Analysis of the Cause(s) of the Collapse of a 3-Storey Building in Ile-Ife, Osun State, Nigeria

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Abstract— This study sets out to investigate the remote cause(s) leading to the collapse of a 3-Storey Building in Ile-Ife. The study analyzed the original design submitted for approval and construction, the report of the Lagos State Materials Testing Laboratory (LSMTL) and the research re-design of the Structural elements required for the stability of the building. The results show that the building was undesigned in critical areas of the building elements such as; columns, beams, and slabs. Also, the contractor did a shoddy work either out of greed, incompetence or lack of experience. It was also noted that inappropriate materials were used by the contractor in the construction of the building. Reinforcement provided at the first, second and third floors of the building were adequate but the design did not take into consideration hogging and cantilevers. This may result in over-turning leading to collapse of the building. The results show that the reinforcement bars provided for the beams were inadequate; it was also observed that the characteristic compressive strength; $f_{cu}(20N/mm^2)$ recommended for reinforced concrete was not met by the constructors. The survey therefore concludes that; under-design, use of low grade concrete, poor concrete mix, poor workmanship, lack of proper supervision among other things contributed to the demise of the 3-storey building.

Index Terms— Lagos State materials Testing Laboratory (LSMTL), Structural Details, Structural Calculations, Investigation Report, Building, Structure, Pad Foundation, Average Soil Bearing Pressure (ASBP).

1 INTRODUCTION

Failure of structures generally have different severity such as; cracks, uneven-settlement, etc. the ultimate is collapse.

According to [14] deterioration or decay in a structure especially in vigor or usefulness can be categorized as a failure but total loss of bearing capacity, resulting in sudden breakdown, physical depletion and/or falling apart is termed a collapse.

Among contributing factors are; structural under-design, greed, incompetence (designers and constructors), corruption, poor planning, poor workmanship, lack or improper supervision, poor or non-enforcement of prevailing codes, inadequate public awareness and education and limited financial and technical expertise among other factors.

These incidents have resulted in the conclusion, how effective are building constructors’ in Nigeria? It has also cast aspersion on the competence of the Nation’s built-environment professionals; especially, architects, builders and engineers. The blame should not be laid solely at the door steps of the professionals but must be equally shared by building owners and constructors from derailing from approved plans, structural designs etc. and relying on un-scientific imagination and fantasy.

Secondly most building owners in Nigeria, shun professional advice in order to cut cost forgetting the saying “Penny Wise Pound Foolish”. Thirdly, the un-regulated economy which gives rise to the high cost of building materials, leads greedy contractors to focus on profit alone and cut corners to the detriment of proper construction methodology.

This also leads to the use of sub-standard and inappropriate construction materials. This practice has contributed majorly to the failure of buildings in Nigeria. Seeley [28] suggests that all potential building sites need investigation to determine their suitability or otherwise for situating a building; nature and extent of preliminary works that may be required. He reiterates that particular attention should be paid to the type of soil and its probable load-bearing capacity, as there may be variations over the site due to non-homogeneity of soil.

Lambe and Whitman [19] define foundation as the part of the structure in direct contact with the ground and transmit the building loads to the ground and play an important role in the construction of a building. Foundation carries all the live, dead and wind loads of a structure and transmits such loads directly to the soil/ground on which the building/structure rest in such a way that there is even distribution of the loads to prevent failure.

On the other hand, [30] reiterates that poor concrete materials mixture do not make good concrete. The result of poor concrete mix is building collapse. Steel reinforcement is used in concrete to make to make up for tensile strength (lacking in concrete) for this purpose, steel reinforcement bars must be bent to regulated standards according to relevant design codes of practice. Otherwise, there will be a structural failure and subsequent collapse.

Oyewande[27] identifies deficient structural drawings as accounting for 50% of collapse of engineering structures in Nigeria. There are many other problems that have been ascribed to the causes of building collapse by many authors such as; absence of proper supervision, alteration of approved drawings, building without an approved building plans, approval of technically deficient building plans by approving authorities, illegal alteration of existing building without recourse to the as-built-drawings, greed, absence of engineering and town planning of the building during construction, lack of monitoring/proper supervision of the building during construction, clients’ penchant for cutting corners and numerous other factors have been identified as causing building collapse [21].

Hall[16], Aniekwuru and Orie[9] all posited that sub-standard materials, especially reinforcement rods/bars, steel sections and impure cement contribute immensely to build-
2 MATERIALS AND METHODS

Firstly, soil test was carried out at the site of the collapsed building using the penetrometer test method to determine the load-bearing capacity of the soil on which the foundation of the building rests before collapse.

Secondly the submitted structural and design details of the building given to the contractor by the design engineer was reviewed. Thirdly, findings of the investigation report (after the collapse) submitted by the Lagos State materials testing laboratory was reviewed. Finally, in order to ensure that there is no bias in the comparison of the actual structural design and the report of the investigation by the LSMTL, the researchers, re-designed the building structure based on the architectural design submitted to the planning authority for approval. This will enable the researchers to discover if there are areas of discrepancies that may have contributed to the collapse of the 3-Storey building.

3 RESULTS

Soil test result from the building site taken at five different locations and obtained at 1.5m depth, reveals that the soil is fine grained, firm, moist, brown, medium-plasticity, clayey and smooth-textured. Based on this result, a pad foundation was chosen for the building.

<table>
<thead>
<tr>
<th>S/No.</th>
<th>Sample Location</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plastic Index</th>
<th>Bearing Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Loc. 1</td>
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<td>30</td>
<td>10</td>
<td>100</td>
</tr>
<tr>
<td>2.</td>
<td>Loc. 2</td>
<td>38</td>
<td>26</td>
<td>12</td>
<td>95</td>
</tr>
<tr>
<td>3.</td>
<td>Loc. 3</td>
<td>40</td>
<td>30</td>
<td>10</td>
<td>100</td>
</tr>
<tr>
<td>4.</td>
<td>Loc. 4</td>
<td>35</td>
<td>25</td>
<td>10</td>
<td>90</td>
</tr>
<tr>
<td>5.</td>
<td>Loc. 5</td>
<td>50</td>
<td>40</td>
<td>15</td>
<td>100</td>
</tr>
</tbody>
</table>

Therefore, average soil bearing pressure (ASBP) = 100+95+100+85+100/5 = 96.0 KN/M²

From this result, pad foundation was chosen as the best fit foundation to carry the dead, live and wind loads imposed on the building.

<table>
<thead>
<tr>
<th>S/No.</th>
<th>Building Element</th>
<th>Grnd. Floor</th>
<th>1st Floor</th>
<th>2nd Floor</th>
<th>3rd Floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Column</td>
<td>Rectangular</td>
<td>Rectangular</td>
<td>Rectangular</td>
<td>Rectangular</td>
</tr>
<tr>
<td>2.</td>
<td>Column Size(mm)</td>
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<td>225x225</td>
<td>225x225</td>
<td>225x225</td>
</tr>
<tr>
<td>3.</td>
<td>Req. Col. Bar Size(mm)</td>
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<td>Y16</td>
<td>Y16</td>
<td>Y16</td>
</tr>
<tr>
<td>5.</td>
<td>Link Spacing(mm)</td>
<td>240</td>
<td>240</td>
<td>240</td>
<td>240</td>
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</tbody>
</table>
### Table 5: Comparing the Three Structural Calculations

<table>
<thead>
<tr>
<th>Floors</th>
<th>Structural Design at Inception</th>
<th>Structural Findings after Collapse</th>
<th>Structural Re-Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ground</td>
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<td>Second</td>
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<tr>
<td>Column Type</td>
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<td></td>
<td></td>
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<tr>
<td>Column Size</td>
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<td>225×225</td>
<td>225×225</td>
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<tr>
<td>Column Bar size</td>
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<td>Y16</td>
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<td>Column Bar Number</td>
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</tr>
<tr>
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<td>T-beam</td>
<td>T-beam</td>
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<td>Beam Size</td>
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<td>225×450</td>
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<td>200</td>
<td>200</td>
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<tr>
<td>Slab Link Spacing</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
</tbody>
</table>

### 4 Discussion

The tables above show some discrepancies in the original design, findings of the LSMTL and the researchers’ results (see appendix for detailed structural re-design of the building).

The survey deduced from the results that; low grade quality concrete employed by the contractor, poor concrete mix, poor workmanship, and poor supervision among many other factors contributed to the demise of the 3-Storey building. The study believes that there was an attempt on the part of the contractor to cut-corners whether with the knowledge of the owner or not; he reduced the sizes and numbers of reinforcement bars in some critical elements of the building.

Opara[34] identifies poor workmanship, use of cheap and inferior materials, wrong interpretation of building design plan, inadequate supervision, non-adherence to due process in building construction, lack of maintenance culture, greedy attitude of contractors, professional incompetence, activities of quacks, and use of plan approved for one storey building to build multi-storey building as major causes of building failure in Nigeria.

Oloyede et al[33] on the other hand attribute factors responsible for building collapse as; use of low quality building materials, employment of incompetent artisans and weak supervision of workmen on site; included are non-compliance with specifications/standards, use of substandard building materials and equipment. The study also points to non-enforcement of existing laws and endemic poor work ethics of the average Nigerian worker.

Alamu and Gana[32] attributes the rising incidents of building collapse to the use of sub-standard building materials and incompetent professionals in construction activities, inadequate supervision, faulty building foundation, refusal of the wider society to recognize professionalism and pay for the services and the attitude of contractors and other stakeholders as the major factors contributing to building collapse in Nigeria.

Results from other works corroborate the findings of the present study in identifying the underlying causes of collapse of the 3-storey building in Ile-Ife, Osun State, Nigeria. This study went further to use scientific analysis in identifying structural deficiency as a major contributor to the collapse and the scientific approach and measurement applied by the research also, contributed to knowledge of underlying cause of the building collapse. The specific research methodology of using structural engineering approach to identify the structural defect in the building design before collapse, helped in avoiding generalization of the contributing factors of the 3-storey building.

### 5 Conclusion

In conclusion, the study finds that shoddy work, lack of proper supervision, under-design of structural members and contractor’s greed led to the collapse of the building in Ile-Ife, Osun State Nigeria.
6 Appendix

6.1 Design Criteria

Design was in accordance with BS 8110, ultimate loads, shear force and bending moment acting on individual member was calculated, then bending ULS was checked for and this is done to determine an adequate depth for the beam and area of both compression and tension reinforcement required.

The design shear stress \( v \) at any cross-section was also calculated from:

\[
v = \frac{V}{b_d}
\]

In no case should shear stress \( v \) exceed \( 0.8 \sqrt{f_{cu}} \) or 5 N/mm\(^2\), whichever is the lesser, whatever shear reinforcement is provided.

Spacing of links

The spacing of links in the direction of the span was designed not to exceed 0.75d. At right-angles to the span, the horizontal spacing was such that no longitudinal tension bar is more than 150 mm; this spacing should in any case not exceed d. Minimum links provide a design shear resistance of 0.4N/mm\(^2\).

Solid slabs

In general the recommendations given for beams were applied and also to solid slabs.

Simply-supported slabs

When simply-supported slabs do not have adequate provision to resist torsion at the corners, and to prevent the corners from lifting, the maximum moments per unit width are given by the following equations:

\[
ms_x = \propto sxnlx^2 \\
ms_y = \propto synlx^2
\]

Shear stresses

The design shear stress ‘\( v \)’ at any cross-section should be calculated from equation 21:

\[
v = \frac{V}{b_d}
\]

In no case should ‘\( v \)’ exceed \( 0.8 \sqrt{f_{cu}} \) or 5 N/mm\(^2\), whichever is the lesser, and whatever shear reinforcement is provided.

Deflection

Deflections was calculated and compared with the serviceability requirements given but in all normal cases, it will be sufficient to restrict the span/effective depth ratio. The ratio for a two-way (2-way) spanning slab should be based on the shorter span.

Columns: Analysis of sections

In the analysis of a column cross-section to determine its design ultimate resistance to moment and axial force, the same assumptions was made as when analyzing a beam.

Short and slender columns

A column may be considered as short when both the ratios \( l_{cr}/h \) and \( l_{cr}/b \) are less than 15 (braced) and 10 (un-braced). It should otherwise be considered as slender.

Nominal eccentricity of short columns resisting moments and axial forces

Short columns usually need only to be designed for the maximum design moment about the one critical axis. Due to the nature of a structure, a column cannot be subjected to significant moment it may be designed so that the design ultimate axial load does not exceed the value of N given by:

\[
N = 0.4f_{cu}A_c + 0.8A_{se}f_y
\]

Short braced columns supporting an approximately symmetrical arrangement of beams

The design ultimate axial load for a short column of this type may be calculated using the equation below:

\[
N = 0.35f_{cu}A_c + 0.7A_{se}f_y
\]

Foundation

Various soil tests such as Penetrometer, Atterberg limits, Compressive Strength tests were carried out to obtain the Soil bearing capacity which was used to re-design the foundation footing.

Design moment on a vertical section taken completely across a pad footing

The design moment on a vertical section taken completely across a pad footing was taken as that due to all external design ultimate loads and reactions on one side of that section. No redistribution of moments was made.

Design shear

The design shear is the algebraic sum of all design ultimate vertical loads acting on one side of or outside the periphery of the critical section. The shear stress at the column face should not be less than the lower of 5N/mm\(^2\) or 0.8 \( \sqrt{f_{cu}} \). Then, punching shear stress was checked for which is 1.5d away from the face of the column. Reinforcement to resist bending at the column face which is the critical section is then calculated using the equation below:

\[
As = \frac{M}{0.87f_{yz}}
\]

The overall stability at the ultimate limit state was checked.

<table>
<thead>
<tr>
<th>Structural Element</th>
<th>Average Compressive Strength (N/mm(^2))</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>COLUMN 1</td>
<td>11.9</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 2</td>
<td>12.1</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 3</td>
<td>10.7</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 4</td>
<td>10.3</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 5</td>
<td>8.8</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 6</td>
<td>14.0</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 7</td>
<td>10.1</td>
<td>POOR</td>
</tr>
</tbody>
</table>

TABLE 6: REPORT OF THE LSMTL

GROUND FLOOR
### TABLE 7
**FIRST FLOOR**

<table>
<thead>
<tr>
<th>Structural Element</th>
<th>Average Compressive Strength (N/mm²)</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
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### TABLE 8
**SECOND FLOOR**

<table>
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<th>Remark</th>
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</thead>
<tbody>
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<td>COLUMNS 1-6</td>
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### TABLE 9
**THIRD FLOOR**

<table>
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**TABLE 7**

<table>
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<th>Average Compressive Strength (N/mm²)</th>
<th>Remark</th>
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<td>POOR</td>
</tr>
<tr>
<td>COLUMN 2</td>
<td>14.0</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 3</td>
<td>17.0</td>
<td>GOOD</td>
</tr>
<tr>
<td>COLUMN 4</td>
<td>18.0</td>
<td>GOOD</td>
</tr>
<tr>
<td>COLUMN 5</td>
<td>15.0</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 6</td>
<td>12.2</td>
<td>POOR</td>
</tr>
<tr>
<td>COLUMN 7</td>
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<td>POOR</td>
</tr>
<tr>
<td>COLUMN 8</td>
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<td>POOR</td>
</tr>
<tr>
<td>COLUMN 9</td>
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<tr>
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<td>COLUMN 11</td>
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<tr>
<td>SLAB 1</td>
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**TABLE 8**

<table>
<thead>
<tr>
<th>Structural Element</th>
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<tr>
<td>SLAB 4</td>
<td>16.3</td>
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</tr>
</tbody>
</table>
SLAB DESIGN
Materials used = Grade 20 concrete and high yield 410mm bars
Depth of slab: 150mm
Study adopts the effective depth (d) = 119mm

 Loads
Self-weight of slab = (24 x 0.15) = 3.6 kN/m²
Finishes = 1.5 kN/m²
Partition = 1.0 kN/m²
Total working load = w = 6.1 kN/m²
Imposed load = 2 kN/m²
Therefore maximum design ultimate load = (1.4Gk+1.6Qk)
= (1.4 x 6.1 +1.6 x 2) x 1m
= 11.74 kN/m

Minimum design ultimate load = (1.4 x 6.1) x1m= 8.54 kN/m

Steel reinforcement
Moment = 15.86KNm
K = M/bd²fcu = 15.86 x 10⁶ / 1000 x 119² x 20 = 0.06
Z = d [0.5 + √(0.25 – k/0.9)] = 119[0.5 + √(0.25 – 0.06/0.9)]
Z = 110.45mm

As required = \[
\frac{M}{0.87f_{yz}} = \frac{15.86 \times 10^6}{0.87 \times 410 \times 110.45} = 402.56 \text{mm}^2 > A_{s_{\text{min}}}
\]
Minimum requirement = \[
\frac{0.1366}{100} = 0.13 \times 1000 \times 150
\]
495mm² OK

Provide 12mm bar @200 center to center (As = 566mm²)

Deflection check
Actual span / effective depth =26
4045 / 119 = 33.99

\[
\frac{F_s}{8} = \frac{5}{8} f_y = \frac{5}{8} \times 410 \times \frac{179.80}{566} = 182.25 \text{N/mm}^2
\]

Modification factor = 0.55 + \(\frac{477}{120(0.9 + \frac{m}{bd²})}\)
\(\frac{M}{bd²} = 15.86 \times 10^6 / 1000 \times 119^2 = 1.12\)
\(M_t = 1.77\)
From table 3.14 of BS 8110, basic span to effective depth ratio is 26
Permissible span / effective depth = basic ratio x \(M_t = 26 \times 1.77 = 46.02 > 33.99\) OK

End support
Moment = 4.75KNm
K = \(M/bd²fcu = 4.75 \times 10^6 / 1000 \times 119^2 \times 20 = 0.02\)

\[
d\left[0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right]
Z =
\]
\[
z = \left[0.5 + \sqrt{0.25 - \frac{0.02}{0.9}}\right]
Z = 0.977d, \text{ use } 0.95d = 113.05
\]
As req. = \[
\frac{M}{0.87f_{yz}} = \frac{4.75 \times 10^6}{0.87 \times 410 \times 113.05} = 179.80 \text{mm}^2
\]
Provide 12mm @ 300 center to center (As = 377mm²)

Deflection check
Actual span / effective depth = 900/119 = 7.56
\[
F_s = \frac{5}{8} f_y = \frac{5}{8} \times 410 \times \frac{179.80}{377} = 122.21 N/mm^2
\]
\(M_f = 0.55 + 477 - f_s / 120(0.9 + m/bd²)
\(M / bd² = 4.75 \times 10^6 / 1000 \times 119^2 = 0.34\)
\[ M_f = 0.55 + \frac{477 - 122.21}{120(0.9 + 0.34)} \]

\[ M_f = 3.23 \]

Permissible span / effective depth = \(7 \times 3.23 = 22.61 > 7.56\) OK

Critical load arrangement for centre columns
Column size=230x300mm

Maximum Ultimate Load at each floor = \(3.225(1.4Gk+1.6Qk)\)
\[ = 3.225(1.4 \times 26.9135 + 1.6 \times 6.6) \]
\[ = 155.57\text{kN/m} \]

Minimum Ultimate load at each floor = \(3.225(1.4Gk)\)
\[ = 3.225 \times 14 \times 26.9135 \]
\[ = 121.51\text{kN/m} \]

Column Loads
2nd and 3rd floors = \(2 \times 155.57 \times 3.225 = 1003.4256\text{kN} \)
First Floor = \(155.57 \times 3.225 \times \frac{2}{2} + 121.51 \times 3.225 \times \frac{2}{2} = 446.7915\text{kN} \)

Column Self weight = \(2 \times 14 = 28\text{kN} \)
\[ 1478.218\text{kN} \]

Column Moments
Member stiffness’s are:
\[ KAB/2 = \frac{bh^2}{12LAB} \]
\[ = 0.23 \times 0.3^2 \]
\[ = 0.3225 \times 1.2 \times 3.225 = 0.8 \times 10^{-4} \]
\[ KBC/2 = KAB/2 \]
\[ = 0.5 \times 0.23 \times 0.3^2 \]
\[ = 1.568 \times 10^{-4} \]

Total stiffness = \((1.568 + 1.568 + 0.8 + 0.8) \times 10^{-4} \)
\[ = 4.736 \times 10^{-4} \]

Distribution factor at joint B
\[ D_{BE} = \]
\[ \frac{(1.568 \times 10^4) \times (-4)}{(4.736 \times 10^4) \times (-4)} = 0.33 \]
\[ D_{BE} = D_{BH} \]
\[ D_{AB} = D_{BC} = 0.8 / 4.736 = 0.169 \]

<table>
<thead>
<tr>
<th>Joint</th>
<th>E</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>H</th>
</tr>
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<tbody>
<tr>
<td>D.F</td>
<td>EB</td>
<td>AB</td>
<td>BA</td>
<td>BE</td>
<td>BH</td>
</tr>
<tr>
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<td>0</td>
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<td>0.33</td>
<td>0.33</td>
<td>0.17</td>
</tr>
<tr>
<td>(f_{EM})</td>
<td>-</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>BAL</td>
<td>-</td>
<td>-</td>
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<td>9.74</td>
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<tr>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>4.87</td>
<td>137.35</td>
<td>129.82</td>
<td>9.74</td>
<td>9.74</td>
</tr>
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</table>

Column Loads
2nd and 3rd floors = \(2 \times 155.57 \times 3.225 = 1003.4256\text{kN} \)
First Floor = \(155.57 \times 3.225 \times \frac{2}{2} + 121.51 \times 3.225 \times \frac{2}{2} = 446.7915\text{kN} \)

Column Self weight = \(2 \times 14 = 28\text{kN} \)
\[ 1478.218\text{kN} \]

Column Moments
Member stiffness’s are:
\[ KAB/2 = \frac{bh^2}{12LAB} \]
\[ = 0.23 \times 0.3^2 \]
\[ = 0.3225 \times 1.2 \times 3.225 = 0.8 \times 10^{-4} \]
\[ KBC/2 = KAB/2 \]
\[ = 0.5 \times 0.23 \times 0.3^2 \]
\[ = 1.568 \times 10^{-4} \]

Total stiffness = \((1.568 + 1.568 + 0.8 + 0.8) \times 10^{-4} \)
\[ = 4.736 \times 10^{-4} \]

Distribution factor at joint B
\[ D_{BE} = \]
\[ \frac{(1.568 \times 10^4) \times (-4)}{(4.736 \times 10^4) \times (-4)} = 0.33 \]
\[ D_{BE} = D_{BH} \]
\[ D_{AB} = D_{BC} = 0.8 / 4.736 = 0.169 \]

<table>
<thead>
<tr>
<th>Joint</th>
<th>E</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.F</td>
<td>EB</td>
<td>AB</td>
<td>BA</td>
<td>BE</td>
<td>BH</td>
</tr>
<tr>
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<td>0</td>
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<td>0.33</td>
<td>0.33</td>
<td>0.17</td>
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<tr>
<td>(f_{EM})</td>
<td>-</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>BAL</td>
<td>-</td>
<td>-</td>
<td>5.02</td>
<td>9.74</td>
<td>9.74</td>
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<tr>
<td>CO</td>
<td>4.87</td>
<td>2.51</td>
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<td>-</td>
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</tr>
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<td>-</td>
<td>4.87</td>
<td>137.35</td>
<td>129.82</td>
<td>9.74</td>
<td>9.74</td>
</tr>
</tbody>
</table>
Design moment in the lower column at section X-X;
Design Axial load, N=1478.218kN
Moment = 9.74kNm
Column is braced.
Assume the effective height of column section at BH=3.3m

X-X direction \[ \frac{3300}{230} = 14.35 \]
Y-Y direction \[ \frac{3300}{300} = 11 \]

Column is short.
Assume a bar diameter=20mm
Cover=25mm, link=8mm
Effective depth, \( d = (300-25-8-10) = 257mm \)

Using design chart, Fig 3.94 from BS8110
\[
\frac{N}{bh} = (1478.218 \times 10^3/230 \times 300)^{1/3} = 2.42
\]
\[
\frac{M}{bh^2} = (9.74 \times 10^3/230 \times 300)^{2/3} = 0.47
\]
From design chart;
\[ \text{Asc} = 2.2 \times 230 \times 300 / 100 = 1518 \text{mm}^2 \]

Provide 6Y20 (Asc=1890mm²)
Minimum link diameter ≥0.25x largest compression bar
=0.25x20=5
8mm≥5mm
Spacing of links ≤ (12x smallest compression bar)
=12x20=240mm
Provide 10mm diameter links at 240mm Spacing.
### Foundation Footing Design

#### Footing (f1) Loading

**Dead load** = 86.80 kN/m  
**Live load** = 21.29 kN/m  
**Serviceability load**

- **Maximum load** = 1.0Gk + 1.0Qk  
  = 1.0 x 86.8 + 1.0 x 21.29  
  = 108.08 kN/m  

- **Minimum load** = 1.0Qk  
  = 1.0 x 86.8  
  = 86.80 kN/m  

#### Bearing pressure

- **Area of footing** = \( \frac{1000}{100} = 10.1136 \text{m}^2 \)  
- Assume the overall depth of footing = 600mm

---

**Total column load**

2nd and 3rd floors = 2 x 108.08 x 3.225 = 697.12 kN  
1st floor = 108.08 x 2 x 3.225 = 314.244 kN  

Total load = 1011.36 kN

**Bearing pressure** = 100 kN/m²

**Area of footing** = 1011.36 \( \div \) 100 = 10.1136 m²

Provide a 3.2m square base (plan area = 10.24 m²)

---

#### Design Calculations

<table>
<thead>
<tr>
<th>Reference</th>
<th>Beam Calculations</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/24 Beam</td>
<td>18.93 kN/m</td>
<td>46.66 kN/m</td>
</tr>
<tr>
<td>Wall Load</td>
<td>7.99 kN/m</td>
<td>Bf(1st) = 796 mm</td>
</tr>
<tr>
<td></td>
<td>8.00 kN/m</td>
<td>Bf(CELL) = 513 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Span Rein</th>
<th>20.55 kNm</th>
<th>Designed as a T-Section</th>
<th>Bf = 513 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>k = 0.15</td>
<td>fcu = 20 N/sq mm</td>
<td>fy = 410 N/sq mm</td>
<td></td>
</tr>
</tbody>
</table>

**Deflection Check**

- As Prev. = 402 sq mm  
  - Mbd = 2 = 0.62  
  - FS = 2 x 217.8 x 410  
  - Max. M.F = 2.00  
  - Used M.F = 2.00  

**M.F = 427.14**

**Depth Reqd = 87.9 < 255 mm Satisfactory!**

**Shear Reinforcement**

- \( v_x = 3.60 \text{N/mm} \)
- \( V_{bd} = 0.62 \text{N/mm} \)

- Provide \( V = \frac{15}{2} = 7.5 \text{ mm} \)

---

**Total column load**

- **2nd and 3rd floors** = 2 x 108.08 x 3.225 = 697.12 kN
- **1st floor** = 108.08 x 2 x 3.225 = 314.244 kN

**Total load** = 1011.36 kN

**Bearing pressure** = 100 kN/m²

**Area of footing** = 1011.36 \( \div \) 100 = 10.1136 m²

Provide a 3.2m square base (plan area = 10.24 m²)

Assume the overall depth of footing = 600mm
Bending Reinforcement

Total ultimate load (W) = 1478.218kN

Earth pressure (ps) = \[
\frac{1478.218}{10.24} = 144.36 \text{kN/m}^2
\]

Maximum design moment occur at the face of the column = 144.36 x 3.2 x 1.62 / 2 = 591.30kNm
Assume cover = 50mm, Bar diameter = 25mm
Effective depth = 600-50-25 = 525mm

Ultimate moment, Mu = 0.156 x 25 x 1000 x 525^2 = 860kNm

Since Mu > M, no compression reinforcement is required.

Design moment = 591.30KNm

K = \[
\frac{M}{bd^2 f_{cu}} = \frac{591.30 \times 10^6}{1000 \times 525^2} = 0.11
\]

Z = \[
\left[ 0.5 + \sqrt{0.25 - \frac{k}{0.9}} \right] = 0.86d
\]

Z = 0.86 x 525 = 451.5mm

As req = \[
\frac{M}{0.87 f_{yz}} = \frac{591.30 \times 10^6}{410} x 451.5 = 3671.5 \text{mm}^2
\]

Provide 25mm @ 125mm center to center (As = 3930mm^2)
As minimum = 0.13%bh = 780 mm^2 < As OK

Fig. 6. Punching Shear

The critical section for checking punching is at a distance 1.5d

Critical perimeter = column perimeter + 8 x 1.5d = 4 x 300 + 8 x 1.5 x 525 = 7500mm

Area within perimeter is;

(300 + 3d)^2 = (300 + 3 x 525)^2 = 3.52 x 10^6 mm^2

Ultimate punching force, V, is

V = load on shaded area = 144.36 x (10.24 - 3.52) = 970.10 kN

Design punching shear stress, \(\nu\), is

\[
\nu = \frac{V}{Perit \times d} = \frac{970.10 \times 10^3}{7500 \times 525} = 0.25 N/mm^2
\]

Design concrete shear stress, \(v_c\), is;

\[
v_c = \left( \frac{f_{cu}}{25} \right)^{1/3} = \left( \frac{20}{25} \right)^{1/3} \times 0.57 = 0.53 N/mm^2
\]

Since \(v_c > \nu\), punching failure is unlikely and a 600 mm depth is acceptable.

Fig. 7. Column Base Design

Footing (f2) Loading
Dead load = 59.88kN/m
Live load = 14.69kN/m
Serviceability load

Maximum load = 1.0Gk + 1.0Qk = 1.0 x 59.88 + 1.0 x 14.69 = 74.57kN/m

Minimum load = 1.0Qk = 1.0 x 59.88 = 59.88 kN/m

Total column load
2nd and 3rd floors = 2 x 74.57 x 2.225 = 331.84kN
1st floor = 74.57 x \[
\frac{2.225}{2} + \frac{59.88 \times 2.225}{2} = 149.58 kN
\]
Total load = 481.42kN
Bearing pressure = 100kN/m²

Area of footing = \( \frac{481.42}{100} = 4.816 \text{m}² \)

Provide a 2.2m square base (plan area=4.84 m²)
Assume the overall depth of footing = 400mm

**BENDING REINFORCEMENT**
Total ultimate load \( W \) = 718.31kN

Earth pressure \( ps \) = \( \frac{84}{31 \times 718} = 148.41 \text{kN/m}² \)

Maximum design moment occur at the face of the column = 148.41 x 2.2 x 1.1²/2 = 197.53 kNm
Assume cover = 50mm, Bar diameter = 25mm
Effective depth = 400-50-25 = 325mm
Ultimate moment, \( Mu \) = 0.156 x 25 x 1000 x 325²
= 411.94 kNm

Since \( Mu > M \), no compression reinforcement is required.

Moment = 197.53 kNm
\( K = \frac{M}{bd²fcu} = \frac{197.53 x 10^6}{1000 x 325^2 x 20} = 0.094 \)

\[ Z = d\left[0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right] \]
\[ Z = d\left[0.5 + \sqrt{0.25 - \frac{0.11}{0.9}}\right] = 0.88d \]
\[ Z = 0.88 x 325 = 286.50 \text{mm} \]

As req = \( \frac{M}{0.087fyz} = 197.53 x 10^6 / 0.87 x 410 x 286.50 = 1932.88 \text{mm}² \)
Provide 25mm @ 250mm center to center \( (A_s = 1960 \text{mm}²) \)

As minimum = 0.13%bh = 520 \text{mm}² < As \hspace{1cm} \text{OK}

The critical section for checking punching is at a distance 1.5d
Critical perimeter = column perimeter + 8 x 1.5d = 4 x 300 + 8 x 1.5 x 325 = 5100mm

Area within perimeter is;
\((300 + 3d)² = (300 + 3 x 325)² = 1.63 x 10^6 \text{mm}² \)

Ultimate punching force, \( V \), is
\( V = \text{load on shaded area} = 148.41 x (4.84 - 1.63) = 476.40 \text{kN} \)
Design punching shear stress, \( v \), is
\( v = \frac{V}{Pc_{crit} x d} = 476.40 x 10^3 / 5100 x 325 = 0.29 \text{N/mm}² \)
\( \frac{100As}{bd} = \frac{100 x 1960}{1000 x 325} = 0.60 \text{N/mm}² \)

Design concrete shear stress, \( v_c \), is
\( v_c = \frac{f_{cu} / 25}{1/3} = (20/25)^{1/3} x 0.54 = 0.50 \text{N/mm}² \)
Since \( v_c > v \), punching failure is unlikely and a 600 mm depth of slab is acceptable.
TABLE 11
VALUES OF DESIGN CONCRETE SHEAR STRESS, VC (N/MM²)

<table>
<thead>
<tr>
<th>100(A_e/bd)</th>
<th>Effective depth (d) mm</th>
<th>125</th>
<th>150</th>
<th>175</th>
<th>200</th>
<th>225</th>
<th>250</th>
<th>300</th>
<th>&gt; 400</th>
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<tbody>
<tr>
<td>0.15</td>
<td>0.45</td>
<td>0.43</td>
<td>0.41</td>
<td>0.38</td>
<td>0.36</td>
<td>0.34</td>
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<td>0.30</td>
<td></td>
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<tr>
<td>0.25</td>
<td>0.53</td>
<td>0.51</td>
<td>0.49</td>
<td>0.47</td>
<td>0.45</td>
<td>0.43</td>
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<tr>
<td>0.50</td>
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<td>0.61</td>
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<td>0.75</td>
<td>0.97</td>
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<td>1.00</td>
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<td>2.20</td>
<td>2.18</td>
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<tr>
<td>≥ 3.00</td>
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<td>2.87</td>
<td>2.85</td>
<td>2.83</td>
<td>2.81</td>
<td>2.79</td>
<td>2.77</td>
<td>2.75</td>
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TABLE 12
CROSS-SECTIONAL AREAS OF GROUP OF BARS (MM²)

<table>
<thead>
<tr>
<th>Bar size (mm)</th>
<th>Number of bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
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</tr>
<tr>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>22</td>
</tr>
<tr>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>16</td>
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</tr>
<tr>
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<tr>
<td>32</td>
<td>50</td>
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<tr>
<td>40</td>
<td>60</td>
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</table>

TABLE 13
VALUES OF \(A_{sv}/S_v\)

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Spacing of ties (mm)</th>
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<tr>
<td>8</td>
<td>11.01</td>
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<tr>
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<td>18.84</td>
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<td>21.45</td>
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TABLE 14
CROSS-SECTIONAL AREA PER METRE WIDTH FOR VARIOUS BAR SPACING (MM²)

<table>
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<tr>
<th>Bar size (mm)</th>
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<tr>
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<tr>
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Fig. 11. Column Base design
Fig. 12. Column design chart
REFERENCES


