amplitude scaling) is computed for a chosen intensity level and same multiplication factor is applied to the other orthogonal

Bidirectional analysis of actual building and shear frame

For checking the accuracy of the proposed method, a seven storey building is designed in the Chapter-4, for that building the syorey stiffness is estimated in X and Y direction from the expression proposed in section 3-4, and a seven storey shear building is designed with the calculated storey stiffness.

Time history analysis is performed for all three events in both E-W and N-S direction. The E-W component is given always in X-direction and N-S component is given in N-S direction, maximum displacement at each floor level is compared between actual frame and shear frame, and the results are shown in Fig-5.2, 5.3 and 5.4.

Floor displacement Comparison:

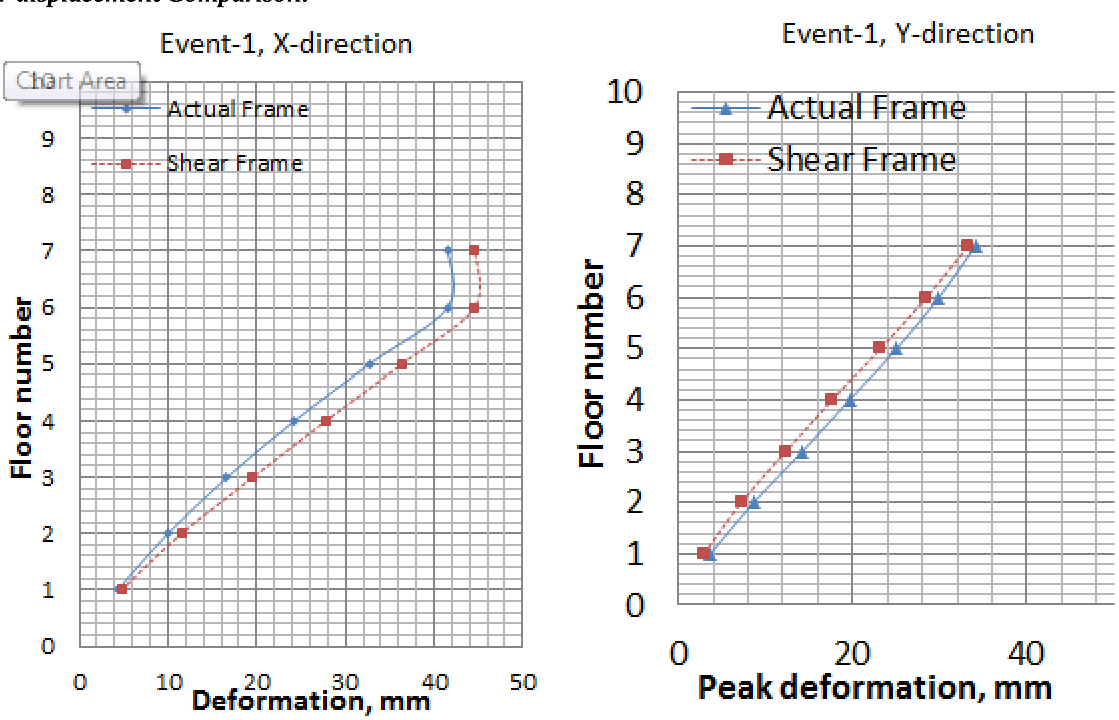

Fig Displacement comparison for the Event-1

The lateral floor displacement throughout the time history at the Centre of Mass (CM) of respective floor is noted. Maximum displacement is identified as the absolute peak displacement at that floor level. Note that the peak floor displacement at different floor levels may not occur at the same time instant. Variation of peak floor displacement across the height is defined as the peak floor displacement profile. The peak floor displacement profile is compared for the building with and without damper.

During the Event1 roof displacement in *X* direction for the actual frame is 42and for the shear frame it is 45 mm, at 6th floor in *X* direction for the actual frame is 41 mm and for the shear frame it is 43 mm, at 5th floor in *X* direction for the actual frame is 36 and for the shear frame it is 32 mm, at 4th floor in *X* direction for the actual frame is 28 mm and for the shear frame it is 26 mm, at 3rd floor in *X* direction for the actual frame is 20 and for the shear frame it is 18 mm, at 2 nd floor in *X* direction for the actual frame is 11 and for the shear frame it is 10 mm,
During the Event1 roof displacement in *Y*direction for the actual frame is 35and for the shear frame it is 34.32 mm, at 6th floor in *Y*direction for the actual frame is 29 mm and for the shear frame it is 28.3 mm, at 5th floor in *Y*direction for the actual frame is 24 and for the shear frame it is 23 mm, at 4th floor in *Y*direction for the actual frame is 28 mm and for the shear frame it is 26.7 mm, at 3rd floor in *Y*direction for the actual frame is 20 and for the shear frame it is 18 mm, at 2 nd floor in *Y*direction for the actual frame is 14and for the shear frame it is 13.7 mm

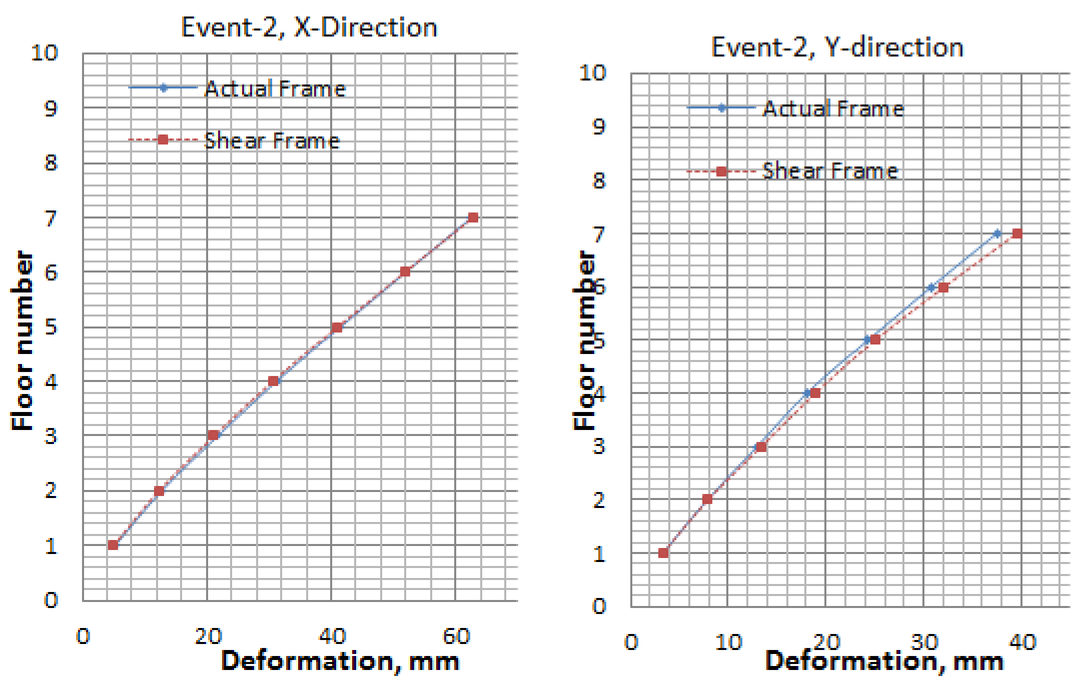

Fig- Displacement comparison for the Event-2

During the Event2 roof displacement in *X* direction for the actual frame is 63and for the shear frame it is 62.31 mm, at 6th floor in *X* direction for the actual frame is 51 mm and for the shear frame it is 51.3 mm, at 5th floor in *X* direction for the actual frame is 40.3and for the shear frame it is 40.25 mm, at 4th floor in *X* direction for the actual frame is 28 mm and for the shear frame it is 26 mm, at 3rd floor in *X* direction for the actual frame is 20 and for the shear frame it is 18 mm, at 2nd floor in *X* direction for the actual frame is 11 and for the shear frame it is 10 mm,
During the Event1 roof displacement in *Y*direction for the actual frame is 40and for the shear frame it is 39 mm, at 6th floor in *Y*direction for the actual frame is 32 mm and for the shear frame it is 31 mm, at 5th floor in *Y*direction for the actual frame is 28and for the shear frame it is 28 mm, at 4th floor in *Y*direction for the actual frame is 19 mm and for the shear frame it is 18.4 mm, at 3rd floor in *Y*direction for the actual frame is 8.2 and for the shear frame it is 8 mm, at 2 nd floor in *Y*direction for the actual frame is 1.8and for the shear frame it is 2 mm

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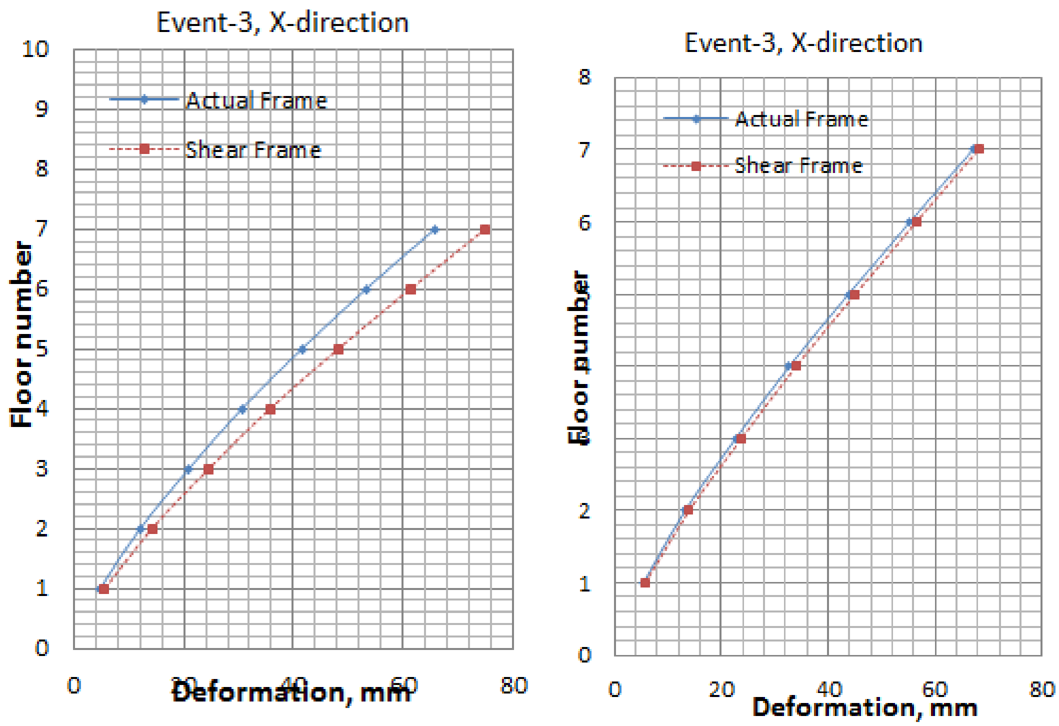

Fig- Displacement comparison for the Event-3

During the Event3 roof displacement in *X* direction for the actual frame is 63and for the shear frame it is 69 mm, at 6th floor in *X* direction for the actual frame is 52 mm and for the shear frame it is 61 mm, at 5th floor in *X* direction for the actual frame is 42.3 and for the shear frame it is 47 mm, at 4th floor in *X* direction for the actual frame is 31 mm and for the shear frame it is 34 mm, at 3rd floor in *X* direction for the actual frame is 20 and for the shear frame it is 22 mm, at 2 nd floor in *X* direction for the actual frame is 15 and for the shear frame it is 17 mm,

During the Event1 roof displacement in *Y*direction for the actual frame is 68and for the shear frame it is 67.4 mm, at 6th floor in *Y*direction for the actual frame is 59.2 mm and for the shear frame it is 58 mm, at 5th floor in *Y*direction for the actual frame is 42and for the shear frame it is 4103 mm, at 4th floor in *Y*direction for the actual frame is 31 mm and for the shear frame it is 30.8 mm, at 3rd floor in *Y*direction for the actual frame is 24.2and for the shear frame it is 23.1 mm, at 2 nd floor in *Y*direction for the actual frame is 18.4and for the shear frame it is 17.2 mm.

Overall comments: From the Figs. it is observed that the deformation profile of the actual frame and the shear frame are giving close results. However there is some degree inconsistency is there in Event-3 X-direction.However, the design criterion of building is done based on response spectrum analysis, and the displacement results for response spectrum analysis (Section 3-8) are matching closely.

IV. CONCLUSION

Based on the numerical results, it can be concluded that the proposed method for estimating the lateralstiffness of building systems is more efficient than existing methods. Thus it can be used effectively forapproximate analysis of building systems subjected to lateral loads in different cases, including lateral displacement calculations, frequency estimation, and especially for final checking of designs. Although the application of the proposed method to the preliminary design of structural elements has not been included in the paper, it should be noted that to use the proposed method for this purpose there is no need to guess the absolute values of member properties, only their relative values are needed. The following observations and conclusions were made during the course of this study.

a. The apparent lateral stiffness of a story is not a stationary property, it can be accurately modeled by a single value for frames that resist lateral loads with regular distributions.
b. The approximate expression derived from Benjamin's work (1959), andthat proposed by Blume et al. (1961) were found to be very inaccurate with columns stiffer than girders (a < 1), especially if there are largedifferences in the heights of adjacent stories.
c. Muto's expression (1971) for individual columns performs well for intermediatestories of frames with equal height stories.
d. The correction factors *%*, *Cs*, and £, enable the proposed expression (11)to provide reasonably good estimates of story lateral stiffness, even for frameswith columns that are as much as ten times stiffer than girders, and even whenstory heights and member stiffnesses*(Ic*and *Ig)* differ by as much as 50% fromone story to the next. For the frames considered, story stiffness estimates wereusually *5%* of the exact solution (always within 7%), and top displacementestimates did not exceed 3% error in most cases (never exceeded 5% error)

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