OVERALL EFFECT OF FRAME AND SHEAR WALL-FRAME SYSTEMS INTERACTION THROUGH LARGE GIRDER AND STIFF SLAB

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Dedication

To

my Father's soul

my beloved Mother

and my Brother

Akram

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Abstract

An analytical study had been undertaken to investigate the overall effect of "Girder (Ring Deep Beam) Around Stiff Slabs" on the structural response of reinforced concrete buildings.

The study goes through three main stages: the first stage was modeling the structures using the STAAD Pro 2000 software. A total of eight typical three-dimensional models of reinforced concrete building have been developed.

The second stage was extraction of which produce from the output file of STAAD Pro2000 into Excel sheets through word document:

The third stage was conducting the analysis of the results using Excel formulae.

The aspect that had been studied was the lateral displacement in X and Z directions, the vertical displacement, the axial force, shear force in Y and Z directions, torsional force and moment force in Y and Z directions.

The main results of this study were the great reduction of the lateral displacements through providing a linkage to mobilize the longitudinal stiffness of the peripheral column in resisting lateral loads.

Chapter One

Introduction

1-1 General

Usually deep beams are used in multi-story buildings to provide columns offsets, walls of rectangular tanks and bins, floor diaphragms, silos and bunkers with pyramid hoopers, in foundation walls supporting strip footings or raft slab, pile caps and in parapet walls.

In this study the effect of this special type of girder is to improve the resistance for the lateral loads by mobilizing the longitudinal stiffness to improve its participation in the lateral stiffness and to reduce the lateral displacement so as to improve the overall behavior of high rise buildings.

The effect of strong earthquakes, its destructiveness on high rise building structures and the method for minimizing this effect is one of the main current research fields.

Linear analysis of concrete structures is generally considered adequate in engineering practice because structures designed on the basis of simplified analysis methods have behaved satisfactory. Nevertheless the adequacy of linear analysis method has been questioned in a number of applications, notably where seismic loads are involved, and because of the complexity of reinforced concrete structures behavior such as the non-homogeneous nature of construction, nonlinear response of material loads, relative slip which occur between concrete and reinforcement and progressive destructiveness of bond in local areas.

In multi-story structures slabs transmit gravity loads to the vertical structural system such as rigid frames and shear walls, as well as acting integrally with the vertical system to resist gravity loads and lateral loads. These functions for floor slabs, which we aim to improve by this modification, are usually reflected to frame action which is controlled by the out – of – plan flexural characteristic. Another function of floor slabs is the diaphragm action, which is controlled by the in-plan characteristics of the floor slabs. In the analyses of three dimensional structural systems, the diaphragms are usually assumed to be rigid, which reduces considerably the complexity of the problem. Under the rigid diaphragm assumption the distribution of the lateral loads among vertical structural system depends only on their relative stiffness for lateral displacement.

A realistic evaluation of the stiffness, strength and ductility of the building depends not only on the inelastic characteristics of its vertical load resisting elements (beam-column frames and shear walls), but also on the seismic behavior of the reinforced concrete floor slabs and their capability to introduce the required interaction with the rest of the structural elements. Only after a preliminary evaluation the designer can make a rational decision regarding the selection of the desirable objectives.

Thus an understanding of the in-plan behavior of reinforced concrete floor slabs and their influence on the dynamic characteristic and response of the structures i.e., fundamental period and distribution of the lateral forces in vertical elements is necessary. These effects are usually ignored in an analysis when rigid floor is assumed.

1-2 Literature Review

Colunga et. al., (1996) Studied the influence of floor flexibility on the seismic response of the building structures, through comparison of the computed seismic response for structures with flexible diaphragm and counterpart structures with rigid diaphragm. The analytical study showed that, in some cases, diaphragm and shear wall accelerations can increase with the flexibility of the diaphragm, torsional forces could be reduce considerably as diaphragm flexibility increases.

Unemori et. al., (1980) used a finite element scheme to examine the cross wall building system including diaphragm flexibility and found that slab flexibility should be taken into consideration for relatively low buildings, with five or fewer stories. The analysis was carried out in the elastic range.

Panahshahi et. al., (1991) studied the effect of flexible floor diaphragms on the inelastic seismic response of reinforced concrete buildings. A combined experimental analytical approach was utilized. A macro-model approach was used, where the effects of both in plane flexural and shear are included. The correlation between the analytical predictions obtained using the developed models and experimental response was examined.

Tareq Tarabieh, (2000) evaluated the effect of diaphragm flexibility and strength on three-dimensional structural response. Several structures were evaluated based on the results of monotonic analysis of the designed structure.

Kunnath et. al., (1991) presented a simplified macro-modeling scheme to incorporate the effect of inelastic floor flexibility in the seismic response analysis of RC buildings. The study showed that the in-plane deflections of floor slab impose a larger demand on strength and ductility of flexible frames than that of rigid or elastic slabs. These may in turn lead to a failure of the gravity –load resisting system.

1-3 Objective and scope of the study

The overall objective of this study is to evaluate the effect of girder (ring deep beam) with stiff floor on the overall performance of high rise buildings. The following eight models of reinforced concrete building structure were examined:

1. Square shape model using frame type structure see Figure (1-1)
2. Modified Square shape model using frame type structure see Figure (1-2)
3. Square shape model using frame type structure with shear core in the middle see Figure (1-3)
4. Modified Square shape model using frame type structure with shear core in the middle see Figure (1-4)
5. T-shape model using frame type structure see Figure (1-5)
6. Modified T-shape model using frame type structure see figure (1,6)
7. T-shape model using frame type structure with shear core in the middle see Figure (1-7)
8. Modified T-shape model using frame type structure with shear core in the middle see Figure (1-8)

The models were selected to represent regular and irregular structures. The effect of the girder on the following factors (forces) has been studied: Shear force in X and Y directions, torsional force moment in X and Y directions, axial force, lateral displacements in X, Y, Z directions

The study was implemented on the UBC-94 code majors loading and cases of loading to conduct the comparison, which are:

1- (Dead Load) 2- (Live Load)

3- (1.4 Dead Load +1.7Live Load)

4- (0.9 Dead Load +1.43 EQ ± ve X)

5- (0.9 Dead Load +1.43 EQ ±ve Z)

6- 0.75×(1.4 Dead Load +1.7 Live Load +1.87 EQ ±ve X)

7- 0.75× (1.4 Dead Load +1.7 Live Load +1.87 EQ ±ve Z)

The comparison was based on the percentage of absolute change from the original value using the following formula

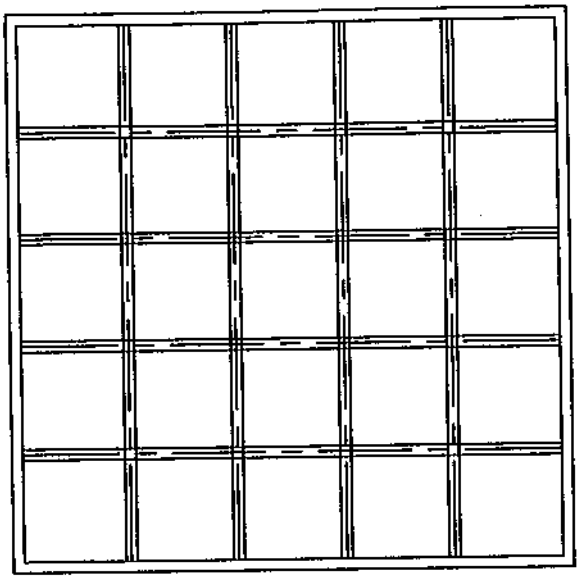
$$\text{Percent of Change} = \frac{|\text{Modified Value}| - |\text{Original Value}|}{\text{Original Value}} \times 100\% \qquad (1\text{-}1)$$

The comparison was conducted between the results which come out of the original models and modified models for all the member-end forces and all the node displacements of each model for all types of forces previously mentioned using also all the above mentioned cases of loading, and all the node displacement of each model for all types of displacement previously mentioned using all the above mentioned cases of loading.

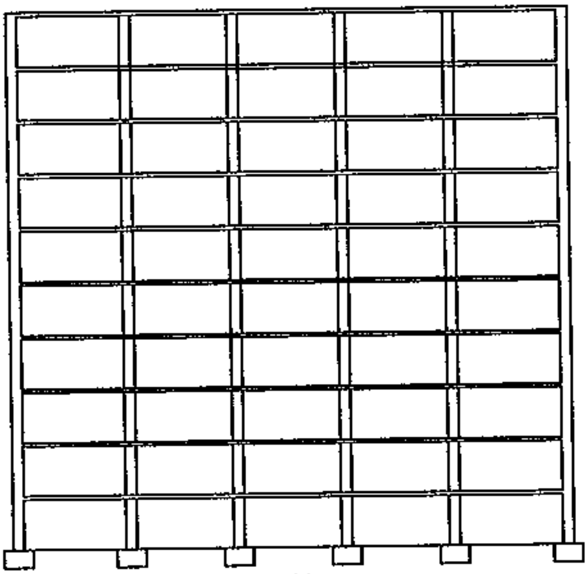
1-4 Examples of previous buildings

One of the first reinforced concrete building utilizing this concept was the 51-story Place Victoria, Victoria Towers in Montreal Figure (1-9) which was constructed in 1964 in which the X-Shaped core is linked at four levels (Mechanical floors) by story high girder to the heavy corner columns (Mark Fintel, 1974).

The U.S. Steel Building in Pittsburgh completed in1970, is another example with heavy girder at the roof level connecting the core with the exterior columns (Mark Fintel, 1974)

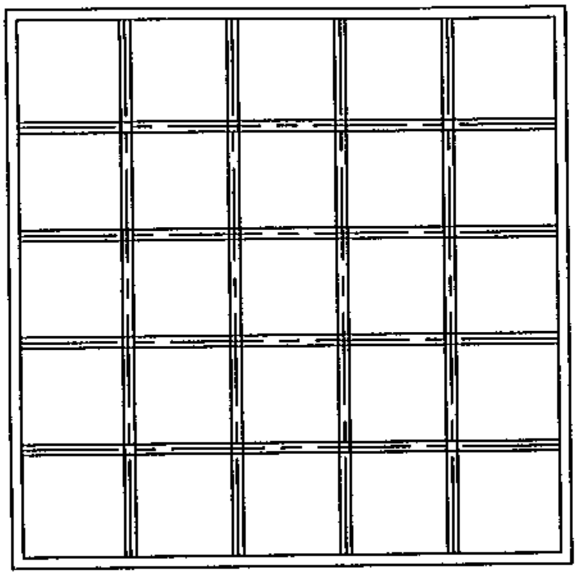


(a) Plan



(b) Profile

Figure (1-1)
Square Shape Frame Structure
Before Modification



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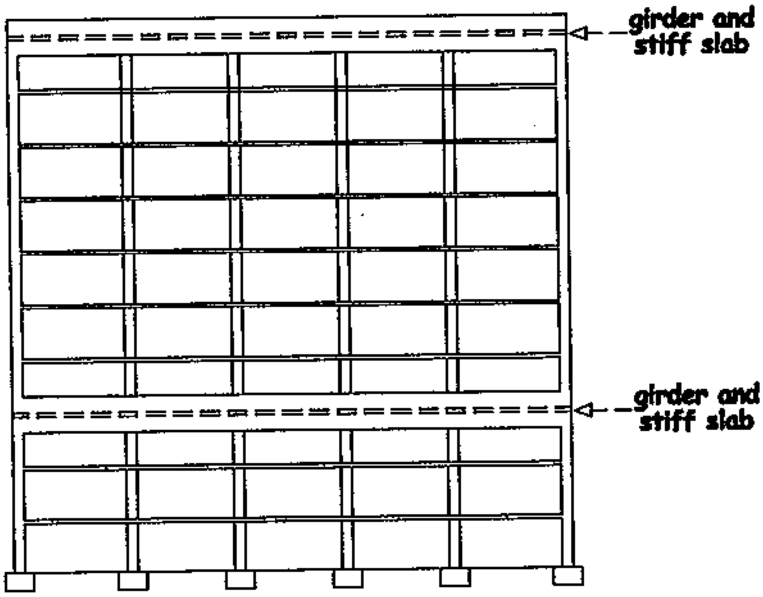
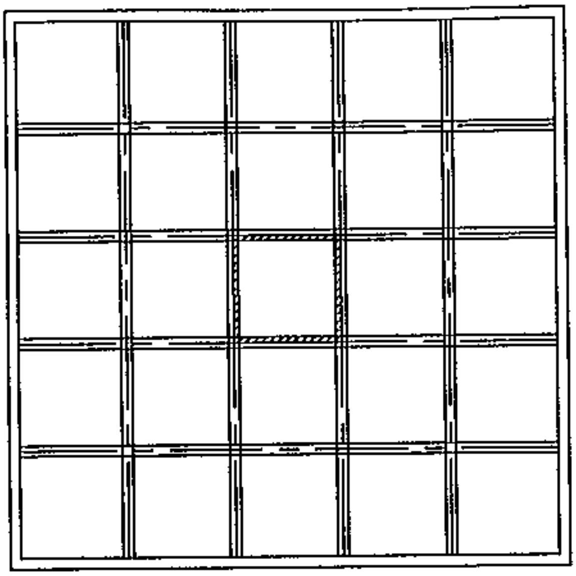


(b) Profile

Figure (1-2)

Square Shape Frame Structure
After Modification



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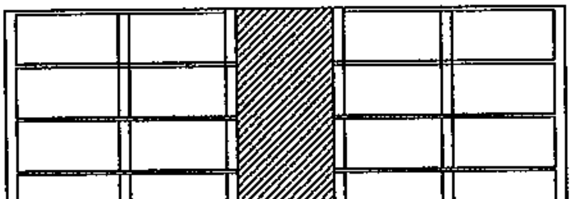
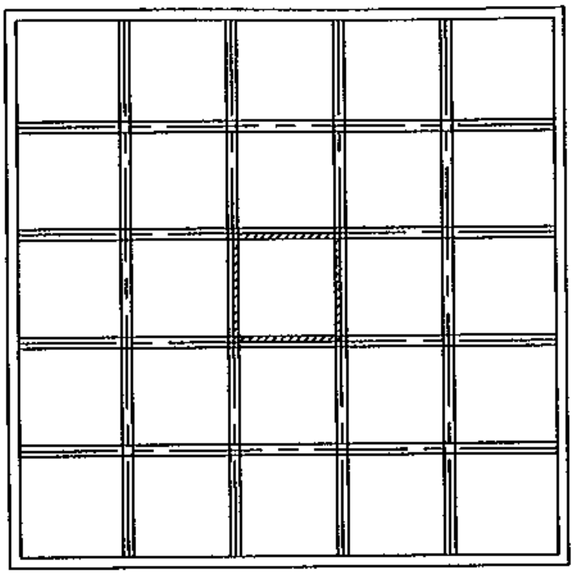
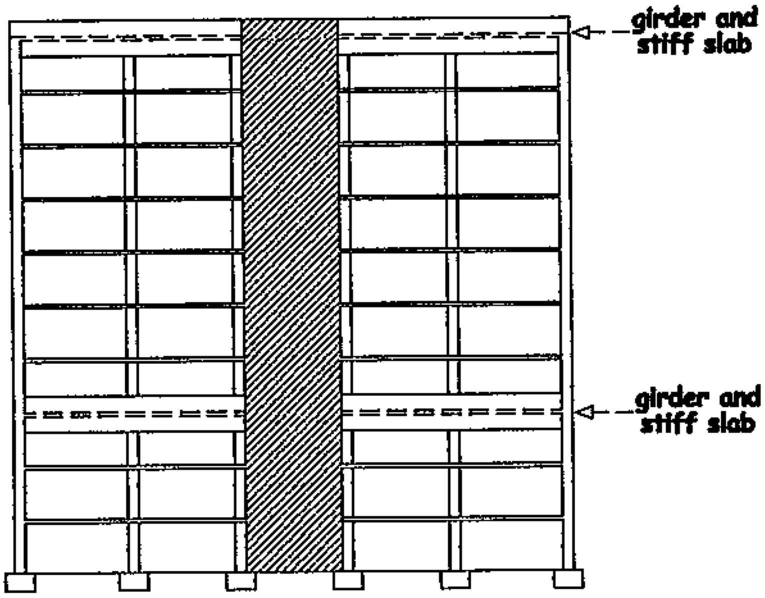


(b) Profile

Figure (1-3)
Square Shape Shear Wall Structure
Before Modification



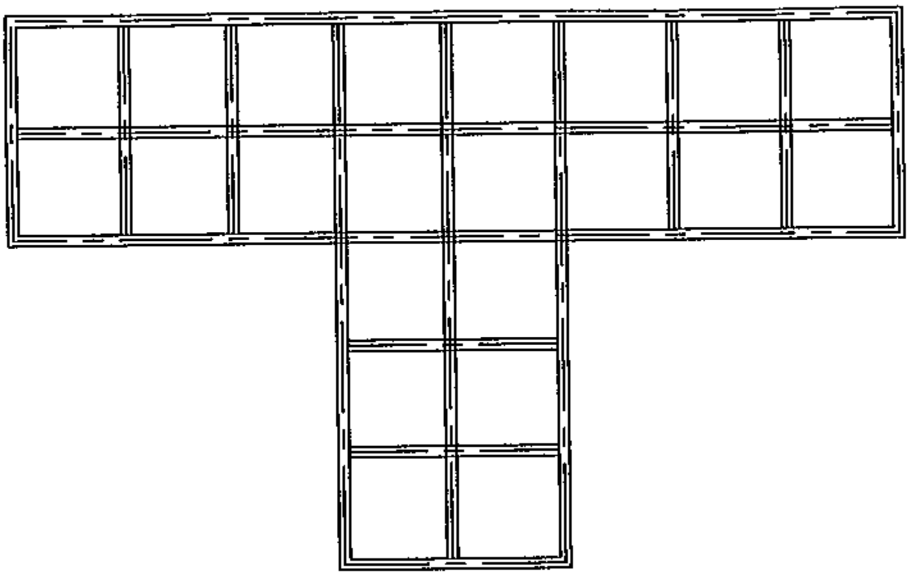
(a) Plan



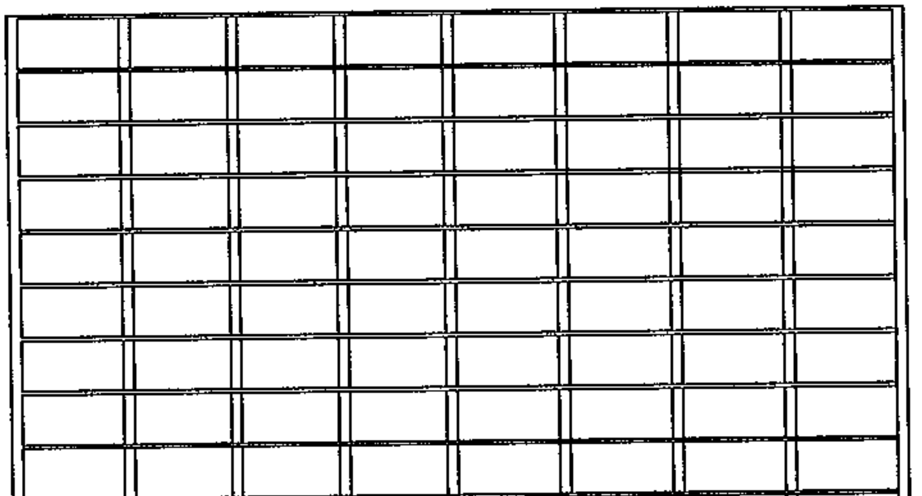
(b) Profile

Figure (1-4)

Square Shape Frame Wall Structure
After Modification

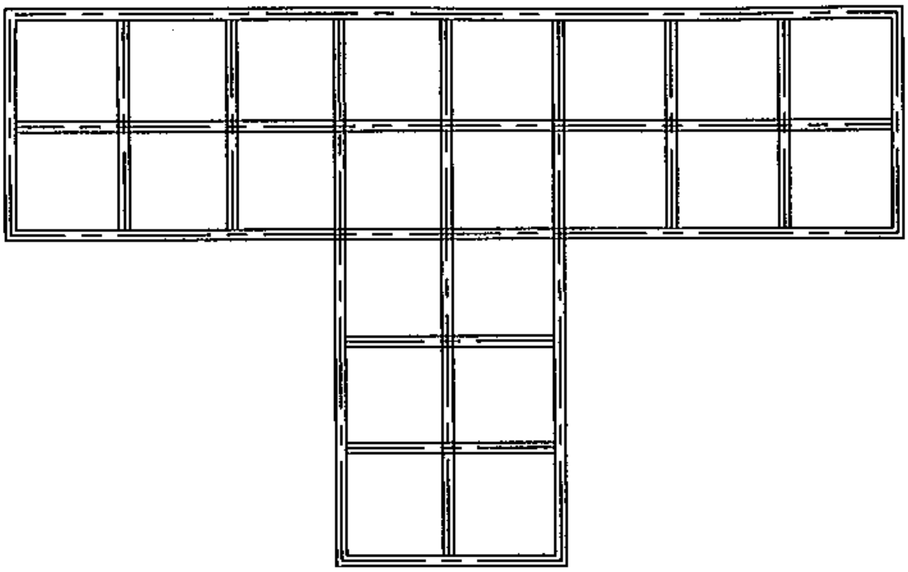


(a) Plan

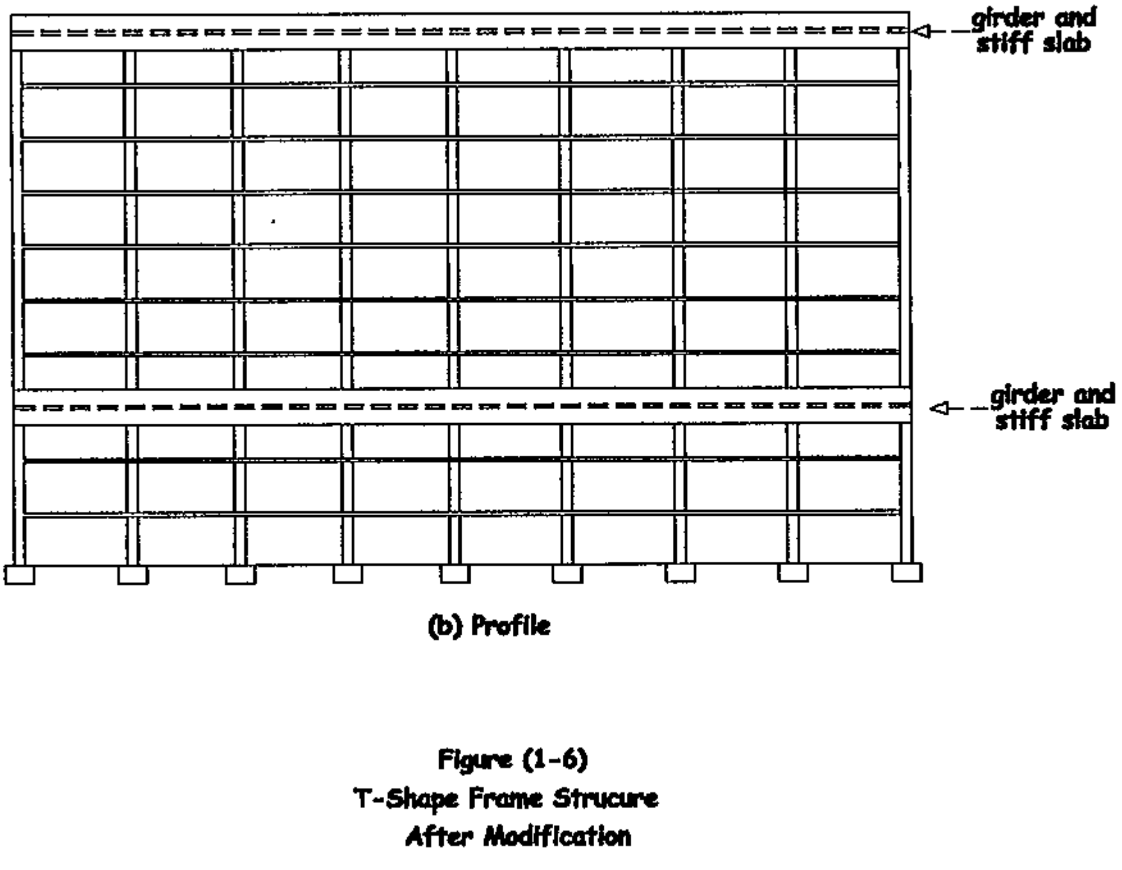


(b) Profile

Figure (1-5)
T-Shape Frame Strucure
Before Modification

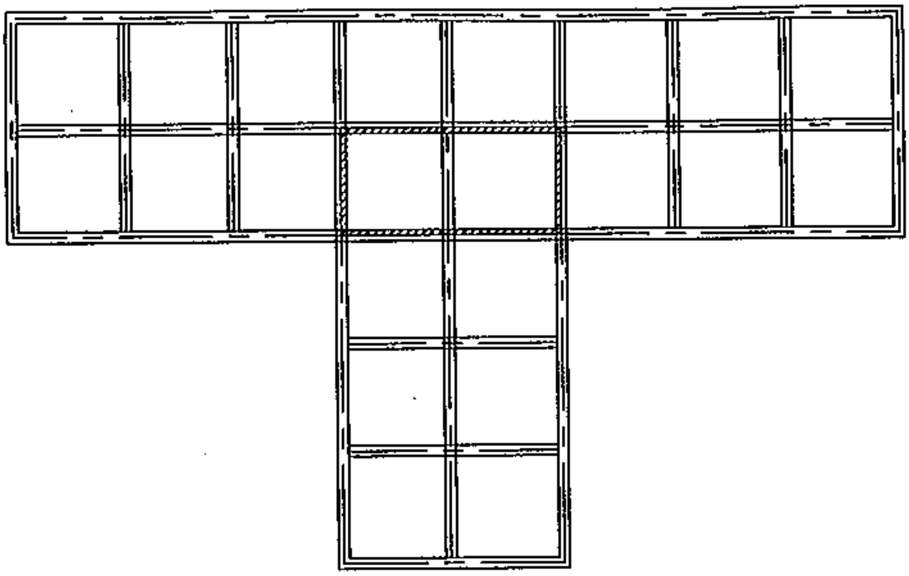


(a) Plan

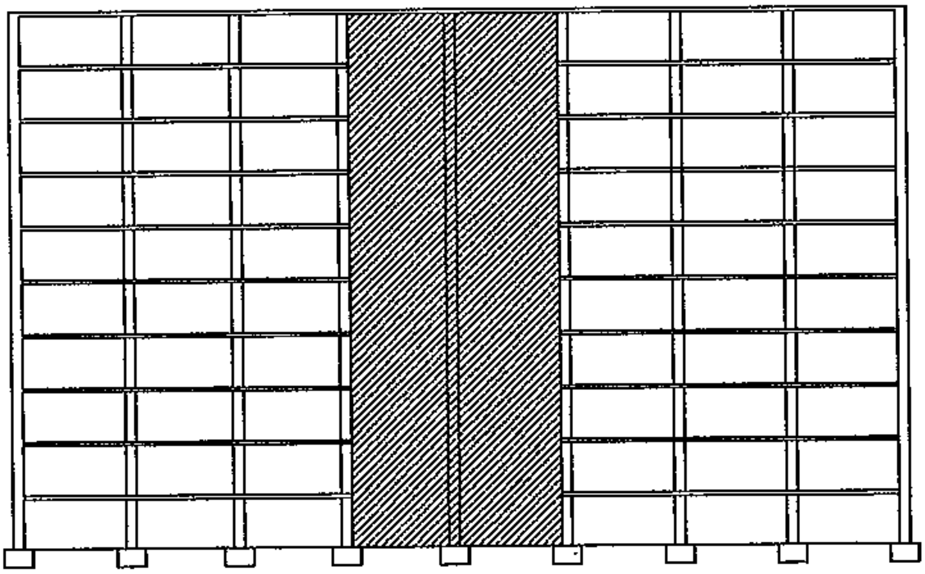


(b) Profile

Figure (1-6)
T-Shape Frame Strucure
After Modification

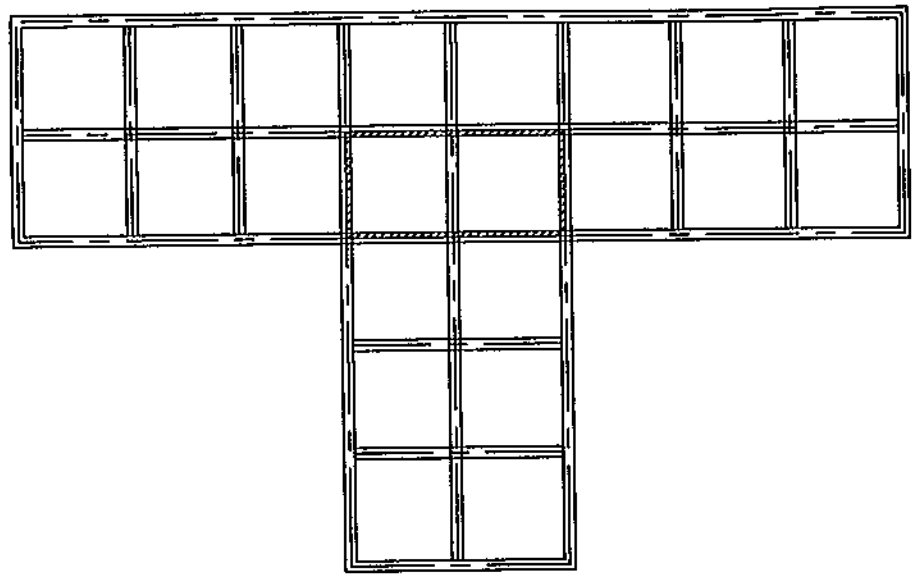


(a) Plan

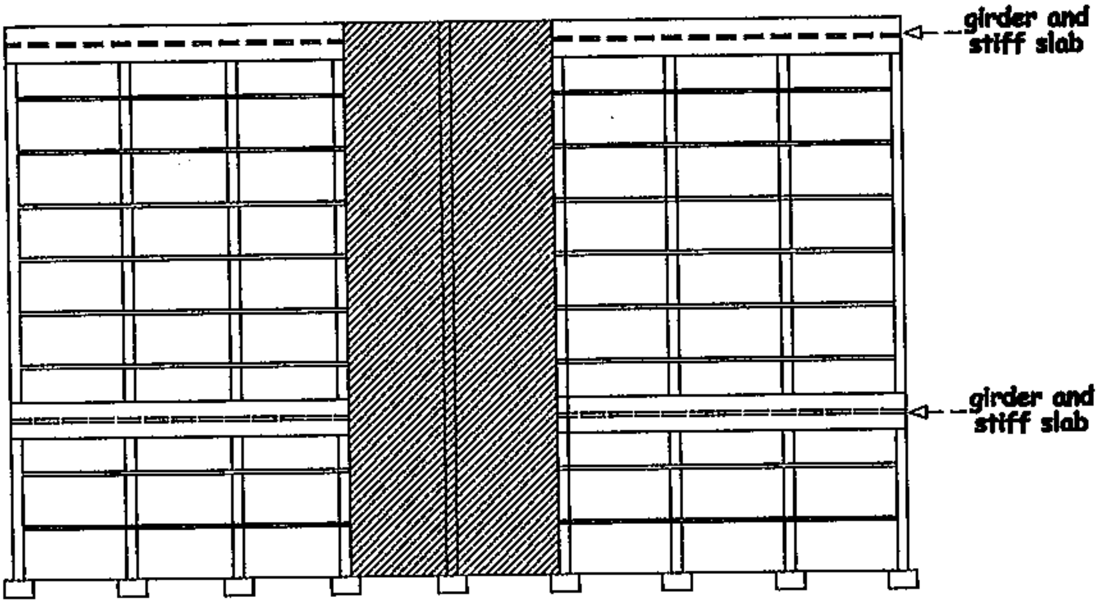


(b) Profile

Figure (1-7)
T-Shape Frame - Shear Wall Strucure
Before Modification



(a) Plan



(b) Profile

Figure (1-8)
T-Shape Frame -Shear Wall Strucure
After Modification

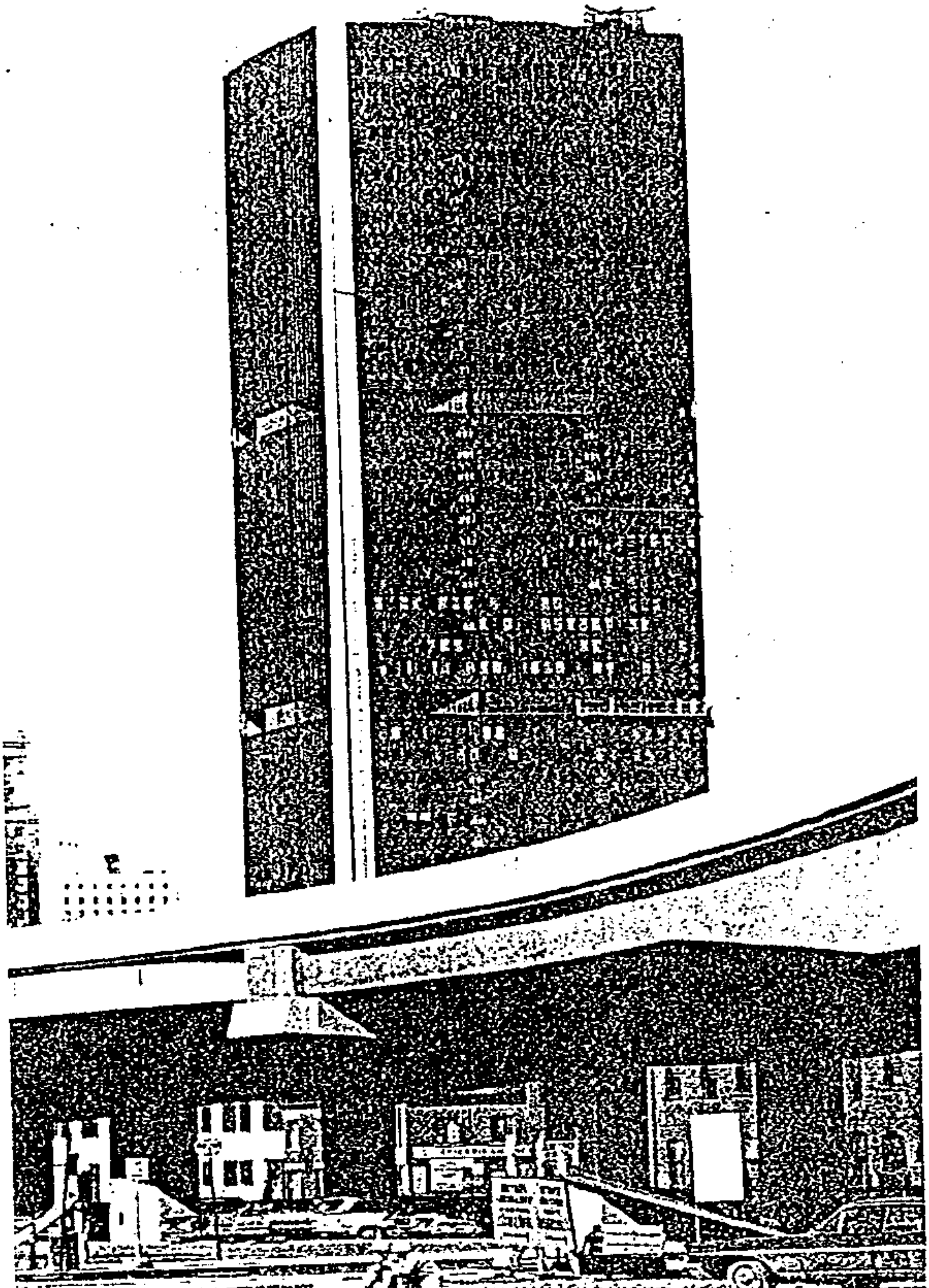


Figure (1-9) the 51-Story High Place Victoria, Monterial Canada

Chapter Two

Modeling

2-1 General Structural Modeling

Usually the elements of structures are idealized as straight-line segments. The intersections of these elements are the joints (nodes) with moment resisting rigid connections. The cladding elements either they are structural elements or nonstructural elements are assumed to contribute only to the static load of the structure. Windows frames and partitions can be considered only for initial stiffness for the structures, but under sever deformation these elements lose their scriptural capacity by brittle failure, in which case the structural framework would then sustain the applied loads.

The wide use of computers made the analysis of various structural problems more flexible and accurate. Methods of analysis such as the finite element method were developed primarily to facilitate the analysis of complex structures used in the modern aircraft industry. The method is used in a more routine way in the analysis of large and small problems and is considered as the most powerful and versatile tool in structural analysis. In the application of the finite element method, it is possible to select types of elements, which differ in various aspects, such as degree of freedom at nodes and any other aspects.

A finite element model of three-dimensional structure is constructed by choosing a global coordinate system in which all the individual elements are defined by nodes at its ends. Each joint is allowed to rotate and translate freely in the space; this movement in the three orthogonal directions represents the degree of freedom of the joints, thus there are a maximum of six independent degrees of freedom, three transitional and three rotational, that can be specified at each joint.

One of the ways to approximate a three dimensional structure is to separate it into a series of one-bay deep two dimensional intersecting frames, to be analyzed individually as plane frame. The rationale for cutting the space frame into plan frame is that the cutting planes are planes of zero shear. This is a good assumption for the vertically loaded interior frames in buildings having regular column layout, if the frames are far from parallel discontinuous edges and have similar stiffness. As proximity to a discontinuous parallel edge, variations in frame stiffness, irregularities in column layout, floor shape load pattern, and number and size of openings make the two-dimensional approximation less acceptable.

One of problems that faces the engineer is the modeling of the shear cores which is know to be a paramount issue in modern structural analysis, especially when the case under consideration involves lateral loading such as resulting from wind or seismic action. Modeling of shear core prior to the introduction of digital computers was more often carried out using simple fixed ends cantilever beam idealization. This inherently disregarded the interaction between the shear core and other elements of the structures. With the current status of both hardware and software dealing with the structural modeling, it is definitely worth while taking into consideration the structure in its totality for analysis and subsequently. For designer the issue now is which path to follow when it comes to the idealization of the shear core. The current state-of-art subdivides shear core modeling into three main ways.

The first : model the core as continuum of finite elements, which is used in this study.

The second : takes the shear core as individual column taking an equal share of the cross sectional properties of the core.

The third : collects the property of the cross section of the shear core into one central column.

2-2 System Discretization

The following elements are used in the three-dimensional reinforced concrete frame-shear wall structural models:

1. Beam element
2. Column element
3. Shear wall element
4. Slab element

2-2-1 Beam element:

The beam elements are typically transverse bars connected between column elements with the axis of the beam lying in the plane of the floor slab and running parallel to the direction of the loading which activates the in plane with the slab. The presence of floor slab thus constrains the bending of the beam in plane with the slab to very small deformation.

The beam stiffness matrix is shown in (equation 2-1)

The stiffness matrix shows the number degree of freedom. The time variable components of the stiffness matrix include the influence of nonsymmetrical beam properties, and nonlinear inelastic behavior.

Some complex non-linearities are present. Some of these effects are:

$$[K_b]=\begin{bmatrix} \frac{AE}{L} & & & & & & & \\ 0 & \frac{12EIz(t)}{L^3} & & & & & & \\ 0 & 0 & \frac{GJ}{L} & & & & & \\ 0 & \frac{6EIz(t)}{L^3} & 0 & \frac{4EIz(t)}{L} & & & & \\ -\frac{AE}{L} & 0 & 0 & 0 & \frac{AE}{L} & & & \\ 0 & -\frac{12EIz(t)}{L^3} & 0 & \frac{-6EIz(t)}{L^2} & 0 & \frac{12EIz(t)}{L^3} & & \\ 0 & 0 & -\frac{GJ}{L} & 0 & 0 & 0 & \frac{GJ}{L} & \\ 0 & \frac{6EIz(t)}{L^2} & 0 & \frac{2EIz(t)}{L} & 0 & \frac{6EIz(t)}{L^2} & 0 & \frac{4EIz(t)}{L} \end{bmatrix}$$

Equation (2-1)

Beam Stiffness Matrix

1. Yielding of flexural steel

Under load reversals, yielding may occur in both top and bottom steel. When flexural steel yields in tension for load in one direction, one or more large cracks open up. In order for these cracks to close again, the steel must yield in approximately equal amount in compression under the opposite loading. Even then, complete gap closure is not assured and open crack may propagate over the full beam depth, leading to a decrease in stiffness (Mayer.C., 1991)

2. Flexural cracking

This varies along the beam length and mainly is a function of bending moment being most pronounced in the plastic hinges. Because cracking is also affected by bond slip, shear forces can be important.

3. Buckling of flexural steel

Supporting the flexural steel by sufficiently closed spaced stirrups will prevent or decrease the buckling in compression for flexural reinforcement.

4. Cyclic yielding of flexural steel

Standard reinforcing steel exhibits hysteretic behavior under cyclic loading (Chopra A.K., 1995) , including pronounced strain hardening and Bauschinger effects.

5. Crushing of concrete

Under strong cyclic load the concrete strength decreases gradually. As a result the concrete cover may crush and spill off.

6. Cracking and yielding under shear

The effects of the shear on concrete and steel are complex especially when both shear and moment are present since shear failures are typically sudden. It is common practice to detail reinforced concrete members such that only the more ductile flexural failures can occur (Park R. and Pauley T., 1975).

2-2-2 Column

A line element connecting vertically between nodes. It resists forces due to load that result in the four principal deformations of members axial, bending, torsion and shear.

Each node has six degrees of freedom, so each element has 12 degrees of freedom. A typical stiffness matrix for columns is shown in equation (2-2) with flexural rigidity EI(t) as a function of time and axial stiffness assumed to remain elastic throughout the analysis. Torsional stiffness of columns is neglected by assigning zero value and the shear deformations in two orthogonal directions are ignored.

All phenomena which cause non-linearity in beams are present in columns, with the following additions:

1. P-delta effect

In tall or flexible building the P-delta effect due to lateral deformation adds significant additional over turning force which can't be ignored (Wilson et. al., 1973).

2. Biaxial bending

Common columns for the orthogonal frames of building are subjected to bending forces around both axis, which decrease the column capacity significantly.

3. Interaction of axial force and bending moment

This is usually represented through conventional interaction curves.

2-2-3 Shear Wall

Shear Walls usually have very large stiffness in-plane and small stiffness out of plane. Usually shear walls attract larger lateral forces than columns.

Essentially all phenomena, which cause nonlinearity in beams and columns apply to shear walls in addition to the following factors:

1. Confinement of concrete:

Due to difficulty and impracticality of confining the concrete over the entire section depth, usually the amount of confined concrete is very small, which affects both flexural and shear behavior.

2. Migration of neutral axis:

Due to cracks and crushing of concrete the neutral axis changes its position. This effect is very obvious in wide shear walls. During cyclic loading the neutral axis tends to be close to the compression face, so the wall tends to tilt first about one edge, then the other, with significant vertical movement at the wall centerline.

3. **Shear failure:**

The amount of moment capacity of wall is very. This high capacity will increase the potential for shear failure, since shear stresses are relatively low. Flexural cracks near the base tend to be horizontal, which reduces the wall shear capacity, as only little effective truss actions develop and sliding shear failure become a very high possibility (Li K. et.al., 1975).

$$[K_c]=\begin{bmatrix}
\frac{AE}{L} \\
0 & \frac{12EIz(t)}{L^3} \\
0 & 0 & \frac{12EIy(t)}{L^3} \\
0 & 0 & 0 & \frac{GJ}{L} \\
0 & 0 & -\frac{6EIy(t)}{L^2} & 0 & \frac{4EIy(t)}{L} \\
0 & \frac{6EIy(t)}{L^2} & 0 & 0 & 0 & \frac{4EIy(t)}{L} \\
-\frac{AE}{L} & 0 & 0 & 0 & 0 & 0 & \frac{AE}{L} \\
0 & -\frac{12EIz(t)}{L^3} & 0 & 0 & 0 & \frac{-6EIz(t)}{L^2} & 0 & \frac{12EIz(t)}{L^3} \\
0 & 0 & -\frac{12EIy(t)}{L^3} & 0 & \frac{6EIy(t)}{L^2} & 0 & 0 & 0 & \frac{12EIy(t)}{L^3} \\
0 & 0 & 0 & -\frac{GJ}{L} & 0 & 0 & 0 & 0 & 0 & \frac{GJ}{L} \\
0 & 0 & \frac{6EIy(t)}{L^2} & 0 & \frac{2EIy(t)}{L} & 0 & 0 & 0 & \frac{6EIy(t)}{L^2} & 0 & \frac{4EIy(t)}{L} \\
0 & \frac{6EIz(t)}{L^2} & 0 & 0 & 0 & \frac{2EIz(t)}{L} & 0 & \frac{6EIz(t)}{L^2} & 0 & 0 & 0 & \frac{4EIz(t)}{L}
\end{bmatrix}$$

Equation (2-2)

Column Stiffness Matrix

2-2-4 Slab Element:-

The modeling of reinforced concrete slabs using finite element analysis is classified into two different approaches:

1. **Modified stiffness approach:**

This is based on an empirical moment-curvature relationship where different flexural rigidities are assumed for different material states (Karadogan H. F. et. al.,1982).

2. **Layered model:**

This is based on the basic nonlinear stress-strain law. The finite element is divided into imaginary concrete layers, each layer having different material properties corresponding to the material state.

2-3 Static analysis of the models

2-3-1 Static analysis under gravity loads

The analysis of structural elements under dead and live loads is determined by static analysis. Loads may be specified as uniformly distributed load or concentrated loads.

The following equation is applicable for the static analysis with gravity load represented by:

$$\{F\} = [K]\{u\} \quad \text{--------------------------------} (2\text{-}3)$$

where

F: Load vector with equivalent node forces

K: Global stiffness matrix

u: displacement vector at nodes

2-3-1-1 Generation of element stiffness matrix

Defining the unknown degree of freedom at each node of the element and displacement field, the stiffness matrix of the element is evaluated based on the virtual work concept according to the following equation

$$K = \int [B]^T [E] \times [B] dv \text{ --(2-4)}$$

Where

K: element stiffness matrix

B: d (N) , where N : Shape function matrix

E: Modulus of elasticity matrix

2-3-1-2 Generation of Nodal Vector

Forces distributed throughout an element or over its surface should be converted to load at element nodes using the following formula

$$\{u\} = \int [N] T \{f_b\} dv + \int [N] T \{f_s\} ds + \sum [N] T \{ F\} \text{-------------(2-5)}$$

where :

u : element load vector

N : shape function matrix

f_b :body force vector

f_s :surface force vector

F : concentrated force vector

Element load vector will be converted to a common coordinate references system that is the global axis of the system. Then the global element load vector will be added to the forces that are applied directly to the element nodes referenced to the global system. The result will be the global node force vector.

2-3-1-3 Structural analysis of element assemblage.

The element stiffness matrix [K] for each element will be transformed to the global coordinate system. By assembling all the matrices of the elements, the result will be a system of equations for each node, i.e., the sum of contribution for all elements meeting at the same node. Applying the boundary condition of the system and solving the equilibrium equations will give the global structure nodal displacement at all inter element node. With these displacement established, the components of strain and stress can be evaluated.

2-4 Dynamic Analysis

The step-by-step procedure is a general approach to dynamic response analysis, and it is well suited to analysis of nonlinear response, because it avoids any use of superposition.

There are many different step-by-step methods, such as Newmark method ,but in all of them the loading and response history are divided into a sequence of time intervals or “steps”. The response during each step is calculated from the initial conditions (displacement and velocity) existing at the beginning of the step and from the history of loading during the step. Thus the response of each step is an independent analysis problem and there is no need to combine response contribution within the step.

Nonlinear behavior may be easily considered by this approach merely by assuming that the structural properties remain constant during each step, and causing them to change in accordance with any specified form of behavior from one step to the next. Hence nonlinear analysis actually is a sequence of linear analysis of a changing system.

Any desired degree of refinement in the nonlinear behavior may be achieved in this procedure by making the time steps short enough. Also it can be applied to any type of nonlinearity, including change in mass and damping properties, as well as the more common nonlinearity due to change in stiffness.

Step-by-step analysis provides the only complete general approach to analysis of nonlinear response; however, the methods are equally valuable in the analysis of linear response because the same algorithms can be applied regardless of whether the structure is behaving linearly or not. Moreover the procedure can be used to solve single-degree-of freedom structure and can easily be extended to deal with multidegree systems merely by replacing scalar quantities by matrices. In fact these methods are so effective and convenient that time-domain analysis almost always are done by some form of step-by-step analysis regardless of whether or not the response behavior is linear (Clough R. W. et. al., 1975)

The equation of motion in terms of the relative displacements of the mass points can be written in an incremental form as follows:

$$\{df_I\} + \{df_D\} + \{df_R\} = \{dP\} \text{ -- } (2\text{-}6)$$

Where:

$\{df_I\}$ = Incremental inertia force vector = $[M]\ \{d\ddot{U}_t\}$

$\{df_D\}$ = Incremental damping force vector = $[C]\ \{d\dot{U}_t\}$

$\{df_R\}$= Incremental restoring force vector = $[R]\ \{dU_t\}$

$\{dP\}$ = Incremental driving force = $\{dP_t\}$

For the solution of Equation (2-6), a direct step-by-step integration procedure using the Newmark method is adopted (Newmark N.M., 1959). The following assumption can be used as an extension of the linear acceleration:

$\{\acute{U}\}_{t+dt} = \{\acute{U}\}_t + dt[(1-\delta)\{\ddot{U}\}_t + \delta\}\ \ddot{U}\}_{t+dt}$ ---------------------------------------(2-7)

$\{U\}_{t+dt} = \{U\}_t + dt\{\acute{U}\}_t + (dt)^2\,[(1/2 - \beta)\{\ddot{U}\}_t + \beta\{\ddot{U}\}_{t+dt}]$ -------------------(2-8)

Where β and δ are parameters that can be determined to obtain integration accuracy and stability.

Newmark originally proposed as an unconditionally stable scheme the constant average-acceleration method, in which β = 14 and δ = ½. The incremental velocity and the incremental acceleration can be expressed with the suggested â and β and equation (2-7) and equation (2-8) in the following form:

$\{\acute{U}\}_{t+dt} = (dt/2)\{\ddot{U}\}_t + (2/dt)\ \{dU\}_{t+dt} - 2\{\acute{U}\}_t - dt\{\ddot{U}\}_t$ --------------------(2-9)

$\{\Delta\ddot{U}\}_{t+dt} = (4/(dt)^2)\{\ dU\}_{t+dt} - (4/(dt))\{\acute{U}\}_t - 2\{\ddot{U}\}_t$ -------------------------(2-10)

Substituting equations (2-9) and (2-10) into the dynamic equation of equilibrium in equation (2-6) yields the incremental displacement at the current time step.

$\{U\}_{t+dt} = [K^*]^{-1}\ \{dF^*\}_{t+dt}$ --(2-11)

Where [K*]-1 and {dF*} are the dynamic stiffness matrix and dynamic load vector as follow:

$[K^*] = (4/(dt)^2)\ [M] + (2/dt)\ [C] + [K]$ --(2-12)

$\{dF^*\} = \{dF^*\}_{t+dt} + \{(4/(dt))\ [M] + 2\ [C]\}\ \{\acute{U}\}_t + \{2[M] + (dt/2)\ \}\{\ddot{U}\}_t$ ----(2-13)

Once the incremental relative displacement vector has been obtained the incremental relative velocities and accelerations are calculated from equation (2-9) and (2-10), respectively.

The dynamic analysis of a consistent mass system generally requires more computational effort than that of a lumped mass system. This is because the lumped mass matrix is diagonal, while the consistent mass matrix has many off-diagonal terms for mass coupling (Paz M., 1997).

Small amount of viscous damping can substantially reduce the response of elastic systems. In reinforced concrete structures most of the damping is derived from inelastic loading reversals. The contribution of viscous damping is obviously negligible when compared to the energy that will be dissipated through hysteretic behavior (Paz M., 1997)(Chopra A. K., 1995).

The following assumption that there is no change in the properties of the structure is used in the incremental step-by-step numerical integration, and any change in the element stiffness yields an overshooting in the strength-deformation relationship; this overshooting normally generates residual forces, which violate the equilibrium requirement. Instead of adopting an iterative procedure the unbalances are transferred or considered at the next time 'step' of the analysis.

2-5 Diaphragm Theory

2-5-1 Diaphragm Action

The story shear which can be calculated by equations (3-4) and (3-6) are assumed to be applied to a lumped mass representing the floor/ceiling layer in a building. The ceiling does not actually resist the story shear, but it does distribute the force among the resisting frames, and other structural elements (e.g., columns, moment resisting elements, shear walls, and any other structural elements).

Ceilings and floors that transmit lateral forces to the resisting elements are known as horizontal diaphragms. The diaphragm's function of distributing the story shears is known as diaphragm action. It is common to refer to the story shear as diaphragm force. However, it should be recognized that the diaphragm force might include some of the story shears for the level and above.

Generally concrete slab floors and diaphragms are considered to be rigid.

2-5-2 Seismic wall and diaphragm forces

When discussing seismic forces in structures with diaphragms, it is important to distinguish between forces in parallel and perpendicular walls. The forces in parallel walls are shear forces, while the forces in perpendicular walls are normal forces.

The seismic shear forces acting on the parallel walls depend on the mass being accelerated. The diaphragm mass accounts for the largest portion and it is usually assumed to be half of the total weight. The diaphragm weight includes the weight of anything suspended inside the building from the roof, and anything mounted on the upper half of the walls. The wall weight includes the weight of any parapet that projects above the roofline.

Forces are calculated as F = ma = (W/g) a, this formula depends on the weight, W, of the structure. Openings such as windows and doors are disregarded when calculating the weight of the wall.

The total seismic force resisted by the two parallel walls near the ground level is the sum of seismic forces resulting from the diaphragm and wall weight.

The portion of the seismic load originating from the acceleration of the perpendicular walls is given by Equations (2-14) and (2-15).

$$F_{\text{per. walls}} = \left(\frac{1}{2}\right) Z\,I\,C_p\,W_{\text{per. walls}} \text{ --(2-14)}$$

$$F_{\text{per. walls}} = \left[\left(\frac{1}{2g}\right) W_{\text{per. walls}}\right] a \text{ --(2-15)}$$

The portion of seismic load originating from the acceleration of the parallel wall is given in equations (2-16) and (2-17).

$$F_{\text{parl. walls}} = \left(\frac{1}{2}\right) Z\,I\,C_p\,W_{\text{parl. walls}} \text{ --(2-16)}$$

$$F_{\text{parl. walls}} = \left[\left(\frac{1}{2g}\right) W_{\text{parl. walls}}\right] a \text{ --(2-17)}$$

To calculate the seismic force on the diaphragm at the diaphragm-to-parallel wall connection equation (2-18) and (2-19) are used.

$$F_{\text{diaphram}} = F_{\text{per.}} + ZIC_pW_{\text{diaphragm}} \text{ --(2-18)}$$

$$F_{\text{diaphram}} = F_{\text{per.}} + \left[\left(\frac{1}{g}\right) W_{\text{diaphragm}}\right] a \text{ --(2-19)}$$

Equation (2-20) is used to calculate the total shear force on the parallel walls.

$$F_{\text{total}} = F_{\text{diphragm}} + F_{\text{paral. walls}} \text{ ---(2-20)}$$

There are two reasons for calculating the forces from the diaphragm and parallel walls separately. The first reason is to distinguish between the two for the purpose of subsequent calculation; that is, the parallel wall force doesn't contribute to chord loads and diaphragm shear where the diaphragm is flexible.

The second is to emphasize the timing difference that occurs in a real earthquake. The perpendicular and parallel walls experience an almost immediate force due to ground acceleration. However, the parallel walls receive the diaphragm force only after some delay. Unfortunately, an accurate analysis of this aspect of seismic behavior is almost impossible. For simple structures with three or fewer floors, the static method of adding all forces together must suffice.

2-5-3 Rigid diaphragm action

A rigid diaphragm doesn't change its plan shape when subjected to lateral loads. It remains the same size, and square corners remain square. There is no internal bending. The same deflection is experienced by all parts of the diaphragm. Rigid diaphragms are capable of transmitting torsion to major resisting elements. The lateral story shear is distributed to the resisting elements in proportion to the rigidities of those elements.

2-5-4 Flexible diaphragms

A flexible diaphragm changes shape when subjected to lateral loads. Its forward edges bend outward, and its back edges bend inward, with a deflection shape similar to that of a simply supported beam loaded uniformly. Flexible diaphragms are assumed to be incapable of transmitting torsion to the resisting elements.

A flexible diaphragm distributes the diaphragm force in proportion to the tributary areas of the diaphragm, as opposed to distributing it in portion to the rigidities of the vertical resisting elements, as does a rigid diaphragm.

The UBC 94 define the flexible diaphragm as one that has a maximum lateral deflection more than two times the average story drift. To determine if a diaphragm is flexible, the in-plane deflection at the midpoint of the diaphragm is compared to the story drift of the adjoining vertical resisting elements under equivalent tributary load.

Chapter Three

Methodology and Study Procedure

3-1-General

In this Chapter a general description of the procedure that the study followed, the tools which have been used, material description, structure layout, members dimensions, loads, load combinations, and the problems which faced the study is described.

3-2-Tools

The following software have been used as tools for this study

- ☐ STAAD PRO2000
- ☐ Microsoft Word
- ☐ Excel Spread Sheet

3-3-Methodology

The study had gone through two main stages and one transition stage

1.Stage I :The modeling Stage (Modeling the structures using STAAD PRO2000)

2.Stage II :The transition Stage (Transfer raw data from STAAD PRO2000 output files to Excel Spread Sheets through world document)

3.Stage III :The analysis stage (Analyzing Data which has been transferred to Excel Sheet using Excel formulas)

3-3-1 Stage I : The modeling Stage:

3-3-1-1 Material Description

The mechanical properties of the concrete which had been used in constructing the models are as follows:

☐	Concrete Strength	$f'_c = 25\ N/mm^2$
☐	Modulus of elasticity	$Ec = 200{,}000\ N/mm^2$
☐	Poisson's ratio	$= 0.2$

3-3-1-2 Structural Layout

In this study two different building layouts have been investigated, square plan for regular structures as shown in Figure (1-1) and T shaped structure for irregular structures as shown in Figure (1-5). Each layout has been modeled twice, first as frame buildings consisting only of beams, columns and slabs, and the other model as frame buildings with a shear core in the middle of the structure.

Figure (1-1) shows the plan and elevation of a typical square structure, without shear walls. The selected structures are ten-Stories high and number of bays is five, the bay width and length selected to be 6 m .

Figure (1-3) shows the plan and elevation of a typical square structure, with a shear core in the middle and having the same dimension as the above-mentioned structure.

Figure (1-5) shows a T-shape building structure which is used to represent irregular structures. No shear walls have been used and the number of bays in the X direction is eight and in the Z direction six. The width and length of the bays is 6m, with building also consisting of ten stories.

Figure (1-7) is the same as figure (1-5) except for the presence of a shear core in the middle of the structure.

3-3-1-3 Designation of the structural elements

For the designation of the structures an alphapatical code is used to ease identification.

Since there are eight different structures, the coding system use three characters for each structure:

The first part of the code uses one character which identify the structural layout of the structure. **S** is used for the square structures and **T** for the T-Shape structures.

The second part of the code uses two characters which identify the structural type of the building. **FR** is used for the structures without shear core and **SH** for the structures which have shear core.

The third part of the code uses one character represents whether the structure has the added girder (ring deep beam) and the stiff slab or not **R** represents structures that had the added girder and **O** for the structures that don't have the added girder.

3-3-1-4 Member Dimensions:

The dimensions of the elements of the structures are unified as follows:

The height of each floor is 3100 mm, the column dimensions are 800 mm × 800 mm and the shear walls are 200 mm thick. There are two types of slabs ordinary slabs with 200 mm thickness which is in all floors except the tenth and the third floor for the modified structures which have the girder (ring deep beam) these slab has 300 mm thickness. The beam dimensions are 300 mm ×600 mm, the special girder is 2000 mm×500 mm and the bay dimension is 6000 mm in depth and width.

Figure (3-1) shows the detailed dimensions mentioned above.

3-3-1-5 Loads and Loads Combinations:

∢ **Gravity loads**

The gravity design loads consist of the dead load of the self-weight of the element of the structure and the live load specified in the building design codes. In this case the live load is taken as 5 kN/m2.

The dead load had been calculated on the following basis:

Reinforced concrete is 24 kN/m3, tile is 25 kN/m3, mortar is 24 kN/m3, filling material is 14kN/m3 and the partition is 1 kN/m2.

∢ **Seismic loads:**

There are two methods in UBC-94 CODE for determining the seismic loading, static and dynamic. In general, any structure may be designed using the dynamic method and some structures must be designed only by the dynamic method.

The static method may be used for buildings with the following characteristics:

- Structures in seismic zone 1 and 2, with standard occupancy category four, irrespective of whether they are regular or irregular.
- Regular structures under 73 m high using one of the lateral force resisting systems, except regular structures located on soil profile S4 which have natural periods greater than 0.7s.
- Irregular structures less than or equal to five stories or 20 m in height.
- Structures with flexible upper portion, supported on a rigid lower portion if the following three conditions are met.

1. Both portions, when considered individually, are regular

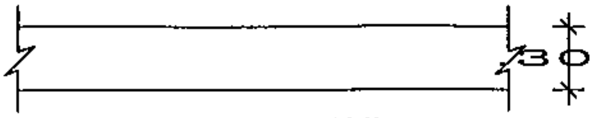


Figure (3-1(a))
Typical Cross Section
in the Modified Slab

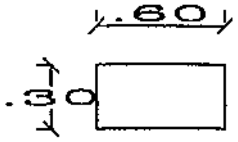


Figure (3-1(b))
Typical Cross Section
in the Ordinary beam

Figure (3-1(d))
Typical Cross Section
in the shear wall

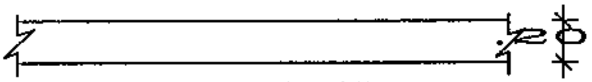


Figure (3-1(c))
Typical Cross Section
in the Ordinary Slab

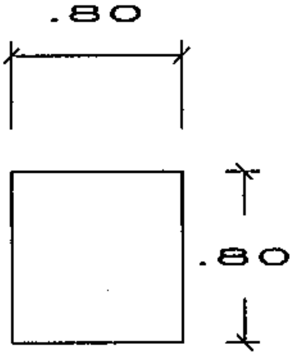


Figure (3-1(e))
Typical Cross
Section in the column

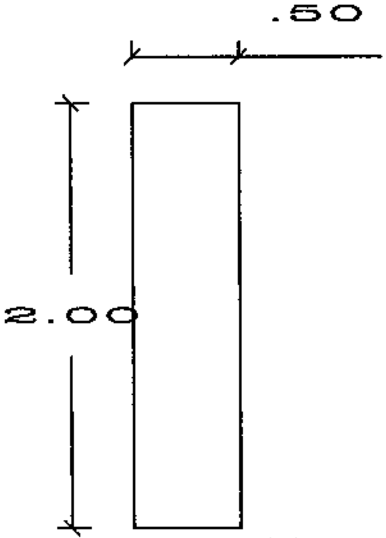


Figure (3-1(f))
Typical Cross Section
in the Special Deep beam

Figure (3-1)

2. The average story stiffness of the lower portion is at least ten times the average story stiffness of the upper portion.

3. The period of the entire structures is no more than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

All structures not meeting these requirements, including irregular buildings, must be designed using the dynamic method.

The base shear 'V' which is the total lateral inertial force imposed on the structure at its base by an earthquake, is the sum of all the inertial story shears. Rather than calculating the story shears individually and then summing them to obtain the base shear, the base shear is calculated from the total weight of the structure and then apportioned to stories, using the following formula (UBC-94).

$$V = \frac{ZICW}{R_w} \quad \text{--}(3\text{-}1)$$

This formula is used for the entire structure and cannot be used for a part of the structure.

Z : Seismic zone factor, accounts for the amount of seismic risk present in the building seismic zone.

I : Seismic importance factor depending on how critical it is that the structure survives the earthquake.

C : Coefficient accounting for the period of the vibration of the building and the supporting soil characteristics.

$$C = \frac{1.25 \times S}{T^{2/3}} \quad \text{------(3-2)}$$

T : Building period

S : Site coefficient

$$T = C_t (h_n)^{(3/4)} \quad \text{------(3-3)}$$

Ct : Coefficient depends on the type of structure

h_n : Actual height

R_w : factor accounts for the different energy absorbing characteristics of the various types of structures in cyclic load.

W : Weight which is normally the total dead load of the structure.

The base shear 'V' is distributed to the (ç) stories in accordance with equation (3-4) and equation (3-6). The Fx forces increase linearly with height above the base as figure (3-2) illustrates. Ft is an additional force that is applied to the top level. In addition to the Fx force at that level the Ft accounts for higher mode effect (UBC-94).

Ft is zero for **T** less or equal **0.7 s**

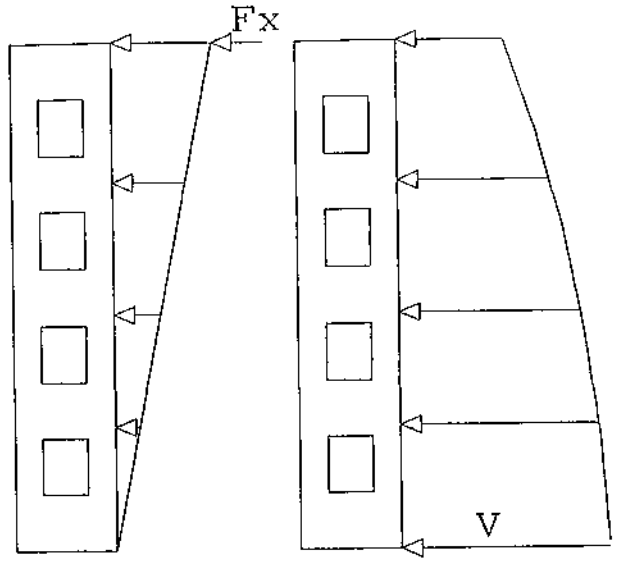
Ft=0.07*TV [Ft<0.25V] ------(3-4)

Ft=0 [T less or equal 0.7 s] ------(3-5)

$$F_x = \frac{(V - F_t) \times W_x \times h_x}{\sum_{i=1}^{n} W_i h_i} \quad \text{------(3-6)}$$

V=Ft + ΣFx ------(3-7)

R_w which used in the analysis of the models assumed the same before and after modification



(a)Loading Diagram (b)Cumulative Diagram

Figure (3-2) Distribution of Story shears

≺ **Load Combinations:**

The following load combinations were considered in the structural analysis:

1. (1.4 Dead Load +1.7Live Load)
2. (0.9 Dead Load +1.43 EQ ±ve X)
3. (0.9 Dead Load +1.43 EQ ±ve Z)
4. 0.75*(1.4 Dead Load +1.7 Live Load +1.87 EQ ±ve X)
5. 0.75*(1.4 Dead Load +1.7 Live Load +1.87 EQ ±ve Z)

3-3-2 Stage II: The Transformational Stage:

This stage deals with transferring the data generated from the STAAD Pro2000 output file to a word document and then to an Excel Sheet in order to analyze the data in an effective way.

The most important problem in this stage is the accuracy during transforming the data due to the amount of data to be analysed. To solve this problem a conditional Excel formulae had been insert in the Excel sheets to check that all the data are in proper order.

3-3-3 Stage III: Analysis of the Raw Data:

The first step after transferring raw data of the STAAD output file to an Excel sheet was to sort the data according to type of force or the direction of the translation. Each type was placed in separate file. Then each file was split into different sheets according to the case of loading. After that put together the sheets that have been produced each two opposite sheets in one sheet so as to compare it -the opposite sheets are the sheet produced from the same model before modification and after modification-.

The analysis of the raw data was the dominant problem in this study due to the amount of data needed to be analysed. The analysis was based on the computing the percentage of change of results, the percentage of change was computed by dividing the results before modification by the results after modification, using the following formula.

$$\text{Percent of Change} = \left|\left(\frac{X_j}{X_i} - 1\right)\right| \times 100\% \text{ ------------------------------------(3-8)}$$

X_i : The value of force or displacement from the original model

X_j : The value of force or displacement from the modified model

The percent of change has been categorized into the following categories:

1. No change category where there is no change in the Percentage of change =0%.

2. Slightly increase category where the percent of change of the member end force or the translation of the node range from 0% to 10% above the original value.

3. Highly increase category where the percent of change of the member end force or the translation of the node between is 10% and 50% above the original value.

4. Severely increase category where the percent of change of the member end force or the translation of the node is more than 50% above the original value.

5. Slightly decrease category where the percent of change of the member end force or the translation of the node ranges from 0% to 10% below the original value.

6. Highly decrease category where the percent of change of the member end force or the translation of the node is between 10% and 50% below the original value.

7. Severely decreases category where the percent of change of the member end force or the translation of the node is more than 50% below the original value.

Conditional Excel formula were written to compare the percentage of change for each of the categories, to assign 1 for the cases that met the condition and 0 value for the cases didn't. Then by calculating the summation of the one's in each column, which represent the number of member end forces or nodal displacement which met that condition and then by dividing that value by the total number of the ones for all categories the percentage of the element which met that condition were obtained.

During conducting the analysis it was discovered that the following special cases need separate consideration:

Column 5. Used to calculate the 10% -50% change category above the original value using the following formula =IF(AND(C>1.1,C<=1.5),1,0)

Column 6. Used to calculate more than 50% change category above the original value using the following formula =IF(AND(C>1.5),1,0)

Column 7. Used to calculate the 0% -10% change category below the original value using the following formula =IF(AND(C>0.9,C<1),1,0)

Column 8. Used to calculate the 10% -50% change category below the original value using the following formula =IF(AND(C>0.5,C<=0.9),1,0)

Column 9. Used to calculate more than 50% change category below the original value using the following formula =IF(AND(C<=0.5),1,0)

Column 10. Used to identify where A and B have Zero values (0/0) which mean that there was no change =IF(AND(A=0,B=0),1,0)

Column 11. Used to identify the Zero Value for A column (1), and non-zero value for column B (2) which gives the following (Non zero/0)=α (IF (AND(C=K, A not equal to 0) 1,0)) which is added to the more than 50% increase category.

Column 12. used to identify the 0% change category using the following Excel formula =IF(C=1,1,0)

Notes: -

- The summation of Columns 10 and 12 represent the No Change Category
- The summation of Columns 11 and 6 represent the 50% Change above the original value.

Table (3-1) Example of Comparison Tables

No.	(1) (A)	(2) (B)	(3) (C)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	33.5	23	**0.68**	0	0	0	0	1	0	0	0	0
2	-34	-23	**0.68**	0	0	0	0	1	0	0	0	0
3	16.6	6.3	**0.38**	0	0	0	0	0	1	0	0	0
4	-17	-6.3	**0.38**	0	0	0	0	0	1	0	0	0
5	10.7	6.5	**0.60**	0	0	0	0	1	0	0	0	0
6	-11	-6.5	**0.60**	0	0	0	0	1	0	0	0	0
7	28.7	24	**0.85**	0	0	0	0	1	0	0	0	0
8	-29	-24	**0.85**	0	0	0	0	1	0	0	0	0
9	28.4	23	**0.82**	0	0	0	0	1	0	0	0	0
10	-28	-23	**0.82**	0	0	0	0	1	0	0	0	0
11	6.95	2.5	**0.35**	0	0	0	0	0	1	0	0	0
12	-7	-2.5	**0.35**	0	0	0	0	0	1	0	0	0
13	7.69	4.9	**0.63**	0	0	0	0	1	0	0	0	0
14	-7.7	-4.9	**0.63**	0	0	0	0	1	0	0	0	0
15	18.8	16	**0.85**	0	0	0	0	1	0	0	0	0
16	-19	-16	**0.85**	0	0	0	0	1	0	0	0	0
17	33.7	31	**0.91**	0	0	0	1	0	0	0	0	0
18	-34	-31	**0.91**	0	0	0	1	0	0	0	0	0
19	34.1	31	**0.92**	0	0	0	1	0	0	0	0	0
20	-34	-31	**0.92**	0	0	0	1	0	0	0	0	0
21	16.8	14	**0.84**	0	0	0	0	1	0	0	0	0
22	-17	-14	**0.84**	0	0	0	0	1	0	0	0	0
23	11.5	0	**0.00**	0	0	0	0	0	1	0	0	0
24	-11	-9.4	**0.82**	0	0	0	0	1	0	0	0	0
25	0	27	**K**	0	0	0	0	0	0	0	1	0
26	-30	-27	**0.89**	0	0	0	0	1	0	0	0	0
27	29.5	25	**0.85**	0	0	0	0	1	0	0	0	0
28	-30	-25	**0.85**	0	0	0	0	1	0	0	0	0
29	0	0	**K**	0	0	0	0	0	0	1	0	0
30	-6.5	-2	**0.30**	0	0	0	0	0	1	0	0	0
31	6.95	3.1	**0.44**	0	0	0	0	0	1	0	0	0
32	-7	-3.1	**0.44**	0	0	0	0	0	1	0	0	0
33	18.4	15	**0.80**	0	0	0	0	1	0	0	0	0
34	-18	-15	**0.80**	0	0	0	0	1	0	0	0	0
35	34.3	31	**0.90**	0	0	0	1	0	0	0	0	0
36	-34	-31	**0.90**	0	0	0	1	0	0	0	0	0
37	34.5	32	**0.94**	0	0	0	1	0	0	0	0	0
38	-34	-32	**0.94**	0	0	0	1	0	0	0	0	0
39	16.8	15	**0.90**	0	0	0	1	0	0	0	0	0
40	-17	-15	**0.90**	0	0	0	1	0	0	0	0	0

Table (3-1) Example of Comparison Tables

No.	(1) (A)	(2) (B)	(3) (C)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
41	12.5	7.7	**0.61**	0	0	0	0	1	0	0	0	0
42	-13	-7.7	**0.61**	0	0	0	0	1	0	0	0	0
43	32.4	28	**0.85**	0	0	0	0	1	0	0	0	0
44	-32	-28	**0.85**	0	0	0	0	1	0	0	0	0
45	31.8	26	**0.83**	0	0	0	0	1	0	0	0	0
46	-32	-26	**0.83**	0	0	0	0	1	0	0	0	0
47	7.37	2.4	**0.32**	0	0	0	0	0	1	0	0	0
48	-7.4	-2.4	**0.32**	0	0	0	0	0	1	0	0	0
49	7.68	4.5	**0.59**	0	0	0	0	1	0	0	0	0
50	-7.7	-4.5	**0.59**	0	0	0	0	1	0	0	0	0
51	19.1	15	**0.81**	0	0	0	0	1	0	0	0	0
52	-19	-15	**0.81**	0	0	0	0	1	0	0	0	0
53	35.8	32	**0.88**	0	0	0	0	1	0	0	0	0
54	-36	-32	**0.88**	0	0	0	0	1	0	0	0	0
55	35.6	32	**0.89**	0	0	0	0	1	0	0	0	0
56	-36	-32	**0.89**	0	0	0	0	1	0	0	0	0
57	16.6	12	**0.75**	0	0	0	0	1	0	0	0	0
58	-17	-12	**0.75**	0	0	0	0	1	0	0	0	0
59	11	-2.3	**0.21**	0	0	0	0	0	1	0	0	0
60	-11	2.3	**0.21**	0	0	0	0	0	1	0	0	0
61	28.2	16	**0.57**	0	0	0	0	1	0	0	0	0
62	-28	-16	**0.57**	0	0	0	0	1	0	0	0	0
63	27.5	21	**0.75**	0	0	0	0	1	0	0	0	0
64	-27	-21	**0.75**	0	0	0	0	1	0	0	0	0
65	4.52	3.3	**0.73**	0	0	0	0	1	0	0	0	0
66	-4.5	-3.3	**0.73**	0	0	0	0	1	0	0	0	0
67	4.19	0.7	**0.16**	0	0	0	0	0	1	0	0	0
68	-4.2	-0.7	**0.16**	0	0	0	0	0	1	0	0	0
69	15.3	11	**0.70**	0	0	0	0	1	0	0	0	0
70	-15	-11	**0.70**	0	0	0	0	1	0	0	0	0
71	30.6	21	**0.68**	0	0	0	0	1	0	0	0	0
72	-31	-21	**0.68**	0	0	0	0	1	0	0	0	0
73	30	18	**0.60**	0	0	0	0	1	0	0	0	0
74	-30	-18	**0.60**	0	0	0	0	1	0	0	0	0
75	13.4	0.7	**0.06**	0	0	0	0	0	1	0	0	0
76	-13	-0.7	**0.06**	0	0	0	0	0	1	0	0	0
77	32.9	-0.1	**0.00**	0	0	0	0	0	1	0	0	0
78	-33	0.1	**0.00**	0	0	0	0	0	1	0	0	0
79	63.1	48	**0.76**	0	0	0	0	1	0	0	0	0
80	-63	-48	**0.76**	0	0	0	0	1	0	0	0	0

Table (3-1) Example of Comparison Tables

No.	(1) (A)	(2) (B)	(3) (C)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
81	59.7	35	0.58	0	0	0	0	1	0	0	0	0
82	-60	-35	0.58	0	0	0	0	1	0	0	0	0
83	21	-18	0.85	0	0	0	0	1	0	0	0	0
84	-21	18	0.85	0	0	0	0	1	0	0	0	0
85	19.8	-17	0.86	0	0	0	0	1	0	0	0	0
86	-20	17	0.86	0	0	0	0	1	0	0	0	0
87	36.2	7.6	0.21	0	0	0	0	0	1	0	0	0
88	-36	-7.6	0.21	0	0	0	0	0	1	0	0	0
89	62.4	49	0.78	0	0	0	0	1	0	0	0	0
90	-62	-49	0.78	0	0	0	0	1	0	0	0	0
91	63	54	0.86	0	0	0	0	1	0	0	0	0
92	-63	-54	0.86	0	0	0	0	1	0	0	0	0
93	33.6	9.9	0.29	0	0	0	0	0	1	0	0	0
94	-34	-9.9	0.29	0	0	0	0	0	1	0	0	0
95	-30	-41	1.37	0	1	0	0	0	0	0	0	0
96	29.7	41	1.37	0	1	0	0	0	0	0	0	0
97	-34	-42	1.21	0	1	0	0	0	0	0	0	0
98	34.5	42	1.21	0	1	0	0	0	0	0	0	0
99	-39	-45	1.16	0	1	0	0	0	0	0	0	0
100	38.8	45	1.16	0	1	0	0	0	0	0	0	0
101	-19	-22	1.19	0	1	0	0	0	0	0	0	0
102	18.8	22	1.19	0	1	0	0	0	0	0	0	0
Total				0	8	0	10	60	22	1	1	0
				0	8	1	10	60	22			1
Percent of Change				0.0%	7.8%	1.0%	9.8%	58.8%	21.6%			1.0%

3-4-Some numbers and facts:

Table (3-2) Number of elements, nodes, and members in each model

Structure	No. of Elements	No. of Joints	No. of members
SFR	3840	4396	2800
SSH	4150	4528	2720
TFR	3840	4449	2870
TSH	3980	4527	2810

Table (3-3) Number of comparisons done for each structure

Structure	No. of Comparisons for Each Case of Loading		Total No. of Comparisons for Each Structure for one Force or Displacement for all Cases of Loading	
	Type of Force	Displacement	Type of Force	Displacement
SFR	5,600	4,396	39,200	30,772
SSH	5,440	4,528	38,080	31,696
TFR	5,740	4,449	40,180	31,143
TSH	5,620	4,527	39,340	31,689

Table (3-4) Total number of comparisons done

Structure	**Total No. of Comparisons for Each Structure for the all Forces / Displacement for all Cases of Loading**		**Total No. of comparisons**
	Force	**Displacement**	
SFR	235,200	92,316	327,516
SSH	228,480	95,088	323,568
TFR	241,080	93,429	334,509
TSH	236,040	95,067	331,107
Total No. of Comparisons			**1,316,700**

Chapter Four

Discussion of Results

4-1-General

The discussion is based upon the result obtained from the Excel analysis of the outputs of the models analysed using STAAD Pro 2000.

4-2-Axial Forces

The effect of modification on the axial force was mainly on the lower part of vertical members (columns) which represent about 10%- 15% of the total number of element in the models. The effect on the other elements was very limited. Most of the elements which show increases above the original value in the severe increase category were columns.

Also the effect on buildings with shear core wasn't different from building without shear core since the percentages is almost the same.

The main cause for this increase was due to additional loads due to the girder and the additional slab thickness.

Tables (4-1), (4-2), (4-3), (4-4), (4-5), (4-6) and Table (4-7) show the percent of change in each model for each loading and case of loading.

4-3-Moment in Y-Direction

The effect of the girder on the moment in Y-direction can be divided into two parts:

The first part: effect on the beams of the third and tenth slab was sever with a percent of change above the original value of more than 50%, due to additional load of the girders and the additional slab thickness.

The second part: the effect on the vertical elements (columns) which showed a great reduction below the original value due to the reduction on the lateral displacement of the nodes.

The effect on building with shear core was more effective -through reducing the amount of lateral displacement- than buildings without shear core. This effect was due to more effective cantilever action which is produced by the core.

Tables (4-8), (4-9), (4-10), (4-11), (4-12), (4-13) and Table (4-14) show the percent of change in each model for each loading and case of loading.

4-4-Moment in Z-Direction

The effect on moment in Z-direction is almost the same as the effect on moment in Y-direction.

Tables (4-15), (4-16), (4-17), (4-18), (4-19), (4-20) and Table (4-21) show the percent of change in each model for each loading and case of loading.

4-5-Shear in Z-Direction

It is obvious that the overall effect of the modification on the shear in the Z-direction wasn't good, due to the significant effect of the additional load of the girders and the additional slab thickness, which affected mainly the beam for the tenth and third slabs, which represent more than 15% of the element of the study.

The reduction on the lateral displacement also affects the shear in the Z-direction in an acceptable manner for the vertical elements.

Tables (4-22), (4-23), (4-24), (4-25), (4-26), (4-27) and Table (4-28) show the percent of change in each model for each loading and case of loading.

4-6-Shear in Y-Direction

The effect of the modification on the shear on Y-direction is almost the same as the shear in Z-Direction due to the same reasons mentioned above.

Tables (4-29), (4-30), (4-31), (4-32), (4-33), (4-34) and Table (4-35) shows the percent of change in each model for each loading and case of loading.

4-7-Torsion

The effect on torsion was acceptable since the percent of element which shows increases above the original value in most of the models is not more than 41%, while 55% of the elements show reduction after the modification was introduced.

Tables (4-36), (4-37), (4-38), (4-39), (4-40), (4-41) and Table (4-42) shows the percent of change in each model for each loading and case of loading.

4-8-Translation on the X-Direction

The effect of the interaction of frame or shear wall-frame buildings through the large girder was very obvious since more than 95 % of elements showed a reduction in the lateral displacement. Most of the elements had percent of reduction of more than 50 %.

This effect is due to the ability of the girder and the thick slab to mobilize the longitudinal (axial) stiffness of the columns and the shear wall in resisting the lateral loads.

The effect was more obvious in buildings with shear core than the frame buildings due to the large longitudinal stiffness of the shear walls.

Tables (4-43), (4-44), (4-45), (4-46), (4-47), (4-48) and Table (4-49) show the percent of change in each model for each loading and case of loading.

4-9- Translation on the Y-Direction

The effect of the modification on the translation in the Y-direction wasn't very satisfactory. Since the average percent of reduction after the modification is 48%, and the average percent of nodes which show increase above the original value is about 50%, while there was no change in 2% of the element.

Tables (4-50), (4-51), (4-52), (4-53), (4-54), (4-55) and Table (4-56) show the percent of change in each model for each loading and case of loading.

4-10- Translation on the Z-Direction

The effect of the modification on the translation in the Y-direction is the same as the effect on the translation in the X-direction for similar reasons that affect there.

Tables (4-57), (4-58), (4-59), (4-60), (4-61), (4-62) and Table (4-63) show the percent of change in each model for each loading and case of loading.

Table (4-1)
Axial force
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	5.8%	7.4%	38.8%	9.8%	30.2%	4.1%	3.8%
S SH	14.5%	11.0%	29.9%	16.9%	17.4%	7.9%	2.5%
T FR	5.3%	11.5%	30.0%	18.4%	24.8%	9.4%	0.7%
T SH	8.6%	15.1%	22.6%	15.9%	27.4%	9.8%	0.6%

Table (4-2)
Axial force
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	5.7%	8.6%	37.7%	10.7%	29.3%	6.0%	2.0%
S SH	13.7%	11.1%	29.9%	15.8%	17.5%	7.9%	4.2%
T FR	4.9%	9.9%	31.9%	17.9%	24.9%	9.8%	0.8%
T SH	6.1%	12.3%	32.3%	9.8%	28.1%	10.6%	0.7%

Table (4-3)
Axial force
(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	4.0%	12.3%	35.7%	7.4%	30.6%	9.4%	0.6%
S SH	11.0%	14.1%	31.5%	16.1%	17.9%	8.2%	1.1%
T FR	5.3%	14.5%	33.5%	15.2%	12.4%	18.9%	0.3%
T SH	5.6%	17.0%	33.2%	6.6%	16.8%	20.7%	0.1%

Table (4-4)
Axial force
(.9DL+1.43 $EQ^{+}VE$ X) and (.9DL+1.43 $EQ^{-}VE$ X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	8.3%	12.2%	30.9%	13.3%	23.0%	11.5%	0.9%
S SH	11.6%	13.8%	31.6%	15.6%	17.8%	8.2%	1.5%
T FR	5.6%	14.3%	31.8%	13.8%	12.9%	20.8%	0.8%
T SH	6.7%	17.7%	37.7%	7.7%	18.5%	11.3%	0.6%

Table (4-5)
Axial force
(.9DL+1.43EQ+VE Z) and (.9DL+1.43EQ-VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	8.3%	12.2%	30.9%	13.3%	23.0%	11.5%	0.9%
S SH	11.6%	13.8%	31.6%	15.6%	17.8%	8.2%	1.5%
T FR	5.6%	14.3%	31.8%	13.8%	22.9%	10.8%	0.8%
T SH	6.7%	17.7%	37.7%	7.7%	18.5%	11.3%	0.6%

Table (4-6)
Axial force
0.75(1.4DL+1.7LL+1.87EQ^{+}VE X) and 0.75(1.4DL+1.7LL+1.87EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	6.3%	13.4%	39.0%	16.7%	15.0%	9.1%	0.6%
S SH	5.9%	16.0%	41.6%	10.6%	17.0%	8.6%	0.2%
T FR	5.3%	14.6%	33.4%	5.2%	22.2%	19.0%	0.3%
T SH	5.6%	16.9%	33.2%	6.4%	26.9%	10.8%	0.1%

Table (4-7)

Axial force

0.75(1.4DL+1.7LL+1.87 EQ +VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ -VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	6.3%	13.4%	39.0%	16.7%	15.0%	9.1%	0.6%
S SH	5.9%	16.0%	41.6%	10.6%	17.0%	8.6%	0.2%
T FR	5.3%	14.6%	33.4%	20.2%	17.2%	9.0%	0.3%
T SH	5.6%	16.9%	43.2%	6.4%	16.9%	10.8%	0.1%

Table (4-8)
Moment-Y Direction
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.5%	4.7%	33.3%	5.5%	13.4%	5.0%	35.6%
S SH	7.5%	10.1%	33.2%	6.0%	7.3%	6.6%	29.4%
T FR	2.4%	7.5%	30.0%	3.3%	16.3%	18.8%	21.7%
T SH	6.1%	14.2%	38.3%	6.6%	16.6%	10.9%	7.3%

Table (4-9)
Moment-Y Direction
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.6%	4.8%	37.7%	5.1%	3.1%	3.4%	43.4%
S SH	6.1%	8.8%	30.4%	5.4%	6.0%	5.8%	37.4%
T FR	2.1%	6.9%	55.3%	1.0%	6.5%	10.2%	18.0%
T SH	4.3%	12.4%	44.8%	3.3%	9.9%	10.4%	14.9%

Table (4-10)

Moment-Y Direction

(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.1%	6.7%	33.6%	16.0%	10.1%	8.9%	22.6%
S SH	9.3%	13.0%	34.7%	7.5%	13.2%	6.0%	16.3%
T FR	2.4%	10.2%	39.5%	1.9%	19.2%	20.8%	6.0%
T SH	5.6%	15.5%	43.5%	5.3%	15.6%	11.6%	2.9%

Table (4-11)

Moment-Y Direction

(.9DL+1.43 $EQ^{+}VE$ X) and (.9DL+1.43 $EQ^{-}VE$ X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.1%	8.6%	35.2%	13.1%	7.4%	20.5%	13.1%
S SH	8.4%	12.5%	33.8%	6.9%	11.7%	5.7%	21.0%
T FR	1.3%	7.1%	38.0%	13.2%	8.0%	19.1%	13.4%
T SH	5.2%	13.7%	38.3%	6.3%	17.1%	11.7%	7.7%

Table (4-12)

Moment-Y Direction

(.9DL+1.43EQ^{+}VE Z) and (.9DL+1.43EQ^{-}VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.1%	8.6%	35.2%	3.1%	17.4%	20.5%	13.1%
S SH	8.4%	12.5%	33.8%	6.9%	11.7%	5.7%	21.0%
T FR	1.3%	7.1%	38.0%	3.2%	18.0%	19.1%	13.4%
T SH	5.2%	13.7%	38.3%	6.3%	17.1%	11.7%	7.7%

Table (4-13)

Moment-Y Direction

0.75(1.4DL+1.7LL+1.87EQ^{+}VE X) and 0.75(1.4DL+1.7LL+1.87EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.3%	9.6%	34.1%	4.1%	18.3%	19.3%	12.2%
S SH	4.6%	12.8%	37.4%	6.4%	12.1%	19.4%	7.2%
T FR	2.2%	10.4%	38.8%	1.6%	18.8%	21.0%	7.2%
T SH	5.4%	15.7%	33.0%	5.2%	15.4%	21.6%	3.8%

Table (4-14)
Moment-Y Direction
0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.3%	9.6%	44.1%	4.1%	8.3%	19.3%	12.2%
S SH	4.6%	12.8%	37.4%	6.4%	12.1%	19.4%	7.2%
T FR	2.2%	10.4%	48.8%	1.6%	8.8%	21.0%	7.2%
T SH	5.4%	15.7%	33.0%	5.2%	15.4%	21.6%	3.8%

Table (4-17)
Moment -Z Direction
(1.4Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	31.4%	7.6%	9.0%	33.9%	8.0%	9.6%	0.6%
S SH	29.2%	8.0%	9.4%	37.0%	7.6%	8.1%	0.7%
T FR	17.8%	8.7%	18.0%	23.7%	19.9%	11.7%	0.2%
T SH	27.4%	9.7%	14.2%	28.5%	11.4%	8.3%	0.5%

Table (4-18)
Moment -Z Direction
(.9DL+1.43 EQ$^+$VE X) and (.9DL+1.43 EQ$^-$VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	23.6%	8.1%	12.1%	26.2%	17.4%	11.5%	1.1%
S SH	29.5%	8.3%	9.1%	37.0%	7.1%	8.1%	0.9%
T FR	20.4%	10.7%	16.8%	22.7%	17.5%	11.0%	0.9%
T SH	24.1%	12.5%	14.0%	27.2%	12.6%	8.4%	1.1%

Table (4-19)
Moment -Z Direction
(.9DL+1.43EQ^{+}VE Z) and (.9DL+1.43EQ^{-}VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	23.6%	8.1%	12.1%	26.2%	17.4%	11.5%	1.1%
S SH	29.5%	8.3%	9.1%	37.0%	7.1%	8.1%	0.9%
T FR	20.4%	10.7%	16.8%	22.7%	17.5%	11.0%	0.9%
T SH	24.1%	12.5%	14.0%	27.2%	12.6%	8.4%	1.1%

Table (4-20)
Moment -Z Direction
0.75(1.4DL+1.7LL+1.87EQ^{+}VE X) and 0.75(1.4DL+1.7LL+1.87EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	19.3%	12.4%	14.7%	24.4%	19.2%	9.0%	1.0%
S SH	29.3%	7.4%	9.7%	34.4%	9.2%	9.0%	1.0%
T FR	17.7%	8.6%	18.0%	23.6%	20.0%	11.7%	0.4%
T SH	27.2%	9.8%	14.2%	28.3%	11.4%	8.3%	0.8%

Table (4-21)
Moment -Z Direction
0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	24.3%	8.5%	12.8%	27.8%	14.5%	11.4%	0.8%
S SH	29.3%	7.4%	9.7%	34.4%	9.2%	9.0%	1.0%
T FR	17.7%	8.6%	18.0%	23.6%	20.0%	11.7%	0.4%
T SH	27.2%	9.8%	14.2%	28.3%	11.4%	8.3%	0.8%

Table (4-22)
Shear Z-Direction
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	1.9%	4.9%	34.9%	6.0%	15.4%	4.3%	32.7%
S SH	8.0%	11.9%	22.4%	6.8%	8.3%	17.6%	25.0%
T FR	2.4%	7.5%	30.6%	3.4%	26.0%	18.4%	11.7%
T SH	6.4%	13.9%	28.5%	16.8%	16.7%	11.1%	6.5%

Table (4-23)
Shear Z-Direction
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	1.9%	4.0%	19.7%	5.4%	13.7%	14.6%	40.7%
S SH	5.9%	9.2%	11.7%	15.8%	6.4%	17.5%	33.5%
T FR	1.3%	11.5%	25.7%	1.0%	14.5%	18.5%	27.5%
T SH	4.2%	13.4%	22.7%	3.4%	21.4%	20.3%	14.7%

Table (4-24)

Shear Z-Direction

(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	1.1%	7.0%	35.0%	6.0%	20.6%	7.4%	22.9%
S SH	8.6%	14.6%	24.6%	8.0%	11.8%	16.6%	15.9%
T FR	2.4%	13.8%	39.9%	1.9%	16.1%	20.6%	5.3%
T SH	5.3%	15.1%	32.5%	6.8%	15.5%	11.4%	13.5%

Table (4-25)

Shear Z-Direction

(.9DL+1.43 EQ$^+$VE X) and (.9DL+1.43 EQ$^-$VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	3.2%	8.9%	36.9%	3.2%	17.8%	13.5%	16.5%
S SH	7.9%	13.6%	24.4%	7.2%	11.4%	17.2%	18.1%
T FR	1.7%	7.1%	29.8%	23.4%	7.2%	18.2%	12.6%
T SH	5.8%	12.6%	19.1%	25.9%	17.0%	11.7%	7.9%

Table (4-28)

Shear Z-Direction

0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	2.4%	9.6%	35.9%	4.3%	18.4%	18.9%	10.5%
S SH	4.7%	14.2%	26.6%	6.3%	22.6%	18.9%	6.7%
T FR	2.2%	13.5%	39.8%	1.8%	15.9%	20.3%	6.6%
T SH	5.1%	14.7%	22.7%	6.4%	25.4%	21.4%	4.3%

Table (4-29)
Shear Y-Direction
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	37.7%	5.6%	7.4%	35.0%	8.7%	3.0%	2.6%
S SH	37.9%	5.4%	6.8%	36.2%	8.8%	2.3%	2.6%
T FR	26.9%	15.2%	12.1%	28.9%	10.6%	5.5%	0.8%
T SH	35.1%	7.3%	9.8%	35.2%	9.3%	2.3%	1.0%

Table (4-30)
Shear Y-Direction
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	37.3%	5.6%	7.4%	33.5%	8.7%	3.0%	4.5%
S SH	36.8%	5.4%	6.8%	35.1%	8.8%	2.4%	4.7%
T FR	26.0%	9.3%	13.5%	27.4%	16.7%	3.8%	3.3%
T SH	33.1%	6.7%	11.5%	32.8%	10.0%	3.0%	2.9%

Table (4-31)
Shear Y-Direction
(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	36.3%	6.3%	8.0%	35.0%	9.7%	3.0%	1.7%
S SH	34.3%	6.2%	8.5%	39.7%	9.2%	1.9%	0.2%
T FR	25.1%	14.4%	14.7%	30.1%	10.5%	5.3%	0.0%
T SH	36.8%	7.9%	11.1%	32.8%	9.0%	2.2%	0.2%

Table (4-32)
Shear Y-Direction
(.9DL+1.43 EQ$^+$VE X) and (.9DL+1.43 EQ$^-$VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	26.2%	15.9%	10.6%	27.4%	15.4%	4.2%	0.3%
S SH	35.0%	6.0%	8.3%	39.5%	9.0%	1.9%	0.3%
T FR	26.9%	16.1%	12.9%	26.7%	11.9%	5.2%	0.3%
T SH	35.6%	7.8%	10.9%	32.3%	10.3%	2.5%	0.6%

Table (4-33)
Shear Y-Direction
(.9DL+1.43EQ^{+}VE Z) and (.9DL+1.43EQ^{-}VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	26.2%	15.9%	10.6%	27.4%	15.4%	4.2%	0.3%
S SH	35.0%	6.0%	8.3%	39.5%	9.0%	1.9%	0.3%
T FR	26.9%	16.1%	12.9%	26.7%	11.9%	5.2%	0.3%
T SH	35.6%	7.8%	10.9%	32.3%	10.3%	2.5%	0.6%

Table (4-34)
Shear Y-Direction
0.75(1.4DL+1.7LL+1.87EQ^{+}VE X) and 0.75(1.4DL+1.7LL^{-}1.87EQ+VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	29.1%	14.0%	9.9%	32.1%	8.8%	5.9%	0.3%
S SH	36.6%	6.5%	8.2%	37.4%	8.6%	2.1%	0.7%
T FR	25.1%	14.4%	14.6%	30.0%	10.5%	5.3%	0.1%
T SH	36.7%	8.0%	11.1%	32.7%	9.0%	2.2%	0.4%

Table (4-35)
Shear Y-Direction
0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	29.1%	14.0%	9.9%	32.1%	8.8%	5.9%	0.3%
S SH	36.6%	6.5%	8.2%	37.4%	8.6%	2.1%	0.7%
T FR	25.1%	14.4%	14.6%	30.0%	10.5%	5.3%	0.1%
T SH	36.7%	8.0%	11.1%	32.7%	9.0%	2.2%	0.4%

Table (4-36)
Torsion
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	18.9%	6.6%	20.0%	24.8%	10.0%	1.7%	18.0%
S SH	18.1%	11.9%	16.5%	25.1%	11.0%	6.2%	11.3%
T FR	13.2%	15.7%	20.2%	15.2%	24.6%	9.6%	1.5%
T SH	19.4%	19.6%	16.8%	19.0%	15.9%	7.2%	2.0%

Table (4-37)
Torsion
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	16.9%	7.1%	20.0%	23.1%	10.6%	1.4%	20.9%
S SH	15.9%	10.9%	16.4%	23.4%	9.9%	6.9%	16.6%
T FR	13.0%	7.6%	21.8%	14.0%	22.7%	8.1%	12.8%
T SH	18.7%	11.0%	19.8%	20.5%	11.7%	5.8%	12.5%

Table (4-38)
Torsion
(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	10.6%	8.3%	21.6%	18.6%	22.6%	8.9%	9.5%
S SH	9.6%	16.9%	17.0%	19.3%	24.7%	7.9%	4.6%
T FR	14.5%	12.6%	16.3%	18.4%	28.4%	9.5%	0.2%
T SH	19.6%	21.3%	17.6%	16.7%	16.9%	7.0%	0.9%

Table (4-39)
Torsion
(.9DL+1.43 EQ^{+}VE X) and (.9DL+1.43 EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	12.2%	5.6%	24.8%	15.8%	30.1%	10.3%	1.1%
S SH	9.6%	16.4%	16.5%	20.5%	22.7%	8.0%	6.3%
T FR	9.7%	14.5%	19.6%	12.7%	30.8%	11.4%	1.2%
T SH	14.6%	23.5%	18.0%	13.0%	19.9%	9.8%	1.2%

Table (4-40)

Torsion

(.9DL+1.43EQ^{+}VE Z) and (.9DL+1.43EQ^{-}VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	12.2%	5.6%	24.8%	15.8%	30.1%	10.3%	1.1%
S SH	9.6%	16.4%	16.5%	20.5%	22.7%	8.0%	6.3%
T FR	9.7%	14.5%	19.6%	12.7%	30.8%	11.4%	1.2%
T SH	14.6%	23.5%	18.0%	13.0%	19.9%	9.8%	1.2%

Table (4-41)

Torsion

0.75(1.4DL+1.7LL+1.87EQ^{+}VE X) and 0.75(1.4DL+1.7LL+1.87EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	12.4%	5.9%	25.2%	17.4%	28.4%	9.2%	1.6%
S SH	16.7%	12.6%	16.8%	22.3%	22.1%	7.8%	1.7%
T FR	14.3%	12.8%	16.3%	18.3%	28.5%	9.5%	0.3%
T SH	19.8%	20.9%	17.7%	16.4%	17.0%	7.0%	1.2%

Table (4-42)

Torsion

0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	12.4%	5.9%	25.2%	17.4%	28.4%	9.2%	1.6%
S SH	16.7%	12.6%	16.8%	22.3%	22.1%	7.8%	1.7%
T FR	14.3%	12.8%	16.3%	18.3%	28.5%	9.5%	0.3%
T SH	19.8%	20.9%	17.7%	16.4%	17.0%	7.0%	1.2%

Table (4-43)
X-Translation
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.1%	0.2%	1.7%	0.2%	70.5%	3.5%	23.8%
S SH	1.3%	4.0%	3.0%	4.7%	75.1%	1.4%	10.6%
T FR	0.0%	0.0%	0.9%	9.9%	89.2%	0.0%	0.0%
T SH	2.4%	9.3%	1.0%	44.8%	41.8%	0.0%	0.7%

Table (4-44)
X-Translation
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.0%	0.1%	8.9%	0.2%	58.2%	1.3%	31.2%
S SH	0.3%	4.3%	2.9%	4.0%	72.9%	1.7%	14.0%
T FR	0.0%	1.8%	8.9%	0.1%	25.9%	17.5%	45.7%
T SH	0.7%	11.2%	9.4%	0.3%	32.6%	15.3%	30.4%

Table (4-45)
X-Translation
(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.1%	0.1%	3.7%	0.1%	56.7%	6.0%	33.3%
S SH	1.8%	3.8%	3.1%	8.6%	75.9%	1.2%	5.5%
T FR	0.0%	0.0%	0.0%	9.9%	89.2%	0.0%	0.9%
T SH	4.2%	9.9%	0.0%	47.2%	32.1%	5.0%	1.7%

Table (4-46)
X-Translation
(.9DL+1.43 $EQ^{+}VE$ X) and (.9DL+1.43 $EQ^{-}VE$ X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.0%	0.0%	0.0%	10.0%	89.1%	0.0%	0.8%
S SH	1.9%	3.8%	3.1%	7.3%	75.6%	1.3%	7.1%
T FR	0.0%	0.0%	0.0%	9.9%	89.2%	0.0%	0.9%
T SH	2.2%	9.3%	0.0%	45.2%	41.8%	0.0%	1.6%

Table (4-49)

X-Translation

0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.2%	1.5%	5.5%	0.3%	49.6%	14.1%	28.8%
S SH	3.4%	3.6%	4.6%	6.7%	72.4%	7.0%	2.3%
T FR	0.0%	0.0%	0.0%	9.9%	89.2%	0.0%	0.9%
T SH	9.0%	9.9%	0.0%	47.1%	32.1%	0.0%	1.9%

Table (4-50)
Y-Translation
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	27.2%	1.1%	0.0%	54.0%	16.7%	0.2%	0.8%
S SH	26.6%	1.1%	0.0%	53.8%	16.4%	0.2%	1.9%
T FR	25.8%	3.9%	1.1%	50.6%	17.6%	0.7%	0.4%
T SH	27.8%	4.2%	1.3%	45.7%	19.4%	0.8%	0.8%

Table (4-51)
Y-Translation
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	27.0%	1.1%	0.0%	54.0%	16.7%	0.2%	1.0%
S SH	26.3%	1.1%	0.0%	53.6%	16.4%	0.2%	2.3%
T FR	20.7%	5.6%	0.9%	52.0%	18.3%	0.9%	1.6%
T SH	20.7%	4.3%	0.1%	55.9%	16.2%	0.6%	2.2%

Table (4-52)
Y-Translation
(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	31.0%	18.7%	0.9%	23.8%	14.4%	10.0%	1.1%
S SH	30.6%	18.1%	0.9%	25.5%	13.7%	10.0%	1.3%
T FR	37.9%	23.6%	1.8%	13.4%	12.3%	10.0%	1.1%
T SH	39.5%	24.7%	2.0%	8.5%	13.6%	10.3%	1.4%

Table (4-53)
Y-Translation
(.9DL+1.43 EQ$^+$VE X) and (.9DL+1.43 EQ$^-$VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	28.7%	28.0%	3.3%	18.9%	9.6%	10.0%	1.4%
S SH	32.1%	16.3%	0.9%	25.1%	14.2%	10.0%	1.4%
T FR	32.4%	36.7%	3.5%	7.9%	8.5%	10.0%	0.9%
T SH	27.3%	40.8%	4.2%	5.6%	9.7%	11.1%	1.4%

Table (4-54)

Y-Translation

(.9DL+1.43$EQ^{+}VE$ Z) and (.9DL+1.43$EQ^{-}VE$ Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	28.7%	28.0%	3.3%	18.9%	9.6%	10.0%	1.4%
S SH	32.1%	16.3%	0.9%	25.1%	14.2%	10.0%	1.4%
T FR	32.4%	36.7%	3.5%	7.9%	8.5%	10.0%	0.9%
T SH	27.3%	40.8%	4.2%	5.6%	9.7%	11.1%	1.4%

Table (4-55)

Y-Translation

0.75(1.4DL+1.7LL+1.87$EQ^{+}VE$ X) and 0.75(1.4DL+1.7LL+1.87$EQ^{-}VE$ X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	31.3%	19.3%	1.1%	24.1%	13.4%	10.0%	0.9%
S SH	25.4%	19.6%	1.5%	27.3%	14.5%	10.3%	1.2%
T FR	37.9%	23.6%	1.8%	13.4%	12.3%	10.0%	1.0%
T SH	39.5%	24.8%	2.0%	8.5%	13.6%	10.3%	1.3%

Table (4-56)

Y-Translation

0.75(1.4DL+1.7LL+1.87 EQ $^{+}$VE Z) and 0.75(1.4DL+1.7LL+1.87 EQ $^{-}$VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	31.3%	19.3%	1.1%	24.1%	13.4%	0.0%	10.9%
S SH	25.4%	19.6%	1.5%	27.3%	14.5%	0.3%	11.2%
T FR	37.9%	23.6%	1.8%	13.4%	12.3%	0.0%	11.0%
T SH	39.5%	24.8%	2.0%	8.5%	13.6%	0.3%	11.3%

Table (4-57)
Z -Translation
Dead Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.0%	0.2%	0.7%	0.2%	50.4%	3.5%	45.0%
S SH	1.3%	0.7%	0.9%	5.7%	85.8%	0.7%	5.0%
T FR	0.7%	12.5%	8.0%	0.8%	72.7%	3.6%	1.7%
T SH	15.7%	14.2%	10.6%	15.1%	38.1%	5.7%	0.7%

Table (4-58)
Z -Translation
Live Load

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.0%	0.1%	2.0%	0.2%	38.2%	1.3%	58.1%
S SH	1.0%	0.5%	0.8%	1.9%	86.9%	0.8%	8.1%
T FR	0.0%	0.0%	0.0%	9.9%	89.2%	0.0%	0.9%
T SH	11.5%	8.6%	0.0%	76.3%	2.4%	0.0%	1.3%

Table (4-59)
Z -Translation
(1.4*Dead Load+1.7Live Load)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.1%	0.1%	1.9%	0.1%	56.7%	6.1%	35.0%
S SH	1.5%	1.3%	0.9%	7.1%	86.3%	0.6%	2.4%
T FR	0.0%	0.0%	0.0%	9.9%	89.2%	0.0%	0.9%
T SH	9.7%	12.8%	0.4%	25.2%	20.6%	30.0%	1.3%

Table (4-60)
Z -Translation
(.9DL+1.43 EQ^{+}VE X) and (.9DL+1.43 EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.1%	1.8%	4.0%	0.2%	45.8%	12.3%	35.8%
S SH	1.3%	1.1%	1.1%	6.8%	86.4%	0.6%	2.8%
T FR	0.8%	12.0%	5.4%	1.3%	73.4%	3.0%	3.9%
T SH	5.4%	20.9%	0.3%	6.0%	41.3%	24.5%	1.6%

Table (4-61)
Z -Translation
(.9DL+1.43EQ^{+}VE Z) and (.9DL+1.43EQ^{-}VE Z)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.0%	0.0%	0.0%	10.0%	89.1%	0.0%	0.8%
S SH	1.9%	3.8%	3.1%	7.3%	75.6%	1.3%	7.1%
T FR	0.8%	12.0%	5.4%	1.3%	73.4%	3.0%	3.9%
T SH	5.4%	10.9%	10.3%	6.0%	51.3%	14.5%	1.6%

Z -Translation
Table (4-62)
0.75(1.4DL+1.7LL+1.87EQ^{+}VE X) and 0.75(1.4DL+1.7LL+1.87EQ^{-}VE X)

Type of Building	Percent of Change						
	Increase Above the Original Value			Decrease Below the Original Value			No Change
	0-10%	10% -50%	More Than 50%	0-10%	10% -50%	More Than 50%	
S FR	0.2%	1.5%	5.5%	0.3%	49.6%	14.1%	28.8%
S SH	3.4%	3.6%	4.6%	6.7%	72.4%	7.0%	2.3%
T FR	0.0%	0.0%	0.0%	9.9%	89.2%	0.0%	0.9%
T SH	9.7%	12.8%	0.4%	25.1%	50.6%	0.0%	1.4%

التأثير الكلي لتحزيم المباني العالية بواسطة جسر عميق و بلاطة ذات جساءة عالية

إعداد

أكرم علي محمد القهيوي

إشراف

الأستاذ الدكتور سميح قاقيش

ملخص

جرى في هذا البحث دراسة تفصيلية لأثر تحزيم المباني العالية بواسطة جسر عميق و بلاطة ذات جساءة عالية ، حيث تمت إضافة هذا التعديل على المستوى الثالث و الأخير (العاشر).

مرت هذه الدراسة خلال ثلاث مراحل: المرحلة الأولى إعداد النماذج بواسطة برنامج (STAAD PRO 2000) حيث تم تصميم ثمانية نماذج

المرحلة الثانية كانت استخراج النتائج من ملف المخرجات من برنامج STAAD PRO 2000) و نقلها إلى برنامج (Excel).

المرحلة الثالثة و هي اهم مرحلة حيث تم فيها تحليل النتائج المستخرجة بواسطة برنامج (Excel).

حيث تم دراسة التغيير الحاصل للقوة و الإزاحات التالية: القوة المحورية، قوة القص باتجاه المحور السيني .و المحور الزيني ، عزم اللي.عزم الانحناء باتجاه المحور السيني و المحور الزيني ، الإزاحة الجانبية باتجاه المحور السيني ،المحور الصادي و المحور الزيني.

من أهم النتائج الملاحظة في هذا البحث هي الأثر الإيجابي على الإزاحة، الجانبية حيث قلة هذه الإزاحة بشكل كبير لمعظم أجزاء الهياكل الأربعة التي تمت الدراسة عليها.