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<td>B.Sc Textile Technology, M.Sc. Technical Science Ph.D. in Industrial Management. The College of Textile Design, Technology and Management, Belgrade, Serbia</td>
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<th><strong>Dr. Cesar M. A. Vasques</strong></th>
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<td>Ph.D., Mechanical Engineering, Department of Mechanical Engineering, School of Engineering, Polytechnic of Porto Porto, Portugal</td>
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<th><strong>Dr. Maurizio Palesi</strong></th>
<th><strong>Dr. Jun Wang</strong></th>
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<td>Ph.D. in Architecture, University of Hong Kong, China Urban Studies City University of Hong Kong, China</td>
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<th><strong>Dr. Salvatore Brischetto</strong></th>
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<th><strong>Dr. Togay Ozbakkaloglu</strong></th>
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<td>B.Sc. in Civil Engineering, Ph.D. in Structural Engineering, University of Ottawa, Canada Senior Lecturer University of Adelaide, Australia</td>
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<th><strong>Dr. Ananda Kumar Palaniappan</strong></th>
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<td>B.E., Ph.D. in Mechanical Engineering University of Sciences and Technology of China, China Professor, Faculty of Health Sciences, University of Macau, China</td>
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Effect of Seismic Load on Column Forces in RC Structures by Response Spectrum Analysis

By Mahdi Hosseini
Nanjing Forestry University

Abstract- In the present research work 30 story building with different type of RC Shear wall at the center in concrete frame structure with fixed support conditions under different type of soil for high seismic zone are analyzed.

This paper aims to study the effect of seismic load on column forces in different type of RC shear walls in concrete frame structures under different type of soil condition and different load combination. Estimation of column forces such as; column axial force, column moment, column shear force, column torsion, time period and frequency and modal load participation ratios is carried out. In dynamic analysis; Response Spectrum method is used. It was found that the axial force and moment in the column increases when the type of soil changes from hard to medium and medium to soft. Since the column moment increase as the soil type changes, soil structure interaction must be suitably considered while designing frames for seismic force.

Keywords: seismic load, linear dynamics analysis, column forces, high seismic zone.

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Keywords: seismic load, linear dynamics analysis, column forces, high seismic zone.

1. INTRODUCTION

a) Structural Systems

In the earliest structures at the beginning of the 20th century, structural members were assumed to carry primarily the gravity loads. Today, however, by the advances in structural design/systems and high-strength materials, building height is increased, which necessitates taking into consideration mainly the lateral loads such as wind and earthquake. Understandably, especially for the tall buildings, as the slenderness, and so the flexibility increases, buildings suffer from the lateral loads resulting from wind and earthquake more and more. As a general rule, when other things being equal, the taller the building, the more necessary it is to identify the proper structural system for resisting the lateral loads. Currently, there are many structural systems that can be used for the lateral resistance of tall buildings[2,3].

Structural systems of tall buildings can be divided into two broad categories: interior structures and exterior structures.

This classification is based on the distribution of the components of the primary lateral load-resisting system over the building.

b) Shear Wall Structure

Shear Wall-Frame Systems (Dual Systems), The system consists of reinforced concrete frames interacting with reinforced concrete shear walls are adequate for resisting both the vertical and the horizontal loads acting on them.

c) Necessity of Shear Walls

Shear wall system has two distinct advantages over a frame system.

• It provides adequate strength to resist large lateral loads with-out excessive additional cost.
• It provides adequate stiffness to resist lateral displacements to permissible limits, thus reducing risk of non-structural damage.

d) Seismic Load

The seismic weight of building is the sum of seismic weight of all the floors [8]. The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, the latter being that part of the imposed loads that may reasonably be expected to be attached to the structure at the time of earthquake shaking. Earthquake forces experienced by a building result from ground motions (accelerations) which are also fluctuating or dynamic in nature, in fact they reverse direction somewhat chaotically[2,3]. In theory and practice, the lateral force that a building experiences from an earthquake increases in direct proportion with the acceleration of ground motion at the building site and the mass of the building. As the ground accelerates back and forth during an earthquake it imparts back-and-forth (cyclic) forces to a building through its foundation which is forced to move with the ground [1].

e) Geo-Technical Consideration

The seismic motion that reaches a structure on the surface of the earth is influenced by local soil conditions. The subsurface soil layers underlying the building foundation may amplify the response of the building to earthquake motions originating in the bedrock.

Bearing Capacity of Foundation Soil

Three soil types are considered here:

I. Hard - Those soils, which have an allowable bearing capacity of more than 10t/m2.
II. Medium - Those soils, which have an allowable bearing capacity less than or equal to 10t/m2.
III. Soft - Those soils, which are liable to large differential settlement or liquefaction during an earthquake.

The allowable bearing pressure shall be determined in accordance with IS: 1888-1982 load test (Revision 1992).

II. Methodology

a) To understand and evaluation building structures and aims to the effect of Seismic load on column Forces in Different Type of RC Shear Walls in Concrete Frame Structures under Different Type of Soil Condition with seismic loading.

b) Modeling a G+29 story high building for five different cases [9-11].

c) Analyzing the building dynamic analysis using linear, i.e. Response Spectrum Analysis [1-3].

d) Analyzing the results and arriving at conclusions.

a) Dynamic Analysis

Dynamic analysis may be executed to get the design seismic force, and its spread in different levels through the height of the building, and also various lateral load resisting element[1-2-3,8].

b) Response Spectrum Method

This method is executed to design spectrum, where as it is specified with a code for specific- site design can be used for a project site for the purposes of dynamic analysis of steel and reinforce concrete buildings, the values of damping for building may be taken as 2 and 5 percent of the critical, respectively. response spectrum method is typically implemented in linear elastic procedures and also very much easier to use. This also called as or mode superposition method or model method. It also made on the idea of the superposition of responses given by the building through various modes of vibrations, each vibration modes is recorded as with its own particular deformed shape, with its own modal damping and its own frequency [7,8].

III. Modeling of Building

a) Details of the Building

A symmetrical building[15] of plan 38.5m X 35.5m located with location in high Seismic zone considered. Four bays of length 7.5m & one bays of length 8.5m along X - direction and four bays of length 7.5m & one bays of length 5.5m along Y - direction are provided. Shear is provided the center inner core of model building.

Struct I: G+29 story’stall building with Plus shape RC shear wall at the center of structure.

Struct II: G+29 story’s tall building with Box shape RC shear wall at the center of structure.

Struct III: G+29 story’s tall building with C- shape RC shear wall at the center of structure.

Struct IV: G+29 story’s tall building with E- shape RC shear wall at the center of structure.

Struct V: G+29 story’s tall building with I- shape RC shear wall at the center of structure.

b) Load Combinations

As per IS 1893 (Part 1): 2002 Clause no. 6.3.1.2, the following load cases have to be considered for analysis:

"1.2 (DL + IL ± EL)"

"1.5 (DL ± EL)"

"EQXP&EQYP"

Earthquake load must be considered for +X, -X, +Y and –Y Directions [5-7].

c) The Building Details

Type of frame: Special RC moment resisting frame fixed at the base, Number of storeys: G+29, Floor height: 3.5 m, Depth of Slab: 225 mm, Size of beam: (300 × 600) mm, Size of column (exterior): (1250×1250) mm up to story five, Size of column (exterior): (900×900) mm Above story five, Size of column (interior): (1250×1250) mm up to story ten, Size of column (interior): (900×900) mm Above story ten, Live load on floor: 4 KN/m2, Floor finish: 2.5 KN/m2, Wall load: 25 KN/m, Grade of Concrete: M 50 concrete, Grade of Steel: Fe 500, Thickness of shear wall: 450 mm, Seismic zone: V, Important Factor: 1.5, Density of concrete: 25 KN/m3, Type of soil: Type I=Soft Soil, Type II=Medium Soil, Type III= Hard Soil, Response spectra: As per IS 1893(Part-1):2002. Damping of structure: 5 percent & All the analyses has been carried out as per the Indian Standard code books [4-8].
Figure 1: Plan of the Structure I

Figure 2: 3D view showing shear wall location for Structure I
Figure 3: Plan of the Structure II

Figure 4: 3D view showing shear wall location for Structure II
Figure 5: Plan of the Structure III

Figure 6: 3D view showing shear wall location for Structure III

Figure 7: Plan of the Structure IV
Figure 8: 3D view showing shear wall location for Structure IV

Figure 9: Plan of the Structure V

Figure 10: 3D view showing shear wall location for Structure V
**Parametric results in column forces such as column axial force, column moment, column shear force & column torsion with different load combination/load Cases such as 1.2 (DL+LL+EQXP), 1.2 (DL+LL+EQYP), 1.5 (DL+EQXP), 1.5 (DL+EQYP), EQXP & EQYP in different type of soil conditions (soft, medium and hard) were considered, in this regard we compared all column forces in different type of soil condition of structures II, III, IV, V with structure I (plus shape shear wall), also compared forces in hard and medium soils with soft soil for all five structures.**

**Table 1: Column Axial Force, P for structures with the load combination 1.2 (DL+LL+EQXP) & 1.2 (DL+LL+EQYP), All value in "kN"**

<table>
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<th>Story</th>
<th><em>Column</em></th>
<th><em>Unique - Name</em></th>
<th><em>Load Case-Combo</em></th>
<th><em>Station</em>m</th>
<th>Struct I</th>
<th>Struct II</th>
<th>Struct III</th>
<th>Struct IV</th>
<th>Struct V</th>
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**Column Axial Force, P in Medium Soil**

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**Column Axial Force, P in Hard Soil**

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Column Moment, M in Hard Soil

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Table 3: Column Shear, V for structures with the load combination 1.2 (DL+LL+EQXP) & 1.2 (DL+LL+EQYP), All value in “KN”

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Column Shear, V in Medium Soil

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Column Shear, V in Hard Soil

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<td>71</td>
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Table 4: Column Torsion, T for structures with the load combination 1.2 (DL+LL+EQXP) & 1.2 (DL+LL+EQYP), All value in “kN-m”

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<th>&quot;Station&quot;/m</th>
<th>&quot;T&quot;/kN-m</th>
<th>&quot;T&quot;/kN-m</th>
<th>&quot;T&quot;/kN-m</th>
<th>&quot;T&quot;/kN-m</th>
<th>&quot;T&quot;/kN-m</th>
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<td>0</td>
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<td>1.2(DL+LL+EQXP)</td>
<td>1.45</td>
<td>-41.6175</td>
<td>-29.3334</td>
<td>-44.901</td>
<td>-42.3525</td>
<td>-43.8436</td>
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<td>2.9</td>
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<td>31.9525</td>
<td>48.8724</td>
<td>46.1375</td>
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<td>46.1375</td>
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<td>45.3145</td>
<td>31.9525</td>
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Table 5: Column Axial Force, P for structures with the load combination 1.5 (DL+EQXP) & 1.5 (DL+EQYP), All value in “kN”

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<th>&quot;Station&quot;/m</th>
<th>&quot;P&quot;/kN</th>
<th>&quot;P&quot;/kN</th>
<th>&quot;P&quot;/kN</th>
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<td>-25098.9089</td>
<td>-25270.435</td>
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### Table 6: Column Moment, M for structures with the load combination 1.5 (DL + EQXP) & 1.5 (DL + EQYP), All value in “kN-m”

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<th>Struct II</th>
<th>Struct III</th>
<th>Struct III</th>
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<th>Struct V</th>
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<td>&quot;M2&quot;</td>
<td>&quot;M3&quot;</td>
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<th>Struct II</th>
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<td>&quot;M2&quot;</td>
<td>&quot;M3&quot;</td>
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Table 7: Column Shear, V for structures with the load combination 1.5 (DL+EQXP) & 1.5 (DL+EQYP), All value in “kN”

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<td>&quot;V3&quot;</td>
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Column Shear, V in Medium Soil

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<td>&quot;Station’m&quot;</td>
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<td>&quot;V5&quot;</td>
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Column Shear, V in Hard Soil

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<td>&quot;V5&quot;</td>
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Table 8: Column Torsion, T for structures with the load combination 1.5 (DL+EQXP) & 1.5 (DL+EQYP). All value in “kN-m”

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Table 9: Column Axial Force, P for structures with the load Cases EQXP & EQYP. All value in “kN”

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### Table 10: Column Moment, M for structures with the load Cases EQXP & EQYP, All value in “kN-m”

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### Column Moment, M in Medium Soil

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### Column Moment, M in Hard Soil

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Table 11: Column Shear, V for structures with the load Cases EQXP & EQYP, All value in “kN”

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Table 12: Column Torsion, T for structures with the load Cases EQXP & EQYP, All value in “kN-m”

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<td>67</td>
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Table 13: Modal Load Participation Ratios

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<th>Struct I</th>
<th>Struct II</th>
<th>Struct II</th>
<th>Struct III</th>
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Graph 1: Modal Load Participation Ratios of Structures

Table 14: Modal Periods and Frequencies

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<th>Period</th>
<th>Frequency</th>
<th>Period</th>
<th>Frequency</th>
<th>Period</th>
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<td>0.589</td>
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<td>0.423</td>
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**Table 15:** Compared of column axial forces in soft soil of structures II, III, IV, V with structure I

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<th>Column</th>
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<th>Station&quot;m</th>
<th>&quot;P&quot;</th>
<th>&quot;P&quot;</th>
<th>&quot;P&quot;</th>
<th>&quot;P&quot;</th>
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<tbody>
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<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>0%</td>
<td>2%</td>
<td>1%</td>
<td>1%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>-2%</td>
<td>-1%</td>
<td>-1%</td>
<td>-1%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>1%</td>
<td>2%</td>
<td>1%</td>
<td>1%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>-2%</td>
<td>-1%</td>
<td>-1%</td>
<td>-1%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
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<td>8%</td>
<td>19%</td>
<td>11%</td>
<td>9%</td>
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<td>C34</td>
<td>67</td>
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### Table 16: Compared of column axial forces in medium soil of structures II, III, IV, V with structure I

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<th>&quot;Station&quot;</th>
<th>&quot;P&quot;</th>
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<td>0.145,2.9</td>
<td>1%</td>
<td>2%</td>
<td>2%</td>
<td>1%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
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<td>-1%</td>
<td>-1%</td>
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<td>3%</td>
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<td>1%</td>
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<td>0.145,2.9</td>
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<td>-1%</td>
<td>-1%</td>
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<td>C34</td>
<td>67</td>
<td>EQXP</td>
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<td>19%</td>
<td>16%</td>
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<td>-11%</td>
<td>-10%</td>
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### Table 17: Compared of column axial forces in hard soil of structures II, III, IV, V with structure I

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<td>3%</td>
<td>2%</td>
<td>1%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
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<td>0.145,2.9</td>
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<td>-1%</td>
<td>-1%</td>
<td>-1%</td>
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<tr>
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<td>C34</td>
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<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
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<td>4%</td>
<td>3%</td>
<td>2%</td>
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<td>0.145,2.9</td>
<td>-3%</td>
<td>-1%</td>
<td>-1%</td>
<td>-2%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>8%</td>
<td>19%</td>
<td>16%</td>
<td>9%</td>
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<tr>
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<td>C34</td>
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<td>EQYP</td>
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<td>-11%</td>
<td>-10%</td>
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### Table 18: Compared of column moment in soft soil of structures II, III, IV, V with structure I

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<th>&quot;Station&quot;</th>
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<th>&quot;M&quot;</th>
<th>&quot;M&quot;</th>
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<td>-35%</td>
<td>-42%</td>
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<td>4%</td>
<td>-1%</td>
<td>4%</td>
</tr>
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<td>1.5(DL+EQYP)</td>
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<td>-39%</td>
<td>-47%</td>
<td>-47%</td>
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<td>C34</td>
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<td>41%</td>
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<td>13%</td>
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### Table 19: Compared of column moment in medium soil of structures II, III, IV, V with structure I

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<th>&quot;Station&quot;</th>
<th>&quot;M&quot;</th>
<th>&quot;M&quot;</th>
<th>&quot;M&quot;</th>
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<td>-35%</td>
<td>-34%</td>
<td>-46%</td>
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<td>5%</td>
<td>5%</td>
<td>4%</td>
</tr>
<tr>
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<td>67</td>
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<td>-39%</td>
<td>-38%</td>
<td>-47%</td>
</tr>
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<td>45%</td>
<td>44%</td>
</tr>
<tr>
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<td>C34</td>
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<td>EQYP</td>
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**Table 20:** Compared of column moment in hard soil of structures II, III, IV, V with structure I

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<th>&quot;M&quot;</th>
<th>&quot;M&quot;</th>
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<td>5%</td>
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<tr>
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<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>-55%</td>
<td>-35%</td>
<td>-34%</td>
<td>-46%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
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<td>6%</td>
<td>6%</td>
<td>5%</td>
</tr>
<tr>
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<td>-39%</td>
<td>-38%</td>
<td>-47%</td>
</tr>
<tr>
<td>1ST</td>
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<td>45%</td>
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**Table 21:** Compared of column shear in soft soil of structures II, III, IV, V with structure I

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<th>&quot;Station&quot;m</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
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</thead>
<tbody>
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<td>14%</td>
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<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>-12%</td>
<td>20%</td>
<td>8%</td>
<td>-18%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>166%</td>
<td>148%</td>
<td>155%</td>
<td>955%</td>
</tr>
<tr>
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<td>C34</td>
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<td>EQXP</td>
<td>0.145,2.9</td>
<td>13%</td>
<td>41%</td>
<td>30%</td>
<td>-5%</td>
</tr>
</tbody>
</table>

**Table 22:** Compared of column shear in medium soil of structures II, III, IV, V with structure I

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique - Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;m</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>-10%</td>
<td>21%</td>
<td>14%</td>
<td>-17%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>173%</td>
<td>152%</td>
<td>157%</td>
<td>2154%</td>
</tr>
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<td>C34</td>
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<td>0.145,2.9</td>
<td>-5%</td>
<td>26%</td>
<td>18%</td>
<td>-15%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>142%</td>
<td>132%</td>
<td>134%</td>
<td>325%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>13%</td>
<td>41%</td>
<td>34%</td>
<td>-5%</td>
</tr>
</tbody>
</table>

**Table 23:** Compared of column shear in hard soil of structures II, III, IV, V with structure I

<table>
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<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique - Name&quot;</th>
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<th>&quot;Station&quot;m</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
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</thead>
<tbody>
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<td>67</td>
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<td>0.145,2.9</td>
<td>-6%</td>
<td>25%</td>
<td>18%</td>
<td>-15%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>154%</td>
<td>140%</td>
<td>143%</td>
<td>521%</td>
</tr>
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<td>C34</td>
<td>67</td>
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<td>0.145,2.9</td>
<td>-2%</td>
<td>29%</td>
<td>21%</td>
<td>-13%</td>
</tr>
<tr>
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<td>67</td>
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<td>124%</td>
<td>125%</td>
<td>226%</td>
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<tr>
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<td>C34</td>
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<td>EQXP</td>
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<td>13%</td>
<td>41%</td>
<td>34%</td>
<td>-5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>83%</td>
<td>86%</td>
<td>86%</td>
<td>64%</td>
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</table>
### Table 24: Compared of column torsion in soft soil of structures II, III, IV, V with structure I

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<th>&quot;Column&quot;</th>
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<th>&quot;Station&quot;m</th>
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<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>2%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>2%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>2%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>1%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.1,45,2.9</td>
<td>-41%</td>
<td>8%</td>
<td>2%</td>
<td>7%</td>
</tr>
</tbody>
</table>

### Table 25: Compared of column torsion in medium soil of structures II, III, IV, V with structure I

<table>
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<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;m</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
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</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>7%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>8%</td>
<td>7%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>7%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>8%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>7%</td>
<td>5%</td>
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<tr>
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<td>0.1,45,2.9</td>
<td>-41%</td>
<td>8%</td>
<td>8%</td>
<td>7%</td>
</tr>
</tbody>
</table>

### Table 26: Compared of column torsion in hard soil of structures II, III, IV, V with structure I

<table>
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<th>&quot;Column&quot;</th>
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<th>&quot;Station&quot;m</th>
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<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
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<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>7%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>8%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>7%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>8%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.1,45,2.9</td>
<td>-42%</td>
<td>7%</td>
<td>7%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.1,45,2.9</td>
<td>-41%</td>
<td>8%</td>
<td>8%</td>
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</table>

### Table 27: Compared of column axial forces of medium soil and hard soil with soft soil for Structure -I

<table>
<thead>
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<th>&quot;Station&quot;m</th>
<th>&quot;P&quot;</th>
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<td>6%</td>
</tr>
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<td>C34</td>
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<td>1.2(DL+LL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>2%</td>
<td>4%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.1,45,2.9</td>
<td>4%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.1,45,2.9</td>
<td>3%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.1,45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.1,45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>
Table 28: Compared of column axial forces of medium soil and hard soil with soft soil for Structure-II

<table>
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<th><em>Column</em></th>
<th><em>Unique-Name</em></th>
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<th><em>Station</em>m</th>
<th><em>Medium soil</em></th>
<th><em>Hard soil</em></th>
</tr>
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<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>3%</td>
<td>6%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>2%</td>
<td>4%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>4%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>2%</td>
<td>4%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
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<td>40%</td>
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</tbody>
</table>

Table 29: Compared of column axial forces of medium soil and hard soil with soft soil for Structure-III

<table>
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<tr>
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<th><em>Station</em>m</th>
<th><em>Medium soil</em></th>
<th><em>Hard soil</em></th>
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<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>4%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>2%</td>
<td>4%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>4%</td>
<td>8%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>3%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 30: Compared of column axial forces of medium soil and hard soil with soft soil for Structure-IV

<table>
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<th><em>Column</em></th>
<th><em>Unique-Name</em></th>
<th><em>Load Case-Combo</em></th>
<th><em>Station</em>m</th>
<th><em>Medium soil</em></th>
<th><em>Hard soil</em></th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>4%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>3%</td>
<td>4%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>5%</td>
<td>8%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>3%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>31%</td>
<td>44%</td>
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</table>

Table 31: Compared of column axial forces of medium soil and hard soil with soft soil for Structure-V

<table>
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<th><em>Unique-Name</em></th>
<th><em>Load Case-Combo</em></th>
<th><em>Station</em>m</th>
<th><em>Medium soil</em></th>
<th><em>Hard soil</em></th>
</tr>
</thead>
<tbody>
<tr>
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<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>3%</td>
<td>6%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>2%</td>
<td>4%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>4%</td>
<td>7%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>3%</td>
<td>5%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>
Table 32: Compared of column moment of medium soil and hard soil with soft soil for Structure -I

<table>
<thead>
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<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;m</th>
<th>&quot;Medium soil&quot;</th>
<th>&quot;Hard soil&quot;</th>
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<tbody>
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<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>20%</td>
<td>32%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>-7%</td>
<td>-14%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>21%</td>
<td>34%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>-11%</td>
<td>-23%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 33: Compared of column moment of medium soil and hard soil with soft soil for Structure -II

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;m</th>
<th>&quot;Medium soil&quot;</th>
<th>&quot;Hard soil&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>25%</td>
<td>38%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>32%</td>
<td>46%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>39%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>29%</td>
<td>43%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 34: Compared of column moment of medium soil and hard soil with soft soil for Structure -III

<table>
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<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;m</th>
<th>&quot;Medium soil&quot;</th>
<th>&quot;Hard soil&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>27%</td>
<td>41%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>25%</td>
<td>39%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 35: Compared of column moment of medium soil and hard soil with soft soil for Structure -IV

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;m</th>
<th>&quot;Medium soil&quot;</th>
<th>&quot;Hard soil&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145,2.9</td>
<td>30%</td>
<td>43%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145,2.9</td>
<td>33%</td>
<td>46%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145,2.9</td>
<td>31%</td>
<td>43%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
<tr>
<td></td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
</tbody>
</table>
Table 36: Compared of column moment of medium soil and hard soil with soft soil for Structure -V

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;M&quot;</th>
<th>&quot;M&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145.2.9</td>
<td>22%</td>
<td>35%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145.2.9</td>
<td>415%</td>
<td>169%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145.2.9</td>
<td>23%</td>
<td>36%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145.2.9</td>
<td>71%</td>
<td>82%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 37: Compared of column shear of medium soil and hard soil with soft soil for Structure -I

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145.2.9</td>
<td>30%</td>
<td>44%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145.2.9</td>
<td>29%</td>
<td>42%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 38: Compared of column shear of medium soil and hard soil with soft soil for Structure -II

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0.145.2.9</td>
<td>56%</td>
<td>61%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145.2.9</td>
<td>40%</td>
<td>52%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 39: Compared of column shear of medium soil and hard soil with soft soil for Structure -III

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
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</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
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<td>1.2(DL+LL+EQXP)</td>
<td>0.145.2.9</td>
<td>31%</td>
<td>45%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0.145.2.9</td>
<td>29%</td>
<td>43%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0.145.2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>
Table 40: Compared of column shear of medium soil and hard soil with soft soil for Structure -IV

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>36%</td>
<td>48%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>31%</td>
<td>43%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>34%</td>
<td>47%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>31%</td>
<td>43%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
</tbody>
</table>

Table 41: Compared of column shear of medium soil and hard soil with soft soil for Structure -V

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;V&quot;</th>
<th>&quot;V&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>30%</td>
<td>44%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>29%</td>
<td>43%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 42: Compared of column torsion of medium soil and hard soil with soft soil for Structure -I

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
</tr>
</thead>
<tbody>
<tr>
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<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 43: Compared of column torsion of medium soil and hard soil with soft soil for Structure -II

<table>
<thead>
<tr>
<th>&quot;Story&quot;</th>
<th>&quot;Column&quot;</th>
<th>&quot;Unique-Name&quot;</th>
<th>&quot;Load Case-Combo&quot;</th>
<th>&quot;Station&quot;</th>
<th>&quot;T&quot;</th>
<th>&quot;T&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>
Table 44: Compared of column torsion of medium soil and hard soil for Structure -III

<table>
<thead>
<tr>
<th>Story</th>
<th>Column</th>
<th>Unique-Name</th>
<th>Load Case-Combo</th>
<th>Station*m</th>
<th>Medium soil</th>
<th>Hard soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

Table 45: Compared of column torsion of medium soil and hard soil for Structure -IV

<table>
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<tr>
<th>Story</th>
<th>Column</th>
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<th>Load Case-Combo</th>
<th>Station*m</th>
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<th>Hard soil</th>
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<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.2(DL+LL+EQXP)</td>
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<td>31%</td>
<td>44%</td>
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<td>31%</td>
<td>44%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
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<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
<tr>
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<td>C34</td>
<td>67</td>
<td>(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>31%</td>
<td>44%</td>
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<tr>
<td>1ST</td>
<td>C34</td>
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<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>31%</td>
<td>44%</td>
</tr>
<tr>
<td>1ST</td>
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<td>EQYP</td>
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</table>

Table 46: Compared of column torsion of medium soil and hard soil for Structure -V

<table>
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<th>Story</th>
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<th>Load Case-Combo</th>
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<th>Hard soil</th>
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<td>67</td>
<td>1.2(DL+LL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>1.5(DL+EQXP)</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>(DL+EQYP)</td>
<td>0,1.45,2.9</td>
<td>27%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQXP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
<tr>
<td>1ST</td>
<td>C34</td>
<td>67</td>
<td>EQYP</td>
<td>0,1.45,2.9</td>
<td>26%</td>
<td>40%</td>
</tr>
</tbody>
</table>

a) Discussion on Results

When a structure is subjected to earthquake, it responds by vibrating. An example force can be resolved into three mutually perpendicular directions—two horizontal directions (X and Y directions) and the vertical direction (Z) [8]. This motion causes the structure to vibrate or shake in all three directions; the predominant direction of shaking is horizontal. All the structures are primarily designed for gravity loads—force equal to mass times gravity in the vertical direction. Vertical acceleration should also be considered in structures with large spans those in which stability for design, or for overall stability analysis of structures. The basic intent of design theory for earthquake resistant structures is that buildings should be able to resist minor earthquakes without damage, resist moderate earthquakes without structural damage but with some non-structural damage. To avoid collapse during a major earthquake, Members must be ductile enough to absorb and dissipate energy by post elastic deformation. Redundancy in the structural system permits redistribution of internal forces in the event of the failure of key elements. When the primary element or system yields or fails, the lateral force can be redistributed to a secondary system to prevent progressive failure.

When a structure is subjected to an earthquake excitation, it interacts with the foundation and the soil, and thus changes the motion of the ground [2,8]. This means that the movement of the whole ground-structure system is influenced by the type of soil as well as by the type of structure. Understanding of soil structure interaction will enable the designer to design structures that will behave better during an earthquake.
V. Conclusions

From the above results and discussions, the following conclusions can be drawn:

- The shear wall and its position have a significant influence on the time period, the time period is not influenced by the type of soil, in tall building with box shape shear walls is showing the low time period which shows a very significant performance.
- Shear is affected marginally by placing of the shear wall, grouping of shear wall and type of soil. The shear is increased by adding shear wall due to the increase in the seismic weight of the building.
- The Axial force and Moment in the column increases when the type of soil changes from hard to medium and medium to soft. Since the column moment increases as the soil type changes, soil structure interaction must be suitably considered while designing frames for seismic force.
- It is evident that the maximum column axial force is various with type of soil and placing of the shear wall.
- It is evident that the maximum column shear force in X-direction is influenced by the type of soil and placing of the shear wall.
- It is evident that the maximum column shear force in Y-direction has no influence on the type of soil and placing shear wall.
- It is evident that the maximum column torsion is the same for all columns in a structure, but is influenced by the type of soil and placing shear wall.
- It is evident that the maximum column moment in X-direction has no influence on the type of soil and placing shear wall.
- It is evident that the maximum column moment in Y-direction is influenced by the type of soil and placing shear wall.
- It is evident that the maximum column moment in Y-direction is influenced by the type of soil and placing shear wall.
- It is evident that the results from 1.2 (DL + IL ± EL) combination load is closed to the 1.5 (DL + EL) and there is no more difference between these combination load.
- Based on the analysis and discussion, shear wall are very much suitable for resisting earthquake induced lateral forces in multistoried structural systems when compared to multistoried structural systems without shear walls. They can be made to behave in a ductile manner by adopting proper detailing techniques.
- According to IS-1893:2002 the number of modes to be used in the analysis should be such that the total sum of modal masses of all modes considered is at least 90 percent of the total seismic mass. Here the maximum mass is for the tall building with box shape RC shear wall.
- ETABS is the robust software which is utilized for analyzing any kind of multi building structures.

Acknowledgments

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Conflict of Interest

The author declare no conflict of interest.

Data Availability Statement

All data generated or analysed during this study are included in this article.

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A Novel Analytical Approach for Axial Load Capacity Evaluation of Stiffened Hollow Steel Columns Utilizing Finite Element Method

By Ebrahim Makled

Abstract- This study presents a finite element (FE) investigation of stiffened and unstiffened box hollow columns having compact, non-compact and slender cross-sections for short as well as long columns. The available analytical methods neglected the effect of stiffener’s length when calculating stiffened hollow steel sections axial capacity. Therefore, an extensive study was conducted on the effect of stiffener length on ultimate strength of steel columns having box hollow sections. Also, the effect of different numbers of stiffeners on ultimate strength of steel columns considering five different grades of steel was numerically studied using nonlinear finite element analysis. A nonlinear (FE) analysis of steel columns which accounts the effects of residual stresses and initial local and global imperfections in long columns was performed. The current FEM results and the analytical methods such as effective width equations were compared and discussed. The FE models built in this study is verified against the available experimental data under axial compression and showed good agreement.

Keywords: local buckling - stiffened hollow square sections – nonlinear analysis - slender hollow square columns - stiffener length.

GJRE-E Classification: FOR Code: 090506

Strictly as per the compliance and regulations of:
A Novel Analytical Approach for Axial Load Capacity Evaluation of Stiffened Hollow Steel Columns Utilizing Finite Element Method

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Keywords: local buckling - stiffened hollow square sections – nonlinear analysis - slender hollow square columns - stiffener length.

1. Introduction

In industrial buildings, square hollow steel columns are regularly utilized, but they are employed more regularly in supporting structures for bridge design. These columns are produced in stiffened and unstiffened box hollow columns having compact, non-compact and slender cross sections. The global and local buckling are two different buckling modes that can occur in compression steel elements. The primary effect of local and global buckling is a reduction of the stiffening and loading capacity of the member. The local and global buckling mode is significant affected by the ratio \( B/t \) ratio, \( L_e/r \) ratio, and boundary conditions of the member. The initial imperfections, residual stress, and boundary conditions are critical factors in determining the ultimate strength of square hollow steel columns in compression members[1].

Many investigations on the behavior of square hollow steel columns have been conducted in the recent decades. The stiffened and unstiffened square hollow columns (SHC), rectangular hollow columns (RHC) were investigated by Tao et al. [2]. The stiffened square hollow columns has only two longitudinal stiffeners welded to its longer sides as opposed to the four longitudinal stiffeners that were once present on each side of the stiffened square hollow columns (SHC). One of the most important considerations was the ratio (B/t). The tubes also come with or without stiffeners. Comparing the ultimate strength of the experimental test results are presented.

A comparison experiment test study between unstiffened and stiffened stainless steel hollow columns was presented by Dabaon et al. [3]. The ratio of length-to-depth \((L/B)\) was fixed at a value of 3, but the depth-to-thickness ratio fluctuated from 60 to 90. For stiffened and unstiffened sections, the ultimate strengths of this columns, buckling modes, and axial load verses axial strain are compared.

Somodi and Kövesdi [4] focuses on the experimental measurements of the residual stress on welded box steel hollow columns with different steel grades and different B/t ratios. The aim of this investigation is to estimate the residual stresses, to determine the maximum compressive and tensile residual stresses.

The experimental tests and finite element (FE) method of hollow long steel columns with non-compact and compact unstiffened sections were developed by Khan et al. [5]. The experimental test results concluded that the non-compact sections with \( L_e/r > 24 \) collapsed as a consequence of the combination of global and local buckling (G and L). Moreover, the compact cross-sections having \( L_e/r \) values between 35 to 109 collapsed in accordance with global buckling. According to the test data of estimation of slender non-compact box sections, it is necessary take into account that the reduction factor resulting from the global and local buckling effects.

Javidan et al.[6] studied the behavior and ultimate strength of an innovative steel hollow long column. The suggested innovative columns are made of mild steel plates that are joined at the corners to mild steel tubes. According the test and FE modeling, a special focus is given to the effect of fabrication initial imperfections, and residual stresses, and welding methods on the behavior of the suggested long hollow columns. Because of the compatibility between the steel
plates and tubes in the column, the studied innovative steel hollow column specimens are demonstrated to have excellent compressive behavior, which significantly increases their strength and ductility.

El-Sayed et al. [7] presented a novel polymer-mortar system that strengthened square hollow columns, proving their behavior and strength. According to the square hollow short columns strengthened using polymer-mortar layer with the thickness equal to 6mm, a maximum axial strength improvement of 31.6% was achieved. For long columns, a polymer-mortar layer applied in 6 mm thickness on all four sides resulted in a ultimate strength gain of 76.7%.

Zheng et al. [8] studied the impacts of cold-forming in the behavior of stainless steel cold-formed hollow steel tube columns. In this investigation a total of 19 and 32 specimens were tested for short and long columns, respectively. The buckling modes of the experimental specimens included the global buckling, local buckling, local–global buckling, and material strain hardening after yielding. The experimental test specimens collapsed in four different modes: global, local, both local-global, and plastic strength after yielding.

Cold-formed hollow columns made of lean duplex stainless steel (LDSS) were designed and demonstrated by Anbarasu and Ashraf [9]. These columns that mainly collapsed due to the interaction of flexural and local buckling modes. In this study, the geometric parameters of the (LDSS) hollow column sections were selected so that the local and global buckling stresses are almost equal.

Nassimia et al. [10] developed a novel hollow columns made of ultra-high-strength steel tubes and corrugated plates. The corrugated plates that make up the suggested novel produced columns are welded to ultra-high strength (UHS) steel tubes and have a yield stress of $f_y = 1250 \text{ MPa}$ at the edges. The yield stress of the steel tube The results demonstrated that the suggested novel columns are very efficient and ductile under axial compression loads.

The finite element (FE) investigation of the fixed ended LDSS slender hollow columns with square (SHC), and non-regular hollow columns (NRHCs) was developed by Patton and Singh [11]. The non-regular hollow columns (NRHCs) such as L-(LHC) and T-(THC) shaped cross-sections. The Abaqus software was used to create the finite element (FE) models was conducted to under pure axial compression. The the finite element (FE) results of square hollow columns and non-regular hollow columns were then compared with the design equations by the ASCE 8-02 and EN1993-1-4 requirements. The finite element results and code predictions have been demonstrated to agree significantly.

Schillo and Feldmann [12] investigated the both global and local buckling mods of the steel square hollow columns. The experiments were verified with the FE using Ansys software. The research presents an analytical method for determining a reduction factor that depends on slenderness. The finite element modeling of experimental tests on slender square hollow columns, subject to combined global and local buckling of steel plates was developed by Pavlovic et al. [13]. The parametric investigation used in the FE analysis are the influence of different imperfections, the cross-section geometry for cold-formed and welded columns, and the columns length. According to the results of a study, the initial imperfection can reduce resistance by up to 45% compared with perfect column.

The nonlinear finite element (FE) analysis of the square hollow stiffened and unstiffened high strength stainless steel (HSS) columns was developed by Elliloby[14]. The column ultimate strengths, the axial load-shortening curves, and the collapsed modes were predicted for the unstiffened and stiffened columns. The main objective of this study was to investigate the effects of various section geometries on the columns strength. Hilo et al. [15] studied the FE analysis on the ultimate strength and behavior of polygonal hollow steel tube columns under axial compression load. In this investigation, different cross sections, including rectangular, circular, square, pentagonal, and hexagonal ones, have been provided. The finite element models have been analyzed to find the effect of the different cross-section shape, thickness, and length on the axial load behavior of the polygonal hollow steel tube columns.

The FE analysis on the ultimate strength and behavior of cold-formed steel rectangular and square hollow columns with two opposing circular holes in the center, at the height of the column was studied by Singh et al. [16]. The parametric analysis has been carried out, taking into account a wide range of cross-sectional slenderness and the size of the circular holes.

The mechanical performance of the hollow steel columns is significantly impacted by sectional residual stress. The sectional residual stress have been conducted by Ban et al. [17], Cao et al. [18], and other researchers. These investigation were carried out to study the effect of the residual stress on the behavior and ultimate strength of hollow steel columns. They concluded that the sectional residual stress has a significant effect on the buckling capacity and behavior of hollow columns under axial compression loads.

In case of the stiffened hollow steel columns, it is observed that there has been just a limited amount of study conducted under monotonic loading including the local and global buckling effects. The present study aims to investigate the performance and ultimate strength of stiffened square hollow columns in the short as well as long columns, under axial compression loads.
Based on the non-linear finite element (FE) analysis, this investigation was carried out to evaluate the influence of major steel tube columns parameters such as stiffener length, the ratio $B/t$, and yield strength on the hollow steel short column's performance. The main objective of the parametric study was to develop a novel mathematical equation to predict the ultimate strength of box steel sections, in addition the study was conducted on effect of the stiffeners length and propose a novel equation to calculate the optimal stiffeners length. The comparison between the current (FE) results and analytical methods is presented. In case of the long columns, a comparison was made for the stiffened and unstiffened sections to investigate how the stiffeners affect the columns ultimate strength. The numerical study is briefly described, with the variable parameters being $KL_e/r$ and the ratio $(B/t)$ equals to (50-12.5).

II. Finite Element Modeling

a) General Description

In this investigation, the finite element software ABAQUS [31] is used to create an accurate finite element model for estimating the behavior and ultimate strength of the steel tube columns with regard to axial compression loads.

b) Initial Imperfection

The initial imperfection of the hollow square columns was taken into account in the load-deflection analysis. It is assumed that the first buckling mode shape obtained from the eigen value buckling analysis is the shape of the local and global initial imperfections. The Japan Standard for Highway Bridges (JSHB) [19] prescribes maximum initial global displacement as $(L/1000)$. The AISC 360-05 [20] prescribes maximum initial displacement as $(L/1500)$. According to experimental measurements in [13], the global imperfection in the Y- and X-direction for hollow long columns appeared to be around $(H/1200 - H/1040 - H/1600)$. For square hollow columns in this study, the initial imperfection value was taken as $(0.01B)$ for local buckling and $(0.001L_e)$ for global buckling according to Chinese Standard GB50017-2002 [21].

c) Residual Stresses

In this paper, according to the experimental test result by Somodi and Kövesdi [4]. To estimate the compressive residual stresses, two models of the residual stress are developed. Equations (1) and (2) were developed to provide the best approximation to the average compressive residual stresses that were measured. The “predefined field” for the initial stress option is available in FE software to model residual stress. The typical residual stress distribution of the steel square hollow stiffened and unstiffened sections are shown in Figure 1.

\[
\sigma_{rc} = 70 - 2t + t^2 - (20900 - 3600t) \left(\frac{b}{t}\right)^{-1} \quad MPa
\]  
\[\text{If } t \leq 5 \text{ mm:}\]

\[
\sigma_{rc} = 70 - 2t + t^2 - (4350 - 290t) \left(\frac{b}{t}\right)^{-1} \quad MPa
\]  
\[\text{If } t \geq 5 \text{ mm:}\]

Where $t$ (mm) and $b$ (mm) is the thickness and width of the steel box columns, respectively.

![Figure 1: Residual stress distributions](image)

\[
\sigma_{\tau} = f_y
\]

\[
\sigma_{\tau} = f_y
\]

\[
\sigma_{\tau} = f_y
\]

$d$) Material Model

In this research, the hollow square columns was modeled by the elastic-plastic model, as shown in Figure 2. The Poisson's ratio was considered to be 0.3. In addition, the plastic zone is with a linear hardening and the hardening modulus was considered $0.005E_y$ with $E_y$ is the elastic modulus of steel [22]. The ultimate strength is clearly not significantly affected by the use of $(\sigma-\varepsilon)$ steel relationship, and the load-deformation curve is only slightly affected. Only slightly affects the load-deformation curve at a later stage [23].
The FE model’s accuracy in the present parametric study was verified using previous experimental test results. The verification study was carried out on two short square columns that were tested by Tao et al. [2] and four long columns that were tested by Khan et al. [5].

a) **Material and Geometric Properties**

For short columns, the steel material for the finite element models was assumed to be the elastic-plastic model as shown in Figure 2. The yield strength $f_y = 234.3 \, MPa$, Elastic modulus $E_s = 208 \, GPA$, yield strain ($\%$)0.137, and the ultimate strength $F_{ult} = 343.7 \, MPa$. The investigated specimens’ labels and geometric properties are shown in Table 1 and Figure 3.

**Table 1**: Dimensions of the stiffened and unstiffened short columns tests in Tao et al. [2]

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>$B$ (mm)</th>
<th>$L$ (mm)</th>
<th>$D/t$</th>
<th>$h_s \times t_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SS25</td>
<td>250</td>
<td>750</td>
<td>100</td>
<td>$35 \times 2.5$</td>
</tr>
<tr>
<td>2</td>
<td>US25</td>
<td>250</td>
<td>750</td>
<td>100</td>
<td>---</td>
</tr>
</tbody>
</table>

**Figure 3**: The specimens investigated by Tao et al. [2]

For long columns, the steel material for the finite element models was assumed to be an elastic-plastic model as shown in Figure 2. The yield strength $f_y = 762 \, MPa$, The elastic modulus $E_s = 213 \, GPA$, The yield strain 0.4157 ($\%$), and the ultimate strength $F_{ult} = 819 \, MPa$. The verification was performed for slender welded box sections with $L_e/r = (77, 66, 28, \text{and } 59)$ and $b_s/b = (1.0 \text{ and } 0.8)$. The dimensions of test specimens are shown in Table 2 and the illustration of the experimental test layout is shown in Figure 4.
Figure 4: Illustration of the experimental test layout

Table 2: Dimensions of the test specimens for the long columns tested by Khan et al. [5]

<table>
<thead>
<tr>
<th>Test specimens</th>
<th>B (mm)</th>
<th>t (mm)</th>
<th>B/t</th>
<th>Le (mm)</th>
<th>Le/r</th>
<th>b_e/b</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS15SL2</td>
<td>74.57</td>
<td>4.93</td>
<td>15</td>
<td>2512</td>
<td>77</td>
<td>1.0</td>
</tr>
<tr>
<td>HS25SL3</td>
<td>125.20</td>
<td>4.92</td>
<td>25</td>
<td>3512</td>
<td>66</td>
<td>0.8</td>
</tr>
<tr>
<td>HS25SL1</td>
<td>125.21</td>
<td>4.92</td>
<td>25</td>
<td>1512</td>
<td>28</td>
<td>0.8</td>
</tr>
<tr>
<td>HS20SL2</td>
<td>99.39</td>
<td>4.92</td>
<td>20</td>
<td>2512</td>
<td>59</td>
<td>1.0</td>
</tr>
</tbody>
</table>

b) Loading and Boundary Conditions

In fact, there are two types of loading application methods: force-controlled loading and displacement-controlled loading. In this paper, force-controlled loading technique was used. The square hollow columns were modeled using the Finite Element Analysis (FEA) under monotonic loading. In case of short columns, two loading plates coupled with a steel tube by tie constraints were used. The boundary condition of the finite element (FE) model was set at the loading plate, as shown in Figure 5. The axial load was applied by carrying out a distributed load on the loading plate.

Figure 5: Modeling of the short columns
In the case of long columns, two reference points have been created and constrained to the loading plate of all hollow square columns specimens by rigid body constraints and set the boundary condition of the (FE) model at the reference point. Both column ends were modeled as pinned condition, i.e., both ends were free to rotate. While the upper end was unconstrained in the vertical axis to apply the external load. The square steel tube coupled with the loading plate by tie constraint is shown in Figure 6. The axial load was applied in the form of distributed load on the loading plate.

![Figure 6: Modeling of the long columns](image)

c) **Element Type and Mesh**

The short hollow columns in this study were modeled using 4-node reduced integration doubly curved thin or thick shell element (S4R). The loading plate was modeled using 4-node linear tetrahedron element. The approximate global element size 12 mm was used in this study. While in case of hollow steel long columns, the (C3D4) 4-node linear tetrahedron element was used. In addition, the approximate global size equal to 12 mm was used in this study.

d) **Accuracy of Adopted Models**

i. **Short Columns**

The comparison of the experimental test results and the finite element results for the US25 and SS25 specimens are shown in Figure 7 and Figure 8, respectively. The mean values of the ratio between axial load deduced by FE and test ($\frac{N_{FEM}}{N_{TEST}}$) are 1.02, and 1.01 for the US25 and SS25 specimens, respectively. From this study, the predictions of ultimate strengths are very close to the test given by Tao et al. [2]. In addition, the mode of failure due to buckling obtained from the geometric nonlinear buckling FE analysis is shown in Figure 9.
Figure 7: Comparison of the experimental test results and the finite element results for the (US25) specimen

Figure 8: Comparison of the experimental test results and the finite element results for the (SS25) specimen
ii. **Long Columns**

The comparison of the experimental test results and the finite element (FE) results for the HS15SL2, HS25SL3, HS25SL1, and HS20SL2 specimens are shown in Figure 10, Figure 11, Figure 12, and Figure 13, respectively. The mean values of $N_{FEM}/N_{Test}$ are, respectively, 1.01, 1.03, 1.02, and 0.99 for the HS15SL2, HS25SL3, HS25SL1, and HS20SL2 specimens. From this study, the predicted ultimate strengths are very close to the given by tests in Khan et al. [5]. The HS15SL2 and HS20SL2 specimens with unstiffened compact section with $b_e/b = 1$ and $L_e/r = 77$ and 59, respectively, failed due to global buckling only. The comparison of the buckling modes for the experimental test and FE model is shown in Figure 14 for the HS15SL2 specimen. The HS25SL3 and HS25SL1 specimens, with unstiffened slender sections with $b_e/b = 0.8$ and $L_e/r = 66$ and 28, respectively, failed due to interaction between global and local buckling. The failure buckling modes for the HS25SL3 and HS25SL1 specimens are shown in Figure 15.
**Figure 11:** Comparison of the experimental test results and the finite element results for the HS25SL3 specimen.

**Figure 12:** Comparison of the experimental test results and the finite element results for the HS25SL1 specimen.
Figure 13: Comparison of the experimental test results and the finite element results for the HS20SL2 specimen.

Figure 14: Comparison of the buckling mode for the experimental test and FE model for the HS15SL2 specimen.

Figure 15: The global and local buckling modes for the unstiffened tube column.
IV. Parametric Study

a) General Description

Based on the study of verification, A parametric investigation was carried out to create three-dimensional finite element models that simulate the stiffened and unstiffened hollow steel columns under axial compression. These models focus on the global and local buckling and were divided into two cases. Case 1 studies the local buckling's effects on the behavior and ultimate strength of hollow steel short columns. Case 2 studies the influence of stiffeners on the ultimate strength and performance of hollow steel long columns. The non-linear finite element (FE) analysis is used in the study to understand the effect of main structural parameters such as $L_e/r$, stiffener length, the ratio of width-to-thickness $B/t$, and the yield stress on the hollow steel column performance.

b) Columns Geometry

i. Short Columns

The steel material model for the finite elements was assumed to be an elastic-plastic model as shown in Figure 2, in addition taken as the hardening modulus equal to $0.005E_s$. The Young's modulus $E_s = 200000$ MPa and Poisson's ratio $\nu = 0.3$. The studied parameters were the yield strength $f_y = 240, 360, 460, 560, \text{ and } 779$ MPa, the stiffeners length ($h_s$), and the ratio $B/t$. The columns length is $L = 750$ mm and the columns width is $B = 250$ mm for all the specimens. The thickness of the stiffeners is considered same as the thickness of the tube, as listed in Tables (3 and 4). The shapes and dimensions of the investigated steel columns are shown in Figure 16. The specimen label shows the whether the sections is unstiffened (US), stiffened using single stiffener per section wall (SS), or stiffened using double stiffener per section wall (DS). In addition, the label suffixed by the thickness of the section walls in (mm).

The boundary condition as described previously and shown in Figure 5. For square hollow columns in these investigation, the initial local imperfection's value has been set to $0.01B$. In addition, this study used the distribution of residual stress shown in Figure 1. The columns in this study were modeled using 4-node shell elements (S4R) and the approximate global size of the mesh is about 12 mm.

---

Figure 16: The investigated cross-sections' shapes and dimensions (a) Unstiffened sections US, (b) Stiffened sections with one stiffener SS, (c) Stiffened sections with two stiffeners DS

Table 3: The parameters and dimensions of hollow steel short columns used in the parametric study

<table>
<thead>
<tr>
<th>Unstiffened sections (US)</th>
<th>Stiffened sections with one stiffener (SS)</th>
<th>Stiffened sections with two stiffeners (DS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t$ mm</td>
<td>$f_y$ MPa</td>
<td>$B/t$</td>
</tr>
<tr>
<td>1</td>
<td>240</td>
<td>250</td>
</tr>
<tr>
<td>1.5</td>
<td>240</td>
<td>167</td>
</tr>
<tr>
<td>2</td>
<td>240</td>
<td>125</td>
</tr>
<tr>
<td>3.2</td>
<td>240</td>
<td>78</td>
</tr>
<tr>
<td>5.1</td>
<td>240</td>
<td>48</td>
</tr>
<tr>
<td>7.7</td>
<td>240</td>
<td>32</td>
</tr>
<tr>
<td>10</td>
<td>240</td>
<td>25</td>
</tr>
<tr>
<td>16</td>
<td>240</td>
<td>16</td>
</tr>
<tr>
<td>1</td>
<td>360</td>
<td>250</td>
</tr>
<tr>
<td>1.5</td>
<td>360</td>
<td>167</td>
</tr>
<tr>
<td>2</td>
<td>360</td>
<td>125</td>
</tr>
<tr>
<td>3.2</td>
<td>360</td>
<td>78</td>
</tr>
</tbody>
</table>
Table 4: The stiffeners length ($h_s$) of stiffened hollow steel short columns used in the parametric study, where $f_y = 779$ MPa

<table>
<thead>
<tr>
<th>Stiffened sections with one stiffener (SS)</th>
<th>Stiffened sections with one stiffener (SS)</th>
<th>Stiffened sections with two stiffeners (DS)</th>
<th>Stiffened sections with two stiffeners (DS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t$ mm</td>
<td>$h_s$ mm</td>
<td>$t$ mm</td>
<td>$h_s$ mm</td>
</tr>
<tr>
<td>1</td>
<td>10</td>
<td>1</td>
<td>40</td>
</tr>
<tr>
<td>1.5</td>
<td>10</td>
<td>1.5</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>2</td>
<td>40</td>
</tr>
<tr>
<td>3.2</td>
<td>10</td>
<td>3.2</td>
<td>40</td>
</tr>
<tr>
<td>5.1</td>
<td>10</td>
<td>5.1</td>
<td>40</td>
</tr>
<tr>
<td>7.7</td>
<td>10</td>
<td>7.7</td>
<td>40</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>16</td>
<td>10</td>
<td>16</td>
<td>40</td>
</tr>
<tr>
<td>1</td>
<td>20</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>1.5</td>
<td>20</td>
<td>1.5</td>
<td>50</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>3.2</td>
<td>20</td>
<td>3.2</td>
<td>50</td>
</tr>
<tr>
<td>5.1</td>
<td>20</td>
<td>5.1</td>
<td>50</td>
</tr>
<tr>
<td>7.7</td>
<td>20</td>
<td>7.7</td>
<td>50</td>
</tr>
<tr>
<td>10</td>
<td>20</td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>16</td>
<td>20</td>
<td>16</td>
<td>50</td>
</tr>
<tr>
<td>1</td>
<td>30</td>
<td>1</td>
<td>60</td>
</tr>
<tr>
<td>1.5</td>
<td>30</td>
<td>1.5</td>
<td>60</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>2</td>
<td>60</td>
</tr>
<tr>
<td>3.2</td>
<td>30</td>
<td>3.2</td>
<td>60</td>
</tr>
<tr>
<td>5.1</td>
<td>30</td>
<td>5.1</td>
<td>60</td>
</tr>
<tr>
<td>7.7</td>
<td>30</td>
<td>7.7</td>
<td>60</td>
</tr>
<tr>
<td>10</td>
<td>30</td>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>16</td>
<td>30</td>
<td>16</td>
<td>60</td>
</tr>
</tbody>
</table>
ii. **Long Columns**

The steel material for the finite element models was assumed to be an elastic-plastic with linear hardening model. In addition, the hardening modulus has been set to equal to $0.005E_s$ as shown in Figure 2. The Young's modulus $E_s = 200000 \text{ MPa}$ and the Poisson's ratio $\nu = 0.3$. The boundary condition as described previously and shown in Figure 6. For square hollow columns in these investigation, the initial global imperfection value was taken as $0.001L$. The studied parameters were the columns length $(L)$ and the ratio $(B/t)$, as listed in Tables (5 and 6). In addition, the columns width $B = 250 \text{ mm}$ for all specimens, the yield strength is used in this study $f_y = 779 \text{ MPa}$, and the stiffeners length $h_s = 35 \text{ mm}$. The thickness of the stiffener is the same as the thickness of the tube. The specimens investigated are shown in Figure 17. The specimens’ labels are as follows:

1. (US - t - L) = (US) Unstiffened section - (t) thickness – (L) column length.
2. (SS - t - L) = (SS) Stiffened section with one stiffener-(t) thickness - (L) column length.

**Figure 17:** The cross-sections investigated in the current study and the manufacturing method (a) Unstiffened section, (b) Stiffened section with one stiffener per wall

**Table 5:** The dimensions of the unstiffened section, where the ratio $(B/t) = 12.5$ and 50

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>$(B/t)$</th>
<th>$KL_e \over r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>US-20-2750</td>
<td>12.5</td>
<td>29.2</td>
</tr>
<tr>
<td>2</td>
<td>US-20-3000</td>
<td>12.5</td>
<td>31.8</td>
</tr>
<tr>
<td>3</td>
<td>US-20-4000</td>
<td>12.5</td>
<td>42.4</td>
</tr>
<tr>
<td>4</td>
<td>US-20-5000</td>
<td>12.5</td>
<td>53</td>
</tr>
<tr>
<td>5</td>
<td>US-20-6000</td>
<td>12.5</td>
<td>63.7</td>
</tr>
<tr>
<td>6</td>
<td>US-20-7000</td>
<td>12.5</td>
<td>74.3</td>
</tr>
<tr>
<td>7</td>
<td>US-20-8000</td>
<td>12.5</td>
<td>84.9</td>
</tr>
<tr>
<td>8</td>
<td>US-5-3000</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>9</td>
<td>US-5-4000</td>
<td>50</td>
<td>40</td>
</tr>
<tr>
<td>10</td>
<td>US-5-5000</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>11</td>
<td>US-5-6000</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td>12</td>
<td>US-5-7000</td>
<td>50</td>
<td>70</td>
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<td>13</td>
<td>US-5-8000</td>
<td>50</td>
<td>80</td>
</tr>
<tr>
<td>14</td>
<td>US-5-9000</td>
<td>50</td>
<td>90</td>
</tr>
</tbody>
</table>

**Table 6:** The dimensions of the stiffened sections with one stiffener, where the ratio $(B/t) = 12.5$ and 50

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>$(B/t)$</th>
<th>$KL_e \over r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SS-20-3000</td>
<td>12.5</td>
<td>33.1</td>
</tr>
<tr>
<td>2</td>
<td>SS-20-4000</td>
<td>12.5</td>
<td>44.1</td>
</tr>
<tr>
<td>3</td>
<td>SS-20-5000</td>
<td>12.5</td>
<td>55.1</td>
</tr>
<tr>
<td>4</td>
<td>SS-20-6000</td>
<td>12.5</td>
<td>66.2</td>
</tr>
</tbody>
</table>
c) Results and Discussion of Parametric Study

i. Unstiffened Short Columns FE Results against Analytical Methods

Most standards and specifications use the effective width approach to take into account the local buckling in case of the slender hollow steel tube cross-sections. This theory was developed based on redistribution of the stress on a steel tube with the average ultimate stress $\sigma_u$ as shown in Figure 18. According to Von Karman et al. [28], the effective width $b_e$ is the only part of the width that can resist the loading, but there is no loading on the plate’s central part. The effective width is represented in Figure 18(b).

$$b_e = \alpha \frac{\sigma_{ul}}{\sigma_y}$$  \hspace{1cm} (3)

Where $\alpha = 0.651$ for heavily welded tubes. Which accounts for geometric imperfections and residual stress. The stress of local buckling $\sigma_{ul}$ is presents in Eq. (4), as shown below:

$$\sigma_{ul} = \frac{K \pi^2 E_s}{12(1 - v^2)(b/t)^2}$$  \hspace{1cm} (4)

Where the coefficient of plate buckling ($K$) can be considered as 4 for hollow sections and $N_x = (b_e/b)A_f f_y$. Von Karman et al. [28] developed the first effective width expression in 1932. This expression states that a width of plate ($b$) and effective width ($b_e$) can be used to evaluate the ultimate strength capacity. Von Karman’s effective width can be written in terms of the yield stress $\sigma_y$ and critical stress $\sigma_{CR}$ as follows:

$$\frac{b_e}{b} = \sqrt{\frac{\sigma_{CR}}{\sigma_y}}$$  \hspace{1cm} (5)
Where:

\[
\sigma_{CR} = \frac{K\pi^2Et^2}{12(1-v^2)b^2} \tag{6}
\]

Where the buckling coefficient \(K = 4\) in case of the simply supported plate. Winter [29] subsequently modified von Karman’s equation to:

\[
b_e = \frac{\sigma_{CR}}{\sigma_e} \left( 1 - 0.25 \frac{\sigma_{CR}}{\sigma_e} \right) \tag{7}
\]

The second term within the bracket out Winter equation is mainly at the point where the applied edge stress \(\sigma_e\) and yield stress \(\sigma_y\) are similar. According to the Direct Strength Method (DSM) by ANSI/AISI S100-16 [25], the theoretical equation to estimating the ultimate loads with take into account the local buckling as given in Eq. (8). Where the \(P_{cr,1}\) is the critical elastic local buckling load of the square hollow columns and \(\lambda_i\) is the non- cross-section slenderness of the cross-section and equals to \(\lambda_i = \left( f_y A_g / P_{cr,1} \right)^{0.5} \).

\[
N_{DSM} = \begin{cases} 
\frac{f_y \times A_g}{\lambda_i^3} & \text{for } \lambda_i \leq 0.776 \\
\frac{1}{1-0.15} \frac{1}{\lambda_i^{1.8}} f_y \times A_g & \text{for } \lambda_i > 0.776 
\end{cases} \tag{8}
\]

Where \(f_y\) in (MPa) and \(A_g\) in (mm²).

Fang and Chan [26] modified the direct strength method to give the predictions of safer strength for welded steel hollow columns, as shown in Eq. (9).

\[
N_{DSM}^h = \begin{cases} 
\frac{f_y \times A_g}{\lambda_i^3} & \text{for } \lambda_i \leq 0.707 \\
\frac{0.96}{\lambda_i^{0.9}} \frac{0.22}{\lambda_i} f_y \times A_g & \text{for } \lambda_i > 0.707 
\end{cases} \tag{9}
\]

In this investigation, the main objective of the analytical methods is to study the local buckling’s effects on the steel tube ultimate strength and compared with the current (FE) results. Eight FE models of unstiffened short columns was used in this case, where the \(B/t\) ratio varying from 16 to 250. According to most of the international codes like ANSI/AISC 360-16 [30] the dimensions used in these investigation provide valuable data for slender, non-compact, and compact sections. The hollow steel sections, according to ANSI/AISC 360-16 are classified for local buckling.

If \(\lambda \leq \lambda_p\), The tube is a compact section \(\lambda \geq \lambda_r\). The tube is a non-compact section \(\lambda > \lambda_r\). The tube is slender cross-sections

Where:

\[
\lambda_p = 1.12 \sqrt{E/F_y} \tag{10}
\]

\[
\lambda_r = 1.40 \sqrt{E/F_y} \tag{11}
\]

\[
\lambda = B/t \tag{12}
\]

The comparison of the analytical and the FE results for the unstiffened steel columns is summarized in Figure 19 and Table 7. According to this comparison, the present FE results produces conservative predictions of the steel tube ultimate. In addition that the average variation is around 4% between current FE models and the effective width approach by Uy[24]. As well as that, the average variation is around 6% between current FE models and modified (DSM). From this comparison, the results of present (FE) and the effective width method by Uy[24] approximately similar. This is because the current FE models and the effective width method by Uy[24] take into accounts for geometric imperfections and residual stress. Thus, it can be concluded that the proposed method can accurately predict the ultimate load capacity of short columns.

### Table 7: Dimensions and the FE results of unstiffened sections (US), where \(f_y = 779\) MPa

<table>
<thead>
<tr>
<th>Specimen label</th>
<th>(B/t)</th>
<th>(\sigma_{uu}/f_y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>US-1</td>
<td>250</td>
<td>0.11</td>
</tr>
<tr>
<td>US-1.5</td>
<td>167</td>
<td>0.16</td>
</tr>
<tr>
<td>US-2</td>
<td>125</td>
<td>0.19</td>
</tr>
<tr>
<td>US-3.2</td>
<td>78</td>
<td>0.29</td>
</tr>
<tr>
<td>US-5.1</td>
<td>48</td>
<td>0.40</td>
</tr>
<tr>
<td>US-7.7</td>
<td>32</td>
<td>0.66</td>
</tr>
<tr>
<td>US-10</td>
<td>25</td>
<td>0.82</td>
</tr>
<tr>
<td>US-16</td>
<td>16</td>
<td>0.96</td>
</tr>
</tbody>
</table>
Figur

ii. **Stiffened Short Columns FE Results against Analytical Methods**

This case deals with stiffeners in steel tube fields subjected to axial stress. There are two primary types of stiffeners:

- **Longitudinal stiffeners,** that are aligned with the steel tube length direction.
- **Transverse stiffeners,** that are aligned normal to the length direction of the steel tube.

The stiffeners can be attached to the four walls of the tube, and it is used to control the local buckling this tubes. In this study, the steel tube is without transverse stiffeners, so it is possible that the stiffener could buckle locally or could be ineffective when the stiffener length is small. There are different formulas to account for stiffeners such as the effective plate width according to Norsok standard (N-004) [27]. This standard was developed depends on a steel tube’s redistribution of stress as shown in Figure 20. The effective width $s_x$ for the stiffened sections subjected to longitudinal stress is found from:

$$
\frac{s_x}{S} C_{xx} C_{yx} C_{rs}
$$

The reduction factor in the longitudinal direction, $C_{xx}$, is found from:

$$
C_{xx} = \frac{\bar{\lambda}_p - 0.22}{\lambda_p} \quad \text{if } \bar{\lambda}_p > 0.673
$$

$$
C_{xx} = 1 \quad \text{if } \bar{\lambda}_p \leq 0.673
$$

Where: $\bar{\lambda}_p$

$$
\bar{\lambda}_p = 0.525 \frac{s}{t} \sqrt{\frac{f_y}{E}}
$$

For $B/t \leq 70$, the predictions of ultimate strength by current FE models are very close to the effective width method by Norsok. This is due to the stiffeners most likely not exhibiting any local buckling. In addition, where $B/t > 70$ the current FE results produce conservative predictions of the ultimate strength. This is due to the stiffeners most likely exhibiting local buckling.
iii. Comparison between Stiffened and Unstiffened Hollow Short Steel Columns

The proposed stiffening system may be improved by arranging the stiffeners properly, which can even change the strain softening properties. The steel tubes' dimensions were selected to provide relatively slender, non-compact, and compact sections. The numerical results for the unstiffened and stiffened steel hollow columns are summarized in Tables 8 and Tables 9, respectively. According this results, the strength of the stiffened steel tube hollow columns is remarkably higher than those of the unstiffened columns ones. The unstiffened and stiffened square steel tube columns primarily collapsed due to local buckling but at different modes, as shown in Figure 28 for US-4, SS-4, and DS-4 specimens, respectively. Figure 24 shows the ultimate strength curves of the US, SS, and the DS sections. The ultimate strength $\sigma_{ult}$ are normalized by dividing by $f_y$. The stress distributions for the US-2.5, SS-2.5, and DS-2.5 specimens are shown in Figure 22 and Figure (23a and b), respectively. The proposed stiffening method can enhance the steel tube ultimate ductility and strength. The collapsed modes of the steel columns indicate that the stiffening scheme effectively delays local buckling.
Figure 22: The stress distribution on the unstiffened columns at ultimate load for the US-2.5 specimen, where $f_y = 779 \text{ MPa}$
Figure 23: The stress distribution on the stiffened sections at ultimate load, where \( f_y = 779 \, MPa \) and stiffeners length \( h_s = 35 \, mm \)

Figure 24: Comparison of the unstiffened and stiffened steel tube columns, where \( h_s = 35 \, mm \) and \( f_y = 779 \, MPa \)

Table 8: The dimensions and FE results of the unstiffened columns

<table>
<thead>
<tr>
<th>Specimen label</th>
<th>( B/t )</th>
<th>( \sigma_{ult} / f_y ) at ( f_y ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>US-1</td>
<td>250</td>
<td>0.17, 0.14, 0.12, 0.11, 0.11</td>
</tr>
<tr>
<td>US-1.5</td>
<td>167</td>
<td>0.25, 0.20, 0.18, 0.16, 0.16</td>
</tr>
<tr>
<td>US-2</td>
<td>125</td>
<td>0.33, 0.27, 0.24, 0.22, 0.19</td>
</tr>
<tr>
<td>US-3.2</td>
<td>78</td>
<td>0.54, 0.44, 0.39, 0.35, 0.29</td>
</tr>
<tr>
<td>US-5.1</td>
<td>48</td>
<td>0.75, 0.61, 0.54, 0.49, 0.40</td>
</tr>
<tr>
<td>US-7.7</td>
<td>32</td>
<td>0.99, 0.93, 0.82, 0.74, 0.66</td>
</tr>
<tr>
<td>US-10</td>
<td>25</td>
<td>1.00, 1.00, 1.00, 0.95, 0.82</td>
</tr>
<tr>
<td>US-16</td>
<td>16</td>
<td>1.00, 1.00, 1.00, 1.00, 0.96</td>
</tr>
</tbody>
</table>
Table 9: The dimensions and FE results of the stiffened sections with one and two stiffeners, where the stiffeners length $h_s = 35 \text{ mm}$

<table>
<thead>
<tr>
<th>Specimen label</th>
<th>(B/t)</th>
<th>$\sigma_{ult}/f_y$ at $f_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>240 MPa</td>
</tr>
<tr>
<td>SS-1</td>
<td>250</td>
<td>0.46</td>
</tr>
<tr>
<td>SS-2</td>
<td>125</td>
<td>0.67</td>
</tr>
<tr>
<td>SS-2.8</td>
<td>89</td>
<td>0.81</td>
</tr>
<tr>
<td>SS-4.5</td>
<td>55</td>
<td>0.98</td>
</tr>
<tr>
<td>SS-6.7</td>
<td>37</td>
<td>0.99</td>
</tr>
<tr>
<td>SS-9</td>
<td>28</td>
<td>1.00</td>
</tr>
<tr>
<td>DS-1</td>
<td>250</td>
<td>0.61</td>
</tr>
<tr>
<td>DS-2.5</td>
<td>100</td>
<td>0.91</td>
</tr>
<tr>
<td>DS-4</td>
<td>63</td>
<td>1.00</td>
</tr>
<tr>
<td>DS-6</td>
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<td>1.00</td>
</tr>
<tr>
<td>DS-8</td>
<td>31</td>
<td>1.00</td>
</tr>
</tbody>
</table>

iv. Effect of Yield Strength of Hollow Short Steel Columns

In recent years the yield strengths of structural steel have increased. This study aims to determine the influence of the yield strength on the normalized ultimate strength ($\sigma_{ult}/f_y$) for the stiffened and unstiffened hollow steel columns. The numerical simulations were carried out for yield strength equal to (240, 360, 460, 560, and 779 MPa). Figures (25, 26, and 27) show the results for the normalized ultimate strength ($\sigma_{ult}/f_y$) versus the ratio (B/t).

Figure 25: Effect of yield strength on the normalized ultimate strength ($\sigma_{ult}/f_y$) for unstiffened sections
v. **Failure Modes**

For hollow short steel columns, in the linear buckling analysis, we studied a steel tube model that was undamaged and without any significant deformations, but now with non-linear buckling analysis, we will look at how the stiffeners affected on the capacity and buckling load of steel hollow columns. The local buckling mode should be plotted in form of deformation and stress, it is necessary for the mode that results in collapse. This is done to confirm that the buckling response is physical and that the square hollow steel columns have in real collapsed. The ultimate strength and the stress versus strain curve of model is the main output of the non-linear analysis.

Figure 28 shows the buckling modes for the US-4, SS-4, and DS-4 specimens, respectively. The stiffeners can effectively constrain the local buckling of the steel tube. Finally, the buckling of the steel tube is less obvious with the increasing of the number of stiffeners, and the stiffened steel columns have greater serviceability advantages compared to those unstiffened columns.
For long steel columns, the unstiffened compact sections with \( \frac{b_o}{t} = 1 \), width-to thickness ratio \( B/t = 12.5 \), and \( KL_e/r \) from 31 to 95 failed mainly due to the effect of global buckling without any local buckling. As well as when \( KL_e/r < 31 \), the columns failed due to the full plastic strength, as summarized in Table 13. For the stiffened sections with one stiffener with the tube thickness \( t = 20 \text{ mm} \) and the \( KL_e/r \) from 44 to 99 collapsed due to the global buckling only without any local buckling. As well as when \( KL_e/r < 44 \) the columns failed due to the full plastic strength, as summarized in Table 12. The numerical specimens for unstiffened sections with width-to thickness ratio \( B/t = 50 \) and \( KL_e/r \) from 30 to 80 failed by both of global and local buckling (G and L) as summarized in Table 14. In addition, these columns failed due to the global buckling when \( KL_e/r > 80 \). For stiffened sections with one stiffener, when the columns with \( B/t = 50 \) and \( KL_e/r < 30 \) failed by predominantly local buckling (L). As well as these columns failed due transforms into a combination of the global and local buckling (L and G) when the \( 30 < KL_e/r < 51 \), as summarized in Table 15. The buckling mode for the (US-5-4000) and (US-5-7000) specimens is shown in Figure 32 and Figure 33, respectively.

vi. Effect of the Stiffener Length for Short Columns

The effect of stiffener length on the stiffened sections with one and two stiffeners is shown in Figure 29 and Figure 30. The optimum stiffener length at different tube thickness is calculated using Eq. (17) and Eq. (18) for the stiffened columns with one (SS) and two (DS) stiffeners, respectively. The \( \frac{B/t}{r} \) ratio, the \( \frac{h_s/B}{r} \) ratio, and the finite element normalized stress results are shown in Table 10 and Table 11.

\[
\frac{h_s}{B} = 0.005t^2 - 0.02t + 0.14 \quad (17)
\]
\[
\frac{h_s}{B} = 0.005t^2 - 0.05t + 0.32 \quad (18)
\]

<table>
<thead>
<tr>
<th>( \frac{B}{t} )</th>
<th>0.04</th>
<th>0.08</th>
<th>0.12</th>
<th>0.16</th>
<th>0.20</th>
<th>0.24</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>0.20</td>
<td>0.26</td>
<td>0.30</td>
<td>0.32</td>
<td>0.31</td>
<td>0.25</td>
</tr>
<tr>
<td>175</td>
<td>0.27</td>
<td>0.34</td>
<td>0.39</td>
<td>0.41</td>
<td>0.40</td>
<td>0.35</td>
</tr>
<tr>
<td>125</td>
<td>0.33</td>
<td>0.41</td>
<td>0.47</td>
<td>0.50</td>
<td>0.49</td>
<td>0.45</td>
</tr>
<tr>
<td>78</td>
<td>0.46</td>
<td>0.56</td>
<td>0.63</td>
<td>0.67</td>
<td>0.66</td>
<td>0.62</td>
</tr>
<tr>
<td>48</td>
<td>0.63</td>
<td>0.76</td>
<td>0.85</td>
<td>0.89</td>
<td>0.91</td>
<td>0.83</td>
</tr>
<tr>
<td>32</td>
<td>0.84</td>
<td>0.99</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>25</td>
<td>0.99</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>16</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
**Figure 29**: Effect of the stiffeners’ length on the stiffened sections with one stiffener, where $f_y = 779 \text{ MPa}$

**Table 11**: The $(B/t)$ ratio, the $(h_s/B)$ ratio, and the FE normalized stress results for stiffened columns with two stiffeners, where $f_y = 779 \text{ MPa}$

<table>
<thead>
<tr>
<th>$(B/t)$</th>
<th>$h_s/B$</th>
<th>0.04</th>
<th>0.08</th>
<th>0.12</th>
<th>0.16</th>
<th>0.20</th>
<th>0.24</th>
</tr>
</thead>
<tbody>
<tr>
<td>125</td>
<td>0.40</td>
<td>0.55</td>
<td>0.63</td>
<td>0.64</td>
<td>0.66</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>78</td>
<td>0.51</td>
<td>0.68</td>
<td>0.76</td>
<td>0.78</td>
<td>0.80</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>48</td>
<td>0.63</td>
<td>0.79</td>
<td>0.88</td>
<td>0.89</td>
<td>0.91</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>0.83</td>
<td>0.93</td>
<td>0.99</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>0.91</td>
<td>0.98</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>0.96</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>
vii. The Stiffeners’ Effect on Long Columns

Figure 31 shows the comparison between normalized ultimate strength $\frac{\sigma_{\text{ult}}}{f_y}$ for the stiffened and unstiffened sections obtained from FEA results, where the tube thickness $t = 20$ mm and $t = 5$ mm. The measured normalized ultimate strength $\frac{\sigma_{\text{ult}}}{f_y}$ for these columns is summarized in Tables (12 to 15). For the compact unstiffened sections when $t = 20$ mm and $KL_e/r \leq 29$, $\frac{\sigma_{\text{ult}}}{f_y} = 1$ this means that the steel columns mainly collapsed due to the full plastic strength. In addition, when $KL_e/r > 29$ the $\frac{\sigma_{\text{ult}}}{f_y} < 1$ this means the columns failed due to global buckling. Similarly, for stiffened sections with one stiffener when $t = 20$ mm and $KL_e/r \leq 33$, the $\frac{\sigma_{\text{ult}}}{f_y} = 1$ this means the columns failed due to the full plastic strength. In addition, when $KL_e/r > 29$ the $\frac{\sigma_{\text{ult}}}{f_y} < 1$ this means the columns failed due to global buckling. The ultimate strength curves for the stiffened and unstiffened sections, when $t = 20$ mm are very close. In case of the slender sections when $t = 5$ mm the measured normalized ultimate strength $\frac{\sigma_{\text{ult}}}{f_y}$ equal to 0.43 and 0.92 for unstiffened and stiffened short columns, respectively. The ultimate strength curve for stiffened sections higher than unstiffened sections for all $KL_e/r$ values as shown in Figure 31. According to this study, the stiffeners greatly affect the ultimate strength of slender sections in the long columns.

Table 12: The dimensions and FE results of the stiffened sections with one stiffener, where the ratio $(B/t) = 12.5$

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>$B/t$</th>
<th>$KL_e/r$</th>
<th>$\frac{\sigma_{\text{ult}}}{f_y}$</th>
<th>Buckling mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SS-20-3000</td>
<td>12.5</td>
<td>33.1</td>
<td>1.00</td>
<td>Plastic</td>
</tr>
<tr>
<td>2</td>
<td>SS-20-4000</td>
<td>12.5</td>
<td>44.1</td>
<td>0.84</td>
<td>Global</td>
</tr>
<tr>
<td>3</td>
<td>SS-20-5000</td>
<td>12.5</td>
<td>55.1</td>
<td>0.67</td>
<td>Global</td>
</tr>
<tr>
<td>4</td>
<td>SS-20-6000</td>
<td>12.5</td>
<td>66.2</td>
<td>0.52</td>
<td>Global</td>
</tr>
<tr>
<td>5</td>
<td>SS-20-7000</td>
<td>12.5</td>
<td>77.2</td>
<td>0.39</td>
<td>Global</td>
</tr>
<tr>
<td>6</td>
<td>SS-20-8000</td>
<td>12.5</td>
<td>88.2</td>
<td>0.31</td>
<td>Global</td>
</tr>
<tr>
<td>7</td>
<td>SS-20-9000</td>
<td>12.5</td>
<td>99.2</td>
<td>0.25</td>
<td>Global</td>
</tr>
</tbody>
</table>
Table 13: The dimensions and FE results of the unstiffened sections, where the ratio \( \frac{B}{t} = 12.5 \)

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>( \frac{B}{t} )</th>
<th>( KL_e )</th>
<th>( \frac{\sigma_{ult}}{f_y} )</th>
<th>Buckling mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>US-20-2750</td>
<td>12.5</td>
<td>29.2</td>
<td>1.00</td>
<td>Plastic</td>
</tr>
<tr>
<td>2</td>
<td>US-20-3000</td>
<td>12.5</td>
<td>31.8</td>
<td>0.95</td>
<td>Global</td>
</tr>
<tr>
<td>3</td>
<td>US-20-4000</td>
<td>12.5</td>
<td>42.4</td>
<td>0.86</td>
<td>Global</td>
</tr>
<tr>
<td>4</td>
<td>US-20-5000</td>
<td>12.5</td>
<td>53</td>
<td>0.73</td>
<td>Global</td>
</tr>
<tr>
<td>5</td>
<td>US-20-6000</td>
<td>12.5</td>
<td>63.7</td>
<td>0.56</td>
<td>Global</td>
</tr>
<tr>
<td>6</td>
<td>US-20-7000</td>
<td>12.5</td>
<td>74.3</td>
<td>0.43</td>
<td>Global</td>
</tr>
<tr>
<td>7</td>
<td>US-20-8000</td>
<td>12.5</td>
<td>84.9</td>
<td>0.34</td>
<td>Global</td>
</tr>
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</table>

Table 14: The dimensions and FE results of the unstiffened sections, where the ratio \( \frac{B}{t} = 50 \)

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>( \frac{B}{t} )</th>
<th>( KL_e )</th>
<th>( \frac{\sigma_{ult}}{f_y} )</th>
<th>Buckling mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>US-5-3000</td>
<td>50</td>
<td>30</td>
<td>0.43</td>
<td>Local</td>
</tr>
<tr>
<td>2</td>
<td>US-5-4000</td>
<td>50</td>
<td>40</td>
<td>0.42</td>
<td>L+G</td>
</tr>
<tr>
<td>3</td>
<td>US-5-5000</td>
<td>50</td>
<td>50</td>
<td>0.41</td>
<td>L+G</td>
</tr>
<tr>
<td>4</td>
<td>US-5-6000</td>
<td>50</td>
<td>60</td>
<td>0.39</td>
<td>L+G</td>
</tr>
<tr>
<td>5</td>
<td>US-5-7000</td>
<td>50</td>
<td>70</td>
<td>0.37</td>
<td>L+G</td>
</tr>
<tr>
<td>6</td>
<td>US-5-8000</td>
<td>50</td>
<td>80</td>
<td>0.33</td>
<td>L+G</td>
</tr>
<tr>
<td>7</td>
<td>US-5-9000</td>
<td>50</td>
<td>90</td>
<td>0.29</td>
<td>Global</td>
</tr>
</tbody>
</table>

Table 15: The dimensions and FE results of the stiffened sections with one stiffener, where the ratio \( \frac{B}{t} = 50 \)

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen label</th>
<th>( \frac{B}{t} )</th>
<th>( KL_e )</th>
<th>( \frac{\sigma_{ult}}{f_y} )</th>
<th>Buckling mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SS-5-1000</td>
<td>50</td>
<td>10.3</td>
<td>0.92</td>
<td>Local</td>
</tr>
<tr>
<td>2</td>
<td>SS-5-2000</td>
<td>50</td>
<td>20.6</td>
<td>0.87</td>
<td>Local</td>
</tr>
<tr>
<td>3</td>
<td>SS-5-3000</td>
<td>50</td>
<td>30.9</td>
<td>0.84</td>
<td>Local</td>
</tr>
<tr>
<td>4</td>
<td>SS-5-4000</td>
<td>50</td>
<td>41.2</td>
<td>0.75</td>
<td>L+G</td>
</tr>
<tr>
<td>5</td>
<td>SS-5-5000</td>
<td>50</td>
<td>51.5</td>
<td>0.67</td>
<td>L+G</td>
</tr>
<tr>
<td>6</td>
<td>SS-5-6000</td>
<td>50</td>
<td>61.8</td>
<td>0.57</td>
<td>Global</td>
</tr>
<tr>
<td>7</td>
<td>SS-5-7000</td>
<td>50</td>
<td>72.1</td>
<td>0.45</td>
<td>Global</td>
</tr>
<tr>
<td>8</td>
<td>SS-5-8000</td>
<td>50</td>
<td>82.4</td>
<td>0.35</td>
<td>Global</td>
</tr>
</tbody>
</table>
Figure 31: Comparison of the current FE results for the unstiffened and stiffened columns, where the tube thickness $t = 20\,\text{mm}$ and $t = 5\,\text{mm}$.

Figure 32: The buckling mode for the (US-5-4000) specimen.
V. Development of Novel Analytical Equations

As a major result of the conducted analysis, novel equations to calculate the steel tube ultimate strength with either one or two stiffeners was presented. The proposed equations were deduced from the parametric study using data regression analysis. The strength ratio \( \sigma_{ult} / f_y \), for hollow square sections stiffened with one stiffener can be calculated using Eq. (19). The \( \alpha_y \) and \( \alpha_{hs} \) are strength reduction factors according to the yield strength and stiffeners length, respectively. The values of \( \alpha_y \) and \( \alpha_{hs} \) are determined from the parametric study, as shown in Figure 29 and Figure 26.

\[
\frac{\sigma_{ult}}{f_y} = \frac{7672}{779} \left( \frac{B}{t} \right)^{-0.62} + \alpha_{hs} + \alpha_y
\]  

(19)

Where; \( \sigma_{ult} / f_y \leq 1 \).

The reduction factors can be calculated as follows:

\[
\alpha_y = 0.36 - 0.36 \sqrt{\frac{f_y}{779}}
\]  

(20)

\[
\alpha_{hs} = \left( -106.92 \left( \frac{h_s}{B} \right)^2 + 35.95 \frac{h_s}{B} - 2.88 \right) \left( \frac{B}{t} \right)^{-0.472}
\]  

(21)

Furthermore, the strength ratio, \( \sigma_{ult} / f_y \), for hollow steel sections stiffened with two stiffeners can be calculated using Eq. (22). Where \( \beta_y \) and \( \beta_{hs} \) are strength reduction factors according to the yield strength and stiffeners length, respectively. The value of \( \beta_y \) and \( \beta_{hs} \) are determined from the parametric study, as shown in Figure 30 and Figure 27.

\[
\frac{\sigma_{ult}}{f_y} = \frac{909}{779} \times e^{-\frac{4B}{1000t}} + \beta_{hs} + \beta_y
\]  

(22)

Where; \( \sigma_{ult} / f_y \leq 1 \)

The reduction factors can be calculated as follows:

\[
\beta_y = \sqrt{\frac{f_y}{779}} \left( 0.13 - 0.0027 \frac{B}{t} \right) + 0.0027 \frac{B}{t} - 0.16
\]  

(23)

If \( \frac{B}{t} < 53 \beta_{hs} \) can be calculated as follows

\[
\beta_{hs} = \left( -0.11 \left( \frac{h_s}{B} \right)^2 + 0.04 \frac{h_s}{B} - 0.004 \right) \left( \frac{B}{t} \right)^{1.25}
\]  

(24)
VI. Conclusions

This research aims to predict the behavior of hollow steel columns under monotonic loads and determine the effect of the main parameters on ultimate strength capacity, based on the study of verification. A parametric investigation was carried out to create three-dimensional finite element models that simulate the stiffened and unstiffened hollow steel columns under axial compression loads. These models focused on both the local and global buckling and were divided into two cases. Case 1, study effect of the local buckling on the ultimate strength and behavior for stiffened and unstiffened hollow steel short columns. Case 2, study the stiffeners’ effect on the steel tube ultimate strength for long columns. The non-linear finite element (FE) analysis is used in the study to understand the effect of main structural parameters such as $K/L_e/r$, stiffener length, the ratio of width-to-thickness $B/t$, and the yield stress on the hollow steel column performance. The conclusions that can be drawn are as follows:

1. The simulation of the behavior of hollow square columns using (FE) analysis can be done with about $(1:3)$% grade of accuracy. In addition to this, the (FE) analysis can reduce cost and time when compared with experimental work. The idealized elastic-plastic material model of the steel tube and the actual material models, as well as the actual initial imperfections and manufacturing errors in the real connections employed in experimental studies, were the main causes of the insignificant variations between the FEA results and experimental testing.

2. The study was conducted on the effect of the stiffeners length and proposed a novel equation to calculate the optimal stiffeners length in case of stiffened sections with one stiffener (SS) and stiffened sections with two stiffeners (DS) for short columns.

3. When increasing the width-to-thickness ratio ($h_w/t$) of stiffeners about the optimum stiffener length, the value of $f_{ult}/f_y$ decreases due to local buckling of the stiffener.

4. The current FE model produces good predictions of the steel box columns ultimate strength compared with the analytical methods. For the unstiffened steel tube columns, the average variation in the ultimate strength depending on the results from the present FE models and the effective width method by Uy$^{[24]}$ is about 4%. Furthermore, the average variation in the ultimate strength obtained from present FE models and the modified (DSM) is about 6%. While for stiffened columns, the average variation is around 6% between current FE models and the effective width method by Norsok. Thus, it can be concluded that the proposed method can accurately predict the ultimate load capacity of short columns.

5. As a major result of the conducted analysis, a novel equation to calculate the ultimate strength of box steel sections with one and two stiffeners was presented.

6. The presence of the stiffeners remarkably increases the ultimate strength of slender sections in the long columns. But on the other hand, it has no effect on the ultimate strength of compact sections.

7. The unstiffened columns with the ratio of width to thickness $B/t = 50$ and $K/L_e/r$ from 30 to 80 collapsed as a consequence of the combination of global and local buckling (G and L). In addition, these columns collapsed according to the global buckling when $K/L_e/r > 80$. For stiffened sections with one stiffener, when the columns with $B/t = 50$ and $K/L_e/r < 30$ collapsed by the local buckling only (L). In addition, these columns collapsed as a consequence of the combination of global and local buckling (G and L) when the $30 < K/L_e/r < 51$.

References Références Referencias


5. M. Khan, B. Uy, Z. Tao, and F. Mashiri, “Concentrically loaded slender square hollow and composite columns incorporating high strength...


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Seismic Behaviour of Tall Structures with RC Shear Walls and Columns Configuration by ETABS (2021)

By Bimlendu K Gautam & Abhay K Jha

Abstract- Shear wall System Commonly used in the Tall Structures to resist/Sustain the Lateral forces Exerted due to Winds, Earthquakes, Due to Shear Wall’s high in-Plane Stiffness and Simultaneously has Capacity to take Gravity Loads, Inclusion of Shear Wall has become very inevitable in Tall Structures to resist Lateral forces. It is important & necessary to Find the Structural Configuration is effectively Safe or not. Hence In this article, The Structural analysis is conducted on Basement+G+31 Tall Building of Total Height of 108m in order to determine the Base Shear, Maximum Storey Displacement, Maximum Storey Drifts, Storey Shears, Overturning Moments and Axial Forces over the Critical Load Combination. For this Purpose, different zones are selected to investigate the effect of Lateral Forces, If Building is Either on Shear Wall Configuration or on Column Configuration. A detailed study on behaviour of both columns and RC Shear Wall is conducted with eight model made on CSI ETABS (2021) Software in the present study.

Keywords: RC shear wall, base shear, max. storey drift, max. storey displacement, storey shear, response spectrum method.

GJRE-E Classification: FOR Code: 0905
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Bimlendu K Gautam & Abhay K Jha

Abstract - Shear wall System Commonly used in the Tall Structures to resist/Sustain the Lateral forces Exerted due to Winds, Earthquakes, Due to Shear Wall's high in-Plane Stiffness and Simultaneously has Capacity to take Gravity Loads, Inclusion of Shear Wall has become very inevitable in Tall Structures to resist Lateral forces. It is important & necessary to Find the Structural Configuration is effectively Safe or not. Hence In this article, The Structural analysis is conducted on Basement+G+31 Tall Building of Total Height of 108m in order to determine the Base Shear, Maximum Storey Displacement, Maximum Storey Drifts, Storey Shears, Overturning Moments and Axial Forces over the Critical Load Combination. For this Purpose, different zones are selected to investigate the effect of Lateral Forces, If Building is Either on Shear Wall Configuration or on Column Configuration. A detailed study on behaviour of both columns and RC Shear Wall is conducted with eight model made on CSI ETABS (2021) Software in the present study. Building models, included with Shear Wall Configuration in different seismic Zones and Column Configuration in different Seismic Zone. Each of these models has Subjected to Response Spectrum method of Analysis as referred in IS 1893:2016. Building is assumed as Commercial building located in all Seismic Zones of India. The comparison of analysis results shows that how both Type of buildings are performing under lateral or seismic loads in Different Seismic zones and determining the seismic parameters like base shear, Max. Storey displacement, Max. Storey drifts, Storey Shear, Overturning Moments and Axial forces are checked out under the Critical Load Combination. In this present study software results shown Max. Storey displacement 38% lesser, Max. Storey drifts 38% and Axial reaction 19% lesser in case of shear wall. Storey Shear 9%, Overturning Moments 4% and Base Shear 3% greater in case of shear Wall Configuration, which is marginal.

Keywords: RC shear wall, base shear, max. storey drift, max. storey displacement, storey shear, response spectrum method.

I. INTRODUCTION

Shear walls are essential structural components that effectively withstand both gravity and lateral loads exerted on buildings. Their primary function is to provide lateral stiffness to buildings, thereby effectively resisting seismic forces that may arise from an earthquake. They provide lateral support for buildings. Shear walls are generality important in tall/high-rise buildings subjected to wind, seismic and other lateral forces. They are constructed from the foundation to the top story. Shear walls resist shear forces and uplift forces. Shear walls transfer horizontal forces to other components in the load path. They should be located on each level of the structure. Shear walls can have openings for windows and doors. The size and location of Shear Wall affects the seismic response. Owing to their numerous benefits to structural design, shear walls have become increasingly popular in building construction. However, their placement is crucial and requires careful evaluation. Ideally, in a floor layout, shear walls should be positioned as close as possible to the center of mass to prevent any additional moments that may arise otherwise. Therefore, it is imperative to utilize the appropriate number of shear walls with the appropriate cross-sectional area.

This study investigates the story Response parameters in all seismic zones using Shear Wall and Columns in RC framed structures. G+31 buildings are considered in different seismic seismic of India. Finite element software ETAB v 21 is used for analysis.

II. OBJECTIVE

To analyse/Investigate the Seismic Behavior of the B+G+31 building of 108m height on Shear Wall and Column Configuration by Etabs software and find various parameter such as Max. Story Displacement, peak story shear, base shear, axial forces, Overturning Moment and story drift, in all seismic zones using Linear Dynamic Method on FEM based software as per IS 1893 (Part-I): 2016.

III. METHODOLOGY

In general Structures are analysed in Software for finding more frequent results for multiple iterations, Therefore, A building is analysed in Etabs Software Which is FEM Based and Having good UI. Linear Dynamic Method (Response Spectrum) used for analysis on multiple modes (Shapes).

A General Outline of the method is used as below:

Selected a Real geometry of almost Square Shape and Done the Analytical modelling on Etabs, followed through the assignments of defined x-sections, Loads and their Combinations, Diaphragm, Support

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Conditions at base. And Lastly Analysed the structure on Selected Modes and Load Combinations.

a) Present Work Description

Taking a B+G+31 story Tall building, modelled and Analysed in ETABS 2021. Analysis Done on both option either building will be analysed on columns or on Shear Wall. So, in this Study, comparison of Parameters Like Max. Story Displacement, Max. Story drift, Story Shear, Base Shear, Overturning Moment, Axial Force is done on the both of the Structural System.

Figure 1: Plan With Column

Figure 2: Plan With Shear Wall

Figure 3: Elevation With Column

Figure 4: Elevation With Shear Wall
b) Specification of the Model
Following data used for analysis of as above mentioned RC frame building model.

<table>
<thead>
<tr>
<th>SPECIFICATION</th>
<th>DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td>B+G+31</td>
</tr>
<tr>
<td>Plan Size</td>
<td>43mx44m</td>
</tr>
<tr>
<td>Total Building Height</td>
<td>108m</td>
</tr>
<tr>
<td>Floor to Floor Height</td>
<td>(5.2x1) + (4.2x2) + (3.35x2) + (3.0x3) + (3.2x6) + 3.6 + (3.2x16) + 3.9 + (3.2x1)</td>
</tr>
<tr>
<td>No. of bays along, X-direction</td>
<td>5</td>
</tr>
<tr>
<td>No. of bays along, Y-direction</td>
<td>6</td>
</tr>
<tr>
<td>Column size</td>
<td>1200x1200, 1000x1000, 600 dia.</td>
</tr>
<tr>
<td>Beam size</td>
<td>200x500, 300x500, 300x600, 400x600</td>
</tr>
<tr>
<td>Slab Thickness</td>
<td>150mm</td>
</tr>
<tr>
<td>Shear wall Thickness</td>
<td>400thk., 300thk. (For Lift only)</td>
</tr>
<tr>
<td>Inner Wall Thickness</td>
<td>230mm AAC Block (Density- 1000kg/m3)</td>
</tr>
<tr>
<td>Outer wall</td>
<td>230mm AAC Block (Density- 1000kg/m3)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SPECIFICATION</th>
<th>DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of Concrete</td>
<td>For Column/Shear Wall- M40</td>
</tr>
<tr>
<td></td>
<td>For Slabs/ Beams -M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe500</td>
</tr>
<tr>
<td>Density of Brick</td>
<td>1000Kg/m3</td>
</tr>
<tr>
<td>Unit weight of RCC</td>
<td>2500kg/m3</td>
</tr>
</tbody>
</table>
**Table 3: Seismic Parameters**

<table>
<thead>
<tr>
<th><strong>SPECIFICATION</strong></th>
<th><strong>DATA</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Zone</td>
<td>Zone II, III, IV &amp; Zone V</td>
</tr>
<tr>
<td>Zone Factor</td>
<td>0.1, 0.16, 0.24, 0.36 (as per table 2 of IS:1893(Part-1-2016))</td>
</tr>
<tr>
<td>Importance Factor</td>
<td>1.2 (as per table 6 of IS:1893(Part-1-2016))</td>
</tr>
<tr>
<td>Response Factor</td>
<td>3 (as per table 7 of IS:1893(Part-1)-2016)</td>
</tr>
<tr>
<td>Type of frame</td>
<td>Ordinary RC moment resisting frame (as per Table-7; IS 1893:2016)</td>
</tr>
<tr>
<td>Damping Ratio</td>
<td>5%</td>
</tr>
<tr>
<td>Soil Type</td>
<td>Medium Soil (Type II)</td>
</tr>
</tbody>
</table>

**c) Load Calculations**

1. **Dead load** (Table 2 as per IS 875(part1):1987)
   - **On floor slabs:**
     - Self-weight = 0.150 * 25 = 3.725 KN/m²
     - Partition wall (assumed) = 6.4 KN/m
     - Floor finish (assumed) = 1.5 KN/m²
     - DL on floors = 3.725 + 1.5 = 5.25 KN/m²
     (As per clause 7.3.1, table 8 of IS1893 (part 1): 2016, for imposed uniformly distributed floor loads of 3KN/m² or below, the % age of imposed load is 25%.)
     - Total DL on the floor = 5.25 + (50/100) x 4 = 7.25 KN/m²
   - **On roof slabs:**
     - Self-weight = 0.150 * 25 = 3.725 KN/m²
     - Floor finish (assumed) = 1.5 KN/m²
     - DL on floors = 3.725 + 1.5 = 5.25 KN/m²
     (As per clause 7.3.2 of IS 1893 (part 1): 2016, for calculating the design seismic force of the structure, the superimposed load on top roof need not to be considered.)
     - Total DL on roof = 5.25 KN/m²
2. **Live load** (Table 1 as per IS 875(part 2):1987)
   - Live load on floors = 4 KN/m²
   - Live load on roof = 4 KN/m²
3. **Seismic Calculations (Linear Dynamic Method)**
   - **Response Spectrum Analysis:** \( Q_{ik} = A_k \omega_{ik} P_k W_i \)
   - Where, \( Q_{ik} \) is Design lateral force at the floor \( i \) in mode \( k \).
   - \( A_k \) is the Design horizontal acceleration spectrum value for the mode \( k \) of vibration.
   - \( \omega_{ik} \) is Mode Shape coeff. at a Floor \( i \) in mode \( k \).
   - \( P_k \) Modal Participation factor of the mode \( k \).
   - \( W_i \) Seismic weight of the floor \( i \).

   User will provide \( A_k \) and \( W_i \). In these \( A_k \) can be provided by specifying Seismic parameter configuration. \( W_i \) can be provided by specifying self-weight contribution in X, Y, Z direction with factor 1 and dead load and appropriate live load in all three directions. Response Spectrum Method 53 of analysis shall be performed using the Design Spectrum.

ETABS utilizes following procedure to generate the lateral seismic loads:

- User provides the value of Z I as factors for input spectrum. 2 R
- Program calculates time period for the first 12 modes or as specified by the user.
- Program calculates \( S_v/g \) for each and every mode in respect of time period & damping.
- The program calculates design horizontal acceleration spectrum \( A_k \) for different modes.
- The program also calculates mode participation factor for different modes.
- The Peak lateral seismic force at every floor in each mode is calculated.
- All the response values for each mode are calculated.
The peak response values are combined as per method (ABS or SRSS or CSM or CQC or TEN) as defined by the user to find the final results for modes.

- The Design Base shear VB (Calculated from the Response Spectrum Method) is compared with the base shear Vb (Calculated by empirical formula for fundamental time period).
- If VB is less than Vb, all of the values (Response) are multiplied by Vb/VB as per clause 7.8.2 (IS1893:2016).

Calculation of Time Period

\[ Ta = \frac{0.09h}{\sqrt{d}} \]

Where \( h = 108m \)
\( d = 44 \)
\( Ta = 1.4661 \) sec
As per IS Code 1893 (part-1) – 2016.

The Design Horizontal Seismic Coeff. (Cl. 6.4.2/ IS1983:2016)

\[ Ah = \frac{(Z/2)\times(Sa/g)}{(I/R)} \]
\( I=1.2, R=3, Sa/g = 1.36/Ta \)
Case I (Zone II, \( Z=0.1 \)) \( Ah = 0.115 \)
Case II (Zone III, \( Z=0.16 \)) \( Ah = 0.184 \)
Case III (Zone IV, \( Z=0.24 \)) \( Ah = 0.276 \)
Case IV (Zone V, \( Z=0.36 \)) \( Ah = 0.414 \)

The Design Seismic Acceleration Spectral Value (Cl. 6.4.6/ IS1983:2016)

\[ Av = \frac{(.667\times Z/2)\times 2.5}{(R/I)} \]
\( I=1.2, R=3 \)
Case I (Zone II, \( Z=0.1 \)) \( Av = 0.0670 \)
Case II (Zone III, \( Z=0.16 \)) \( Av = 0.1072 \)
Case III (Zone IV, \( Z=0.24 \)) \( Av = 0.1610 \)
Case IV (Zone V, \( Z=0.36 \)) \( Av = 0.24 \)

\( d) \ Load \ Combos \)

Load combinations that are to be used for Limit state Design (LSM) of reinforced concrete structure are listed below. 1.5(DL+LL)

1. 1.2(DL+LL+EQX)
2. 1.2(DL+LL+EQ-X)
3. 1.2(DL+LL+EQZ)
4. 1.2(DL+LL+EQ-Z)
5. 1.5(DL+EQX)
6. 1.5(DL-EOX)
7. 1.5(DL+EQQ)
8. 1.5(DL-EOQ)
9. 0.9DL+1.5EQX
10. 0.9DL-1.5EQX
11. 0.9DL+1.5EQZ
12. 0.9DL-1.5EQZ

\( IV. \ Result \ and \ Discussion \)

Results are extract and study about the Parameters like Maximum Story Displacements, drifts, story shear, Overturning moments, base shear and axial forces. By these results, These Story Response parameters will be discussed which affect the tall structures.

\( Table \ 4: \ Model \ Cases \)

| 4.1.1. Case I- Shear Wall & Seismic Zone II |
| 4.1.2. Case II- Shear Wall & Seismic Zone III |
| 4.1.3. Case III- Shear Wall & Seismic Zone IV |
| 4.1.4. Case IV- Shear Wall & Seismic Zone V |
| 4.1.5. Case V- Column & Seismic Zone II |
| 4.1.6. Case VI- Column & Seismic Zone III |
| 4.1.7. Case VII- Shear Wall & Seismic Zone IV |
| 4.1.8. Case VIII- column & Seismic Zone V |
a) Story Response - Maximum Story Displacement

Maximum Story Displacement Defined as Displacement occurred at each story level, generally high rise/multi-storey/tall Buildings, has maximum storey displacements at top floors, as height increases story displacements increases. Following are the Result extractions from model done in Etabs 2021, for the story response- Max. Story displacement. This Parameter is being analyzed for Critical Load Combination is 1.5DL-1.5 Eqx.

Table 5: Summary of Max. Storey Displacements

<table>
<thead>
<tr>
<th>SUMMARY OF MAX. DISPLACEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRITICAL CASE 1.5 DL-1.5EQX</td>
</tr>
<tr>
<td>ON SHEAR WALL</td>
</tr>
<tr>
<td>X- DIR</td>
</tr>
<tr>
<td>ZONE II</td>
</tr>
<tr>
<td>ZONE III</td>
</tr>
<tr>
<td>ZONE IV</td>
</tr>
<tr>
<td>ZONE V</td>
</tr>
</tbody>
</table>

![Graph showing maximum story displacement for shear wall and column cases](image-url)
b) Max. Story Drift

Story Response - Maximum Story Drift. Story Drift is defined as relative (Inter-storey) displacement between the stories. Higher Drift Causes the horizontal displacement of the story/building in the case of lateral forces application like earthquake and winds. Building sway laterally in case of higher drift occurred.

Total drift of the i floor = \( \Delta_i \) Inter-storey drift of i floor \((\delta)_i = \Delta_i - \Delta_{(i-1)}\)

Drift Index Drift Index = deflection/height

Following are the Result extractions from model we did in Etabs 2021, for the story response- Max. Story Drift. Followings are the tables and their graphs in eight cases, are formed for the analysis for critical case 1.5DL-1.5Eqx.

Table 6: Summary of Max. Storey Drifts

<table>
<thead>
<tr>
<th>SUMMARY OF MAX. DRIFT</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRITICAL CASE 1.5 DL-1.5Eqx</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ON SHEAR WALL</th>
<th>ON COLUMN</th>
</tr>
</thead>
<tbody>
<tr>
<td>X- DIR</td>
<td>Y- DIR</td>
</tr>
<tr>
<td>ZONE II</td>
<td>0.002127</td>
</tr>
<tr>
<td>ZONE III</td>
<td>0.003182</td>
</tr>
<tr>
<td>ZONE IV</td>
<td>0.004585</td>
</tr>
<tr>
<td>ZONE V</td>
<td>0.006737</td>
</tr>
</tbody>
</table>

Table 7: Summary of Max. Storey Shear

<table>
<thead>
<tr>
<th>SUMMARY OF STORY SHEAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRITICAL CASE 1.5 DL-1.5Eqx</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ON SHEAR WALL</th>
<th>ON COLUMN</th>
</tr>
</thead>
<tbody>
<tr>
<td>X- DIR</td>
<td>Y- DIR</td>
</tr>
<tr>
<td>ZONE II</td>
<td>11643.7931</td>
</tr>
<tr>
<td>ZONE III</td>
<td>18297.9481</td>
</tr>
<tr>
<td>ZONE IV</td>
<td>27147.0281</td>
</tr>
<tr>
<td>ZONE V</td>
<td>40720.1684</td>
</tr>
</tbody>
</table>
d) Overturning Moment

Story Response - Maximum Overturning Moments

Overturning moment defined as the Total moment of building with developed through Lateral forces applications.

Overturning moment in x- direction = Seismic Force in X-Dir x Height of building from N.G.L
Overturning moment in Y- direction = Seismic Force in Y-Dir x Height of building from N.G.L

Following are the Result extractions from model we did in Etabs 2021, for the story response- Max. Overturning Moment.

This is nothing but the torsion generated over the building due to lateral forces.

Basically, this is the moment that turns building with the central axis due to forces causes due to lateral forces at each of the story.

Torsional rigidity can be seen if overturning moments are lesser in below cases.

Followings are the tables and their graphs in eight cases, are formed for the analysis for critical case 1.5DL-1.5Eqx.

Table 8: Summary of Overturning Moment

---

e) Base Shear

Base Shear is defined as Total Force act at foundation level or lowest level of building due Seismic Building Weight.

Base Shear = Seismic Weight of the Building x Design Horizontal Coeff.(A_h)

\[ V_B = A_h \times W \]

\[ A_h = \frac{Z}{2} \frac{1}{R} \frac{Sa}{g} \]
Where,

\( A_e \) is the outline flat seismic coefficient, which relies upon the seismic zone factor (Z), response reduction factor (R), importance factor (I), and the normal reaction speeding up coefficients (\( S_a/g \)). \( S_a/g \) thus establishment relies upon the idea of soil (shake, medium or delicate soil site), characteristic span and damping of the structure.

Followings are the tables and their graphs in eight cases, are formed for the analysis for critical case 1.5DL-1.5Eqx.

**Table 9:** Summary of Base Shear

<table>
<thead>
<tr>
<th>Max. Base Shear in Shear Wall Case</th>
<th>Shear Wall Zone II</th>
<th>Shear Wall Zone III</th>
<th>Shear Wall Zone IV</th>
<th>Shear Wall Zone V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical Case</td>
<td>KN</td>
<td>KN</td>
<td>KN</td>
<td>KN</td>
</tr>
<tr>
<td>1.5DL-1.5EQ+X</td>
<td>11644</td>
<td>18298</td>
<td>27147</td>
<td>40720</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Max. Base Shear in Column Case</th>
<th>Column Zone II</th>
<th>Column Zone III</th>
<th>Column Zone IV</th>
<th>Column Zone V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical Case</td>
<td>KN</td>
<td>KN</td>
<td>KN</td>
<td>KN</td>
</tr>
<tr>
<td>1.5DL-1.5EQ+X</td>
<td>11313</td>
<td>17777</td>
<td>26375</td>
<td>39561</td>
</tr>
</tbody>
</table>

**BASE SHEAR ZONE WISE**

<table>
<thead>
<tr>
<th>BASE SHEAR IN KN</th>
<th>0</th>
<th>5000</th>
<th>10000</th>
<th>15000</th>
<th>20000</th>
<th>25000</th>
<th>30000</th>
<th>35000</th>
<th>40000</th>
<th>45000</th>
</tr>
</thead>
<tbody>
<tr>
<td>ZONE II</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ZONE III</td>
<td>11644</td>
<td>18298</td>
<td>27147</td>
<td>40720</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ZONE IV</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ZONE V</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

f) **Max. Axial Force**

Followings are the tables and their graphs in eight cases, are formed for the analysis for critical case 1.5DL+1.5LL. At node no 29 in critical case, max. Individual axial force is found in all eight cases.

**Table 9:** Summary of Max. Axial Force

<table>
<thead>
<tr>
<th>MAX. FZ (AXIAL REACTION-KN)</th>
<th>On Shear Wall</th>
<th>On Column</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>ZONE III</td>
<td>56726.213</td>
<td>67351.3876</td>
<td>-19%</td>
</tr>
<tr>
<td>ZONE IV</td>
<td>56726.213</td>
<td>67351.3876</td>
<td>-19%</td>
</tr>
<tr>
<td>ZONE V</td>
<td>56726.213</td>
<td>67351.3876</td>
<td>-19%</td>
</tr>
</tbody>
</table>
g) Discussion on Results

1. Max. Storey Displacement- As the seismic zones increases, Maximum story displacement increases in both cases either Building Analyzed on shear wall or on columns, When Critical Load Combination found 1.5DL-1.5EQx, In Various Zones II, III, IV, V the max. Story displacement found 37%, 38%, 38%, 39% lesser Value in case of Building being analyzed on shear wall respectively. Building’s max. Story displacement is under allowable limit. As per IS 1893:2016).

2. Max. Storey Drift- As the seismic zones increases, Maximum story displacement increases in both cases either Building Analyzed on shear wall or on columns, When Critical Load Combination found 1.5DL-1.5EQx, In Various Zones II, III, IV, V the max. Story drift found 37%, 38%, 38%, 39% lesser Value in case of Building being analyzed on shear wall respectively. Building’s max. Story drift is under allowable limit. As per IS 1893:2016).

3. Story Shear- As the seismic zones increases, Story shear increases in both cases either Building Analyzed on shear wall or on columns, When Critical Load Combination found 1.5DL-1.5EQx, In Various Zones II, III, IV, V the max. Story shear found 3%, greater Value in each zone, Building being analyzed on shear wall respectively. Story shear found greatest in value on ground floor.

4. Story Overturning Moment- As the seismic zones increases, No change in Story Overturning Moment in both cases either Building Analyzed on shear wall or on columns, When Critical Load Combination found 1.5DL-1.5EQx, In Various Zones II, III, IV, V the max. Story Overturning Moment found 4%, greater Value in each zone Building being analyzed on shear wall respectively. Story Overturning Moment Found Greatest in Value on Ground floor.

5. Base Shear- As the seismic zones increases, Base shear increases in both cases either Building Analyzed on shear wall or on columns, When Critical Load Combination found 1.5DL-1.5EQx, In Various Zones II, III, IV, V the max. Base shear found 3%, greater Value in each zone, Building being analyzed on shear wall respectively. Base shear found greatest in value on basement level.

6. Axial force/Reaction- As the seismic zones increases, No Change in Axial Force in both cases either Building Analyzed on shear wall or on columns, When Critical Load Combination found 1.5DL+1.5LL, In Various Zones II, III, IV, V the max. Axial force found 19%, lesser Value in each zone Building being analyzed on shear wall respectively. At column/node No. 29 max. Axial force Found.

V. Conclusion/Summary and Findings

Based on the result obtained the following conclusions can be drawn by Etabs 2021.

1. Maximum Storey Displacement found 38% average lesser if Building Analyzes over shear wall in comparison of as on column.

2. Maximum Storey Displacement Found at 108m lvl. For critical case 1.5DL-1.5EQx

3. Max. Storey displacements is min. In Zone II (Shear Wall case) is 182.17mm and maximum in Zone V (Column Case) is 897.968mm. This Building is Safe Up to Zone IV for shear Wall Case and Safe up to Zone III for Column Case. (Refer Table no. 4.10)

4. As Inter Max. storey Displacements or Max. Story Drifts relates with Storey Displacements, Max.
Storey Drift Found 38% average lesser if Building Analyses over Shear Wall in comparison of as on Column.
5. Maximum Storey Drift Found at 46.9 m Lvl. In each of Model Case, For critical case 1.5DL-1.5Eqx
6. Max. Storey Drift is min. In Zone II (Shear Wall case) is .00212 and maximum in Zone V (Column Case) is 0.011027. This Building is Safe Up to Zone IV for Shear Wall Case and Safe up to Zone II for Column Case. (Refer Table no. 4.19)
7. Storey Shear Found 3% greater if Building Analyses over Shear Wall in comparison of as on Column, Which is Marginal.
8. Storey Shear Found Greatest in Value at Ground Floor 0.00 m Lvl. In each of Model Case, For critical case 1.5DL-1.5Eqx
9. Storey Shear is min. In Zone II (Column case) is 11312.6 kN and maximum in Zone V (Shear Wall Case) is 40720.17 kN. (Refer Table no. 4.28)
10. Storey Overturning moment Found 4% greater if Building Analyses over Shear Wall in comparison of as on Column, Which is Marginal.
11. Storey overturning moment Found Greatest in Value at Ground Floor 0.00 m Lvl. In each of Model Case, For critical case 1.5DL-1.5Eqx
12. Storey overturning moment is min. In Column case is 31492751 kN-m and maximum in Shear Wall Case is 32749983 kN-m. (Refer Table no. 4.37) for Each Seismic Zone.
13. Base Shear Found 3% greater if Building Analyses over Shear Wall in comparison of as on Column, which is Marginal.
14. Base Shear is min. in Zone II (Column case) is 11313 kN and maximum in Zone V (Shear Wall Case) is 40720 kN. (Refer Table no. 4.40), Which is Greatest in Value at foundation/Base Floor -5.2 m Lvl. In each of Model Case, For critical case 1.5DL-1.5Eqx.
15. Maximum Axial force/Reaction found 19% Lesser if Building Analyses over Shear Wall in comparison of as on Column, at Node/Column No. 29, for Critical Case 1.5DL+1.5LL. (Refer Table no. 4.41)
16. Hence, If all above conclusion taken in the Consideration, Shear wall performs/behaves better than Column comparatively in case of tall Structure. To provide better safety against Maximum Story displacement, Maximum Story Drift, Story Shear, Maximum Overturning Moment, Base shear and Axial Reaction shear wall Configuration is Recommended to use.
17. For deliver more safety in the building in All Seismic Zone, Parametric Properties of Shear Wall can be improved.

References Références Referencias
1. Anshuman S, Dipendu Bhunia, Bhavin Ramjiyani, Solution of Shear Wall Location in Multi-Storey Building International Journal of Civil & Structural Engineering, 2011, Volume 2, Issue 2, Online ISSN: 0976-4399.84

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Verbs have to be in agreement with their subjects. In a research paper, do not start sentences with conjunctions or finish them with prepositions. When writing formally, it is advisable to never split an infinitive because someone will (wrongly) complain. Avoid clichés like a disease. Always shun irritating alliteration. Use language which is simple and straightforward. Put together a neat summary.

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15. Never start at the last minute: Always allow enough time for research work. Leaving everything to the last minute will degrade your paper and spoil your work.

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17. Never copy others' work: Never copy others' work and give it your name because if the evaluator has seen it anywhere, you will be in trouble. Take proper rest and food: No matter how many hours you spend on your research activity, if you are not taking care of your health, then all your efforts will have been in vain. For quality research, take proper rest and food.

18. Go to seminars: Attend seminars if the topic is relevant to your research area. Utilize all your resources.

19. Refresh your mind after intervals: Try to give your mind a rest by listening to soft music or sleeping in intervals. This will also improve your memory. Acquire colleagues: Always try to acquire colleagues. No matter how sharp you are, if you acquire colleagues, they can give you ideas which will be helpful to your research.

20. Think technically: Always think technically. If anything happens, search for its reasons, benefits, and demerits. Think and then print: When you go to print your paper, check that tables are not split, headings are not detached from their descriptions, and page sequence is maintained.

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21. **Adding unnecessary information:** Do not add unnecessary information like "I have used MS Excel to draw graphs." Irrelevant and inappropriate material is superfluous. Foreign terminology and phrases are not apropos. One should never take a broad view. Analogy is like feathers on a snake. Use words properly, regardless of how others use them. Remove quotations. Puns are for kids, not grunt readers. Never oversimplify: When adding material to your research paper, never go for oversimplification; this will definitely irritate the evaluator. Be specific. Never use rhythmic redundancies. Contractions shouldn’t be used in a research paper. Comparisons are as terrible as clichés. Give up ampersands, abbreviations, and so on. Remove commas that are not necessary. Parenthetical words should be between brackets or commas. Understatement is always the best way to put forward earth-shaking thoughts. Give a detailed literary review.

22. **Report concluded results:** Use concluded results. From raw data, filter the results, and then conclude your studies based on measurements and observations taken. An appropriate number of decimal places should be used. Parenthetical remarks are prohibited here. Proofread carefully at the final stage. At the end, give an outline to your arguments. Spot perspectives of further study of the subject. Justify your conclusion at the bottom sufficiently, which will probably include examples.

23. **Upon conclusion:** Once you have concluded your research, the next most important step is to present your findings. Presentation is extremely important as it is the definite medium though which your research is going to be in print for the rest of the crowd. Care should be taken to categorize your thoughts well and present them in a logical and neat manner. A good quality research paper format is essential because it serves to highlight your research paper and bring to light all necessary aspects of your research.

**Informal Guidelines of Research Paper Writing**

**Key points to remember:**

- Submit all work in its final form.
- Write your paper in the form which is presented in the guidelines using the template.
- Please note the criteria peer reviewers will use for grading the final paper.

**Final points:**

One purpose of organizing a research paper is to let people interpret your efforts selectively. The journal requires the following sections, submitted in the order listed, with each section starting on a new page:

**The introduction:** This will be compiled from reference matter and reflect the design processes or outline of basis that directed you to make a study. As you carry out the process of study, the method and process section will be constructed like that. The results segment will show related statistics in nearly sequential order and direct reviewers to similar intellectual paths throughout the data that you gathered to carry out your study.

**The discussion section:**

This will provide understanding of the data and projections as to the implications of the results. The use of good quality references throughout the paper will give the effort trustworthiness by representing an alertness to prior workings.

Writing a research paper is not an easy job, no matter how trouble-free the actual research or concept. Practice, excellent preparation, and controlled record-keeping are the only means to make straightforward progression.

**General style:**

Specific editorial column necessities for compliance of a manuscript will always take over from directions in these general guidelines.

**To make a paper clear:** Adhere to recommended page limits.

**Mistakes to avoid:**

- Insertion of a title at the foot of a page with subsequent text on the next page.
- Separating a table, chart, or figure—confine each to a single page.
- Submitting a manuscript with pages out of sequence.
- In every section of your document, use standard writing style, including articles ("a" and "the").
- Keep paying attention to the topic of the paper.
• Use paragraphs to split each significant point (excluding the abstract).
• Align the primary line of each section.
• Present your points in sound order.
• Use present tense to report well-accepted matters.
• Use past tense to describe specific results.
• Do not use familiar wording; don’t address the reviewer directly. Don’t use slang or superlatives.
• Avoid use of extra pictures—include only those figures essential to presenting results.

Title page:
Choose a revealing title. It should be short and include the name(s) and address(es) of all authors. It should not have acronyms or abbreviations or exceed two printed lines.

Abstract: This summary should be two hundred words or less. It should clearly and briefly explain the key findings reported in the manuscript and must have precise statistics. It should not have acronyms or abbreviations. It should be logical in itself. Do not cite references at this point.

An abstract is a brief, distinct paragraph summary of finished work or work in development. In a minute or less, a reviewer can be taught the foundation behind the study, common approaches to the problem, relevant results, and significant conclusions or new questions.

Write your summary when your paper is completed because how can you write the summary of anything which is not yet written? Wealth of terminology is very essential in abstract. Use comprehensive sentences, and do not sacrifice readability for brevity; you can maintain it succinctly by phrasing sentences so that they provide more than a lone rationale. The author can at this moment go straight to shortening the outcome. Sum up the study with the subsequent elements in any summary. Try to limit the initial two items to no more than one line each.

Reason for writing the article—theory, overall issue, purpose.
• Fundamental goal.
• To-the-point depiction of the research.
• Consequences, including definite statistics—if the consequences are quantitative in nature, account for this; results of any numerical analysis should be reported. Significant conclusions or questions that emerge from the research.

Approach:
• Single section and succinct.
• An outline of the job done is always written in past tense.
• Concentrate on shortening results—limit background information to a verdict or two.
• Exact spelling, clarity of sentences and phrases, and appropriate reporting of quantities (proper units, important statistics) are just as significant in an abstract as they are anywhere else.

Introduction:
The introduction should "introduce" the manuscript. The reviewer should be presented with sufficient background information to be capable of comprehending and calculating the purpose of your study without having to refer to other works. The basis for the study should be offered. Give the most important references, but avoid making a comprehensive appraisal of the topic. Describe the problem visibly. If the problem is not acknowledged in a logical, reasonable way, the reviewer will give no attention to your results. Speak in common terms about techniques used to explain the problem, if needed, but do not present any particulars about the protocols here.

The following approach can create a valuable beginning:
• Explain the value (significance) of the study.
• Defend the model—why did you employ this particular system or method? What is its compensation? Remark upon its appropriateness from an abstract point of view as well as pointing out sensible reasons for using it.
• Present a justification. State your particular theory(-ies) or aim(s), and describe the logic that led you to choose them.
• Briefly explain the study's tentative purpose and how it meets the declared objectives.

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Approach:

Use past tense except for when referring to recognized facts. After all, the manuscript will be submitted after the entire job is done. Sort out your thoughts; manufacture one key point for every section. If you make the four points listed above, you will need at least four paragraphs. Present surrounding information only when it is necessary to support a situation. The reviewer does not desire to read everything you know about a topic. Shape the theory specifically—do not take a broad view.

As always, give awareness to spelling, simplicity, and correctness of sentences and phrases.

Procedures (methods and materials):

This part is supposed to be the easiest to carve if you have good skills. A soundly written procedures segment allows a capable scientist to replicate your results. Present precise information about your supplies. The suppliers and clarity of reagents can be helpful bits of information. Present methods in sequential order, but linked methodologies can be grouped as a segment. Be concise when relating the protocols. Attempt to give the least amount of information that would permit another capable scientist to replicate your outcome, but be cautious that vital information is integrated. The use of subheadings is suggested and ought to be synchronized with the results section.

When a technique is used that has been well-described in another section, mention the specific item describing the way, but draw the basic principle while stating the situation. The purpose is to show all particular resources and broad procedures so that another person may use some or all of the methods in one more study or referee the scientific value of your work. It is not to be a step-by-step report of the whole thing you did, nor is a methods section a set of orders.

Materials:

*Materials may be reported in part of a section or else they may be recognized along with your measures.*

Methods:

- Report the method and not the particulars of each process that engaged the same methodology.
- Describe the method entirely.
- To be succinct, present methods under headings dedicated to specific dealings or groups of measures.
- Simplify—detail how procedures were completed, not how they were performed on a particular day.
- If well-known procedures were used, account for the procedure by name, possibly with a reference, and that’s all.

Approach:

It is embarrassing to use vigorous voice when documenting methods without using first person, which would focus the reviewer’s interest on the researcher rather than the job. As a result, when writing up the methods, most authors use third person passive voice.

Use standard style in this and every other part of the paper—avoid familiar lists, and use full sentences.

What to keep away from:

- Resources and methods are not a set of information.
- Skip all descriptive information and surroundings—save it for the argument.
- Leave out information that is immaterial to a third party.

Results:

The principle of a results segment is to present and demonstrate your conclusion. Create this part as entirely objective details of the outcome, and save all understanding for the discussion.

The page length of this segment is set by the sum and types of data to be reported. Use statistics and tables, if suitable, to present consequences most efficiently.

You must clearly differentiate material which would usually be incorporated in a study editorial from any unprocessed data or additional appendix matter that would not be available. In fact, such matters should not be submitted at all except if requested by the instructor.
Content:
- Sum up your conclusions in text and demonstrate them, if suitable, with figures and tables.
- In the manuscript, explain each of your consequences, and point the reader to remarks that are most appropriate.
- Present a background, such as by describing the question that was addressed by creation of an exacting study.
- Explain results of control experiments and give remarks that are not accessible in a prescribed figure or table, if appropriate.
- Examine your data, then prepare the analyzed (transformed) data in the form of a figure (graph), table, or manuscript.

What to stay away from:
- Do not discuss or infer your outcome, report surrounding information, or try to explain anything.
- Do not include raw data or intermediate calculations in a research manuscript.
- Do not present similar data more than once.
- A manuscript should complement any figures or tables, not duplicate information.
- Never confuse figures with tables—there is a difference.

Approach:
As always, use past tense when you submit your results, and put the whole thing in a reasonable order.
Put figures and tables, appropriately numbered, in order at the end of the report.
If you desire, you may place your figures and tables properly within the text of your results section.

Figures and tables:
If you put figures and tables at the end of some details, make certain that they are visibly distinguished from any attached appendix materials, such as raw facts. Whatever the position, each table must be titled, numbered one after the other, and include a heading. All figures and tables must be divided from the text.

Discussion:
The discussion is expected to be the trickiest segment to write. A lot of papers submitted to the journal are discarded based on problems with the discussion. There is no rule for how long an argument should be.
Position your understanding of the outcome visibly to lead the reviewer through your conclusions, and then finish the paper with a summing up of the implications of the study. The purpose here is to offer an understanding of your results and support all of your conclusions, using facts from your research and generally accepted information, if suitable. The implication of results should be fully described.
Infer your data in the conversation in suitable depth. This means that when you clarify an observable fact, you must explain mechanisms that may account for the observation. If your results vary from your prospect, make clear why that may have happened. If your results agree, then explain the theory that the proof supported. It is never suitable to just state that the data approved the prospect, and let it drop at that. Make a decision as to whether each premise is supported or discarded or if you cannot make a conclusion with assurance. Do not just dismiss a study or part of a study as "uncertain."
Research papers are not acknowledged if the work is imperfect. Draw what conclusions you can based upon the results that you have, and take care of the study as a finished work.
- You may propose future guidelines, such as how an experiment might be personalized to accomplish a new idea.
- Give details of all of your remarks as much as possible, focusing on mechanisms.
- Make a decision as to whether the tentative design sufficiently addressed the theory and whether or not it was correctly restricted. Try to present substitute explanations if they are sensible alternatives.
- One piece of research will not counter an overall question, so maintain the large picture in mind. Where do you go next? The best studies unlock new avenues of study. What questions remain?
- Recommendations for detailed papers will offer supplementary suggestions.

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Approach:

When you refer to information, differentiate data generated by your own studies from other available information. Present work done by specific persons (including you) in past tense.

Describe generally acknowledged facts and main beliefs in present tense.

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**Segment draft and final research paper:** You have to strictly follow the template of a research paper, failing which your paper may get rejected. You are expected to write each part of the paper wholly on your own. The peer reviewers need to identify your own perspective of the concepts in your own terms. Please do not extract straight from any other source, and do not rephrase someone else's analysis. Do not allow anyone else to proofread your manuscript.

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**BY GLOBAL JOURNALS**

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