# ASSESEMENT OF ADJACENT RC BUILDING IN ADDIS ABABA AGAINST POUNDING

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A thesis submitted to
The Department of Civil and Environmental Engineering.

Presented in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures)

**Addis Ababa University**
**Addis Ababa, Ethiopia**

**July, 2013**

**ADDIS ABABA UNIVERSITY**

**SCHOOL OF GRADUATE STUDIES**

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## Abstract

**Assessments of Adjacent RC Building in Addis Ababa against Pounding**

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Addis Ababa University, 2013

The highly congested building system in many metropolitan cities constitutes a major concern for seismic pounding damage as observed in past earthquakes in different parts of the world. Several buildings located in Ethiopian cities, especially in Addis Ababa, are constructed very closely without adequate seismic separation. This may cause considerable damage or even lead to a structure's total collapse during earthquake excitation. The aim of this research paper is to identify whether the existing separation distance between adjacent buildings is sufficient or not to avoid pounding. For this purpose, three pairs of adjacent buildings are taken as a case study from different part of Addis Ababa to study their vulnerability against pounding effect. In the analysis, all structures have been modeled by using finite element software programs, SAP2000 and ETABS, with response spectrum function. The analysis has been conducted for elastic structures under seismic zone-2. The results of the analysis shows that in the two cases (case-II and case-III) the separation distances in between are insufficient to withstand the pounding effect, according to EBCS-8, 1995. Due to this, a new RC wall has been provided to increase the stiffness of the building and to reduce the lateral displacement or deformation of the entire structure. This mitigation method shows the significant change in lateral displacement and the period of the buildings.

## Acknowledgement

It is my pleasure to express my greatest thanks to my advisor, Dr. .-Ing. Adil Zekaria , for his incredible level of support, advice, supervision, and encouragement at every step in the process. His door was always open to provide me with necessary publications relevant to this work.

I also extend my gratitude towards my friends who in one way or the other helped me in accomplishing this study.

Last but not least I would like to thank my family who are always standing on my side and pray for successful completion of the work.

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## List of Symbols and Abbreviations

$a_i$ = The coefficient as function of aspect ratio Ly /Lx and support conditions

b = unit width of the slab

$c_c$ = Dashpot constant of the connector modeling

$c_p$ = Dashpot constant of the pounding modeling

$C_R$ = Coefficient of restitution

d = depth for serviceability requirement

de = Displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum

ds = Displacement of a point of the structural system induced by the design seismic action.

$E_{cm}$ = Secant modulus of elasticity of concrete

Es = Elastic modulus of reinforcement steel

$f_{ck}$ = Characteristic strength compressive strength of concrete

$f_{cd}$ = Design concrete compressive strength of concrete

$f_{ctd}$ = Design tensile strength of concrete

$f_{ctk}$ = Characteristic tensile strength of concrete

$F_D$ = design load

$F_k$ = Characteristic load

$f_{yd}$ = Design yield strength of steel

$f_{yk}$ = Characteristic yield strength of steel

$F_p$ = Forces of pounding

g = Acceleration due to gravity

$g_d$ = The uniformly distributed design permanent load

$G_k$ = Characteristic value of a permanent action

$g_p$ = Static separation between structures

K = Stiffness

$k_c$ = Spring constant of the connector modelling

$k_f$ = Final stiffness

$k_i$ = Approaching stiffness

$k_p$ = Spring constant of the pounding modeling

$l_e$ = effective length

$L_x$ = the shortest length of the span

$L_y$ = the longest length of the span

M = Mass

$M_{dmax}$ = The maximum design moment of the slab

$m_i$ = The design moment per unit width at the point of reference

$M_f$ = Span moment in two-way slab

$M_{xf}$ = Span moment in the shorter direction of a panel

$M_{ys}$ =Span moment in the longer direction of a panel

$P_d$ = Design load

$Q_k$ = Characteristic value of a single variable action

$q_d$ = The uniformly distributed design live load

$S_d(T)$ =Ordinate of the design spectrum

$V_x$ = the load transferred along the shorter direction

$V_y$ = the load transferred along the longer direction

W = Weight

αi = Moment coefficient

$\alpha_{xf}$ = Moment coefficient of span along the short direction

$\alpha_{xs}$ = Moment coefficient of support along the short direction

$\alpha_{yf}$ = Moment coefficient of span along the longer direction

$\alpha_{ys}$ = Moment coefficient of support along the longer direction

βa = Coefficient for effective depth

$\beta_{vx}$ = The load transfer coefficient along the shorter direction

$\beta_{vy}$ = The load transfer coefficient along the longer direction

$\gamma_c$ = unit weight of concrete

$\gamma_d$ = displacement Behaviour factor

$x_g$ = Ground acceleration

μ = Ductility demand

i = Damping ratio

$\gamma_c$ = Partial safety factor for concrete

$\gamma_s$ = Partial safety factor for steel

$\gamma_f$ = Partial safety factor for loads

DDC = Double Difference Combination

DOF = Degree Of Freedom

MDOF = Multiple Degree Of Freedom

MRF = Moment Resisting Frame

PGA = Peak Ground Acceleration

PRD = Pounding Reduction Device

SDOF = Single Degree Of Freedom

SPD = Spectral Difference Method

SRSS = Square Root of the Sum of the Squares

# 1.0 Introduction

## 1 .1 Statement of the Problem

Building collision, commonly called ‘pounding’, occurs during an earthquake, due to their different dynamic characteristics, adjacent buildings vibrate out of phase and the at-rest separation is insufficient to accommodate their relative motions. Pounding means an instance of rapid strong pulsation and sometimes, like hammering, repeated heavy blows. Because of rapid increase in urban development and the associated increase in real-estate values especially in urban areas many buildings were constructed even up to their property lines. This situation may lead to non-structural and structural damages to the buildings and may also give rise to total collapse of buildings during seismic pounding.

In the past, major earthquakes affecting large metropolitan areas have induced severe pounding damage. In some cases, the additional forces generated by the impact interactions have led to the collapse of structure. In other cases, the buildings presented minor local damages, but indicating that pounding may be a serious threat to the structures if a stronger earthquake take place.

The Ethiopian Building Code Standard gives basic guidelines to prevent pounding effect of adjacent buildings during earthquake. There are lots of adjacent buildings, which are constructed before and still under construction, with no separation distance. This is because the cost of land is very expensive in major town of Ethiopia (especially in Addis Ababa). So it is better to check the separation distance between the adjacent buildings to guarantee whether at-rest separation is sufficient or not for future earthquake. This is the objective of this paper for selected case studies.

## 1.2 Objectives

The overall goal of the paper is to investigate the vulnerability of adjacent high rise buildings, which are constructed with insufficient separation distance in Addis Ababa, to pounding failure. And also, to determine the minimum required separation distance between adjacent buildings. Moreover, to provide the method of mitigation of the adjacent buildings which have no sufficient separation distance in between.

## 1.3 Scope of the Project

This paper will present the effect of pounding between two adjacent buildings. This work is limited only to investigate the minimum required separation distance between adjacent buildings to avoid pounding. In addition to this it provides the method of mitigation for buildings which are vulnerable to pounding due to insufficient gap between adjacent buildings. This can be achieved by taking three adjacent RC buildings (six single buildings which have been constructed and are under construction in Addis Ababa).

## 1.4 Methodology

This research has been achieved in four stages.

The first is literature review, which is related to the topic, has been exhaustively undertaken. In particular, Ethiopian Building Code of practices, Euro-Codes, International Building Codes, and International edited books and scientific journals in Engineering served as important documents to develop the basic formulation pertinent to modeling and analysis of adjacent buildings.

In the second step collecting of the design data, which have already constructed and being constructed in pair, has been done. The data have been collected from the administration of Construction follow up and controlling Office of Addis Ababa City and Bole Sub-City.

The third step is modeling and analysis of these collected designs of adjacent buildings by using finite element software's (SAP2000 v.14 and ETABS-v9.1). During modeling and analysis, the 3D model has been made for the adjacent buildings separately and only one pair (adjacent) building has been modeled together using contact element (gap element/linear spring ) as a sample modeling to show pounding effect. The analysis has been done with response spectrum function.

Finally, method of mitigation has been introduced for the adjacent buildings which have no sufficient separation distance to withstand lateral displacement during earthquake based on the analysis output results.

# 2 Overview of Seismic Pounding

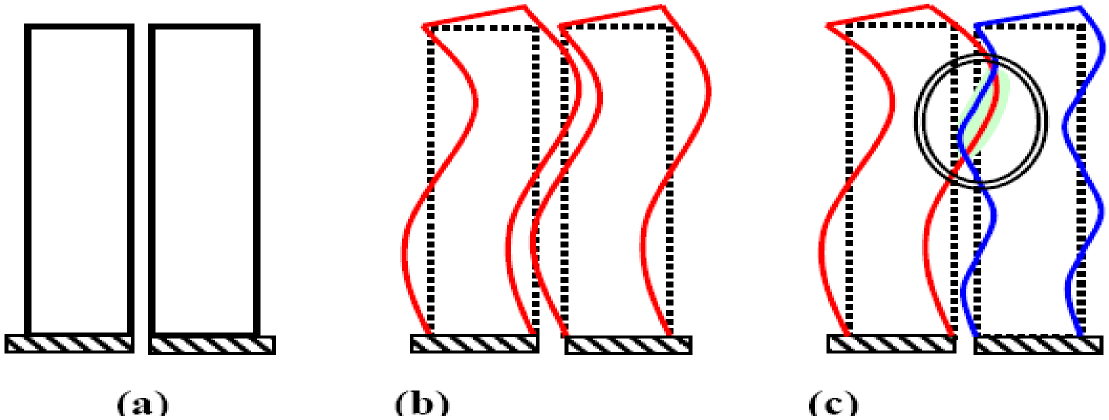
## 2.1 Background

The destructive nature of earthquakes has always been important to structural engineers. The reports on damage after strong earthquakes show that the effect of structural pounding is often one of the reasons for damage. In dense urban settings, the potential for closely spaced buildings to pound against each other exists. Pounding occurs when buildings have different dynamic characteristics those are governed by building stiffness (period of vibration), floor height, mass, number of storey, and building structural type [18,24]. Pounding of adjacent building could have worse damage. The earthquake that struck Alaska (1964), San Fernando (1995), Mexico City (1985), Loma Prieta (1989) and Kobe (1995) are clear examples of the serious seismic hazard due to pounding. Beteo (1985) reported that pounding was present in about 40% of 330 collapsed building during the 1985 Mexico City earthquake and in 15% of all cases it led to collapse. The highly congested building system in many metropolitan cities, such as Addis Ababa, constitutes a major concern for seismic pounding damage. For these reasons, it has been widely accepted that pounding is an undesirable phenomenon that should be prevented or mitigated.

Buildings are often built right up to property lines in order to make maximum use of space. Historically, buildings have been built as if the adjacent structures do not exist. As a result, the buildings may pound during an earthquake. Building pounding can alter the dynamic response of both buildings, and impart additional inertial loads to them [17]. During earthquake building make displacement and deformation for damping earthquake force. As a reason of this, structural elements can be damaged or whole structure can be collapsed. Such collision can induce very high and unexpected accelerations and storey shears in the overall structure. Characters of these displacements and deformations are very complex and it is a function of many variables from ground acceleration to rigidity center of the structure. At strong earthquake adjacent structures that do not have appropriate distance and hit each other, can be cause of pounding. The difference between dynamic properties (mass, hardness and height) of adjacent buildings or structures results different phase oscillations which is the main cause of pounding or impact and the more different is shape of vibration causes strong impact. [5,6]

Each time a collision occurs, the buildings are subjected to short lateral impact forces not specially accounted for in the conventional design process. The pounding forces can be several times greater than the seismic action effects anticipated by building codes. Pounding can cause instant collapse, thus reducing the survival chances of the occupants. These impact forces produce high-amplitude, short-duration, local accelerations which can induce damage in structural members or mechanical/electrical components of the buildings. Furthermore, earthquake pounding can amplify the overall dynamic response of the buildings. Moreover and to a less dangerous manner, closely spaced buildings in dense urban environments are also subjected to the failure of hazardous adjacent buildings or building components. The pounding of one building against another can loosen material, thus endangering pedestrians below, or it can lead to a weakening of either structure concluded in a premature failure. The problem is increased if there is an older adjacent building, built to less stringent seismic codes. Designers should carefully assess neighboring structures and design against possible objects falling from them (e.g., unreinforced parapets, walls or chimneys).

Characterizing pounding requires a detailed knowledge of the dynamic performance of multiple buildings, as well as knowledge of how the buildings will react to very high magnitude but very small duration impulsive forces. Structural poundings happen because of swaying of adjacent buildings with different mode shapes and periods under seismic loads which are not separated from each other properly (Fig. 1) [18, 24]. During earthquakes, structure’s mass and rigidity affect seismic behavior. It is nearly impossible to construct a building which has similar seismic behavior to another building. Many modern building codes may include separation requirement to have sufficient separation distance between adjacent buildings, so that building pounding could be prevented. [24]




**a)** Before earthquake, **b)** Similar seismic behavior, **c)** Different seismic behavior

Figure 1: Seismic behaviors of two adjacent buildings.

Poundings may occur because of structural irregularities. For example eccentricity between mass and rigidity centers cause torsion in the structure (Fig. 2a). If pounding building is regular, an impact surface can be formed between two adjacent buildings (Fig. 2b). [7]

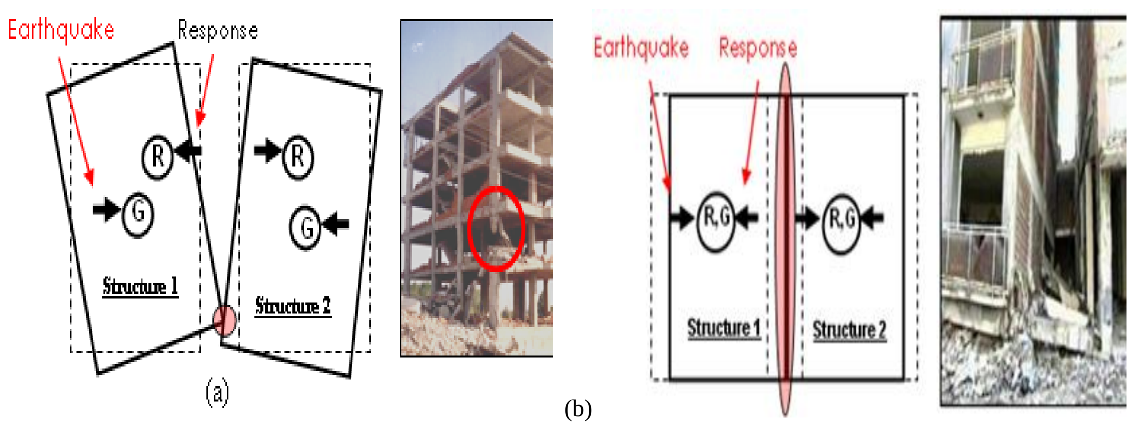



Figure 2: Pounding of regular adjacent buildings.

Some modern building code also gives some guidelines to prevent the pounding of adjacent buildings, such as: [23]

**I. Euro-code 8** stated as follows, Buildings shall be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building. This principle is considered to be satisfied:

*For buildings, or structurally independent units, that do not belong to the same property, if the distance from the property line to the potential points of impact is not less than the maximum horizontal displacement of the building at the corresponding level, calculated in accordance with expression (EC part 4.23);*

*For buildings, or structurally independent units, belonging to the same property, if the distance between them is not less than the square root of the sum- of the squares (SRSS) of the maximum horizontal displacements of the two buildings or units at the corresponding level, calculated in accordance with expression (EC part 4.23).*

*A specification is made: If the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0,7. The "damage limitation requirement" is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement".*

**II. International Building Code (IBC) 2009** also specifies spacing between the adjacent buildings equal to the square root of the sum of squares (SRSS) of the individual building displacements. Following is the excerpt of the specification from IBC 2009. [12]

*1613.6.7 (2009)* ***Building separations****. All structures shall be separated from adjoining structures. Separations shall allow for the displacement* $\delta_M$*. Adjacent buildings on the same property shall be separated by at least,* $\delta_{MT}$ *,*

*Where*

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad \text{.......................... 2.1}$$

*And δM1 and δM2 are the displacements of the adjacent buildings.*

*When a structure adjoins a property line not common to a public way, which structure shall also be set back from the property line by at least the displacement, δM, of that structure.*

***Exception****: Smaller separations or property line setbacks shall be permitted* *when justified by rational analyses based on maximum expected ground motions.*

**III. Ethiopian Building Code Standard (EBCS)** specifies about the separation distance of adjacent buildings as follows:-[23]

**Seismic Joint Condition (2.4.2.7)**

*1. Building shall be protected from collisions with adjacent structures induced by earthquake.*

*2. The requirement of (1) above is deemed to be satisfied if, the distance from the boundary line to the potential point of impact is not less than the maximum horizontal displacement according to equation (2.15) given as:*

$$d_s = \frac{d_e}{\gamma_d} \qquad \text{……………………… 2.2}$$

*Where*

*$\gamma_d$ is displacement behavior factor equal to $\gamma$ and can be expressed as follow.*

$$\gamma = \gamma_o k_D k_R k_W \qquad \text{…………………… 2.3}$$

*$\gamma_o$ is basic value of the behavior factor, depending on the structural type. The values is given in the table 1 below*

Table 1 basic value $\gamma_o$ of behavioral factors (table 3.2 in EBCS-8)

| *Structural type* | | $\gamma_o$ |
|---|---|---|
| *Frame system* | | *0.2* |
| *Dual system* | *Frame equivalent* | *0.2* |
| | *Wall equivalent, with coupled walls* | *0.2* |
| | *Wall equivalent, with uncoupled walls* | *0.2* |
| *Wall system* | *with coupled walls* | *0.2* |
| | *with uncoupled walls* | *0.25* |
| *Core system* | | *0.3* |
| *Inverted pendulum system* | | *0.5* |

*$k_D$ is factor reflecting the ductility class and given as:-*

*1.00 for DC"H"*

*$k_D$ = 1.50 for DC"M" (take this for our case)*

*2.00 for DC"L"*

*$k_R$ is factors reflecting the structural regularity in elevation and shall be given as follows.*

*$k_R$ = 1.00 for regular structures*

*1.25 For non-regular structures (take this for our case)*

*$k_W$ is factors reflecting the prevailing failure made in structural system with walls shall be taken as follows*

*1.00 for frame, frame equivalent dual system.*

*$k_W$ = (2.5-0.5$\alpha_o$) for wall, wall equivalent system*

*≥ 1 and core system*

*Therefore, the value of displacement behavior factor can be calculated using equation 2.3 as follows:-*

$$\gamma_d = \gamma = \gamma_o k_D k_R k_W$$

*by substituting the value of the factors we get:-*

$$\underline{\gamma_d = 0.50}$$

*3. If the floor elevations of a building under design are the same as those of the adjacent building, the distance referred in (2) above may be reduced by a factor of 0.7.*

4. *Alternatively separation distance is not required if, appropriate shear walls are provided on the primate of the building to act as collision walls ("bumpers"). At least two such walls must be placed at each side subject to pounding and must extend over the total height of the building. Then must be perpendicular to the side subject to collisions and they can end on the boundary line. Then the separation distance for the rest of the building can be reduced to 40mm.* 3a-d)

Even though, the building code puts a guide line as stated above, due to the maximum land use often required because of high population density and economic considerations there are many buildings which are already constructed and being constructing in contact with or extremely close to one another in this area (as shown in Fig. 3a-d). It is noted that out-of-phase vibrations may be induced when adjacent buildings are subjected to earthquake loading and pounding may occur if the separation distance is inadequate. As a result, the seismic pounding of adjacent buildings may pose a potentially serious problem in Addis Ababa.


a


b

c

d

Figure 3: Sample adjacent buildings, a, found at Merkato, Haile Yirga BLD, b, found at Mexico, Cheleleq and LTA BLD, c, is found at CMC, in front of Ethiopian Economics Association office, d, found at hayahulet, Ribka BLD**.**

## 2.2 Different cases of pounding

Poundings can be developed between high-rise buildings, between low-rise buildings, as well as between high-rise and low-rise buildings during strong earthquakes. Pounding during earthquake can also take place between a non-structural component and the structure itself as well as between two adjacent components.

Earthquakes cause ground shaking; the ground beneath a building is displaced laterally. The loads in the upper part of the building are generated inertia effects of this displacement. As a result of this, there will be pounding between adjacent buildings. The resulting shear forces and bending moments in a building are (generally) a maximum just above foundation level. [11]

Observation of previous earthquakes shows certain characteristics related to pounding. Buildings of similar height and with similar structural systems tend to suffer less damage than buildings of different height and with different structural systems [32]. This is due to the fact that buildings with the same height will have similar natural frequencies and will tend to move in-phase relative to one another. On the contrary, buildings of different height or with different structural systems will have different natural frequencies and will tend to sway out-of-phase with respect to each other; this may lead to damage that is more serious.

This remark illustrates that pounding problems must be treated case by case.

A classification of the different type of pounding which can appear is presented below

**2.2.1: Adjacent buildings with equal height and with aligned floor levels.**

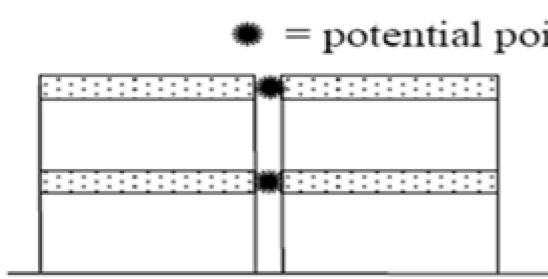



Figure 4: Floor-to-floor

This type of pounding is known as **floor-to-floor pounding:** Buildings that are the same height and have matching floors (see figure 4) are likely to exhibit similar dynamic behavior. If the buildings pound, floors will impact other floors, so damage usually will be limited to nonstructural components [7]. There are buildings which are already built and being constructed in contact or very closely one to another with the same floor level in Addis Ababa as shown in Fig. 5

Figure 5: Sample adjacent building with the same floor level

### 2.2.2: Adjacent buildings of similar or different height, with not aligned floor levels

Adjacent buildings with different floor levels can be case of pounding with the floor of one building with the mid column of the other. Such types of pounding are called **FLOOR-TO-COLUMN POUNDING**: This type of Pounding occurs in some adjacent buildings in which the floors levels are not in the same heights. Therefore, when shaking with different phases occurs, the floor of one building hits the column of another and causes serious damages which can lead to the fracture of the columns of the storey [14]. Since relatively rigid floor or roof diaphragm may impact an adjacent building at or near mid-column height, causing bending or shear failure in the columns, and consequently storey collapse. This type is the most dangerous Pounding that can result in sudden destruction of the structure (Fig. 6a). [3, 5,7]

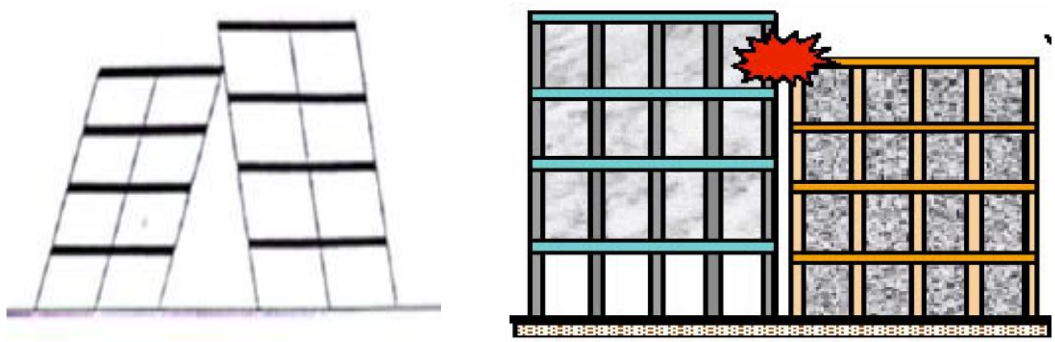


Figure 6: Pounding of building on the adjacent building column

The observation in Addis Ababa shows that such type of adjacent buildings are present as shown in the figure below (Fig. 7).

Figure 7: Sample adjacent building with different floor level. a. found at Habte Gorgis Dildy, Kirtu commercial center, b. found at Bole, K.Z Hotel, Addis Ababa.

### 2.2.3: Adjacent buildings with unequal height and with aligned floor levels

**Pounding of a shorter building on a taller one:** When two structures with different heights are adjacent, because of different dynamic properties, the shorter structure hits the adjacent one, which results in floor shearing in higher levels of impact part. It is important to know that the higher in the impact part level, the greater impact is tolerated more intensive response. Moreover, when buildings are of different heights, the shorter building may act as a buttress for the taller neighbor. The shorter building receives an unexpected load while the taller building suffers from a major discontinuity that alters its dynamic response. Since neither is designed to weather such conditions, there is potential for extensive damage and possible collapse (Fig. 9). [3,7]

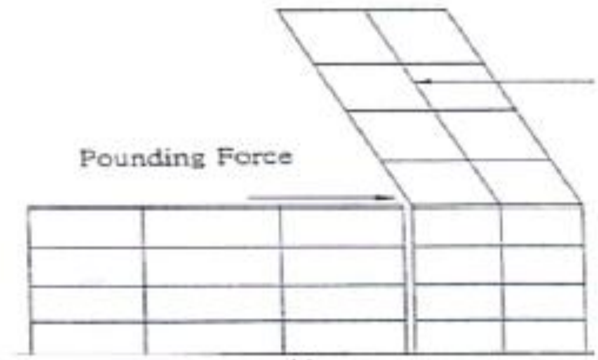



Figure 8: Pounding of shorter building on a taller building

We can observe adjacent buildings which have been constructed in contact or very closely one to another with different heights in Addis Ababa as shown in Fig. 9


a b

Figure 9: Sample adjacent building with different heights, a, found at found at Habte Gorgis Dildy,Kelifa BLD, b,also found at found at Habte Gorgis Dildy, Curtu commertial,

**2.2.4: Pounding of a heavier building on a lighter one:**

Since adjacent buildings may differ in the structural system of floors and/or in their applications, they have different masses, this can cause different phase oscillations, since the lighter building tolerates more intensive response (Fig10). [7]

Figure 10: Pounding of heavier building on a lighter building

**2.2.5: Buildings with an asymmetric structure (torsional pounding).**

**Pounding of two adjacent buildings with non-coaxial mass centers:** In building with non-coaxial mass centers, the structure may pound on the edge of the adjacent structure and cause strong tensional torques, which can lead to seriously damage to the column on the edges and corners of the pounded building (Fig. 11). [7]

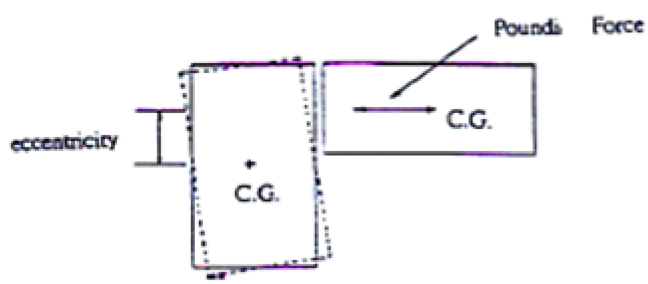



Figure 11: Adjacent buildings with non-coaxial mass centers

## 2.3 Classification of damage

Poundings may cause both architectural as well as structural damages. One type of classification of damage could be [14]:

- Major structural damage (Figure 12 b);
- Failure and falling of building non-structural elements creating a life-safety hazards;
- Loss of building functions due to failure of mechanical, electrical or fire protection systems;
- Architectural, non-structural and/or minor structural damage (Figure 12a).


Figure 12: Damage of adjacent building due to pounding (Mexico 1985) [14]

## 2.4 Analytical models for impact

Pounding between adjacent structures is a very complex phenomenon, which can involve plastic deformation at contact points, local crashing or crushing as well as fracturing at the contact, friction, etc. Due to this complexity, modeling of impact is an extremely difficult task [26]. These non-linear deformations are not easily incorporated into the modeling of pounding. Therefore, idealizations and assumptions have inevitably been used in theoretical models. For example, structures have been idealized as rigid barriers, single-degree-of-freedom oscillators or multi-degree-of-freedom oscillators; pounding between structures have been modeled by linear dashpot-spring system or non-linear impact model. Despite of these simplifications, theoretical analyses have been valuable in providing insight into the pounding mechanisms.

The collisions between adjacent buildings are simulated either by means of special contact elements (of the spring-dashpot type) activated when the bodies come in contact or by applying the impact laws of mechanics for particles (stereomechanical impact), with a coefficient of restitution (CR) for plastic impacts. The first approach, also called *piece-wise impact or simply contact element method*, can provide a better approximation to the real problem, under the condition that appropriate values of the impact element properties be used. Moreover, while these properties are highly uncertain and hence difficult to determine with accuracy, it turns out that the response is quite insensitive to wide changes in their values.

### 2.4.1 Contact element method

The contact element approach is a very widely used formulation because of its easy adaptability and logical nature to model impact. The impact phenomenon is modeled by using a contact element that is activated when the gap between the structures closes. A spring with high stiffness is necessary to provide a realistic estimate of the impact force, ensure small impact duration and limit the penetration or overlapping between adjacent segments. This spring may be used in conjunction with a damping element. However, using a spring of very high stiffness can result in unrealistically high impact forces and lead to numerical convergence problems. The solution difficulties arise from the large changes in stiffness upon impact or contact loss, thus resulting in large unbalanced forces affecting the stability of the assembled equations of motion. [31]

Three types of contact have been used in the past: Linear spring element, Kelvin-Voight element, and the Hertz contact element.

#### 2.4.1.1 Linear spring element

The simplest contact element consists of a linear elastic element (Figure 13). The spring is assumed to have restoring force characteristics such that only when the relative distance between the masses becomes smaller than the initial distance ($g_p$), the spring contracts and generates forces, which enable us to consider the phenomenon of pounding within the framework of an ordinary response analysis. This collision spring is assumed to be the axial stiffness of the floors and the beams in each storey.[5,30]

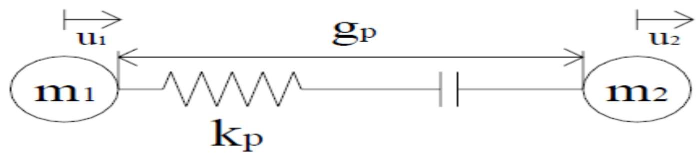
The force in the contact element may be expressed according to:

$$F_p = k_p [u_1 - u_2 - g_p] \qquad \text{if } u1\text{-}u_2\text{-}g_p \geq 0 \quad \text{(Buildings getting closer) .. 2.4a}$$

$$F_p = 0 \qquad \text{if } u1\text{-}u_2\text{-}g_p < 0 \quad \text{(Buildings moving apart) … 2.4b}$$

Where $u_1$ and $u_2$ are the displacements of the impacting bodies, kp is the spring constant of the element and $g_p$ is the static separation between the structures. [5]

The present model for impact has been extensively used by Maison and Kasai [~1990].




This approach can be easily implemented in commercial software. However, energy loss during impact cannot be modeled. Whenever two mechanical systems collide there is an exchange of momentum and energy is dissipated in the high stress region of contact. This energy dissipation is the work involved in damped elastic behavior and also plastic deformation and fracture.

A generalization of the piece-wise linear stiffness contact element considers non-linear stiffness for the contact element. The simplest of the models proposed considers two types of stiffness's, an approaching stiffness $k_i$ and a higher stiffness for separation $k_f$.

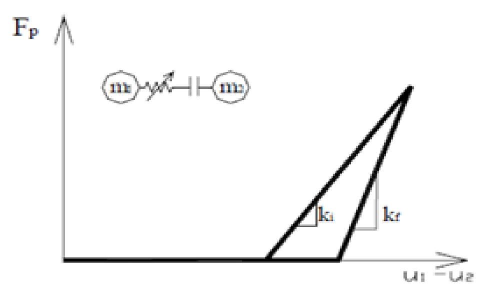



Figure 13: Force displacement for a generalization of the piece wise model

The present model includes some energy dissipation due to hysteretic behavior at the impact element, which is represented by the surface of the triangle in the figure above [5].

#### 2.4.1.2 Kelvin-Voight element

The Kelvin-Voight element is represented by a linear spring in parallel with a damper, as shown in Figure 14. This model has been widely used in some studies [Anagnostopoulos, 1988; Wolf and Skrikerud, 1980; Jankowski, 2004]. The stiffness of the impact spring is typically large and represents the local structural stiffness at the point of impact that will react to the shocks during contact. The constant of the associated dashpot determines the amount of energy dissipated during impact.

The forces in the contact element may be calculated from:

$$F_p = k_p (u_1 - u_2 - g_p) + C_p (\acute{u}_1 - \acute{u}2) \qquad \text{if } u1\text{-}u_2\text{-}g_p \geq 0 \quad \ldots 2.5a$$

$$F_p = 0 \qquad \text{if } u1\text{-}u_2\text{-}g_p < 0 \quad \ldots 2.5b$$

Where Kp and Cp are the spring and dashpot constant of the element, $u_1$ and $u_2$ are the displacement of the impacting bodies, $v_1$ and $v_2$ are the velocities of impacting bodies and $g_p$ is the static separation between the structures.

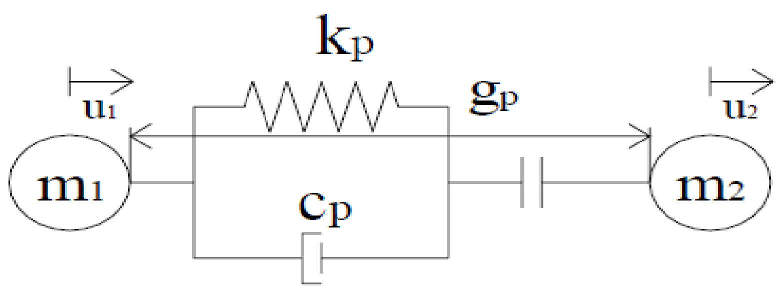



Figure 14: The Kelvin-Voight element

The damping coefficient (cp) can be related to the coefficient of restitution (CR), by equating the energy losses during impact [Anagnostopoulos, 1988]):

$$C_P = 2\xi_i \sqrt{\mathcal{K}_P . \frac{m_1 . m_2}{m_1 + m_2}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.6a$$

$$\xi_i = \frac{-ln C_R}{\sqrt{\pi}^2 + (ln C_R)^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots 2.6b$$

Where $m_1$ and $m_2$ are the colliding masses and ζ is the damping ratio.

Values of ζ and corresponding values of the coefficient of restitution (CR) are listed in table 2. The value of $C_R$ ranges from 0 (for perfectly plastic impact) to 1, 0 (for elastic impact).

Table 2 Value of the damping ratio in function if the coefficient of restitution

| $\xi_i$ | 0.00 | 0.02 | 0.05 | 0.10 | 0.20 | 0.50 | 1.00 |
|---|---|---|---|---|---|---|---|
| $C_R$ | 1.00 | 0.94 | 0.85 | 0.73 | 0.53 | 0.16 | 0.00 |

The disadvantage of the traditional Kelvin element is that the viscous element remains activated when the structures tend to separate, that is, the dashpot in the element opposes the motion of the structure when they come together, but also opposes the motion of the structure when they bounce back, which does not have a physical explanation.

A variation of the Kelvin model may include a non-linear spring and a contact element that only contributes for positive loading; see Figure 15. This model is called the Impact Kelvin element.

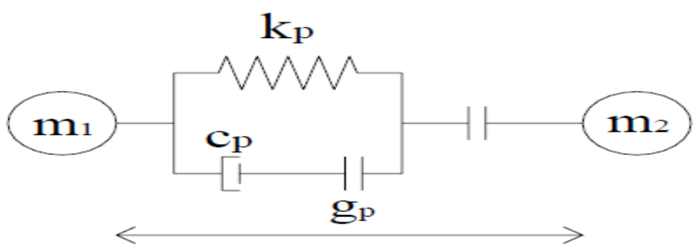



Figure 15: Impact Kelvin Element

#### 2.4.1.3 Hertz contact

In pounding, one would expect the contact area between neigh boring structures to increase as the contact force grows, leading to a non-linear stiffness. In order to model highly non-linear pounding more-realistically, Hertz impact model has been adopted by various authors [Davis, 1992; Jing and Young, 1990 and 1991; Chau and Wei, 2001; Chau et al, 2003]. This model uses the Hertz contact law: a non-linear spring in an impact oscillator. The main restriction of their works is that only pounding of a SDOF oscillator on a stationary barrier or on a barrier moving with 'locked-to-ground-motion' is considered. [31]

The force in the contact element may be expressed as:

$$F_p = k_p \,.\, (u_1 - u_2 - g_p)^{3/2} \qquad \text{if } u1 - u_2 - g_p \geq 0 \qquad \text{...... 2.7a}$$

$$F_p = 0 \qquad \text{if } u1 - u_2 - g_p < 0 \qquad \text{....... 2.8b}$$

Where $k_p$ is the spring constants of the element, $u_1$ and $u_2$ are the displacements of the impacting bodies and $g_p$ is the static separation between the structures. The coefficient $k_p$ depends on material properties and geometry of colliding bodies. [6]

The Hertz contact law corresponds to the solution of static contact of two elastic bodies. Nonetheless, the formula has been extrapolated to the cases of dynamic contact problems [Goldsmith, 1960]. The Hertz contact law, considering elastic bodies, is incapable of taking into account dissipation during impact phenomenon. The value of the Hertz exponent, 3/2, may be different for real pounding, but Davis [1992] has shown that the exact value may be altered without radically changing the oscillator response. [31]

An improved version of the Hertz model, called Hertzdamp model, has been considered by Muthukumar and DesRoches [2004] whereby a non-linear damper is used in conjunction with the Hertz spring. Similar models have been used in other areas such as robotics, and multi-body systems. However, their efficiency in structural engineering has not been considered.

$$F_{\mathrm{p}} = k_{\mathrm{p}}\,.(u_1 - u_2 - g_{\mathrm{p}})^{3/2} + C_{\mathrm{p}}\,(\acute{u}_1 - \acute{u}2) \qquad \text{if } u1\text{-}u_2\text{-}g_p \geq 0 \quad \ldots\ 2.9a$$

$$F_{\mathrm{p}} = 0 \qquad \text{if } u1\text{-}u_2\text{-}g_p < 0 \quad \ldots\ 2.9b$$

Where $k_p$ and $c_p$ are the spring and dashpot constants of the element, $u_1$-$u_2$ -$g_p$ is the relative penetration and $\acute{u}_1$-$\acute{u}_2$ *is the penetration velocity [6]*

A non-linear damping coefficient ($C_p$) is proposed so that the hysteretic loop matches the expected loop due to a compressive load that is applied to and removed from a body within its elastic range at a slow rate:

$$C_{\mathrm{c}} = \zeta.(u_1 - u_2 - g_{\mathrm{p}})\,n \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ 2.10$$

Equating the energy loss during stereo-mechanical impact to the energy dissipated by the damper, the value of $\zeta$ can be related to the spring constant, $k_p$ , the coefficient of restitution, e, and the relative velocities of the bodies at the instant of impact, $\acute{u}_1$-$\acute{u}_2$ as shown below:

$$\xi = \frac{3\mathcal{K}_{\mathrm{P}}(1 - C_{\mathrm{R}}^{2})}{4.(\acute{u}_1 - \acute{u}_2)} \qquad \ldots\ldots\ldots\ 2.11$$

The contact element approach has its limitations, with the exact value of spring stiffness to be used, being unclear. Uncertainty in the impact stiffness arises from the unknown geometry of the impact surfaces, uncertain material properties under loading and variable impact velocities.

### 2.4.2 Stereo-mechanical Impact

The stereo-mechanical theory of impact is the classical formulation to the problem of impacting bodies. The stereo-mechanical approach assumes instantaneous impact and uses momentum balance and the coefficient of restitution to modify velocities of the colliding bodies after impact.

The original theory considered the impacting bodies as rigid; later a correction factor to account for energy losses was introduced. The theory concentrates on determining the final velocities of two impacting bodies depending on their initial velocities and a coefficient of restitution to account for plasticity during impact. Due to macroscopic approach to the problem, the theory does not consider transient stresses and deformations in the impacting bodies. Permanent deformation of the bodies is implicitly accounted for by the coefficient of restitution ($C_R$). The part of the initial kinetic energy that is transformed into post-impact vibrations in one of the impacting bodies is assumed negligible.

The inconvenient of the method is that it is no longer valid if the impact duration is large enough so that significant changes occur in the configuration of the system. This implies that the duration of impact is neglected. Furthermore, it cannot be easily implemented into existing commercial software.

The final velocities (as shown in equation below), when two non-rotating bodies impact and when the contact point and the centre of mass of the bodies lie in the same line (central impact), are given by:

$$\ddot{u}_1 = \acute{u}_1 - (1 + C_R).\frac{m_2\,(\acute{u}_1 - \acute{u}_2)}{m_1 + m_2} \qquad \text{………… 2.12a}$$

$$\ddot{u}_2 = \acute{u}_2 - (1 + C_R).\frac{m_2\,(\acute{u}_1 - \acute{u}_2)}{m_1 + m_2} \qquad \text{………… 2.12b}$$

where $u_1$ and $u_2$ are the initial velocities of the bodies at the onset of impact, $m_1$ and $m_2$ are the masses, and $C_R$ is the coefficient of restitution:

$$C_{\mathrm{R}} = \frac{\ddot{u}_1 - \ddot{u}_2}{\acute{u}_1 - \acute{u}_2} \qquad \text{........... 2.13}$$

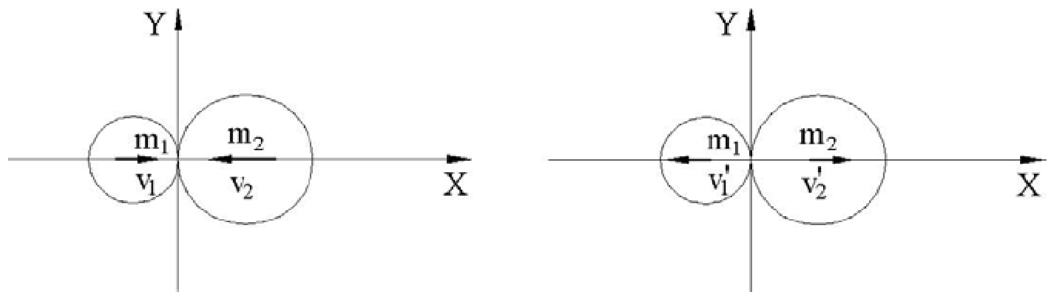



Figure 16: Stereo-mechanical impact: (a) Pre-impact state; (b) Post-impact state.

Traditionally the value of the coefficient of restitution was assumed to depend only on the material properties; however, the influence of the mass, the shapes, and the relative velocities has been recognized. The coefficient of restitution is traditionally determined from observations of rebound height (h*), when a sphere is dropped from a height of h on a massive plate of the same material:

$$C_{\mathrm{R}}{}^2 = \frac{b^*}{b} \qquad \text{................. 2.14}$$

When two spheres of different materials collide, the coefficient of restitution may be estimated from:

$$C_{\mathrm{R}12} = \frac{C_{\mathrm{R}11}.E_2 + C_{\mathrm{R}22}.E_2}{E_1 + E_2} \qquad \text{.................2.15}$$

Where $C_{R11}$ and $C_{R22}$ are the coefficient of restitution for a sphere impacting a plate of the same material, and $E_1$ and $E_2$ are the elasticity modulus of each sphere.

It should be noted that at $C_R$ equal 1, the Kelvin model reduces to the linear spring and the Hertzdamp model reduces to the Hertz model.

Athanassiadou, Penelis and Kappos [1994] modeled the contact between two structures by assuming it instantaneous and the values of velocities to be used at the new time step are calculated using Newton's law in combination with the principle of conservation of momentum. They concluded that the effect of the coefficient of restitution in the realistic range of 0,2 to 0,8 is relatively minor.

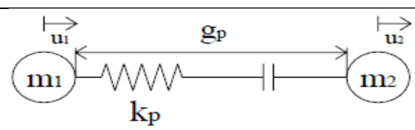
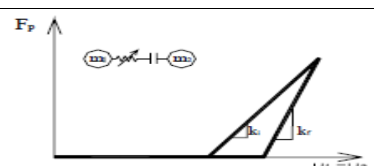
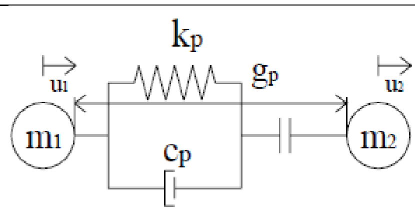
It has been shown that the variation in the coefficient of restitution ($C_R$) has a relatively minor effect on the structural response due to pounding (Athanassiadou et al. [1994]).

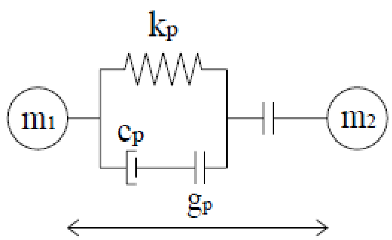
Papadrakakis et al. [1991] proposed an algorithm, which can be incorporated into computer program. The method is based on the Lagrange multiplier approach by which the geometric compatibility conditions due to contact are enforced.

### 2.4.4 Summary on impact model used in Seismic Pounding

Table 3 presents a summary of the different models used to represent pounding phenomenon. The advantages, disadvantages and list of authors who utilized the models are presented.

Table 3 Summary of pounding model

| **Piece-wise model** | | | | |
|---|---|---|---|---|
| **Model** | | **Advantage** | **Disadvantag e** | **Used by** |
| **Linear spring** |  | Easily implemented in software | Energy loss cannot be modeled | Maison & Kasai [1990,1992] |
| Variation: non-linear stiffness |  | Stay simple | Some energy dissipation due to hysteretic behavior | Valles and Reinhorn [1997] |
| **Kelvin-Voight** |  | The constant of the dashpot determines the amount of energy dissipated | The viscous element remains activated when the structures tend to separate | Anagnostopoulos [1988], Wolf and Skrikerud [1980], Jankowski[2004] |

| | | | | |
|---|---|---|---|---|
| Variation: Impact Kelvin element |   | The viscous Element does not remain activated when the structures tend to separate | Time consuming | |
| Hertz contact | $\mathcal{F}_P = \mathcal{K}_P.(u_1 - u_2 - g_P)^{3/2}$ | More realistic | -Energy loss cannot be modeled<br>- Not easily implemented in software | Davis[1992], Jing and Young[1990 and 1991] Chau and Wei [2001] Chau et al [2003] |
| Hertzdamp model | $\mathcal{F}_P = \mathcal{K}_P.(u_1 - u_2 - g_P)^{3/2} + C_P(\acute{u}_1 - \acute{u}_2)$ | The constant of the dashpot determines the amount of energy dissipated | | |
| General comment | The piece-wise model can provide a better approximation than stereo-mechanical to the real problem, under the condition that appropriate values of the impact element Properties are used. | | | |

| **Stereo-mechanical** | | | |
|---|---|---|---|
| **Model** | **Advantage** | **Disadvantage** | **Used by** |
| $\ddot{u}_{i=}\,\acute{u}_i - (1 + C_R).\dfrac{m_j\,(\acute{u}_i - \acute{u}_j)}{m_i + m_j}$ | -Classical formulation of the problem<br>-Permanent deformation is accounted by the coefficient of restitution | -Not valid if the impact duration is large<br>-difficult to implement in software | Athanassiadou et al. [1994], Papadrakakis et al. [1991...] |

## 2.5 Previous Studies on Pounding between Structures

### 2.5.1 SDOF oscillator's studies

Wolf and Skrikerud [1980] investigated two and one-sided impact of a single degree of freedom structure, as sketched at Figure 17 in which ground acceleration is imposed to the basis of an oscillator. Side barriers limit the displacements. They studied pounding using piece-wise linear spring for the contact element. The study focused on elastic impact, the effect of energy dissipation in the impact process was not investigated. They derived expressions giving the effective "natural periods" of such systems and then computed response spectra for harmonic input and for artificial earthquake motion. Owing to the simplicity of their approach, they were able to draw broad insights into the hammering process. They found that pounding hardly changes the global seismic response of the involved structure. For very flexible structures an increase in the global response occurs, whereas for stiff structures a slight decrease is even observed. Significant stresses resultants and displacements occur in the vicinity of the area of impact. The impact force-time history contains substantial amplitudes of high frequency. In addition, because it acts on a small area, modes of higher frequency are strongly excited. This leads to an increase of the in-structure response spectra in the high-frequency domain range throughout the structure.

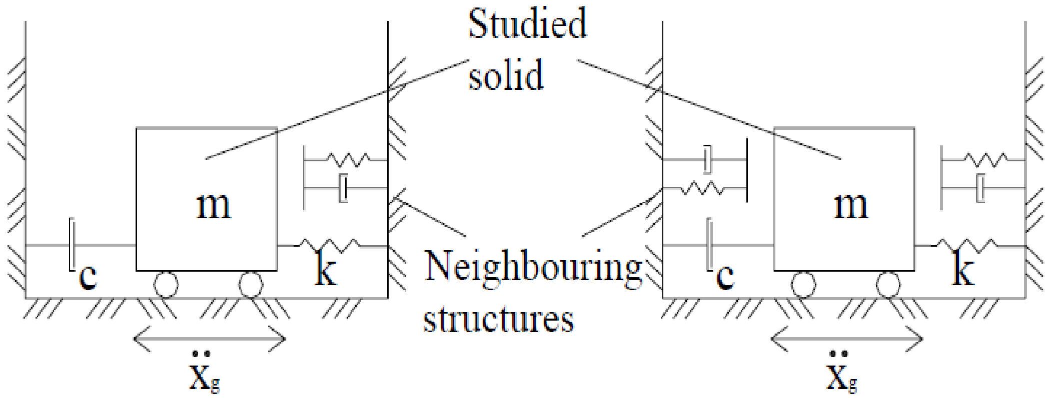



Figure 17: One and two-sided impact of a single degree of freedom structure

Recently Davis [1992] has used a SDOF oscillator interacting with either a stationary or a moving neighboring barrier. Impact forces are described by non-linear Hertz law of contact and results are given in the form of impact velocity spectra for harmonic base input. These spectra are characterized by a strong peak near a period equal to one-half the natural period of a similar non-impacting oscillator. In effect, the impact oscillator has a natural period equal to roughly half the value it would have had if the neighboring structure had not been present.

More recently, Chau and Wei [2001] extended the model of Davis that studied the rigid impact of an oscillator pounding on a stationary barrier. They consider poundings (as non-linear Hertzian impact) between two adjacent structures under harmonic earthquake excitation. When the difference in natural periods between the two oscillators increases, the impact velocity also increases drastically. Parametric studies show that the maximum relative impact velocity is not very sensitive to changes in the contact parameters. The peak of the impact velocity spectrum occurs at an excitation period T* which may be less than both $T_1$ and $T_2$. Thus, the maximum impact velocity may occur at a period less than those of the oscillators may. This is undesirable in view of the fact that higher level of ground excitation at lower period is recommended in nearly all seismic design codes. In addition, unwanted period shift of an existing structure imposed by the construction of a new building in its neighborhood may lead to unprepared and unexpected damages of the former during earthquakes.

One-side impacts SDOF systems have also been studied as a random vibrations problem. Jing and Young [1991] used a Hertzian impact law. As David [1992] found, their expressions show that the effective frequency of the impacting system can be up to two times the natural frequency of the system without impact. This shift could have adverse effects for flexible structures with fundamental period in the region of the descending branch of an earthquake design spectrum.

Pantelides and Ma [1998] considered poundings between a damped SDOF structure with either elastic and inelastic structural behavior and a rigid barrier. The pounding phenomenon is modeled as a Hertz impact force. Artificial as well as actual earthquake excitations were used in numerical evaluations of the seismic response. The response of the inelastic structural system is compared to that of an elastic structure. The inelastic structure has considerably smaller accelerations as compared to the elastic structure and the maximum displacement of the inelastic structure is larger than that of the elastic structure. Moreover, the maximum pounding force and number of pounding occurrences are considerably less in the inelastic case as compared to the elastic case. The inelastic behavior of structures under pounding is less conservative than the elastic behavior assumption. This could be one of the explanations of why in general buildings experiencing pounding have shown satisfactory response in past earthquakes.

The case of several adjacent buildings in a row subjected to pounding has been studied by Anagnostopoulos [1988]. Pounding is simulated using linear viscoelastic impact elements. Elastic and inelastic systems have been examined using a set of five real earthquake motions and a wide variation of the problem parameters. Results are given in terms of displacement amplifications for exterior and interior systems (structures). The results indicate that the displacement of exterior constructions may be considerably amplified, while interior structures may experience amplification or deamplification, depending on the ratio of structural periods. The results of this study corroborate the observed greater damage that corner buildings experience, while interior buildings may exhibit a deamplification in the response. The results show that the effects of pounding diminish as the gap increases. The authors note that the relative masses of the buildings have an important effect on the response, being greater the amplification in the structure with smaller mass. Other parameters, like the stiffness of the contact element, play a minor role in the response. Impact generated accelerations, and to a lesser degree the corresponding velocities, are quite sensitive to changes in the impact element properties, especially to changes in the spring stiffnesses. These accelerations can cause damage to the contents of the building, but have little effect on the displacement response of the colliding masses.

Athanassiadou et al. [1994] have made similar work. They introduce a constant phase difference in the base motion of each system (Figure 18), in an attempt to approximate travelling-wave effects. In this manner, pounding can be induced between buildings having similar or even equal periods. Although this is a gross and perhaps questionable approximation, it may give some rough idea about the effects that travelling waves may have on pounding in long building rows. The buildings are modelled as single degree of freedom structures in series with inelastic load-displacement relation. The contact between two structures is modelled by the stereomechanical impact. Results indicate that pounding induced in this manner has practically negligible effects in a row of 5 buildings. When the number of buildings in the row increases to 8, then for the first five of them pounding leads to reduced response, while for the last three buildings the response increases, though not by much.

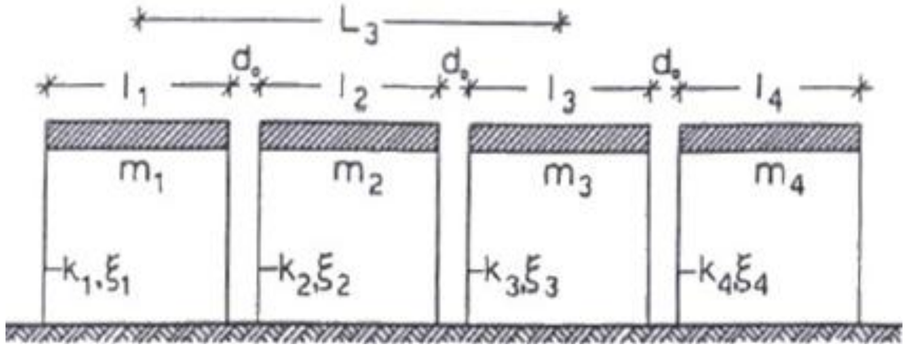



Figure 18: Analytical model for adjacent structures [Athanassiadou et al., 1994]

Valles and Reinhorn [1995] introduced the concept of Pseudo Energy Radius (PER) to study the effect of pounding in buildings. The response of a single degree of freedom system in the state space plane, subjected to seismic input was related to the elastic structural energy (Ee) of the system through the Pseudo Energy Radius as follows.

$$r_{PER} = \sqrt{\frac{2.E_{emax}}{m.\omega^2}} \qquad \text{………… 2.16}$$

Where m is the mass of the structure, ω is the frequency of the ground motion, $E_{emax}$ is the maximum elastic structural energy of the system and $r_{PER}$ is the pseudo energy radius (PER). The Pseudo Energy Radius being expressed as units of displacement could be used to determine the critical gap to preclude pounding ($g_{cr}$) between adjacent structures, as shown below.

$$g_{cr}=\sqrt{r_1^2+r_1^2-2.r_1r_2\rho} \qquad \text{…………… 2.17}$$

Where ρ is the correlation coefficient, and $r_1$, $r_2$ are the pseudo energy radii corresponding to the energy levels of the two structures. Pounding occurred when the pseudo energy radii overlap and the initial separation between the structures ($g_p$) was less than the critical gap ($g_{cr}$). The impact of the structures was assumed to occur at their respective maximum energy levels imposed by the earthquake. The stereo-mechanical approach was then used to determine the post impact states of the colliding masses. The ratio of post impact PER to pre impact PER was an estimate of the amplification effects due to pounding.

However, the concept of PER is based on the maximum elastic structural energy (Ee) of the system. No adjustments are made to include the effects of yielding, which may alter the structural energy of the system considerably, depending on the period of the system and characteristics of the input motion.

Jankowski [2006] proposed an impact force response spectrum for two adjacent structures, which shows the plot of the peak value of pounding force as a function of the natural structural vibration periods. The examples of response spectra show that the selection of structural parameters, such as gap size between structures, their natural vibration periods, damping, mass and ductility as well as the time lag of input ground motion records, might have a substantial influence on the peak pounding force value. [26]

### 2.5.2 MDOF oscillators' studies

Anagnostopoulos and Spiliopoulos [1992] extended their original studies on a series of single-degree-of-freedom systems to the case of multi-degree-of-freedom systems (Figure 19). They investigated the linear as well as the non-linear res ponse of several adjacent buildings in a row under conditions of pounding. They idealized the buildings as lumped mass, shear beam type, MDOF systems with bilinear force-deformation characteristics and with viscoelastic supports. Furthermore, the structural models include foundation compliance by means of a linear spring for translational and rotational motions. The constants for the foundation springs were determined considering a spread footing foundation on a stiff soil. Collisions were simulated by viscoelastic impact elements (a Kelvin solid) and five real earthquake motions were used. The damping constant of the contact element was calculated to yield a coefficient of restitution of 0,5. They found that pounding could cause high overstresses, mainly when the colliding buildings have significantly different heights, periods or masses. When studying the influence of different heights of the impacting structures, the authors found that the taller structure pushed the smaller one, imposing upon it in one direction its own higher amplitude and longer period motion, increasing the plastic deformation of the s maller one, while the displacements of the taller remained almost unchanged. Greater consequences for the tall building can be expected if the lower building were more massive and stronger. The parametric studies on the contact element stiffness indicated that the ductility demands are not sensitive to this parameter, while some sensitivity is observed on the response from variation of the coefficient of restitution. The authors noted that their observations were based on displacements, and not on accelerations or on the shear forces.

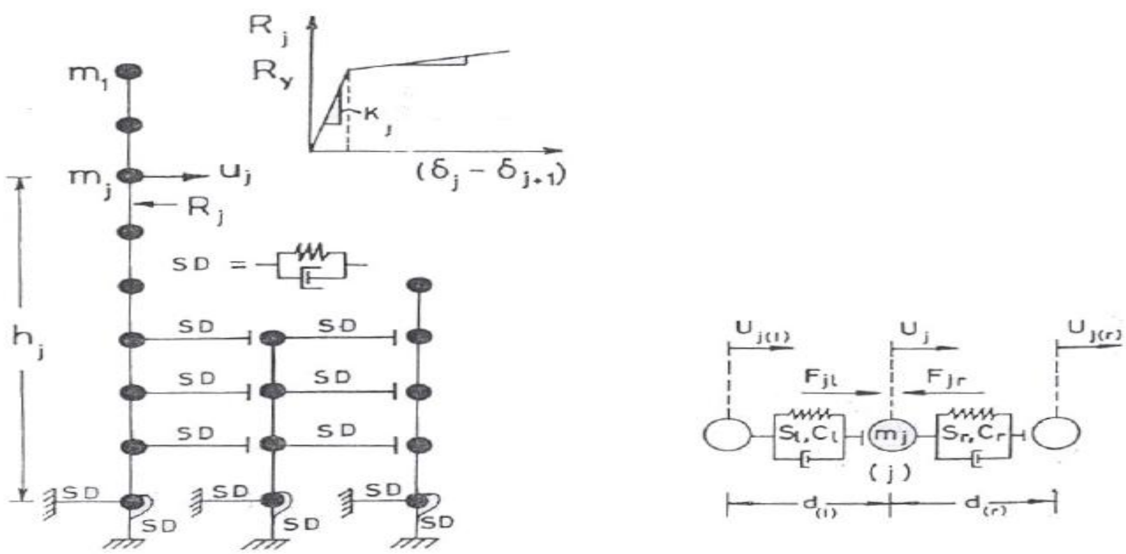



Figure 19: Idealization of adjacent buildings [Anagnostopoulos and Spiliopoulos, 1992]

Maison and Kasai [1990] present a formulation and solution of the multiple degree of freedom equations of motion [15]. The subject building undergoes pounding at a single floor level with a rigid adjacent building. A single linear spring represents the local flexibility of the buildings at their locations of contact. To simplify the problem, it has been formulated and applied by allowing only one pre-selected contact point. They found that even at the relatively large separation (90% of the sum of maximum displacements obtained without pounding) the increases in drifts and shears are significant. In situations where pounding may potentially occur, neglecting its possible effects leads to non-conservative building design/evaluation.

Maison and Kasai [1992] studied the influence of different parameters to pounding between 15-storey and 8-storey buildings [15]. The response of the taller structures becomes larger when the mass of the shorter structure increases, while the response of the shorter structure is hardly influenced by the variation in this parameter. An increase in the separation between the structures reduces the maximum response of the taller structure, while in the shorter structure a decrease in the response away from the pounding side is observed. A response increase is observed in the upper storeys of the taller structure for a decrease in the separation. The results indicated that, for the structure considered, the stiffness and damping of the contact element had little influence on the response parameters being studied, but the authors recognize that the acceleration response may be greatly influenced by the contact element stiffness.

Instead of the spring-dashpot impact element, Conoscente et al. [1992] applied the impact laws of mechanics with the coefficient of restitution approach. With this method, contact is not limited to a single point. Two adjacent 15-storey buildings are evaluated; building 2 is about 20% more massive than the stiffer one. The results indicate the storey shears are dramatically affected by impact but these effects are localized in the areas of impact.

### 2.5.3 Finite element studies

Non-linear material behavior has also been considered. Papadrakakis et al. [1991] based a method for harmonic input on the Lagrange multiplier approach making use of coefficient of restitution for a three-dimensional finite element analysis. A Lagrange multiplier solution was proposed to enforce geometric compatibility when pounding occurs and using the formulas for stereomechanical pounding, imposes the calculated separation velocities as initial condition for the next step. The results demonstrate the amplification effect of pounding on the dynamic response of the less excited structure and the corresponding beneficial effect on the structure vibrating near its resonance. The aggravation, however, is more pronounced for the stiffer structure. They used their method [1996] to a three-dimensional model of two or more adjacent buildings, which takes into account the dynamic contact conditions in space. Two real earthquake motions were used and the effect of various in-plan configurations of adjacent buildings as well as the effect of stiffness irregularities is investigated. The combination of flexible and stiff adjacent buildings results in an amplification effect during pounding on a stiff structure for all cases considered, particularly when the excitation is near the resonance of the flexible building, and in a mitigation effect on the flexible building in the majority cases. Another important result to say is that the tests have demonstrated an amplification effect in the case of three-building orthogonal pounding with irregularities. Papadrakakis and Mouzakis [1995] experimented two-storey building frames. Two series of tests, with and without pounding, were carried out using the shaking table simulator. The experimental results were compared with analytical ones based on a formulation of the contact impact problem by the Lagrange multiplier method. Good agreement between the experimental and the analytical results was achieved.

More recently [2004], Papadrakakis and Mouzakis studied a three-dimensional pounding phenomenon of two adjacent buildings during earthquakes with aligned rigid horizontal diaphragms for linear and non-linear structural response. The interaction process between the two colliding bodies is modeled using the coefficient of restitution and the ratio μ of tangential to normal impulses, which corresponds to the coefficient of friction under certain conditions. The computation of μ is related to the energy loss of collision and is bounded by values that correspond to no sliding at separation and conservation of energy. The value of μ is defined by the condition that the total kinetic energy before and after impact remains the same and that the final relative tangential velocities at the contact point is zero. They concluded that the response of the stiff building during pounding is not substantially affected by the value of the gap separation, while the flexible one suffers larger displacements with larger gap separation.

Liolios and Galoussis [1992] made a numerical approach of the problem. The method is based on formulating the problem by the finite element method as an inequality one and on solving this inequality by average-acceleration method of time-discretization and non-linear mathematical programming. The purpose of their work is to take into account the effect of elastoplastic-softening/fracturing behavior for unilateral contact and P-effects. Two adjacent buildings of respectively 3 and 8 storey are studied. The results show an increased of the stresses in the 3-storey of the small structure and in the 8-storey of the tall one. The most significant increase is for the 4-storey of the taller structure.

Jeng and Tzeng [2000] have made a survey of the seismic separation for Taipei City. A series of pounding analyses of adjacent buildings subjected to simulated earthquakes was realized. The buildings are modeled as elastic moment resisting frames. Two adjacent buildings are studied with same level floors and different heights. The pounding mechanism was simulated by a spring. They found that as long as the separation is small enough to cause pounding, the difference in the separation distance does not produce a substantial change in the pounding effect. The maximum storey shear amplifications were observed above the pounding floor level for the taller building and at the pounding floor level for the smaller one. Storey shear amplification of the other floors is found with smaller magnitude.

Karayannis & Favvata [1998] studied the pounding problems between structures designed according to EC2 and EC8. Nonlinear analyses were performed in order to examine the influence of the pounding to the capacity requirements of the structures. A special purpose one-dimensional element is employed for the modeling of the critical columns in the pounding area. The following results were found: i) Pounding between frame structures increased significantly the ductility requirements of columns in the pounding area. ii) Poundings of an 8-storey frame with a 4-storey frame-wall system were critical and measures for the increase of the available ductility of the columns are required. In 2005, Karayannis & Favvata studied the case where the slabs of the first structure hit the columns of the other. Special attention was paid to the local effect in the external columns of the tall building that suffer impact with the upper floor slab of the adjacent shorter and stiffer structure (3 storeys). The ductility demands in these columns are increased when compared with the ones without the pounding effect. In the case where the two buildings are in contact (e=0) these demands appear to be higher than the available ductility values. Moreover, in all their examined cases the developing shear forces exceed the shear strength of the column many times during the seismic excitation.

Jankowski [2004] studied pounding of buildings modeled as elastoplastic MDOF lumped mass models and using a non-linear viscoelastic model of collisions considering two adjacent structures of 3-storey buildings with the same floor levels. The results indicate that pounding has a significant influence on behavior of a more flexible and lighter structure amplifying its response. On the other hand, the behavior of the heavier and stiffer structure is influenced negligibly. [15]

But for this case, the adjacent building has been modeled separately and the required separation distance between the adjacent buildings has been calculated according to Ethiopian Building Code Standard (EBCS) and International Building Code (IBC) by taking the displacement at the top floor of both buildings in facing direction.

However, for this thesis only the gap element (leaner spring) modeling has been used to model the adjacent buildings to show the pounding effect as sample modeling.

# 3.0 Structural Modeling and Analysis Using Finite Element Software.

## 3.1 General

In order to evaluate the Seismic gap between buildings; it is batter to model the structure in 3D with a help of engineering- software. Under this topic, there are three different cases to be studied with detailed discussion presented under the sub-topic of 3.2. All cases are modeled and analyzed using engineering software.

The finite element analysis software SAP2000 is utilized to create 3D model and run all analyses. The software is able to predict the geometric nonlinear behavior of space frames under static or dynamic loadings, taking into account both geometric nonlinearity and material inelasticity. The software accepts static loads (either forces or displacements) as well as dynamic (accelerations) actions and has the ability to perform Eigen values, Ritz-vector, nonlinear static pushover and nonlinear dynamic analyses.

### 3.1.1 Assigning loads.

After having modeled the structural components, all possible load cases are assigned. These are as follows:

#### 3.1.1.1 Gravity loads

- Gravity loads on the structure include the self weight of beams, columns, slabs, walls and other permanent members. The self weight of beams and columns (frame members) is automatically considered by the program itself.
- The slabs (area sections) and wall loads have been calculated manually.
- Live loads have been assigned as uniform area loads on the slab elements as per EBCS-2 1995
    - Live load on roof 2 KN/m2
    - Live load on all other floors depends on the functional use of the floor.

#### 3.1.1.2 Earthquake lateral loads

Earthquake loads can be considered in the design in two different ways. These are;-

- By calculating the design lateral loads at different floor levels corresponding to fundamental time period and are applied to the model.
- Program calculation method by SAP2000.

In our case the second alternative approach was used. Since the software is capable to consider earthquake load automatically by giving the necessary data's.

## 3.2 Modeling and analyses

Building of adjacent structures with insufficient gap distances is one of the most critical mistakes. In EBCS [8], minimum gap distance between adjacent buildings is given. However, adjacent buildings are still been constructing without considering gap distances. *It is noted that one of the critical objectives of design codes is to restrict the "maximum risk" to a socially acceptable level.*

In this case there are six buildings under three cases, which are constructed and being constructed in the main street of Addis Ababa city. Some of these building have been constructed at different times and some of them are under construction. Buildings are modeled as elastic moment resisting frames with a 5% damping ratio using SAP2000 with Response-spectrum analysis.

Response-spectrum analysis is a statistical type of analysis for the determination of the likely response of a structure to seismic loading. Response- spectrum analysis seeks the likely maxi mum response to these equations rather than the full time history. The earthquake ground acceleration in each direction is given as a digitized response-spectrum curve of pseudo- spectral acceleration Response versus period of the structure.

### 3.2.1 Response-Spectrum Function

The response-spectrum curve for a given direction is defined by digitized points of pseudo-spectral acceleration response versus period of the structure. The shape of the curve is given by specifying the name of a Function. All values for the abscissa and ordinate of this Function must be zero or positive.

For the Seismic pounding effect between adjacent buildings, response spectrum analysis is carried out using the spectra for medium soil as per EBCS-8, 1995.
The spectral acceleration coefficient (Sa/g) values are calculated as follows.

For soil type B,

$$Sa/g = 1 + 10T, \quad (0.00 \leq T \leq 0.10),$$
$$= 2.50, \quad (0.10 \leq T \leq 0.60) \qquad \text{--- 3.1}$$
$$= 1.5/T, \quad (0.60 \leq T \leq 4.00)$$

(T= time period in seconds)

The values of the spectral acceleration coefficient (Sa/g) for the time periods of 0.00 to 4.00 seconds calculated as per the above equations and the plot of spectral acceleration coefficient (Sa/g) Vs. Period are as shown.

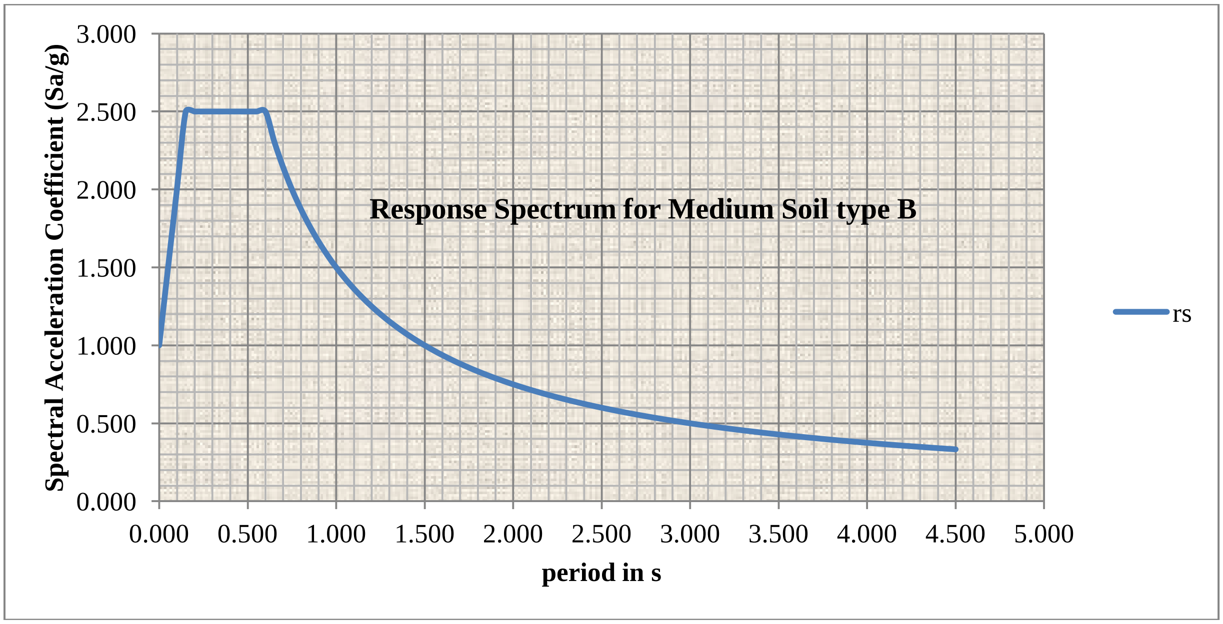


Figure 20: Defining response spectrum function (Sa/g) Vs. Period

### 3.2.2 Response spectrum analysis in SAP 2000

The procedures are mentioned as the following:-

- Defining quake loads under the load type ‘quake’ and naming it appropriately.
- Defining response spectrum function as per required building standards. The values of Sa/g Vs. T can be linked in the program in the form of a data file.

- Modifying the quake analysis case with the appropriate analysis case type, applied loads and scale factors.
- Running the analysis.

For all cases, the required material properties like mass, weight density, modulus of elasticity, shear modulus and design values of the material used can be modified as per requirements. Beams and column members have been defined as 'frame elements' with the appropriate dimensions and reinforcement.

The slabs are designed manually and the loads which transfer to the beam are calculated according to EBCS-2 1995. And this calculated load is loaded as uniformly distributed force for the solid slab and as point load with 400mm spacing for the case of ribbed slab.

During the modeling process the necessary data and the whole dimension of the structural element (frame, structural wall, slab, - - - etc) and non- structural element (wall - - -) has taken as per built or as per design. As mentioned above there are three cases and will be discussed one by one as follow:-

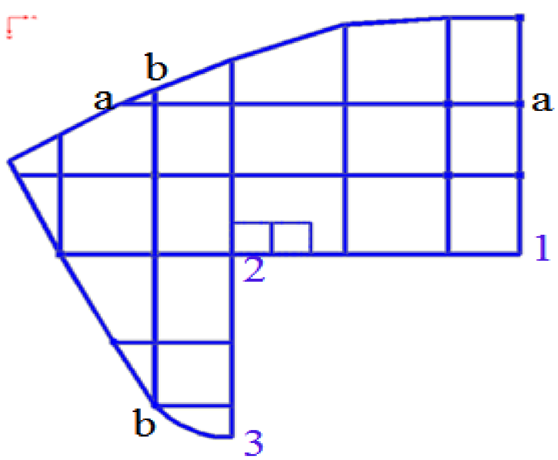
### 3.2.3 Modeling and analyses of the case -I

In the first case it will be discussed the modeling and analysis of the two adjacent buildings which are found at the main street of cross-avenue to Bole airport road. For the purpose of discussion, as shown in fig. 21 below, the two buildings are designated by the symbols A and B.

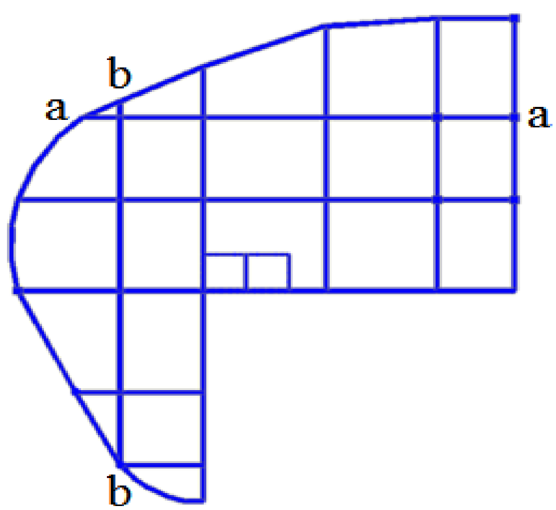
These adjacent buildings are not aligned floor-to-floor; that means, the heights of the storey levels of the two structures are not equal (Figure 21). In this very common case the slabs of the diaphragms of one structure hit the columns of the other structure at a point within the deformable height. This phenomenon is especially intense at the contact point of the upper storey level of the short stiffer structure with the corresponding column of the tall building for adjacent buildings with different total building height. The separation distance between building A and building B can be approximated as 40cm. the aim of this paper is to check that this separation distance is sufficient or not.


Figure 21: Adjacent building found at Bole

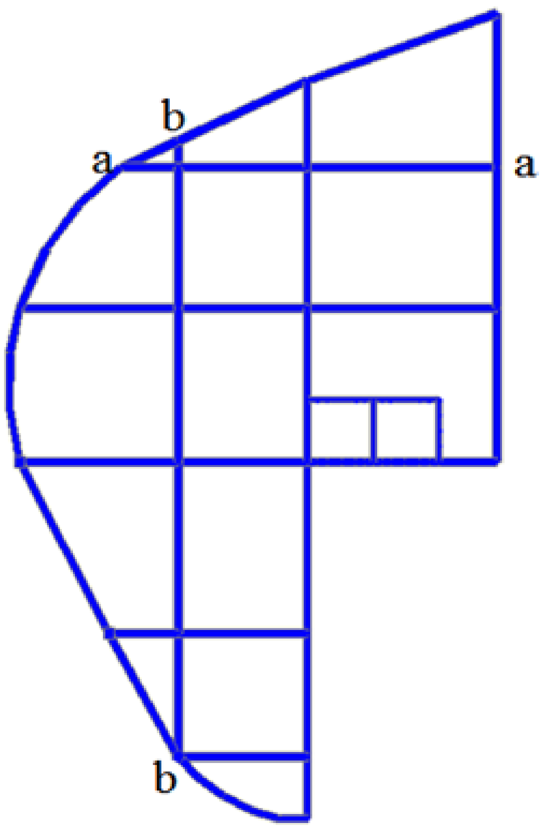
Building A has a total of 11 stories (G+9-1) with the total building height of 33.8m and storey height of 3m (in average). Similarly building B have a total of 11 stories (G+9-1) with the total building height of 32m and story height of 3m (in average).



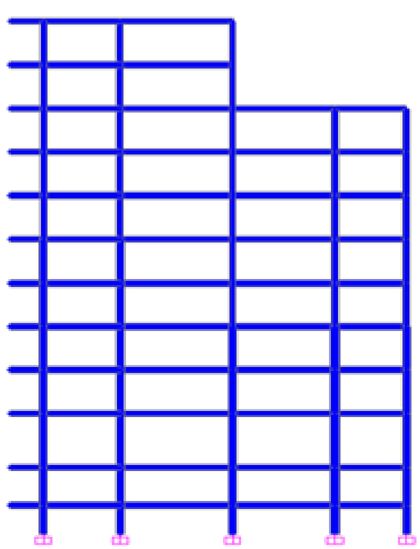

a) Floor plan of basement – 1st floor



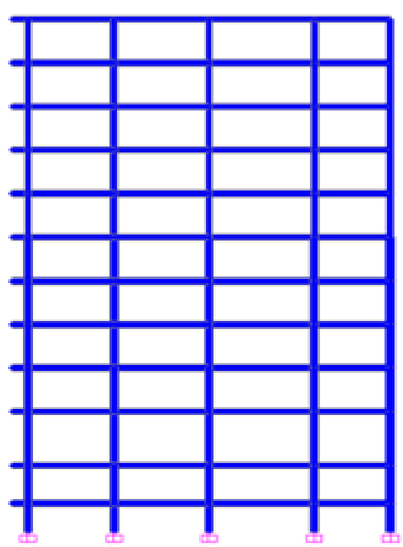

b) Floor plan of 2nd -7th floor



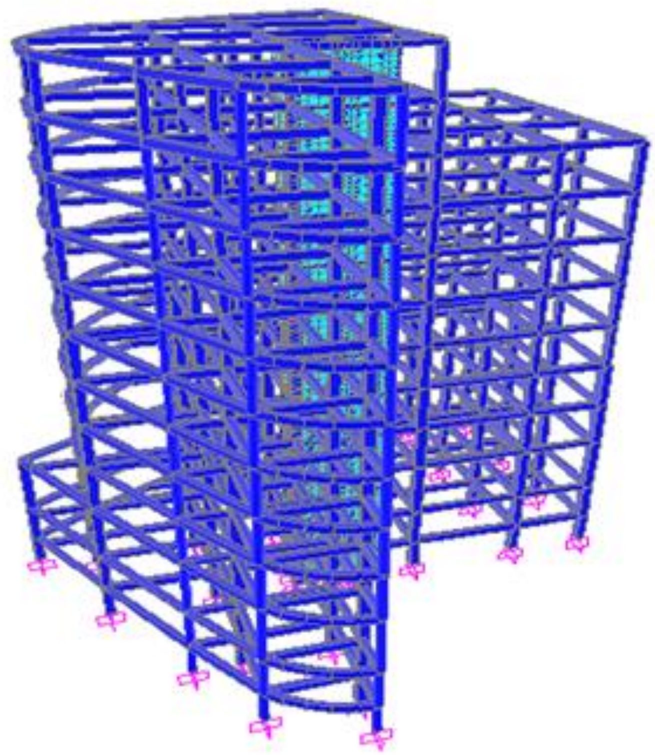

c) Floor plan of 8th-9th floor



d) Elevation along a-a



e) Elevation along b-b

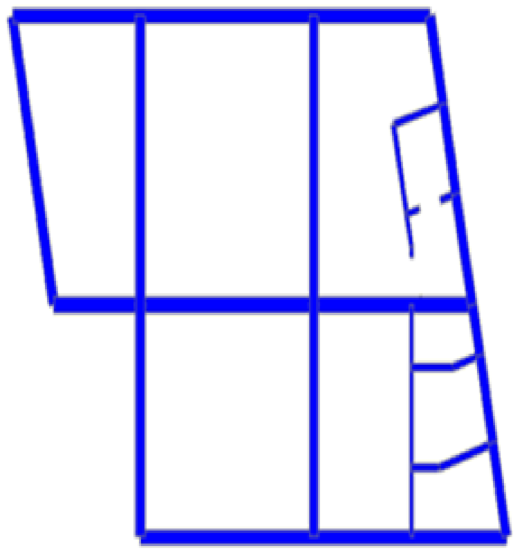


f) 3D model of building A

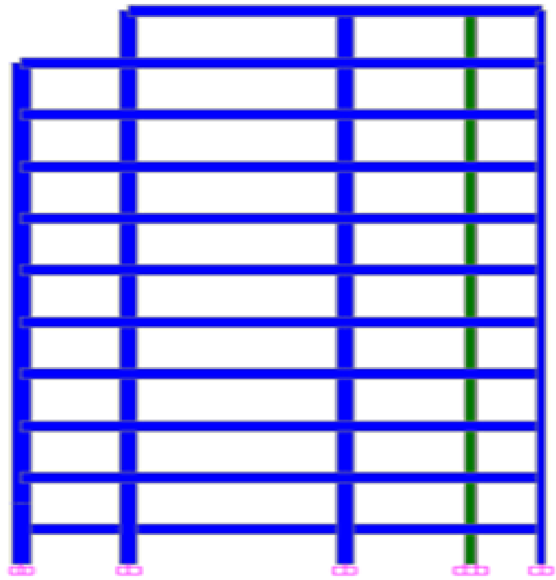
Figure 22: floor plans, elevations and 3D view of the building A

Building A is currently under construction and after the completion of the construction it will be used for hotel service. And it is modeled using SAP2000 as shown in the figure 22 above. As shown from the above figure the model consists of only the frame system and the structural walls. The floor slab is designed and calculated manually according to Ethiopian Building Code Standard Two (EBCS-2-1995) and the loads that transfer to the beam is loaded as a point load with the spacing of 400mm apart since the slab is ribbed slab. In addition to this the load of the non-structural parts of the building has been calculated and loaded as uniformly distributed load.

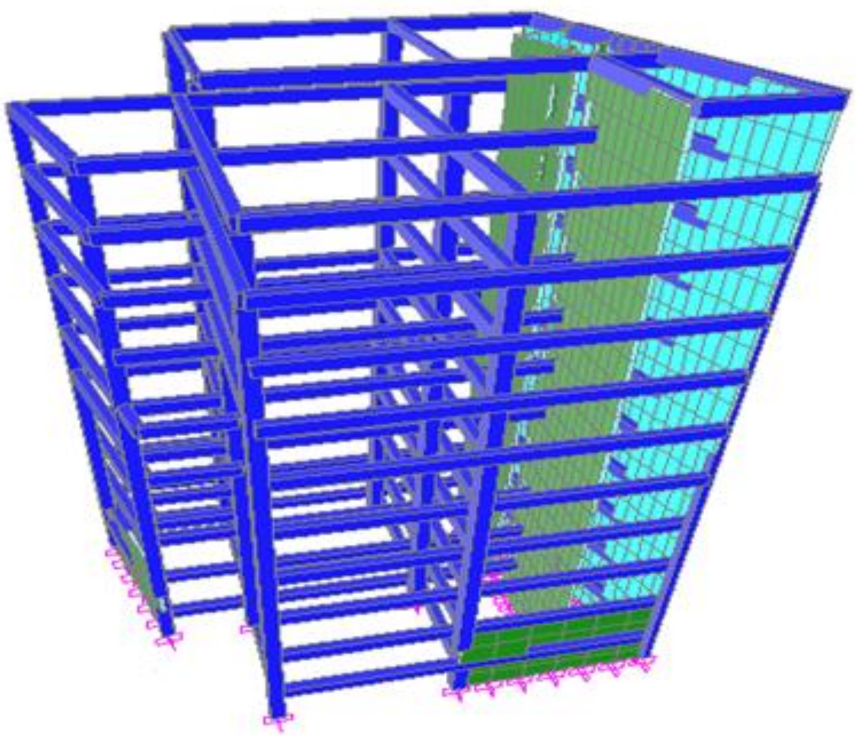
The lines 1-2 and 2-3 are the lines that faced to the adjacent building, which is building B. this indicate that building A and building B are faced in two sides.
Similarly, building B is modeled with SAP2000 as shown in the figure 23 bellow.



a) Typical floor plan



b) Typical elevation



c) 3D model of the building B

Figure 23: Floor plans, elevations and 3D view of the building B

Building B was constructed many years ago and it has been giving hotel service. And this building modeled in a manner as building A was modeled, which consist of only the frame system and the structural wall. The floor slab is designed and calculated manually according to Ethiopian Building Code Standard Two (EBCS-2-1995) and the load is loaded as a uniformly distributed load since the slab is solid slab. In addition to this the load of the non-structural element of the building has been calculated and loaded as uniformly distributed load.

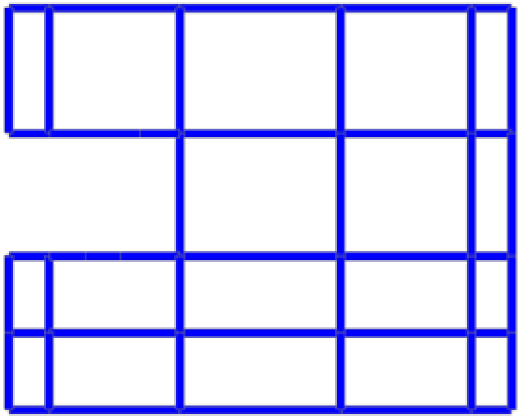
### 3.2.4 Modeling and analyses of the case -II

In this case, it will be discussed the modeling and analysis of the two adjacent buildings which are found at CMC along the main road of Megenagna to Hayat. For the purpose of discussion, as shown in figure 24 below, the two buildings are assigned as building C and building D.

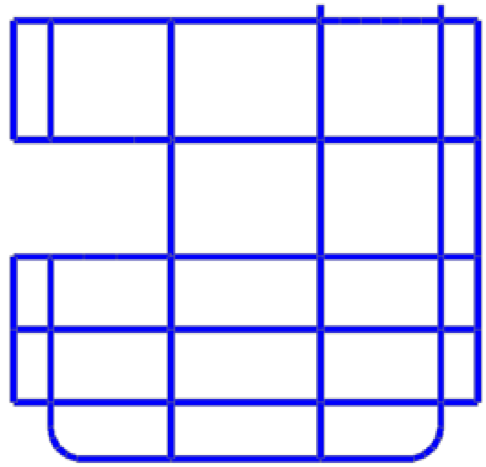

Figure 24: Adjacent building found at CMC

As shown in figure 24, the two buildings were constructed with no separation distance. They have equal floor level (they have been aligned to floor-to-floor).

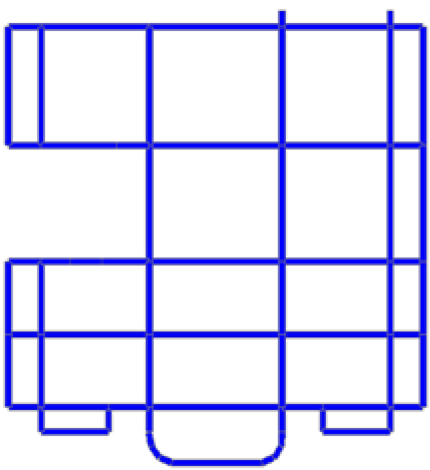
Building C has a total of 6 stories (G+5) with the total building height of 18m with storey height of 3m. Similarly building D has a total of 10 stories (G+8-1) with the total building height of 33.6m and storey height of 3m (in average).



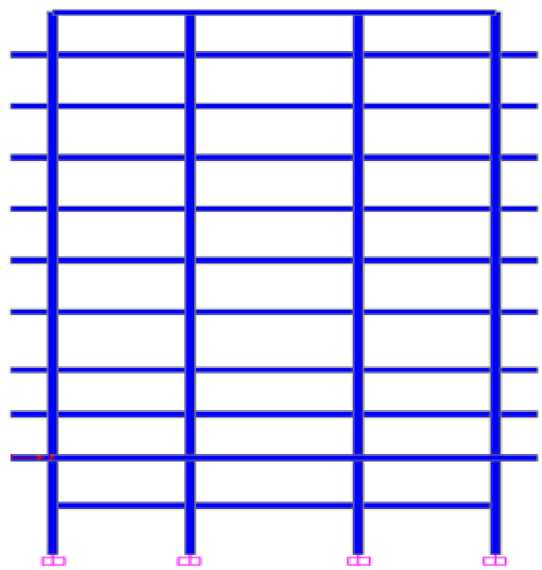
a) Ground floor plan



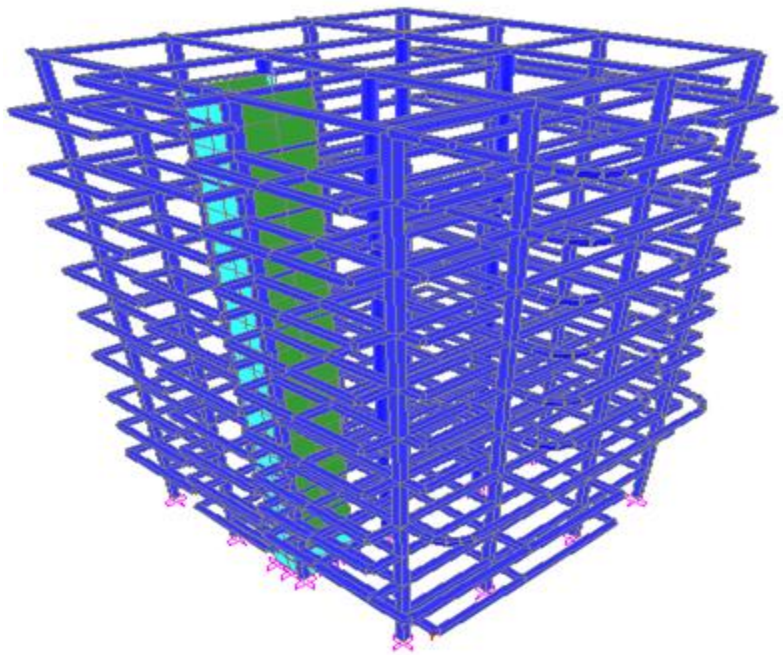
b) 1st and 2nd floor plan



C) 3rd- 8th floor plan



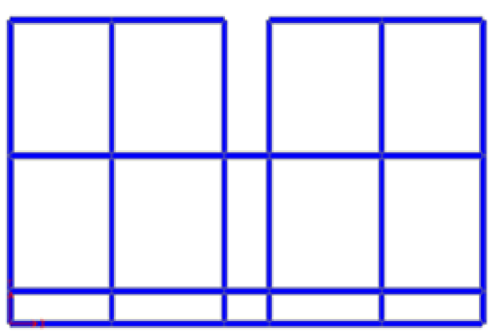
d) Typical elevation plan



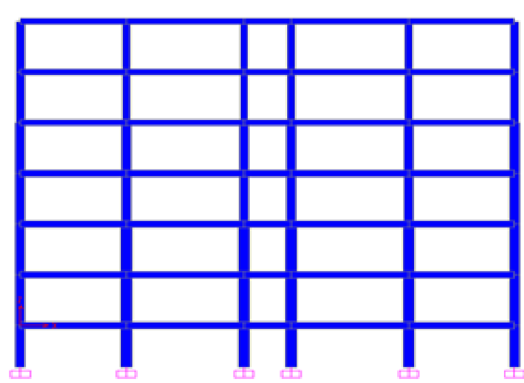
e) 3D model of building D

Figure 25: Floor plans, elevations and 3D view of the building D

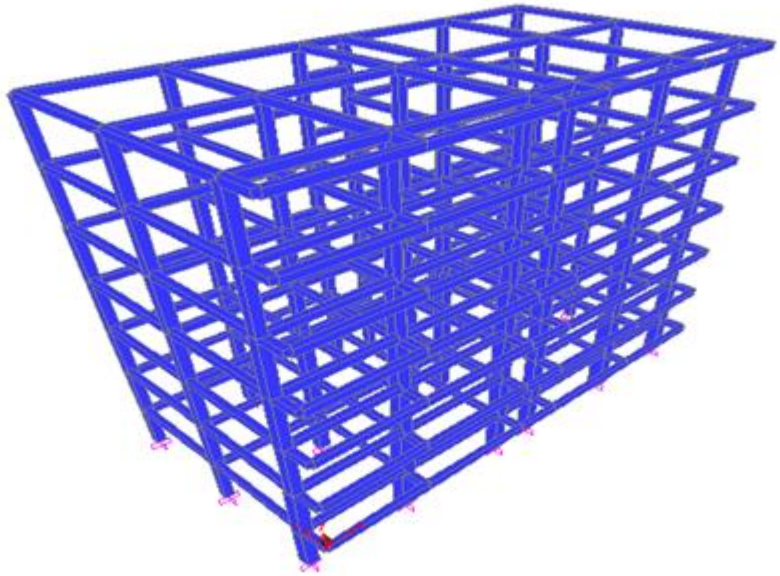
Building D is a mixed used building and it is modeled with the help of SAP2000 as shown in the figure 25 above. From the figure above, the model consists of only the frame system and the structural walls. The floor slab is designed and calculated manually according to Ethiopian Building Code Standard-Two (EBCS-2-1995) and this slab load is loaded as a point load with the spacing of 400mm apart since the slab is ribbed slab. In addition to this the load of non-structural element of the building has been calculated and loaded as uniformly distributed load.



a) Typical floor plan



b) Typical elevation



c) 3D model of the building

Figure 26: Floor plans, elevations and 3D view of the building C

Building C is a mixed used building and medaled in similar manner with buildings mentioned above which consist of only the frame system. The floor slab is designed and calculated manually based on Ethiopian Building Code Standard-Two (EBCS-2-1995) and the load is loaded as a uniformly distributed load since the slab is solid slab. In addition to this the load of the non-structural element of the building has been calculated and loaded as uniformly distributed load.

### 3.2.5 Modeling and analyses of the case -III

In this case, it will be discussed the modeling and analysis of the two adjacent buildings which are found around Tewodrous Avenue along Cherchel Street. For the purpose of discussion, as shown in figure 27 below, the two buildings are assigned as building E and building F.

Figure 27: Adjacent building (before the adjacent building constructed) found around Tewodrouse Avenue

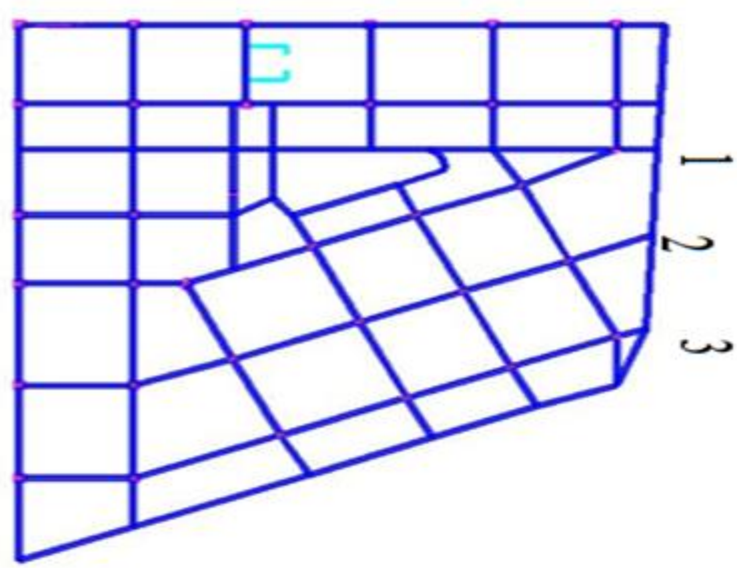

Figure 28: Adjacent building (during the constructed of adjacent building) found around Tewodrouse Avenue

In this case the adjacent buildings are not aligned floor-to-floor; that means, the heights of the storey levels of the two structures are not equal see figure 28. In this very common case the floor of one structure hit the columns of the other structure at a point within the deformable height. The separation distance between the two adjacent buildings is estimated to be about ten centimeter (10 cm).

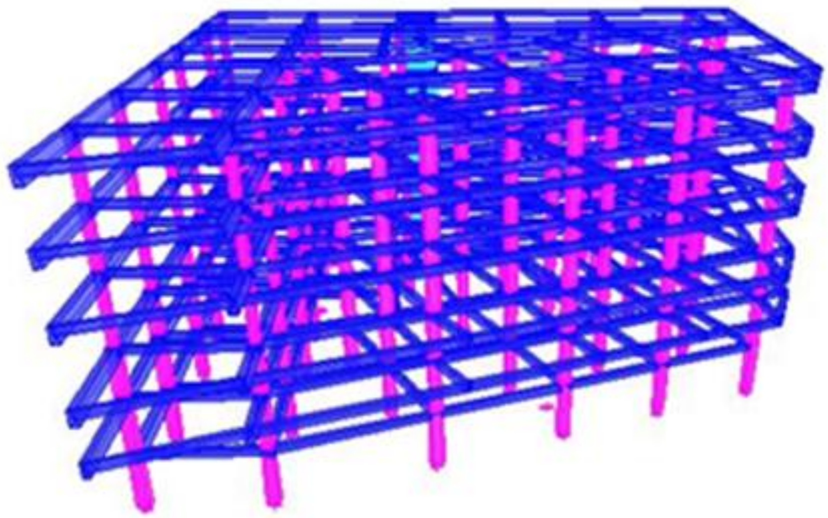
The contacts between these buildings are only up to storey four. This is because of the set back at this story in both buildings as shown in the figure 27 and 28

Building E is a mixed used building and it is modeled with the help of ETABS V9.1 as shown in the figure 29 bellow. From the figure bellow, the model consists of only the frame system and the structural walls. The floor slab is designed and calculated manually according to Ethiopian Building Code Standard-Two (EBCS-2-1995) and since the slab is ribbed slab; slab load is loaded as a point load with the spacing of 400mm apart. In addition to this the load of non-structural element of the building has been calculated and loaded as uniformly distributed load.

There was design modification after it has been started doing this paper. Based on the modified design it was separated in to two by using construction joint. The construction has also got two phases; the first phase has a total of five stories and is under construction as shown in figure 28 above, and it becomes the final level or height.




a) Floor plan

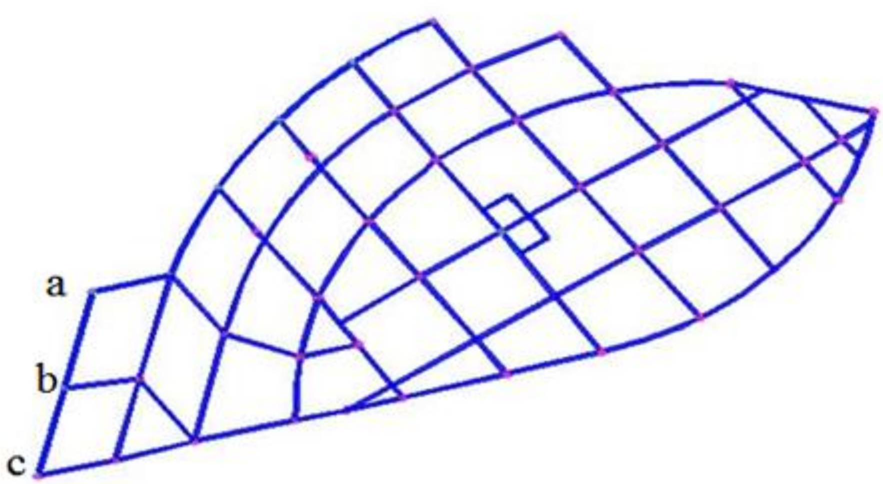


b) 3D view

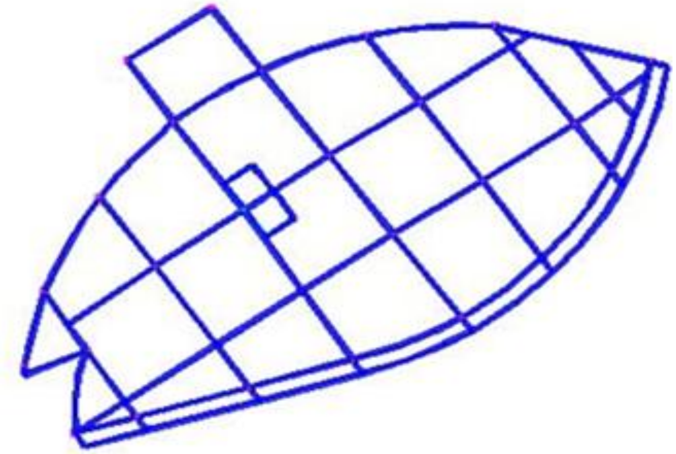
Figure 29: Floor plan and 3D of building E (first phase construction)

The points (1, 2 and 3 see figure 29a) are the expected contact points if the separation distance is inadequate with adjacent building (building F). The second part of the building has a total of ten stories. This part of the building has no contact with adjacent building at all.

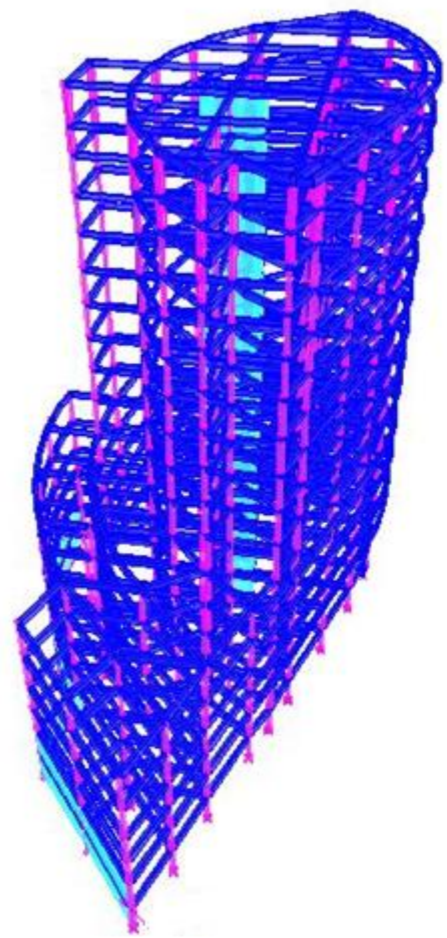
Building F is also mixed used building and it is modeled with the similar way with buildings mentioned above as shown in the figure 30 bellow.




a) Floor plan from ground-3rd floor



b) Floor plan from above 3rd floor



c) 3D view

Figure 30 Floor plan and 3D view of building F

The points (a, b and c see figure 30a) are expected contact points if the separation distance is inadequate with adjacent building (building E) corresponding to the point (1, 2 & 3 see fig. 29). This building has been started service around a year ago. It has a total of fifteen stories but at the adjacent building side, it has a set back at storey five. Due to this the building is closed to building E only up to third floor (about 12m above ground level).

# 4 .0 Results and Discussions

## 4.1 General

SAP2000 is used to compute the response of buildings (A-F) which are mentioned above with response spectrum analysis. Results from Response Spectrum analysis are observed for the period and Displacements of the joints to determine the seismic pounding gap (separation distance) between adjacent structures of two models.

The two results (period and displacement of the joint) have been taken to identify that whether the two buildings oscillate (vibrate) in-phase or out- of-phase and also to check the separation distance between them is sufficient or not, if the building are out-of – phase.

The separation distance between the adjacent buildings could be calculated according to Ethiopian Building Code Standard and International Building code.

The detail discussion will be present hereafter for the three cases one by one.

## 4.2 Result and Discussion for the Three Cases

### 4.2.1 Result and Discussion of Case-I

It has been discussed the modeling and analysis of the adjacent building in the previous topic (chapter); but here it will be interpret and discussed the result of this analysis.

Table 4: Response Spectrum Modal Information of building A

| Step Type | Mode-1 | Mode-2 | Mode-3 | Mode-4 | Mode-5 | Mode-6 | Mode-7 | Mode-8 | Mode-9 | Mode-10 | Mode-11 | Mode-12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period (sec) | 4.403 | 3.770 | 2.976 | 2.610 | 1.635 | 1.551 | 1.436 | 1.313 | 1.301 | 1.229 | 1.020 | 0.989 |

Table 5: Response Spectrum Modal Information of building B

| Step Type | Mode-1 | Mode-2 | Mode-3 | Mode-4 | Mode-5 | Mode-6 | Mode-7 | Mode-8 | Mode-9 | Mode-10 | Mode-11 | Mode-12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period (sec) | 3.175 | 2.690 | 1.441 | 1.107 | 1.078 | 0.958 | 0.933 | 0.852 | 0.744 | 0.693 | 0.685 | 0.653 |

Based on the results in the table 4 and table 5 above the two buildings (building A and B see figure 31and 32 bellow) have different period as it is observed from the first twelve mode of the two buildings of the SAP output; as a result of this, it is possible to say that, the vibrations of these adjacent buildings are out-of-phase. This indicates that the calculation of the separation distance between the two buildings is necessary to check whether the gap in between is sufficient or not.

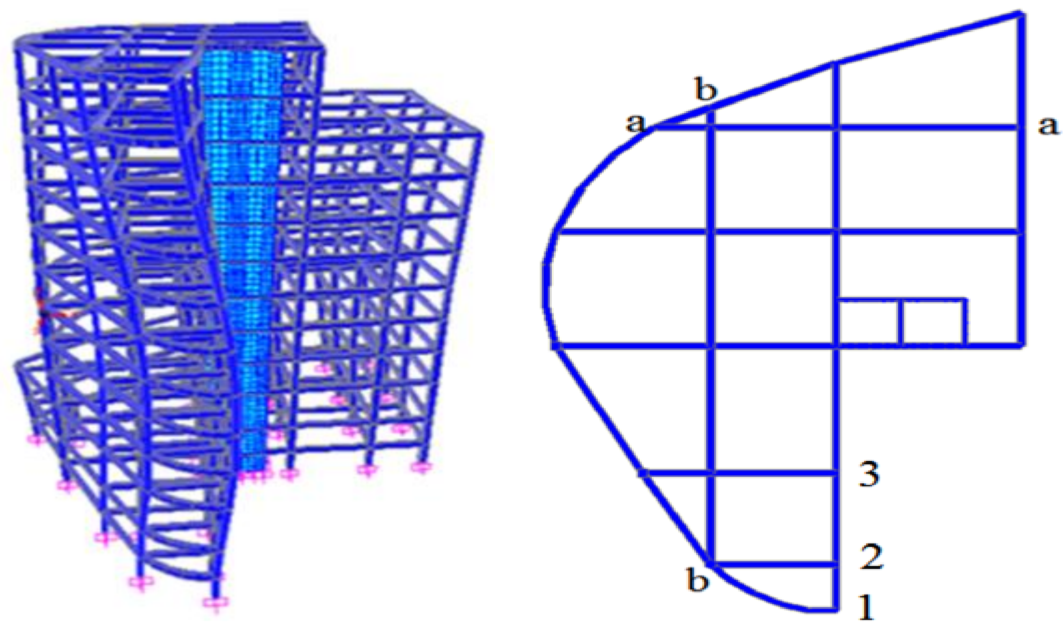



Figure 31: The deformed shape of building A (after analysis at 5th mode)

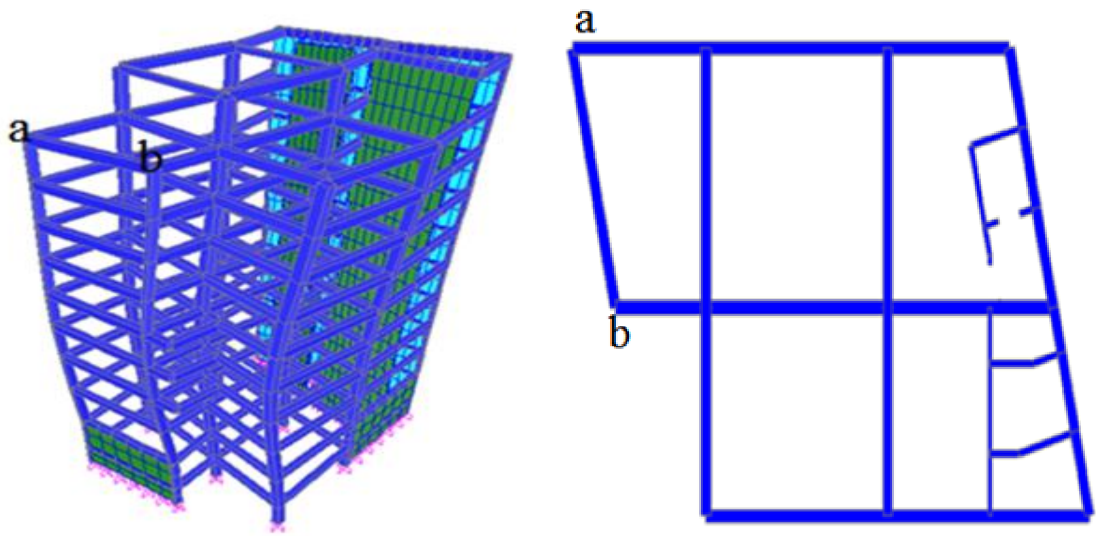



Figure 32: The deformed shape of building B (after analysis at 5th mode)

Table 6: Joint Displacements for building A

| Joint | Output Case | Displacement($d_{eA}$) cm |
|---|---|---|
| 1 | EQX-SPECTRUM | 4.62 |
| 2 | EQX-SPECTRUM | 4.62 |
| 3 | EQX-SPECTRUM | 4.62 |

Table 7: Joint Displacements for building B

| Joint | Output Case | Displacement($d_{eB}$) cm |
|---|---|---|
| **a** | EQX-SPECTRUM | **14.86** |
| **b** | EQX-SPECTRUM | **14.83** |

The joint displacement at the top floor of building A and building B (in facing side) are given in the above table (table 6 and table 7) respectively. Using these results, the minimum required separation distance (d) between the two buildings calculated and summarized in the table 8 bellow (according to EBCS and IBC).

Table 8 Minimum required separation distance

<table>
<tr><th rowspan="2">Joint</th><th rowspan="2">$d_{eA}$<br>(cm)</th><th rowspan="2">$d_{eB}$<br>(cm)</th><th rowspan="2">$d_{sA}$/ ($\delta_{MA}$)<br>(cm)</th><th rowspan="2">$d_{sB}$/ ($\delta_{MB}$)<br>(cm)</th><th colspan="2">Min. required sep. distance (d) according to</th><th rowspan="2">Existing Separation Distance (cm)</th></tr>
<tr><th>EBCS (cm)</th><th>IBC($\delta_{MT}$) (cm)</th></tr>
<tr><td>**a/2**</td><td>4.62</td><td>14.86</td><td>9.24</td><td>29.72</td><td>38.96</td><td>31.12</td><td rowspan="2">40</td></tr>
<tr><td>**b/3**</td><td>4.62</td><td>14.84</td><td>9.24</td><td>29.68</td><td>38.92</td><td>31.09</td></tr>
</table>

Since the floor elevation of the adjacent building is not the same, the result did not factor with reduction factor of 0.7. As a result the minimum required separation distance between the adjacent building (building A and building B) based on EBCS and IBC must be greater than 38.96 cm and 31.12 cm respectively.

### 4.2.2 Result and Discussion of Case-II

The modeling and analysis of these adjacent building was discussed under 3.2.2 (Modeling and analyses of the case –II); but here it will try to discussed the result of this analysis.

Table 9: Response spectrum modal information (period) of building C

| Step Type | Mode-1 | Mode-2 | Mode-3 | Mode-4 | Mode-5 | Mode-6 | Mode-7 | Mode-8 | Mode-9 | Mode-10 | Mode-11 | Mode-12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period (sec) | 2.364 | 2.026 | 1.907 | 1.148 | 1.064 | 0.776 | 0.748 | 0.707 | 0.671 | 0.665 | 0.635 | 0.590 |

Table 10: Response spectrum modal information (period) of building D

| Step Type | Mode-1 | Mode-2 | Mode-3 | Mode-4 | Mode-5 | Mode-6 | Mode-7 | Mode-8 | Mode-9 | Mode-10 | Mode-11 | Mode-12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period (sec) | 4.479 | 2.793 | 2.389 | 1.711 | 1.438 | 1.115 | 1.035 | 1.011 | 0.941 | 0.900 | 0.804 | 0.798 |

Based on the results in the table 9 and table 10 above, the two buildings (building C and D see figure 33 and 34 bellow) have different periods as shown in the above tables which shows the first twelve modes of the two buildings; in order to this, the adjacent buildings vibrations out-of-phase. This indicates that the calculation of the separation distance between the two buildings is necessary to check whether the gap in between is sufficient or not based on EBCS-8.

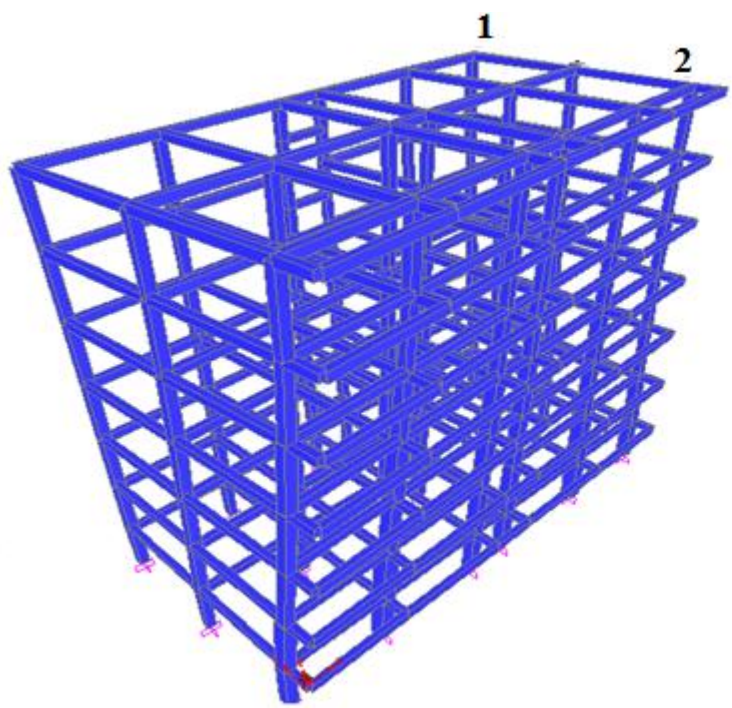



Figure 33: The deformed shape of building C (after analysis at 2$^{nd}$ mode)

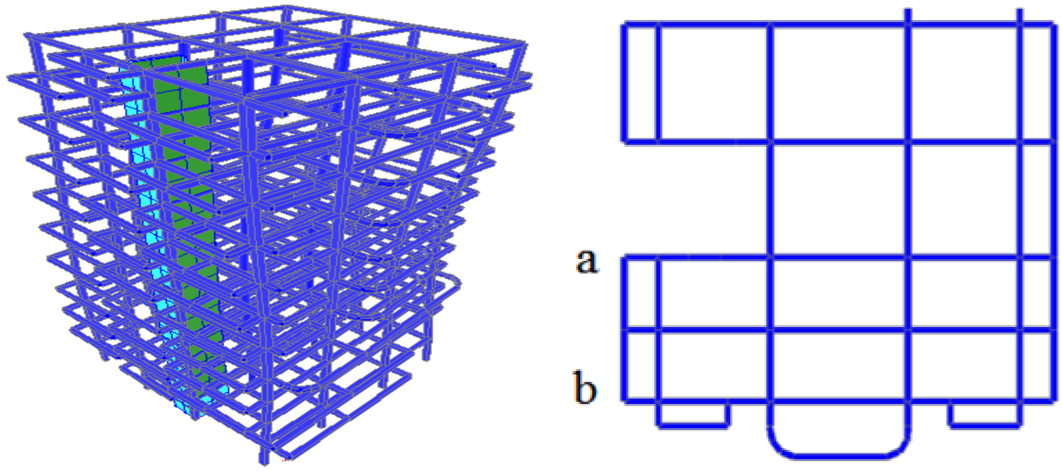



Figure 34: The deformed shape of building D (after analysis at $4^{th}$ mode)

Table 11: Joint displacements for building C

| Joint | Output Case | Displacement($d_{eC}$) cm |
|---|---|---|
| 1 | EQX-SPECTRUM | 11.73 |
| 2 | EQX-SPECTRUM | 12.96 |

Table 12: Joint displacements for building D at the top floor of $5^{th}$ story

| Joint | Output Case | Displacement($d_{eD}$) Cm |
|---|---|---|
| a | EQX-SPECTRUM | 10.67 |
| b | EQX-SPECTRUM | 9.93 |

The joint displacement at the top floor of building C and at the top floor of $5^{th}$ story of building D (in facing side) are given in the above table (table 11 and table 12) respectively. Using these results, the minimum required separation distance (d) between the two buildings is calculated and summarized in the table 13 bellow (according to EBCS and IBC).

Table 13 Minimum required separation distance

| Joint | $d_{eC}$ (cm) | $d_{eD}$ (cm) | $d_{sC}$/($\delta MC$) (cm) | $d_{sD}$/($\delta MD$) (cm) | Min. required sep. distance (d) according to | |
|---|---|---|---|---|---|---|
| | | | | | EBCS (cm) | IBC($\delta_{MT}$) (cm) |
| a/1 | 11.73 | 10.67 | 23.47 | 21.34 | 44.81 | 31.72 |
| b/2 | 12.96 | 9.93 | 25.92 | 19.86 | 45.78 | 32.65 |

Since the floor elevation of the adjacent building is in the same height, the calculated minimum required separation distance between the adjacent building (building C and building D) should be factor with reduction factor of 0.7 based on EBCS. Hence the factored required separation distance is given in the table 14 bellow.

Table 14 Factored minimum required separation distance

| Joint | deC (cm) | $d_{eD}$ (cm) | $d_{sC}$ (cm) | $d_{sD}$ (cm) | $d_{sC}$ (cm) ( factored) | $d_{sD}$ (cm) ( factored) | Min. required sep. distance (d) according to | | Existing Separation Distance (cm) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | EBCS(factored) (cm) | IBC($\delta_{MT}$) (cm) | |
| a/1 | 11.73 | 10.67 | 23.47 | 21.34 | 16.43 | 14.94 | 31.37 | 31.72 | 0.0 |
| b/2 | 12.96 | 9.93 | 25.92 | 19.86 | 18.14 | 13.90 | 32.04 | 32.65 | |

Based on this result the minimum required separation distance to avoid or minimized pounding according to EBCS and IBC is 32.04 cm and 32.65 cm respectively.

### 4.2.3 Result and Discussion of Case-III

The modeling and analysis of these adjacent building (building E and building F) was discussed under 3.2.3 (Modeling and analyses of the case –III); but here the result of this analysis will be discussed. The next table shows the period of the first twelve modes of the adjacent buildings.

Table 15: Response spectrum modal information (period) of building E

| Step Type | Mode-1 | Mode-2 | Mode-3 | Mode-4 | Mode-5 | Mode-6 | Mode-7 | Mode-8 | Mode-9 | Mode-10 | Mode-11 | Mode-12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period (sec) | 2.457 | 1.937 | 1.754 | 1.439 | 1.269 | 0.873 | 0.815 | 0.792 | 0.789 | 0.729 | 0.690 | 0.655 |

Table 16: Response spectrum modal information (period) of building F

| Step Type | Mode-1 | Mode-2 | Mode-3 | Mode-4 | Mode-5 | Mode-6 | Mode-7 | Mode-8 | Mode-9 | Mode-10 | Mode-11 | Mode-12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Period (sec) | 5.089 | 4.931 | 3.815 | 1.839 | 1.797 | 1.693 | 1.652 | 1.519 | 1.207 | 1.173 | 1.156 | 1.147 |

Based on the results in the table 15 and table 16 above, the adjacent buildings (building E and F see figure 35 and 36 bellow) have different fundamental periods (the fundamental period of building E is 2.457 sec. and that of building F is 5.089 sec.); due to this the adjacent buildings vibrations out-of-phase. This indicates that the calculation of the separation distance between the two buildings is necessary to check whether the separation distance between those adjacent buildings is sufficient or not.

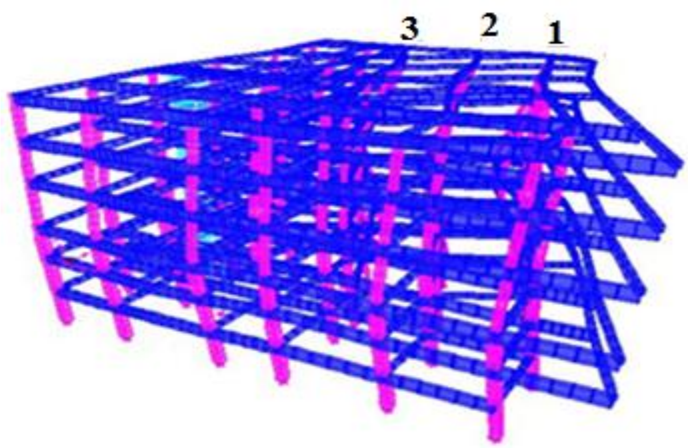



Figure 35: The deformed shape of building E (after analysis at 3rd mode)

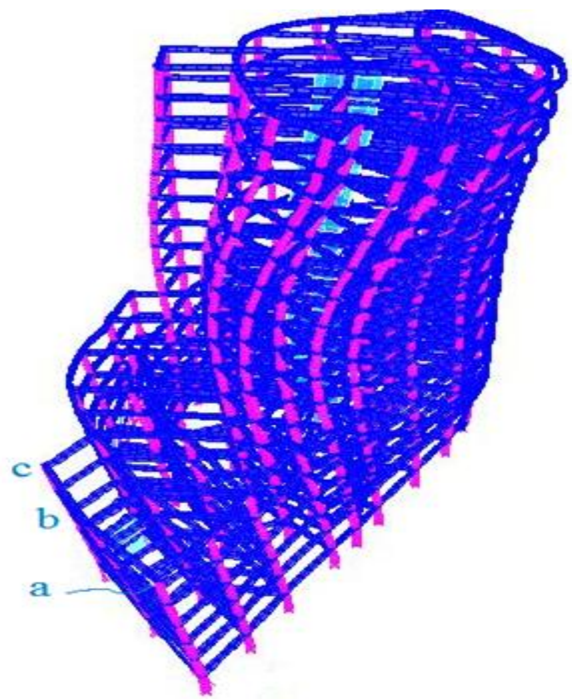



Figure 36: The deformed shape of building F (after analysis at 6th mode)

Table 17: Joint displacements for building E

| Joint | Output Case | Displacement($d_{eE}$) in cm |
|---|---|---|
| 1 | EQX-SPECTRUM | 9.86 |
| 2 | EQX-SPECTRUM | 8.16 |
| 3` | EQX-SPECTRUM | 6.66 |

Table 18: Joint displacements for building F at the top floor of 4th story

| Joint | Output Case | Displacement($d_{eF}$) in cm |
|---|---|---|
| a | EQX-SPECTRUM | 7.69 |
| b | EQX-SPECTRUM | 4.89 |
| c | EQX-SPECTRUM | 3.34 |

The joint displacement at the top floor of building E and at the top floor of $4^{th}$ storey of building F (in facing side) are given above in table 17 and table 18 respectively. Based on this results, the minimum required separation distance (d) between the adjacent buildings has been calculated and summarized in the table 19 bellow (according to EBCS and IBC).

Table 19 Minimum required separation distance

| Joint | $d_{eE}$ (cm) | $d_{eF}$ (cm) | $d_{sE}$/ ($\delta_{ME}$) (cm) | $d_{sF}$/ ($\delta_{MF}$) (cm) | Min. required sep. distance (d) according to | | Existing Separation Distance (cm) |
|---|---|---|---|---|---|---|---|
| | | | | | EBCS (cm) | IBC($\delta_{MT}$) (cm) | |
| a/1 | 9.86 | 7.69 | 19.72 | 15.38 | 35.10 | 25.00 | 10 |
| b/2 | 8.16 | 4.89 | 16.32 | 9.78 | 26.10 | 19.03 | |
| c/3 | 6.66 | 3.34 | 13.32 | 6.68 | 20.00 | 14.90 | |

Because of the difference in floor elevation between the adjacent buildings, the minimum required separation distance is the calculated value as it is without being multiplied by the reduction factor 0.7. So that the minimum required separation distance between the adjacent buildings (building E and building F) based on EBCS and IBC is 35.10 cm and 25.00 cm respectively were as the existing gap in between is 10 cm.

## 4.3 Sample Modeling to Show Pounding Effect

From those three cases, case II has been selected for sample modeling to show pounding effect. This is because; building C and building D are simpler for modeling due to their regularity in plan and also they have more contact points relative to the other. Moreover these pair buildings have aligned floor levels (equal floor level). All these make the selected case simple for modeling.

In this case the two adjacent buildings are different in total building height with aligned floor level. These buildings are modeled using gap element and with a damping coefficient of 5%. According to Jankowski (2005), for concrete-to-concrete impact, the linear viscoelastic model with k = 93,500 kN/m shows good agreement with experimental results obtained by van Mier et al. (1991) [26, 28]. This large value of K has been used to avoid overlapping and to ensure that it works nearly rigidly when the gap is closed. Thus, these values are employed in the present research.

For a comparison purpose, six frame elements (three in each building (1, 2, 3, a, b, and c)) has been selected see figure 37 below. From the analysis output, the value of shear force and bending moment has also been selected to compare it before pounding and after pounding. The value is tabulated below in tables 20 and 21.for building C and building D respectively.

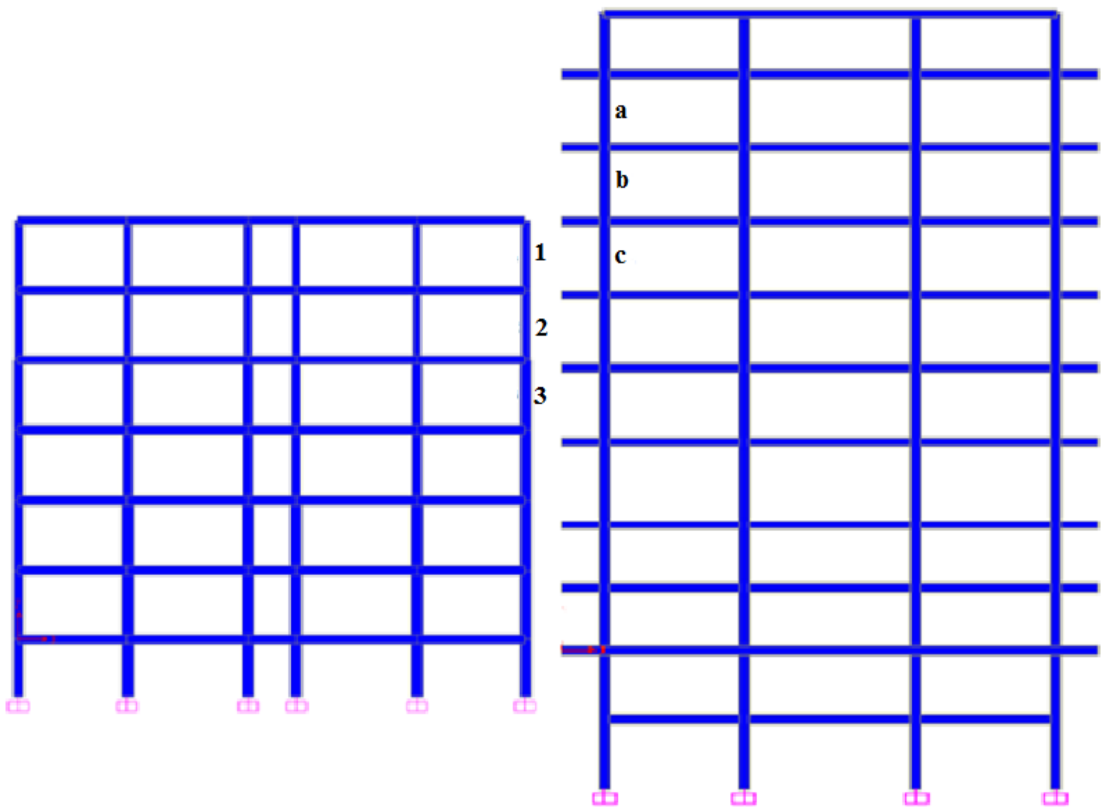



Figure 37 Selected elements for the comparison

Table 20 Shear force & bending moment before and after pounding for the shorter building

| Element | Shear Force & Bending Moment Before Pounding | | Shear Force & Bending Moment After Pounding | |
|---|---|---|---|---|
| | Shear Force (KN) | Bending Moment (KNm) | Shear Force (KN) | Bending Moment (KNm) |
| 1 | 14.67 | 48.86 | 177.36 | 352.59 |
| 2 | 61.08 | 112.79 | 193.03 | 362.22 |
| 3 | 70.06 | 179.19 | 217.13 | 430.22 |

Table 21 Shear force & bending moment before and after pounding for the longer building

| Element | Shear Force & Bending Moment Before Pounding | | Shear Force & Bending Moment After Pounding | |
|---|---|---|---|---|
| | Shear Force (KN) | Bending Moment (KNm) | Shear Force (KN) | Bending Moment (KNm) |
| a | 25.12 | 9.25 | 201.96 | 155.08 |
| b | 38.51 | 38.90 | 372.88 | 645.62 |
| c | 42.66 | 68.14 | 25.15 | 81.07 |

As shown in the above tables, significant increasing in the shear force and bending moment has been observed after pounding. Even if, the gap element has the deficiency to consider the energy dissipation during pounding of the adjacent buildings, the result from the analysis out-put can be considered as evidence that the modeling process is correct. This is because the result shows similar situation with different researches written in different journals

As it is discussed above, when two structures with different heights are adjacent, because of different dynamic properties, the shorter structure hits the adjacent one, which results in floor shearing in higher levels of impact part. When buildings are of different heights; the shorter building may act as a buttress for the taller neighbor. The shorter building receives an unexpected load while the taller building suffers from a major discontinuity that alters its dynamic response. Since neither is designed to such conditions, there is potential for extensive damage and possible collapse [27, 15].

Hence as stated above, the result from the tables above (table 20 &21) shows that the shorter building receives a large loads where as the longer building suffers with discontinuity problem. That means, as shown on the tables, the value of shear force and bending moment for long building is large above the impacting floor or above the top floor of the shorter building. So that based on the reference of those journals the modeling process has become correct. This method has been used as evidence because of the difficulties to verify the modeling process. As it has been discussed above, there are different methods of modeling to consider pounding but except the gap element (linear spring method) the rest is difficult to model using commercial software programs. And almost all of the researches (journals) at hand use modeling different from gap element. That’s why it becomes difficult to verify the modeling process.

# 5.0. Method of Mitigation

## 5.1 General

Often existing buildings do not have sufficient seismic gap between adjacent buildings to withstand the lateral displacement due to earthquake forces and resulting large damage for moderate earthquake and possibly collapses for heavy earthquake. Provision of supplemental strength in form of additional lateral force-resisting elements such as shear walls, braced or moment frames helps to reduce the lateral displacement by increasing the stiffness of the buildings. [1]

Pounding introduces impact load that have be superimposed on these caused by the ground acceleration itself. Since these impact loads from pounding are too high, the structural system has to be modified to reduce the response. Several methods have been opposed to avoid pounding induced collapse of buildings. But, in this paper it is going to be discussed only one method that is: - Increasing the stiffness of one or both buildings

## 5.2 Increasing the Stiffness of One or Both Building

Since the gap between two existing buildings usually cannot be increased, increasing the stiffness of one or both buildings may reduce the seismic deformations to the point where impact is precluded. Increasing the stiffness of the building reduces its period and decreases its displacement. An important thing to recall is that it is not possible to increase stiffness without increasing strength. This method might cause some space rearrangement of the building. To increase the stiffness of the buildings, there are different techniques such as:-[1,13,19]

- ✓ Installation of new RC shear wall
- ✓ Installation of new steel bracing
- ✓ Jacketing of structural elements

### 5.2.1 Installation of New RC Shear Wall

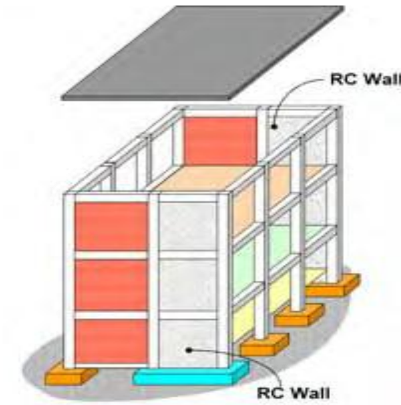
Reinforced concrete (RC) frame buildings can be provided with vertical plate-like RC walls (often called shear walls), in addition to the slabs, beams, columns and infill walls. These RC walls should be continuous throughout the building height starting at the foundation level. They act like vertically-oriented beams that carry earthquake loads downwards to the foundation. RC shear walls provide large strength and stiffness to buildings in the direction of their orientation; this significantly reduces lateral sway of the building and thereby reduces damage to structural and nonstructural components. Since RC shear walls also carry large horizontal earthquake forces, the overturning effects on them are large [1, 19].

The most common, and perhaps the most effective, method for strengthening reinforced concrete frame structures consists of the installation of new RC shear walls, as shown in Figure 38. New RC shear walls must be installed at strategic locations in order to minimize undesirable torsional effects. Also, these walls must be reinforced in such a way as to act together with the existing structure. Careful detailing and material selection are required to ensure an effective connection between the new and existing structure. The addition of shear walls substantially alters the force distribution in the structure under lateral load, and thus normally requires strengthening of the foundations. Figure 39 shows a retrofit concept for RC frames based on the installation of new shear walls [1, 19].

In some cases, installation of new reinforced concrete shear walls is combined with the column jacketing, as shown in Figure 40. Jacketing also has a beneficial effect of increasing the strength and ductility of existing reinforced concrete columns. This technique is usually implemented when it is not possible to achieve an effective connection between the new and the existing structure using the steel dowels.

In general, addition of new reinforced concrete shear walls provides a better option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures. Moreover, symmetrical placement of shear walls along the building perimeter will ensure the best earthquake performance [1].

Figure 38 Installation of new shear walls (source: WHE-2006-03, 2006).



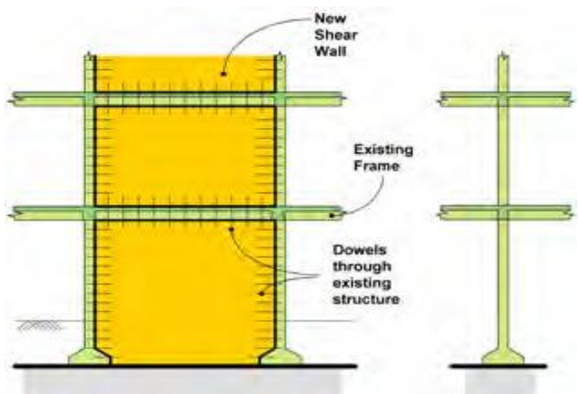




Figure 39 Installation of new RC shear walls in an existing RC frame building– note dowels provided to tie the new and the existing structure (source: WHE-2006-03, 2006).

Figure 40 Retrofit of existing RC building using new RC shear walls and jacketing of the existing columns after the 2003 Boumerdes (Algeria) earthquake (photo: M. Farsi; drawing courtesy of CTC Algiers). Addition of new structural elements.

### 5.2.2. Installation of Steel Bracing

Steel diagonal braces can be added to existing concrete frames. Braces should be arranged so that their center line passes through the centers of the beam-column joints as shown in figure 41 below.

The bracing methods adopted fall into two main categories, namely (i) external bracing and (ii) internal bracing. In the external bracing system, existing buildings are retrofitted by attaching a local or global steel bracing system to the exterior frames. Sekiguchi, et al., Del Vall Calderon, et al. and Badoux and Jirsa have reported on practical examples of retrofitting carried out using this method. Scaled model testing was also reported by Bush et al. Architectural concerns and difficulties in providing appropriate connections between the steel bracing and RC frames are two of the shortcomings of this method [13, 19].

In the internal bracing method, the buildings are retrofitted by incorporating a bracing system inside the individual units or panels of the RC frames. The bracing may be attached to the RC frame either indirectly or directly. In the indirect internal bracing, a braced steel frame is positioned inside the RC frame. As a result, the transfer of load between the steel bracing and the concrete frame is achieved indirectly through the steel frame. Successful retrofits of existing buildings by indirect internal bracing using different forms of X, V and K concentric and eccentric braces within steel frames have been reported. Instead of using a steel frame, Hjelmstad et al. jacketed the RC columns with steel plates and connected the brace to the jackets. Tagawa et al. also carried out model testing on Kbraced frames. They concluded that the combined capacity of the RC frame and the braced steel frame may be assumed as the direct sum of the capacities of the individual frames. As Coated in [13].

Generally, installing the new steel braces can be provided to increase earthquake resistance of these buildings. Figure 42 illustrates a retrofit example from a recent test in Japan.

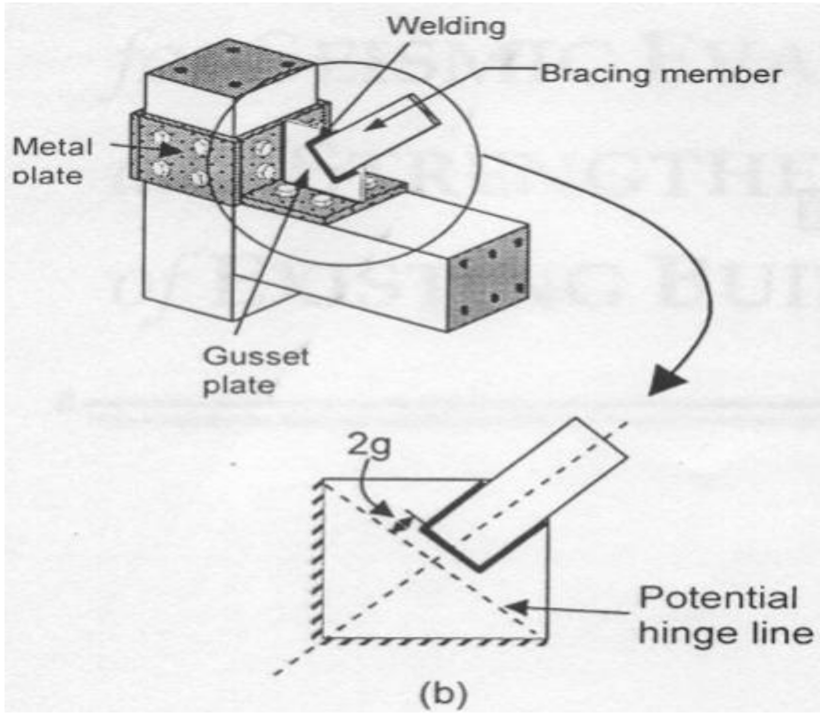



Figure 41 (a) Brace connection details (b) Gusset base connection detail

Figure 42 Retrofit of RC frames with steel braces – shake-table testing at E-Defence, Japan:

### 5.2.3. Jacketing of structural elements

A method in which a concrete column or beam is covered with a steel or concrete jacket Jacketing consists of installing new steel reinforcement bars (lateral ties and vertical bars) in order to increase strength and ductility of existing concrete members (usually columns), as shown in Figure 43. As a result of the jacketing, the column cross section is also enlarged. When new ties are installed in the beam-column joint region, the existing concrete in the joint region must be carefully removed. Figures 44 and 45 show the jacketing of RC frames in Colombia.[1]

Concrete jackets are applied to columns and walls for all or some of the following purposes:

- increasing the bearing capacity,
- Increasing the flexural and/or shear strength,
- Increasing the deformation capacity,
- Improving the strength of deficient lap-splices.

The thickness of the jackets should be such as to allow for placement of both longitudinal and transverse reinforcement with an adequate cover. **Jacketing must be provided continuously through the floor slabs in order to be effective**

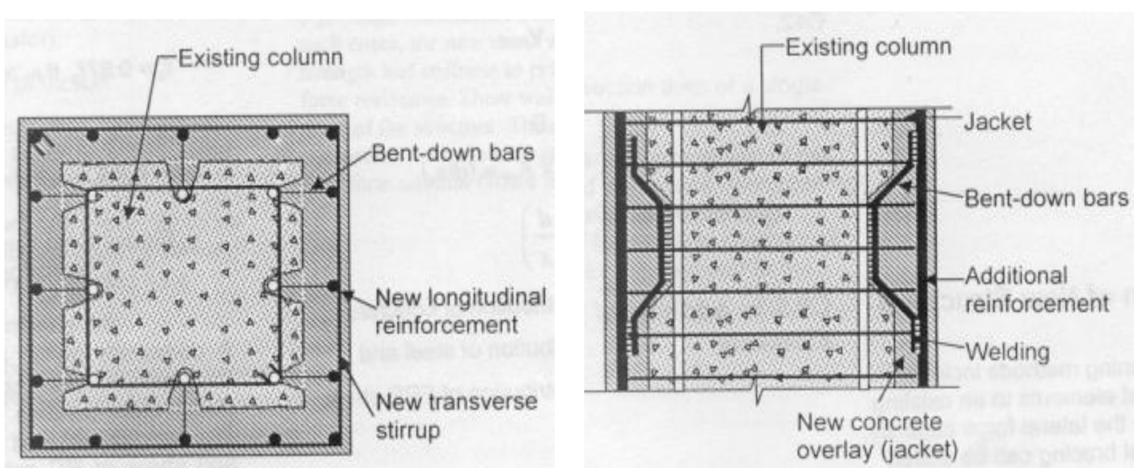



Figure 43 Reinforced Concrete Jacketing`

Figure 44 Installation of reinforced concrete jackets from the foundation level up to the beam soffit; examples from Colombia (source: WHE-2006-03, 2006).

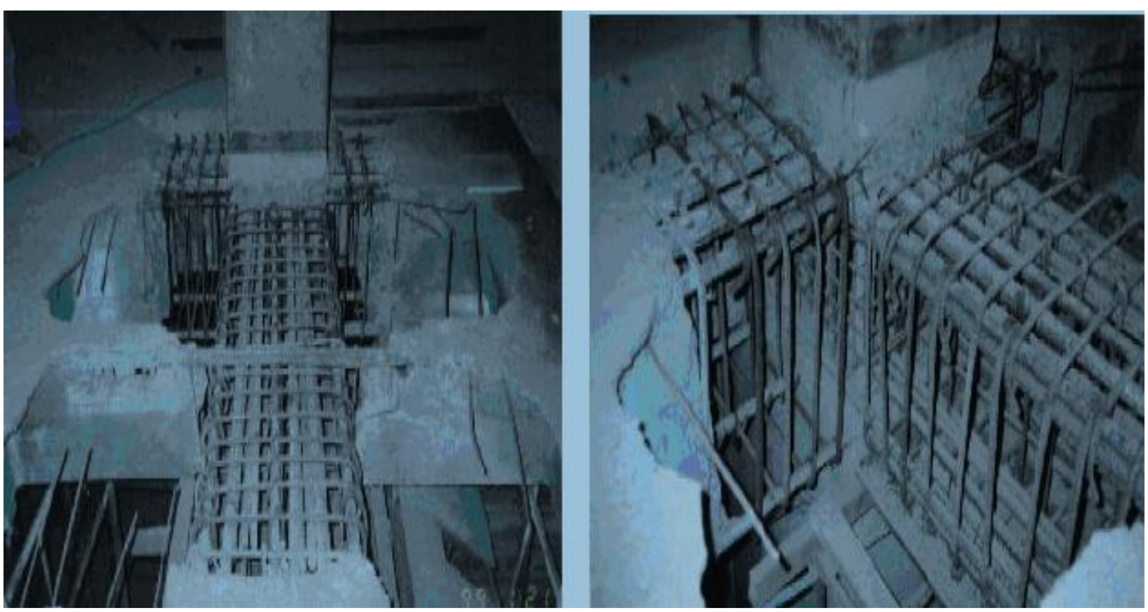


Figure 45 Jacketing of a beam-column joint region; an example from Colombia (source: WHE-2006-03, 2006).

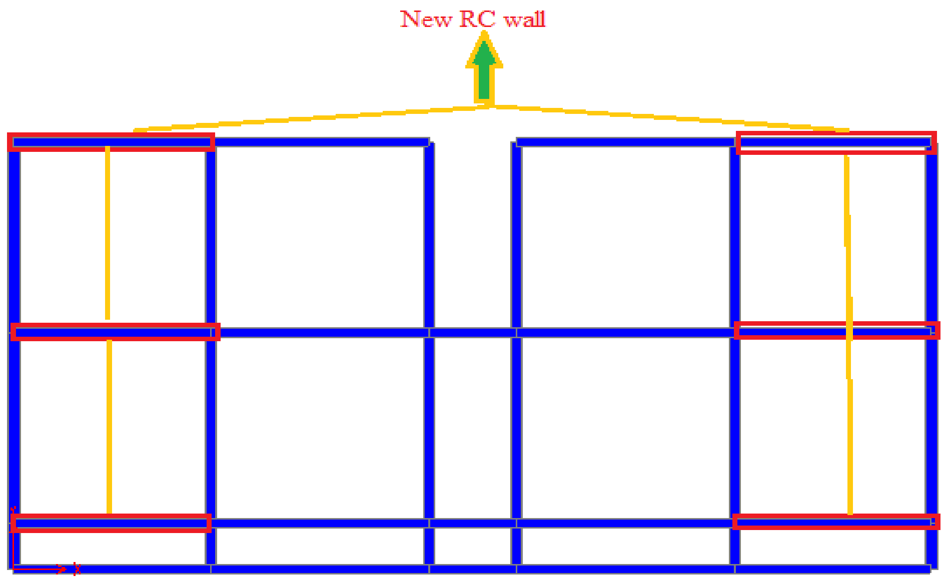
## 5.3 Increasing the Stiffness and Strength of the Adjacent Buildings for Case II and Case III

There are different techniques to increase the stiffness and strength of existing adjacent buildings as discussed above. But in this case, only installation of new RC shear wall has been provided to increase the stiffness and strength of the adjacent buildings in order to reduce the lateral displacement or deformation of the adjacent buildings consequently, to reduce the minimum required separation distance between them. A significant change in lateral displacement has been observed due to introducing of the new element of structural wall in both cases. The new structural wall has been provided in such way that the torsional effect becomes less. The detail discussion will present hereafter.

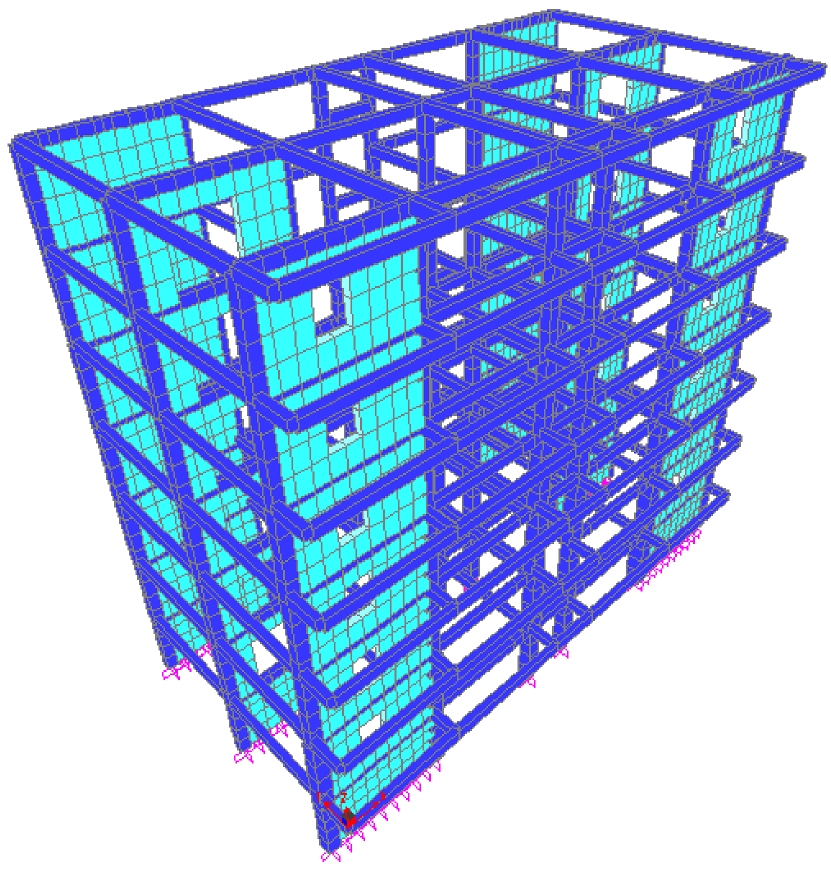
### 5.3.1. Increasing the Stiffness and Strength for Case II

As we have been discussed under the section 3.2.4 *(*Modeling and analyses of the case -II) and 4.2.2 (Result and Discussion of the case-II), that the adjacent buildings has been designated as building C and D (see figure 24). Under the result and discussion section, the result shows that, the two adjacent buildings did not satisfy the minimum required distance according to EBCS-8, 1995. This is because of the lateral displacement of the two adjacent buildings (building C and D) are 6.38cm and 8.27cm respectively. Consequently, the minimum required separation distance between these two adjacent buildings was 20.51 cm, where as the two buildings have been constructed with no gap in between as discussed above.

As we have been discussed above, new RC shear wall has been introduced to reduce the lateral displacement. Accordingly the RC wall layout has been provided in symmetrical manner (as much as possible) to avoid undesirable twisting or torsional effects due to unsymmetrical location of RC walls which cases non uniformly distribution of lateral load. The provided RC shear wall has a thickness of 250mm with dual layer system. Moreover the shear wall has been placed in side subjected to pounding (or perpendicular to the side subjected to collisions) and it has also been extended over the total height of the buildings see figure 46 and 47.

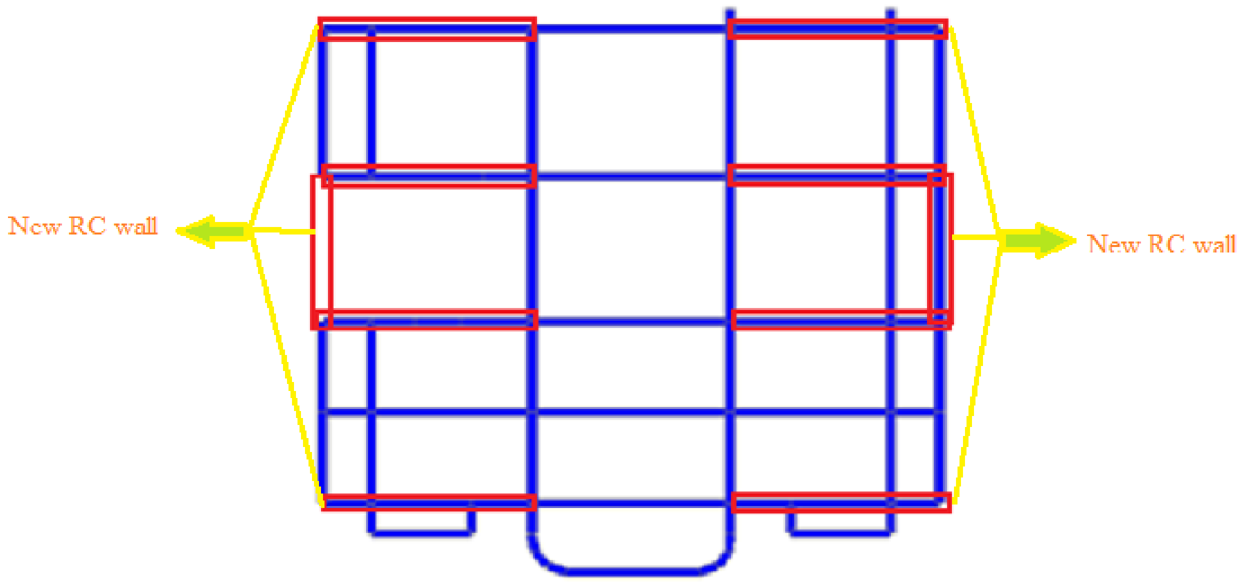



a) Floor plan of building C

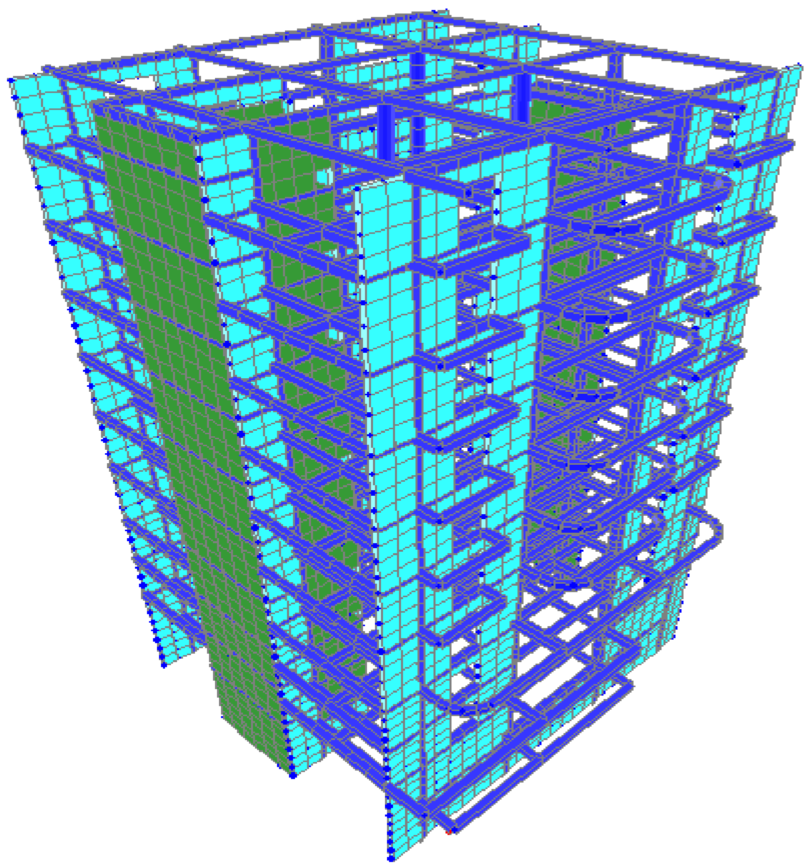


b) 3D view of building C

Figure 46 Plan and 3D view of building C after installation of new RC wall




a) Floor plan of building D



b) 3D view of building D

Figure 47 Plan and 3D view of building D after installation of new RC wall

Hence, after installation of new RC shear wall the lateral displacements of the two adjacent building reduces from 12.96 cm to 0.55 cm for building C and 10.67 cm to 5.07 cm for building D, due to the reduction of lateral displacement in both buildings, the minimum required separation distance between the adjacent buildings reduces from 32.04 cm to 7.46 cm which is about 23.28% of previous calculated separation distance. The joint displacement and the calculated minimum required separation distance are listed in the tables 22, 23 and 24 below.

Table 22 Joint Displacements for building C after installation of RC shear wall

| Joint | Output Case | Displacement($d_{eC}$) cm |
|---|---|---|
| 1 | EQX-SPECTRUM | 0.26 |
| 2 | EQX-SPECTRUM | 0.55 |

Table 23 Joint Displacements for building D after installation of RC shear wall

| Joint | Output Case | Displacement($d_{eC}$) cm |
|---|---|---|
| 1 | EQX-SPECTRUM | 5.07 |
| 2 | EQX-SPECTRUM | 3.08 |

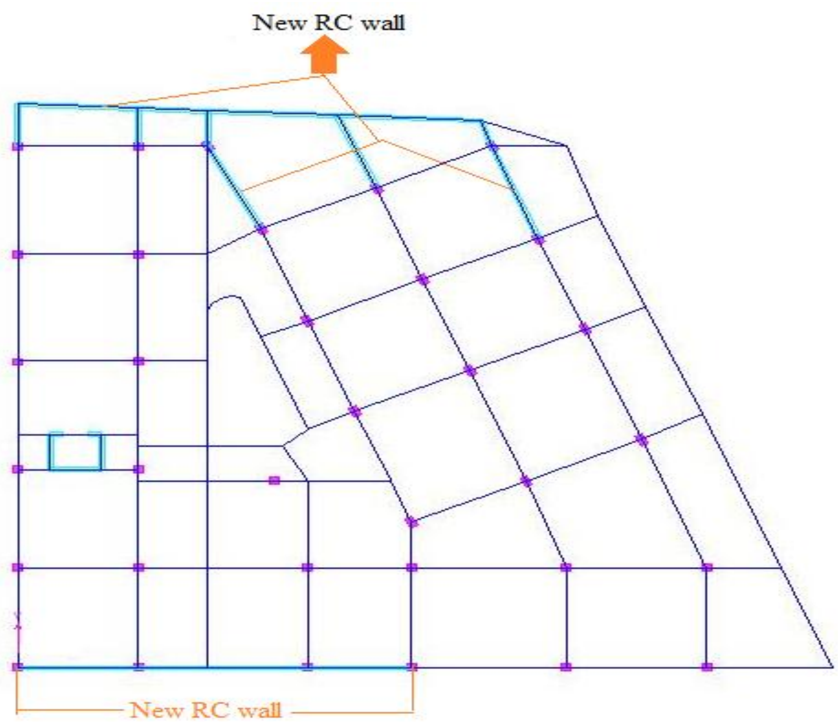
Table 24 minimum required separation distance

| Joint | deC (cm) | $d_{eD}$ (cm) | $d_{sC}$ (cm) | $d_{sD}$ (cm) | $d_{sC}$ (cm) ( factored) | $d_{sD}$ (cm) ( factored) | Min. required sep. distance (d) according to | | Existing Separation Distance (cm) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | EBCS(factored) (cm) | IBC($\delta_{MT}$) (cm) | |
| a/1 | 0.26 | 5.07 | 0.52 | 10.14 | 0.36 | 7.10 | 7.46 | 10.15 | 0.00 |
| b/2 | 0.55 | 3.08 | 1.10 | 6.16 | 0.77 | 4.31 | 5.08 | 6.26 | |

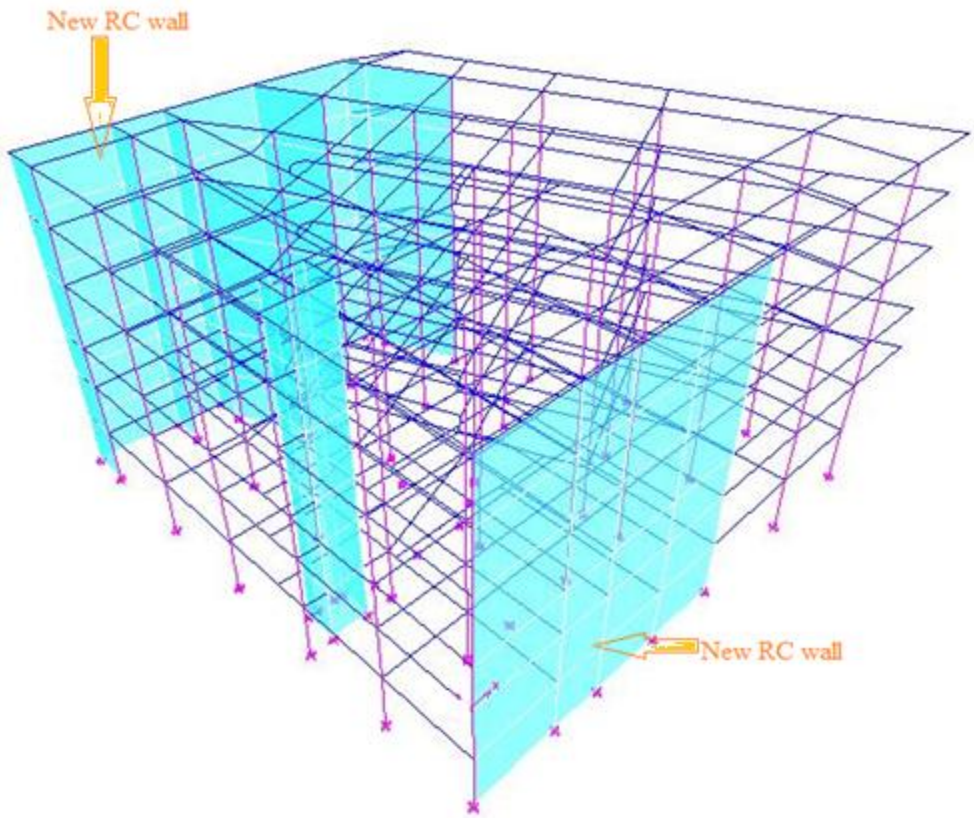
### 5.3.2. Increasing the Stiffness and Strength for Case III

As it has been discussed above for case-III, the adjacent buildings has been designated as building E and F (refer figure 28). The lateral displacements of the adjacent buildings (building E and F) from analysis output are 9.89 cm and 7.69 cm respectively. Consequently, the minimum required separation distance between these two adjacent buildings is 35.10 cm where as the existing separation distance in between is estimated about 10 cm. Based on this result the two adjacent buildings have not been satisfied the minimum required separation distance in between according to EBCS-8, 1995.

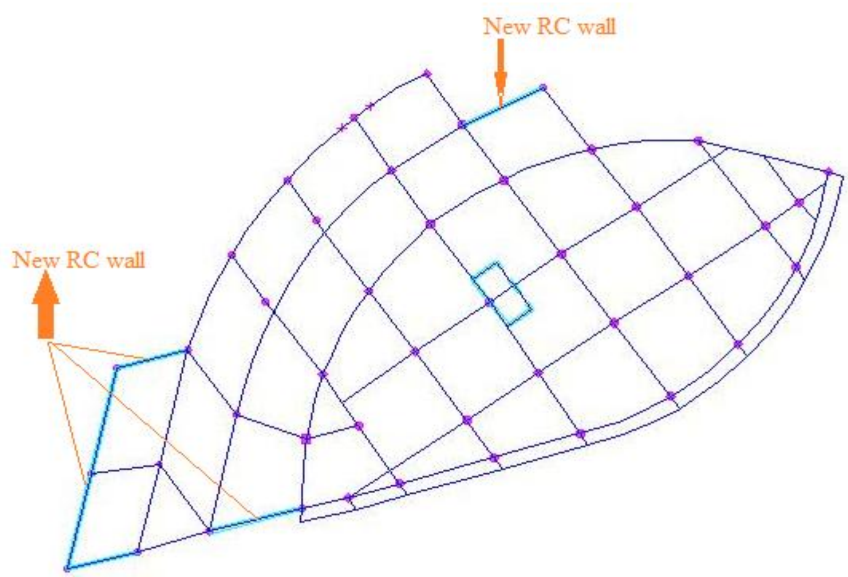
Due to installation of the new RC shear wall all these figure out quantities have been changed, RC wall layout has been provided in symmetrical manner (as much as possible as similar to case-II) to avoid undesirable twisting effects. The provided RC shear wall has a thickness of 250mm with dual layer system see figures 48 and 49.



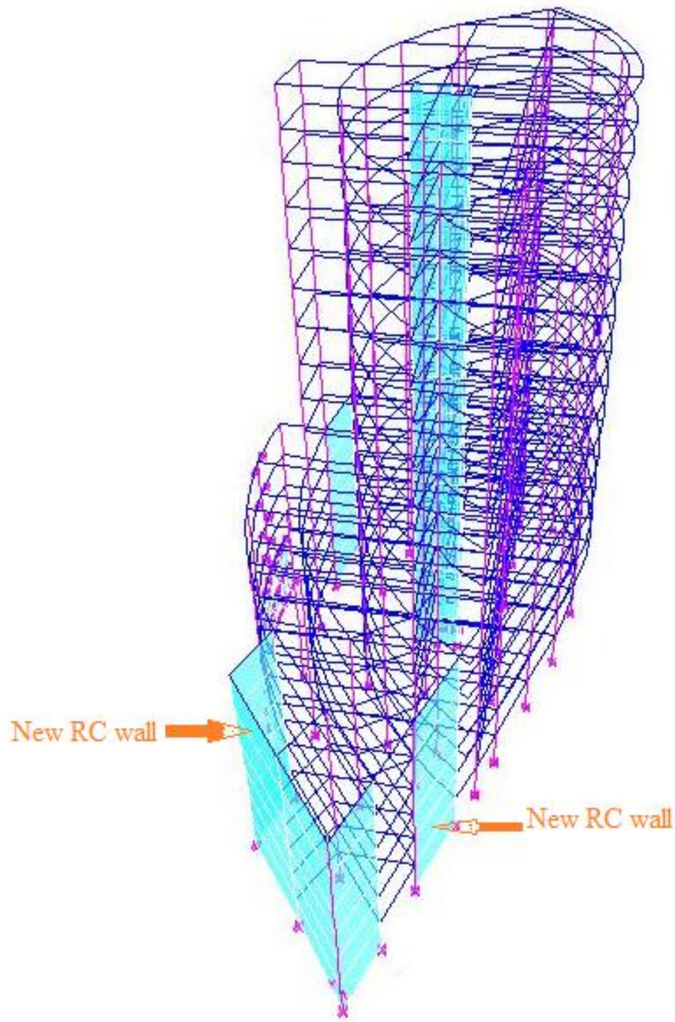

a) Floor plan of building E




b) 3D view of building E
Figure 48 Plan and 3D view of building E after installation of new RC wall




a) Floor plan of building F




b) 3D view of building F

Figure 49 Plan and 3D view of building F after installation of new RC wall

Hence, after installation of new RC shear wall the lateral displacements of the two adjacent building reduces from 9.89 cm to 1.86 cm for building E and 7.69cm to 1.11 cm for building F, due to the reduction of lateral displacement in both buildings, the minimum required separation distance between the adjacent buildings reduces from 35.10 cm to 5.79 cm which is about 16.50% of previous calculated separation distance (reduces by 83.5 %). The joint displacement and the calculated minimum required separation distance are listed in the tables (25-27).

Table 25 Joint Displacements for building E after installation of RC shear wall

| Joint | Output Case | Displacement($d_{eD}$) |
|---|---|---|
| | | Cm |
| 1 | EQX-SPECTRUM | 1.24 |
| 2 | EQX-SPECTRUM | 1.86 |
| 3 | EQX-SPECTRUM | 1.73 |

Table 26 Joint Displacements for building F after installation of RC shear wall

| Joint | Output Case | Displacement($d_{eE}$) |
|---|---|---|
| | | Cm |
| a | EQX-SPECTRUM | 0.93 |
| b | EQX-SPECTRUM | 1.03 |
| c | EQX-SPECTRUM | 1.11 |

Table 27 minimum required separation distance

| Joint | $d_{eE}$ (cm) | $d_{eF}$ (cm) | $d_{sD}$/(δME) (cm) | $d_{sE}$/(δMF) (cm) | Min. required sep. distance (d) according to | | Existing Separation Distance (cm) |
|---|---|---|---|---|---|---|---|
| | | | | | EBCS (cm) | IBC($\delta_{MT}$) (cm) | |
| a/1 | 1.24 | 0.93 | 2.48 | 1.86 | 4.34 | 3.10 | 10.00 |
| b/2 | 1.86 | 1.03 | 3.72 | 2.06 | 5.78 | 4.25 | |
| c/3 | 1.73 | 1.11 | 3.46 | 2.22 | 5.68 | 4.11 | |

# 6.0 Conclusion and Recommendations

## 6.1 Conclusions

In the paper, the idea of minimum required separation distance between adjacent buildings to avoid earthquake-induced structural pounding between them has been considered.

Based on the analysis results of the three cases the following conclusion can be drawn:

In the first case the total building height of the two buildings is approximately the same but their floor elevation are different. Existing gap between building A and building B is approximated to 40cm and the calculated minimum required separation distance between them is approximately 38.96cm. Hence the existing separation distance between these buildings is sufficient to accommodate displacement response of the structures relative one another.

In the second case the two buildings are constructed without any separation distance and these building have different total height and with the same floor elevation. The calculated (according to EBCS) minimum required separation distance between these buildings (building C and building D) is 32.04 cm before installation of new RC wall, but after installation of new RC wall the minimum required separation distance reduces to 7.46 cm. But still this value is not sufficient. Moreover any type of building, according to EBCS-8, 1995, cannot be constructed without any separation distance because the code (EBCS-8) sets a minimum of 40mm. Hence the gap between building C and D cannot be zero but providing of additional shear wall can reduce the risk significantly.

In the third case the adjacent buildings are constructed with separation distance of 10 cm between them but not throughout the width of the building. Moreover, these adjacent buildings have no equal floor elevation and also they have different building height. However, the calculation and the analysis result show that the minimum required separation distance to avoid pounding is 35.10 cm (according to EBCS). To alleviate the problem, RC shear wall has been provided as method of mitigation to increase the stiffness and strength of the buildings. As a result of this, a significant change has been shown in lateral displacement and period and the required separation distance between them reduces to 5.78 cm. If the proposed additional shear wall is going to be provided accordingly, the existing separation distance between these buildings is sufficient to accommodate displacement response of the structures,

## 6.2 Recommendations

The first work for pounding prevention is to establish a good and reliable estimate of the minimum gap required for the design earthquake so that pounding between the structures will not occur. Providing a sufficient gap has been the commonly accepted strategy adopted by building codes throughout the world.

Several modern buildings are constructed and under construction in Addis Ababa with insufficient separation distance; this can cause pounding of adjacent buildings. Addis Ababa city municipality and sub-cities will have a great role in solving such kind of problem, by permitting an appropriate design for construction and supervising the construction process whether constructed or not according to the approved design. This is because, they are the authoritarian body to allow & control the implementations of designs for the construction. It is strictly recommended to practice procedures recommended by local codes to avoid pounding risk for any future earthquake.

Architects can play a great role in minimizing pounding effect through providing a set-back at each storey with a dimension equal to the dimension of respective storey drift in all direction throughout the height of the building.

Adjacent buildings, which have been constructed with nearly closed or totally attached each other, must be treated to withstand the lateral force or earthquake load for minimizing the pounding risk.

***For Further Study***

The thesis is assumed to be continued by some future studies considering the limitations made in this study. Therefore, one can extend this study to include the following points:-

- This study dealt only for a soil type B, thus one can extend it to consider other soil types for further studies.

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