Analysis of Tall Buildings Using A "Moment-Force Transformation Method" As a Simplified Method of Analysis

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***ABSTRACT*–**This paper presents a simplified method of analysis, the moment-force transformation method (MFT), which is a modification of the moment transformation method (MT). In the MFT, the axial deformation in the vertical member is taken into account and the axial forces are transformed from one level to another level together with the transformed moments. With the consideration of the axial deformations in the vertical members, analysis of super-tall buildings, such as tube and outrigger buildings systems, becomes possible and accurate. The final output of the transformation procedure includes rotations and moments, from which shear forces in the vertical members and the lateral displacements are calculated. The axial displacements and axial forces in the vertical members are directly obtained. A number of problems were analyzed using the proposed method and the results obtained, were compared with known results and with those obtained using StaadPro2004 and ETABS. The comparison shows that, the results are in good agreement, thus verifying the accuracy of the developed method.

Keywords*: Simplified Analysis Method, Tall building, Lateral Displacement, Axial Deformation, Vertical Members.*

المستخلص – تعرض هذه الورقة طريقة مبسطة لتحليل المباني العالية والمعرضة لأحمال رأسية و أفقية في آن واحد. الطريقة هي تطوير لطريقة نقل العزوم، مع الأخذ في الإعتبار الإزاحات المحورية في الأعضاء الرأسية، حيث يتم نقل القوى الرأسية مع العزوم المنقولة من مستوى إلى مستوى آخر. بإدخال الإزاحات الرأسية في الحسابات أصبح من الممكن إستخدام الطريقة في التحليل الدقيق للمباني العالية جداً ذوات النظم الأنبوبية و النظم ذوات أذرعة الإمتداد. في هذه الطريقة تحسب الدورانات و العزوم في الأعضاء في كل المستويات أولاً، ومن ثم تحسب القوى القاصة في الأعضاء الرأسية والإزاحات الأفقية لكل مستوى، كما تحسب الإزاحات الرأسية و القوى المحورية في الأعضاء الرأسية كناتج نهائي لعملية النقل. أستخدمت الطريقة المقترحة لدراسة مسائل مختلفة، وقد تمت مقارنة النتائج المتحصل عليها بنتائج معلومة أو بنتائج تم الحصول عليها باستخدام برنامجي StaadPro2004 و ETABS . وقد أثبتت المقارنة أن هنالك توافقاً جيداً بين النتائج ، مما يؤكد دقة الطريقة المطورة.

INTRODUCTION

Simplified methods of static and dynamic analysis for the effects of vertical and horizontal loads on tall building are required, especially in the preliminary design stage when the proposed structural system has to be analyzed several times before the final agreement. Also in the analysis of large structural systems, such as the tall buildings which include huge numbers of unknowns, a lot of difficulties arise such as:

- The capability and capacity of the hardware of the computing machine.
- The machine running time, which is proportional to the number of unknowns?
- The interpretation of the vast amount of the analysis results.
- The need for new rearrangements of members or changing the structural system.

The huge number of conducted researches, as a solution of these problems, can be classified according to different types of problems formulation and solution methods, as:

- Simple manual arithmetical methods, e.g. Portal and Cantilever methods.

- Differential equations and Continuum methods of analysis.
- Simplified finite element and matrix methods of analysis.
- Methods of Simplifying the models and Reduction Techniques.

Each one of the mentioned methods is used with limitations and sometimes tailored for a certain type of structural system. For example the cantilever and portal methods are sought in practice for the specific reason that they do not require cross sectional areas for the analysis. According to Selvam [1], there are two versions of the portal method. One is the simplified portal method and the other is the improved portal method. Both of these two methods assume that the hinges are located in the middle of all the columns and the beams. In the simplified portal method, the storey shear is distributed among the columns considering that each of the outer columns resists half the shear resisted by any of the internal columns.

In the improved portal method, the storey shear is distributed among the columns in proportion to the tributary length of the spans between the columns. Selvam [1], proposed an alternative analysis method that splits vertically the whole frames into separated simple frames, each one containing only one bay subjected to lateral loads which is calculated from the dimensions of all the bays. The results of the proposed method are in harmony with the solution of the improved portal method. These methods are used only for analysis of relatively short un-braced portal frames subjected to lateral wind loads or equivalent static seismic loads. Also, they can't be used to calculate the lateral displacements and the dynamic properties of the frames.

In a continuum model, the horizontal slab and beams connecting the vertical elements are assumed to be smeared as a continuous connecting medium having equivalent distributed stiffness properties. Two common types of structure that can be solved using continuum techniques are a coupled wall and a wall-frame structure. Jaeger and Mamet [3], proposed an analytical theory for the analysis of tall three dimensional multiple shear wall buildings.

The basis of their theory is the continuum approach in which the floors of the building are replaced by an equivalent continuous medium. Their results were compared and verified with results obtained by using the finite element method and experiments. A simplified approach for seismic calculation of a tall building braced by shear walls and thin-walled open section structures was also presented by Meftah and others [2]. based on the continuum technique and D'Alembert's principle. The governing equations of free vibration and the corresponding eigenvalue problem were derived. By applying the Galerkin technique, a generalized method was proposed for the free vibration analysis of a building braced by shear walls.

Simplified formulae were given to calculate the circular frequencies and internal forces of a building structure subjected to earthquakes. Also some researches used different mathematical methods for the analytical solutions such as the work presented by Bozdogan [3]. In his work, free vibration analysis of wall-frame structures were studied. A wall-frame structure was modeled as a cantilever beam and the governing differential equations were solved using the Differential Quadrature Method (DQM).

A Simplified Analytical Method for High-Rise Buildings was presented by Hideo Takabatake [4]. The method is a simple analytical theory for doubly symmetric frame-tube structures applying ordinary finite difference method to the governing equations proposed by the one-dimensional extended rod theory. Instead of calculating the lateral displacements and the stresses in the members directly, another alternative of a simplified analysis of shear-lag in framed-tube structures with multiple internal tubes is presented by Lee et al. [5].

In their work a simple numerical modeling technique is proposed for estimating the shear-lag behavior of framed-tube systems with multiple internal tubes. An approach to Static analysis of tall buildings with a combined tube-in-tube and outrigger-belt truss system subjected to lateral loading is presented by Jahanshahi, et al. [6]. They present an efficient technique for static analysis of the building system while considering shear lag effects. Based on the principle of minimum total potential energy, simple closed form solutions are derived for stress and displacement distributions. Alternative method of solution of the continuum problems is the transfer matrix method used by

Bozdogan and Ozturk [7], [8] in the lateral stability and the vibration analysis of the tall buildings.

There is a lot of development in the proposed simplified methods of solution for the two and three dimensional frames based on the matrix and finite element methods of analysis, as those presented by Clough, King, and Wilson as cited by Ghali and Neville et al. [9] and the Substitute-Frame method derived by Lightfoot [9], which uses a simplified matrix method to analyze two dimensional frames. An iteration method for calculating the side-sway of the frame-shear wall system has been suggested by Khan and Sbarounis as cited by Ghali and Neville et al. [9] ,who give also charts to assist in practical design. A three dimensional analysis of shear wall tall structures was proposed by Ghali and Neville et al. [9].

In their method, they distributed the applied lateral loads among the irregularly arranged shear walls in the building plan, but they neglected the important effects of the out of plane stiffness of the floor slabs and beams. A two-level finite element technique of constructing a frame super-element was created by Leung and Cheung, to reduce the computational effort for solving large scale frame problems.

In the method the nodal displacements of all the nodes are related to those of a small number of selected master joints in the frame by means of global finite element interpolating functions and the frame may be considered as a super-element to be connected to other elements by means of these master nodes. Leung also proposed a method for analysis of two dimensional frames based on distribution factors which are allowed to vary from floor to floor and are determined by using three floors at a time. By means of these distribution factors, the number of degrees of freedom is reduced to three at any one floor.

This two dimensional method was generalized by Leung [10], to a three dimensional analysis method. In order to improve the results another three additional sets of global distribution factors were introduced by Leung and S. C. Wong, to account for the uneven shortening (elongation) of the columns having unevenly distributed stiffness along the height and across the floor plane. Based on Leung's works, C. W. Wong and Lau[11] presented another similar simplified finite element method for analysis of tall buildings.

Also Swaddi et al. used the finite strip method to simplify the analysis of frame-shear wall buildings subjected to lateral loads. A method for lateral static and dynamic analyses of wall-frame buildings using one dimensional finite element was presented by Bozdogan [12]. The study presents an approximate method which is based on the continuum approach and one dimensional finite element to be used for lateral static and dynamic analyses of wall-frame buildings.

There is a lot of development in the simplification of the modeling and the reduction techniques such as the work conducted by TolgaAkis [13] , the main purpose of which was to model and analyze the non-planar shear wall assemblies of shear wall-frame structures. The use of the equivalent cubes to simplify the finite element modeling of multi-storey buildings is presented by Duffield and Hutchinson [14] The used cubes have equivalent stiffness properties, which result in significant reduction of the mesh density of the whole building and accordingly the computational time and the consumed memory.

There are some researchers conducted to study and improve the structural systems of the tall buildings, such as the study conducted by Bayati et al. [15] to optimize the use of multi-outrigger system to stiffen the tall building. Another study was conducted by Moghadam and Aziminejad [16], to study the interaction of torsion and the p-delta effects in tall buildings. An Optimum structural modeling for tall buildings is also studied by Jameel et al. [21]. They studied the global structural strength of the tall buildings provided by the different members such as the columns, shear walls, slabs and beams.

By application of computer program namely COSMOS/M, Marsono and Wee [17], also conducted a nonlinear finite element analysis to study the structural behavior and mode of failure of reinforced concrete tube in tube tall buildings. The moment transformation method is a simple approach presented by Ibrahim and Mohamed [18], [19], used for the analysis of tall buildings subjected to both vertical and horizontal loads, with the axial deformation in the vertical members neglected. The method is development and generalizations of the concepts of no-shear and direct moment distribution to distribute or transform a coupled bundle of moments from one level to another level.

In this work the objective is to develop the moment transformation method to incorporate the axial deformation of the vertical members in the analysis, and the axial forces are bundled and transformed, together with the moments, between the different levels of the building.

MATERIAL AND METHOD:

Transformation of Moments:

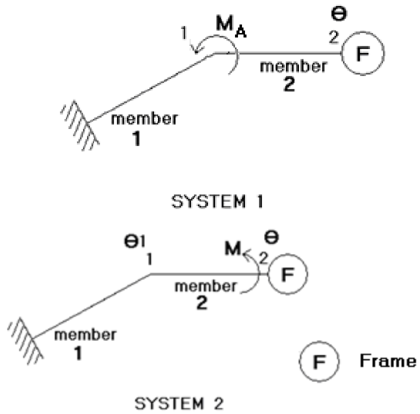


Figure 1: Frame of two members

Assuming the two systems, in Figure 1, M_A: a moment applied at joint 1, M: equivalent moment at joint 2, θ_1 and θ: rotation angles in radians. Applying the slope deflection method for system 2, and the reciprocal theorem for the two systems 1 and 2, yields the following equations:

$$S_e = \left[S_2 - \frac{t_2^{\,2}}{(S_1 + S_2)} \right] \quad (1)$$

$$M = M_A \cdot \left[\frac{-t_2}{(S_1 + S_2)} \right]$$

$$TF = \frac{-t_2}{(S_1 + S_2)} \quad (2)$$

where:

$$S_i = \frac{(4+\alpha)}{(1+\alpha)} (\frac{EI}{L})_i \; t_i = \frac{(2-\alpha)}{(1+\alpha)} (\frac{EI}{L})_i ,$$

$$\alpha = \left(\frac{12EI}{Ga_r L^2} \right)_i$$

Se: is the equivalent stiffness of the members 1 and 2 (the stiffness of the member which can replace the two members 1 and 2). TF: is the transformation factor used to theoretically transform the moment MA from joint 1 towards joint 2.

Transformation of Forces:

Alternative approach has been carried out to incorporate the axial deformation in the vertical members of a building system. Consider the two connected members in Figure 2:

In system 2, the same displacement of the connecting members 1 and 2 at joint 1, yields:

$$R = \frac{-K_2}{(K_1 + K_2)} \quad (3)$$

Since the axial displacement at joint 2 in system 2 is prevented, then the displacements at joint 2 are equal in systems 1 and 3 and since the applied load is unity, then the force (-R) can be looked at as the transformation factor used to transforms the vertical force from joint 1 towards joint 2.

$$TF = \frac{K_2}{(K_1 + K_2)} \quad (4)$$

From the local axial stiffness matrix of a single member, and since the carryover force is the reaction force of the restrained far end, then, t = -K, and:

$$TF = \frac{-t_2}{(K_1 + K_2)} \quad (5)$$

which is similar to equation 2 of the rotation stiffness.

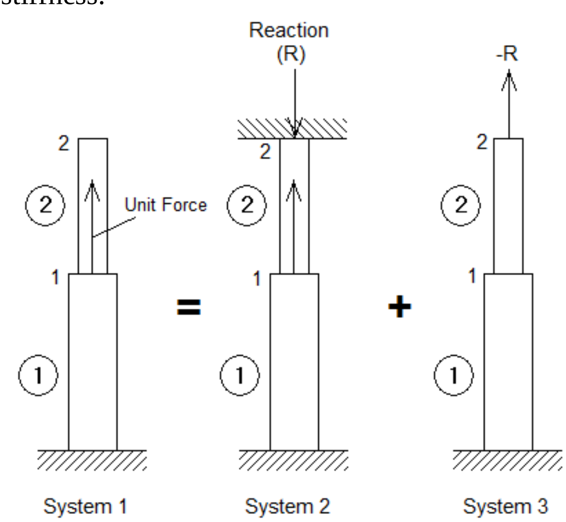


Figure 2: Two axial members system

The stiffness of the equivalent member which can replace the two members 1 and 2, can be calculated using the following equation:

$$\frac{1}{K_e} = \frac{1}{K_1} + \frac{1}{K_2}$$

or,

$$K_e = \frac{K_1.K_2}{(K_1 + K_2)} \quad (6)$$

Substitution for $K_e = S_e$, $S_1 = K_2$, $S_2 = K_2$, and $t_2 = -K_2$ in equation (1), yields the same as in equation 6.

Multi-Bay Multi-Storey Buildings

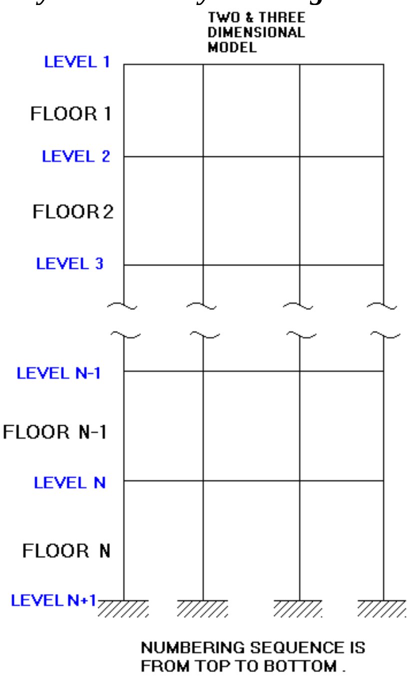


Figure 3: Multi-Storey Two or Three Dimensional Building

By combining the moments and the forces transformation procedures, the moment-force transformation procedure can be generalized to calculate the equivalent stiffness matrix and the transformation factors matrix of the building, as follows: The transformation procedure, gives:

$$[SR] = [NN]_i + [GG]_{i-1} \quad (7)$$

$$[AA] = [A]_i + [SR] \quad (8)$$

$$[FF]_i = -[B]_i^T.[AA]^{-1} \quad (9)$$

$$[GG]_i = [A]_i + [FF]_i [B]_i \quad (10)$$

Where $[NN]^i$: The overall rotation-translation stiffness matrix of level i. $[GG]_{i-1}$: The equivalent rotation-translation stiffness matrix of floor i-1. [SR]: The summation of $[NN]_i$ and $[GG]_{i-1}$. $[A]_i$: The condensed rotation-translation stiffness matrix of floor i. [AA]: The summation of $[A]_i$ and [SR]. $[FF]_i$: The transformation factors matrix of floor i. $[B]_i$: The carry over moment-force matrix of floor i. $[GG]_i$: The equivalent rotation-translation stiffness matrix of floor i.

Condensed Stiffness and Carryover Matrix for Multiple Vertical Members

Considering a system of two vertical members, Figure 4, the stiffness matrix equation corresponding to the three degrees of freedom 1, 2 and 3, condensed into 1 and 2, is as follows:

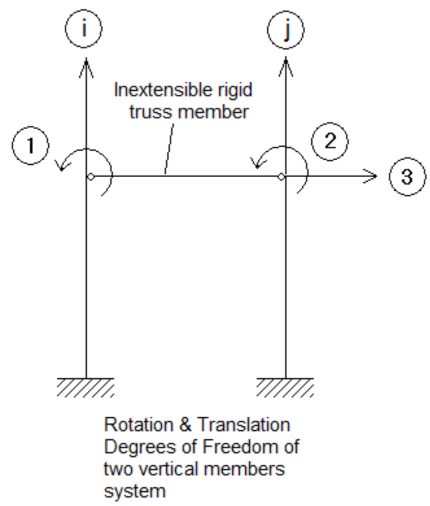


Figure 4: Rotations and Translations DOF s of a Two Vertical Members System

$$\begin{pmatrix} S_{11} & S_{12} & S_{13} \\ S_{21} & S_{22} & S_{23} \\ S_{31} & S_{32} & S_{33} \end{pmatrix} \begin{Bmatrix} 1 & 0 \\ 0 & 1 \\ D_1 & D_2 \end{Bmatrix} = \begin{Bmatrix} S_{11}^* & S_{12}^* \\ S_{21}^* & S_{22}^* \\ 0 & 0 \end{Bmatrix} \quad (11)$$

The internal interaction force, $(F_i)D_j$ is obtained from the different rotational stiffness configurations and hence the elements of the carryover moments matrix are calculated from the following equation:

$$t_{ij}^* = -S_{ij}^* + (F_i)_{Dj}.L \quad (12)$$

where L is the floor height. Since the axial stiffness of the vertical members are uncoupled with each other and also uncoupled with the rotational and the lateral translation stiffness of the members, then the axial stiffness of each member can be added to the condensed rotational stiffness of the members after the condensation procedure.

The rotational and the axial stiffness of the vertical members are coupled with the rotational stiffness and the lateral stiffness of the horizontal floor members during the transformation procedure. Similarly, the axial force carryover elements also should be added to the carryover matrix.

NUMERICAL EXAMPLES

Using the computerized proposed simplified method (MFTProg), two simple portal frames examples were studied. A case study of a fifteen floors symmetric building with non-symmetrical lateral loadings was also carried out. The results were compared with those obtained by MTProg (with axial deformations in the vertical members neglected), and with those obtained by using StaadPro2004 [22] and ETABS [23].

Portal Frames:

Two Floor One-bay Portal Frame:

The bending moments are obtained using the simplified method for a two storey frame under the vertical and horizontal loading shown in the Figure 5.

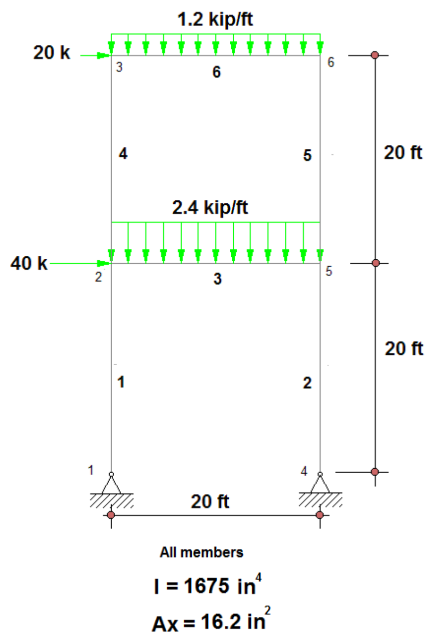


Figure 5: One-bay Frame properties and loading

The bending moments results obtained are compared with the results from reference [20] and are shown in Table 1.

Table 1: Comparison of bending moments (2 Floor 1 bay Frame)

Results	M_3 (kip-ft)	M_6 (kip-ft)	H_4 kip
Reference[20]	141.90	206.30	31.16
StaadPro2004	145.79	209.03	31.18
MFTProg1	145.98	208.84	31.21
MFTProg2	142.06	206.06	31.20

1 Considering shear deformation. 2 Neglecting shear deformation.

The comparison of the results shows very close agreement.

Two Floor Two-bay Portal Frame:

The bending moments are obtained using the simplified method for a two storey frame under the vertical and horizontal loading shown in the Figure 6.

The bending moments results obtained are compared with the results from reference [20] and StaadPro2004 are shown in Table 2.

Table 2: Comparison of bending moments (2 Floor 2 bay Frame)

Results	M_1 (kip-in)	M_3 (kip-in)	M_6 (kip-in)
Reference[20]	1147.00	1462.00	1750.00
StaadPro2004	1150.30	1461.85	1748.63
MFTProg1	1145.71	1463.50	1750.87
MFTProg2	1142.05	1463.70	1752.24

1 Considering shear deformation. 2 Neglecting shear deformation.

Comparison of the published results and the results obtained using StaadPro2004 with those obtained by MFTProg, show very close agreement.

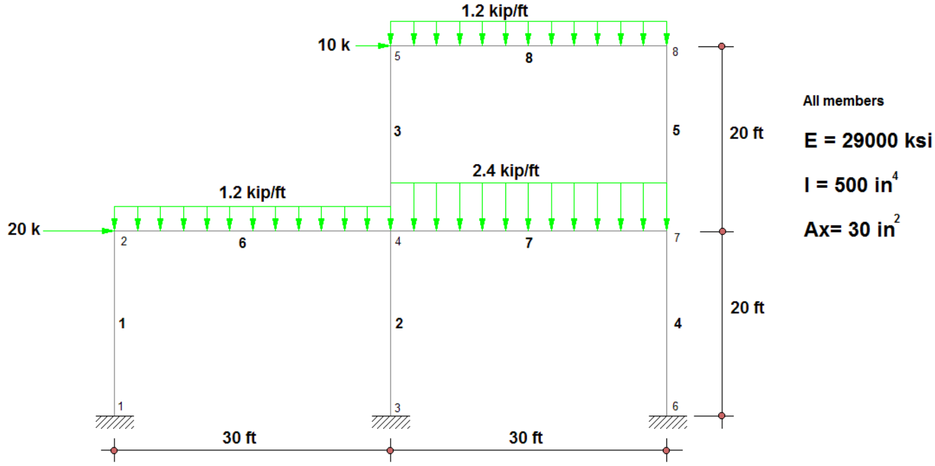


Figure 6: Two-bay Frame properties and loading

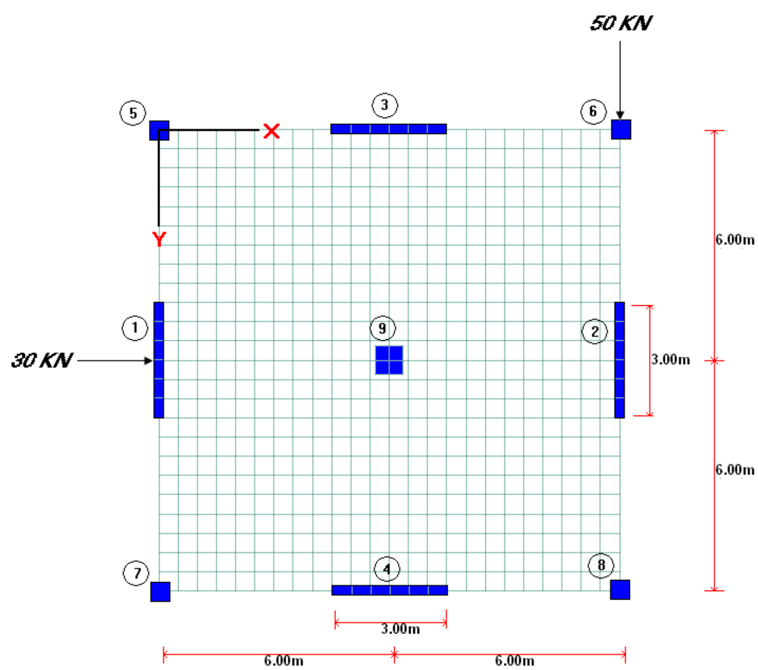


Figure 7: 12m x 12m floor plan for 15 Storey, Square Building

Model of fifteen storey building subjected to unsymmetrical lateral loading:

The plan shown in Figure 7 is for a 12m x 12m floor slab of thickness = 0.25 m. The building is composed of 15 floors of floor height = 3.5 m for all floors except the lower floor which is of height = 5.5 m.

All building members are concrete of elasticity, E = 21718500 kN/m2, and Poisson's ratio, v = 0.17

The section properties of the vertical elements (in meters) are:

Columns: Corners: 0.60 x 0.60 and Interior: 0.85 x 0.85. Shear walls: The lower 7 floors: 0.30 x 3.00 and The upper 8 floors: 0.25 x 3.00. The building is subjected to the lateral loads shown in Figure 7, (30 kN and 50 kN) at all floor levels.

The building has been analyzed by using MFTProg with the axial deformation in the vertical members considered. The accuracy of the results is verified by using MTProg and the structural analysis packages StaadPro2004 [22] and ETABS [23]. Comparison of the displacements of the origin (Column 5), obtained using the proposed method and the different packages is shown in Figures 8, 9 and 10. Comparison of the shear force and bending moment of shear wall 2 is shown in Figures 11 and 12.

In all the comparisons, the difference is found to be very small for large stress values (shear forces and bending moments in shear walls 1, 2 & 3). The largest percentage difference is found in shear wall number 4, but this resists very small stresses compared with its section.

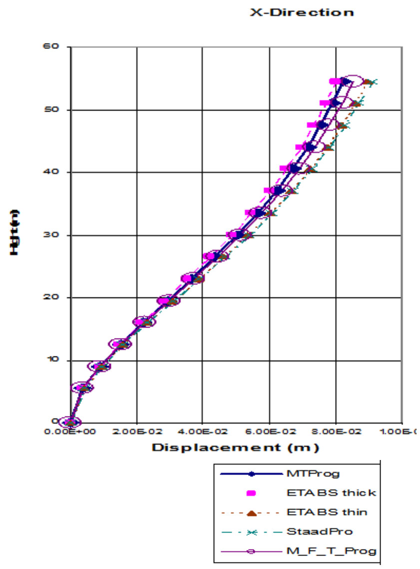
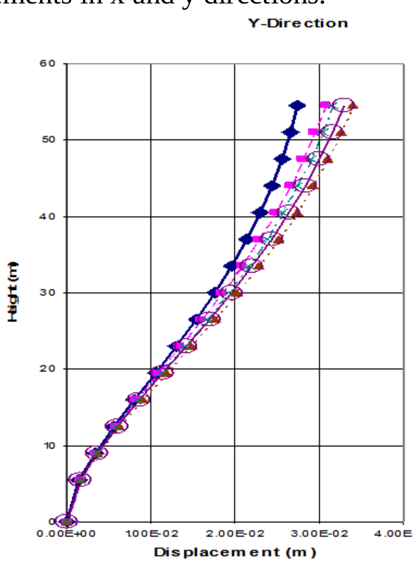


Figure 8: Displacements of the origin in x-direction

As shown in Figures 8 - 12 and Table 3, consideration of the axial deformations in the vertical members has affected the bending moments in all shear walls, and also the displacements in x and y directions.



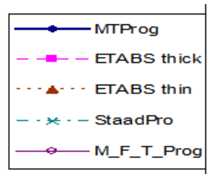


Figure 9: Displacements of the origin in y-direction

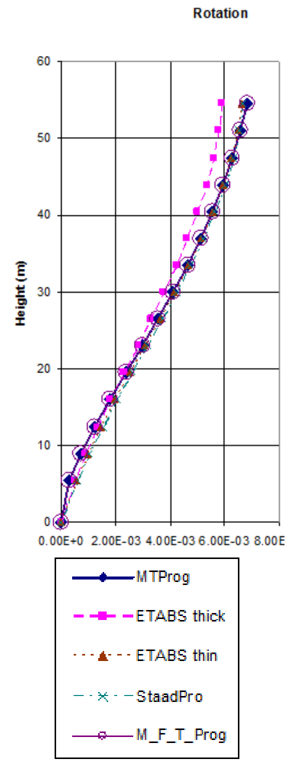


Figure 10: Rotations in radians of the origin

A 160 Floors building:

The same previous floor has been used again in a 160 floors building:

The section properties of the vertical elements (in meters) are:

All Columns: Corners: 0.60 x 0.60 and Interior: 0.85 x 0.85

Shear walls: The lower 80 floors: 0.30 x 3.00 and The upper 80 floors: 0.25 x 3.00

The building is subjected to the lateral loads shown in Figure 7, (30 kN and 50 kN) at all floor levels. The lower floor height= 5.5 m, all the other heights=3.5m

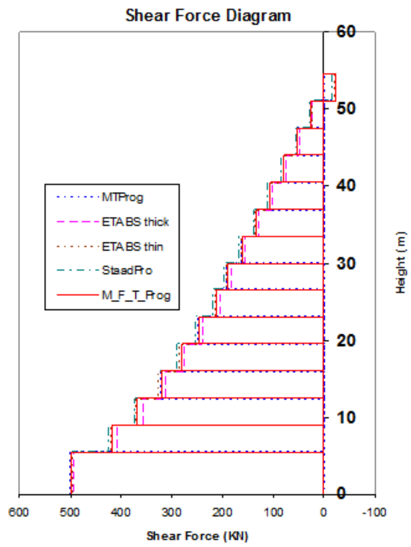


Figure 11: Comparison of S.F.D. for shear wall #2

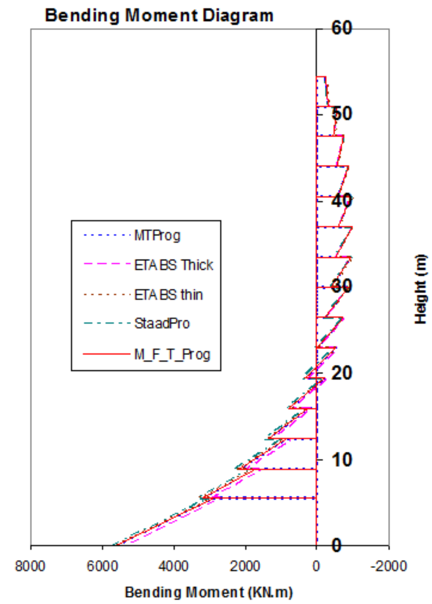


Figure 12: Comparison of B.M.D. for shear wall #2

Table 3: Comparison of the maximum shear force (kN) and bending moment (kN.m)

Wall #	Wall 1				Wall 2			
Package	Shear	% Diff.	Moment	% Diff.	Shear	% Diff.	Moment	% Diff.
MFTProg	155.4	-1.46	1656.1	-2.46	500.3	0.62	5618.9	-1.89
MTProg	155.4	-1.46	1583.7	-6.73	500.3	0.62	5546.6	-3.15
StaadPro2004	157.8	0.06	1651.4	-2.74	497.1	-0.02	5741.2	0.24
ETABS1	155.7	-1.27	1604.8	-5.48	494.6	-0.52	5424.3	-5.29
ETABS2	157.7	0	1697.9	0	497.2	0	5727.2	0

Wall #	Wall 3				Wall 4			
Package	Shear	% Diff.	Moment	% Diff.	Shear	% Diff.	Moment	% Diff.
MFTProg	369.2	0.93	4163.9	-1.81	24.2	-11.03	201.1	-6.29
MTProg	369.1	0.9	4120.5	-2.83	24.2	-11.03	157.7	-26.51
StaadPro2004	365.6	-0.05	4261	0.48	27.3	0.37	174.5	-18.69
ETABS1	364.1	-0.46	4016.9	-5.27	26.1	-4.04	200.7	-6.48
ETABS2	365.8	0	4240.5	0	27.2	0	214.6	0

1 Thick Slab. 2 Thin Slab. All the material same are as before.

The problem was solved by using MFTProg and other commercial structural package. The elapsed running time by MFTProg was 17 seconds and by the commercial package was 41385 seconds. Comparison of the maximum shear force and bending moment are shown in Table 4 and displacements of the origin in Table 5.

Table 4: Comparison of the maximum shear (kN), and bending moment (kN.m)

Wall #	Wall 1		Wall 2	
Package	Shear	Moment	Shear	Moment
MFTProg	1644.9	31623.0	5290.1	82061.7
Commercial	1690.7	31643.0	5240.3	84021.6
% Diff	-2.71	-0.06	0.95	-2.33

Wall #	Wall 3		Wall 4	
Package	Shear	Moment	Shear	Moment
MFTProg	3903.1	59324.8	257.9	8886.1
Commercial	3853.6	60887.0	305.0	8511.7
% Diff	1.28	-2.57	-15.42	4.40

Table 5: Displacements of the origin in mm, (column 5)

Package	x	y	Twist (rad)	Axial
MFTProg	54394.4	74197.5	1.03	1620.0
Commercial	55076.6	73759.3	1.13	1608.5
% Diff	-1.24	0.59	-8.64	0.71

4.5 Comparison of the Number of Unknowns:

In order to show the power of the proposed method, the floor slab idealized by 24 x 24 finite elements with 9 vertical members (shear walls and columns) shown in Figure 7, is studied and compared with the conventional matrix methods.

The total number of unknowns for a building with such a floor and of total N floors is equal to:

a) Conventional matrix methods:

$S_1 = [(25 \times 25 \times N + 9) \times 6]$, For 6 DOF per joint.

b) The Moment-Force Transformation Method:

The unknowns in the proposed method are composed of two parts:

- Coupled unknowns for one floor with 3 DOF per joint solved simultaneously and used to obtain the level stiffness.
- Two Rotations and one axial translation for each wall at all levels and for supports if there are springs. The unknowns solved separately, each (9x3) coupled unknowns for each level.

$S_2 = [25 \times 25 \times 3] + [9 \times 3] \times (N + 1)$

Note: coupled unknowns are in square brackets [ ].

For N, say equal 160 floors:

S1= 600054 Coupled unknowns and, S2 = 6222 unknowns, Ratio= 96 times.

CONCLUSION

The moment-force transformation method yields identical results compared to the exact analyses methods. The saving in computer storage and computing time provided by the developed program MFTProg, allows rapid re-analysis of the building to be accomplished in the preliminary analysis and design stages. The method is suitable for the analysis of super-tall buildings with tubes and outrigger systems. The ease in data preparation and in interpretation of final results, compared with finite element packages, is one of the main advantages of the proposed method. The simplicity in programming the method, when compared with the difficulty in obtaining reliable packages is an added advantage.

REFERENCES

[1] Manicka Selvam V.K. and Bindhu ,K.R, (2011), “Split frame method for lateral load analysis of short frames”, International Journal of Civil and Structural Engineering, Volume 1, No 4, 2011, Research article ISSN 0976 – 4399 835.

[2] Sid Ahmed Meftah, Abdelouahed Tounsi and AddaBedia El Abbas,(2007), “A Simplified Approach for Seismic Calculation of a Tall Building Braced by Shear Walls and Thin-Walled Open Section Structures”, Science-direct.

[3] Kanat Burak Bozdogan, (2011), “Free Vibration Analysis of Wall-Frame Structures by Differential Quadrature Method”, International Journal of Engineering and Applied Sciences (IJEAS), Vol.3, Issue 4, 2011, 7-20.

[4] Hideo Takabatake, (2012), “A Simplified Analytical Method for High-Rise Buildings“, Department of Architecture, Kanazawa Institute of Technology, Institute of Disaster and Environmental Science, Japan, hideo@neptune.kanazawa-it.ac.jp,.

[5] Kang-Kun Lee, Hong Guan and Yew-Chaye Loo, (2000), “Simplified Analysis of Shear-Lag in Framed-Tube Structures with Multiple Internal Tubes”. Griffith University Gold Coast Campus, Australia.

[6] M. R. Jahanshahi, R. Rahgozar, and M. Malekinejad, (2012), “A Simple Approach to Static Analysis of Tall Buildings with a Combined Tube-in-tube and Outrigger-belt Truss System Subjected to Lateral Loading”, IJE Transactions A: Basics Vol. 25, No. 3, July 2012, 289-299, International Journal of Engineering.

[7] Kanat Burak Bozdogan and Duygu Ozturk, (2010), “An approximate method for lateral stability analysis of Wall-frame buildings including shear deformations of walls”, Sadhana Vol. 35, Part 3, June 2010, pp. 241-253. Indian Academy of Sciences, Department of Civil Engineering, Ege University, Izmir, 35040 Turkey, e-mail: kanat.burak.bozdogan@ege.edu.tr

[8] K.B.Bozdogan and D.Ozturk, (2010), ”Vibration Analysis of Asymmetric shear wall structures using the transfer matrix method”, Iranian Journal of Science and Technology, Transaction B, Engineering, Vol. 34, No B1,PP 1-14

[9] A. Ghali, Neville, A. M. and T.G. Brown (2009), “Structural Analysis – A Unified Classical and Matrix Approach”, 6th Edition, Spon Press.

[10] A. Y. T. Leung, (1985), “Micro-Computer Analysis of Three-Dimensional Tall Buildings”, Computers & Structures Vol. 21, No. 4, pp. 639-661.

[11] C. W. Wong and S. L. Lau, (1989), “Simplified Finite Element Analysis of Three-Dimensional Tall

Building Structures", Computers & Structures Vol. 33, No. 3, pp. 821-830.
[12] Kanat Burak Bozdogan, (2011), "A method for lateral static and dynamic analyses of wall-frame buildings using one dimensional finite element", Scientific Research and Essays Vol. 6(3), pp. 616-626, 4 February, 2011. Available online at http://www.academicjournals.org
[13] TolgaAkis, (2004), "Lateral Load Analysis of Shear Wall-Frame Structures", Ph.D. Thesis Dissertation. The Graduate School of Natural and Applied Sciences of The Middle East Technical University.
[14] B. Li, C. F. Duffield and G. L. Hutchinson, (2008),"Simplified Finite Element Modeling of Multi-storey Buildings: The Use of Equivalent Cubes", Electronic Journal of Structural Engineering.
[15] Z. Bayati, M. Mahdikhani and A. Rahaei, (2008), "Optimized Use of Multi-Outriggers System to Stiffen Tall Buildings", The 14th World Conference on Earthquake Engineering, October 12-17, Beijing, China,
[16] A.S. Moghadam and A. Aziminejad., (2004),"Interaction of Torsion and P-Delta Effects in Tall Buildings", 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1-6,Paper No. 799.
[17] Proceedings of the 6th Asia-Pacific Structural Engineering and Construction Conference, (APSEC 2006), 5 - 6 September 2006, Kuala Lumpur, Malaysia, A-161, "Nonlinear Finite Element Analysis of Reinforced Concrete Tube in Tube of Tall Buildings", Abdul Kadir Marsono and Lee Siong Wee.
[18] M. S. Ibrahim and A. E. Mohamed, (2013), "Analysis of Shear Wall Structures using the Moment Transformation Method", Journal of Science and Technology- Engineering and Computer Sciences, Vol. 14, No. 3, June 2013.
[19] M. S. Ibrahim, (2013), "Analysis of Tall Buildings using the Moment Transformation Method", M.Sc. Thesis dissertation, Sudan University of Science and Technology.
[20] Jack McCormac and Rudolf E. Elling, "Structural Analysis - A Classical and Matrix Approach", Harper and Row, published, 1988.
[21] Mohammed Jameel, A. B. M. Saiful Islam, Raja Rizwan Hussain, M. Khaleel and M. M. Zaheer, (2012), "The Structural Design of Tall and Special Buildings, Optimum structural modeling for tall buildings", Published online in Wiley Online Library.
[22] StaadPro2004 – "Structural Analysis and Design Software", Research Engineer International, California.
[23] Wilson, E. L., Dovey, H. H. and Habibullah, A., (2006), "ETABS, Three Dimensional Analysis of Building Systems", Computers and Structures Inc., Berkeley, California.