DESIGN OF SUPPORT SYSTEM FOR REPLACEMENT OF A DEFICIT COLUMN AT INTERNAL BEAM-COLUMN JUNCTION

D. Jawaharlal
Research Scholar, Department of Civil Engineering,
Dr. MGR Educational and Research Institute, Chennai – 600 095, India

Prof. Dr. T. Felix Kala
Additional Dean, Engineering & Technology,
Dr. MGR Educational and Research Institute, Chennai – 600 095, India

ABSTRACT

The structural failures are increasingly happening during the construction stage. One of the prime causes is the lack in quality control. The quality control plays a vital role in any construction project and failure in adherence to required quality parameters lead to catastrophic failure of structure. The quality control is exercised in three phases, pre-construction, during construction and post construction. In the pre-construction phase, the auditing of specifications, design and drawings are the prime parameters looked into. In the post construction phase, non-destructive testing plays a key role in quality audit. The quality control plays a major role during the construction phase and the failure in this task result in endanger of people working on the project and impose a challenging task to design the retrofitting or strengthening of the deficit structural element. This paper analysed the cause of failure of a reinforced cement concrete internal column and derives a viable solution to retrofit the system. Different possibilities to retrofit the system also have been explored.

Key works: Column-Beam Joint, Failure of Concrete, High Strength Concrete, Retrofitting, Structural Steel.

1. INTRODUCTION
The structural audit of a building under construction involves different procedures like verifying the design adequacy, quality control of the construction materials, grade of concrete and so on. The post construction audit involves non-destructive and partial testing methods such as rebound hammer test, ultrasonic pulse velocity method and core compression test method to assess the grade of concrete poured. These tests are performed to counter check the quality control measures undertaken during the construction phase. In a study, it is observed that the grade of concrete is found to be less than the designed grade and the structure has been progressed. The concrete grade used for design was M25 (the characteristic compression strength of the concrete should be 25 Mpa at 28 days curing). The compression test performed on the cylindrical has shown the values ranging from 3.40 Mpa to 20 Mpa for 20 columns which is 1/3rd of the total columns in the floor (total number of columns in the floor is 62). Under this scenario, demolishing the whole structure will be an unwise decision. So, to overcome the situation different strengthening techniques are studied like FRP jacketing, RCC jacketing, steel jacketing and so on. After working on various retrofitting and strengthening schemes, as the building configuration has not permitted the application any of the strengthening methods, a more feasible solution of demolition of the column and re-casting of the same was arrived for which a support system has been designed using the structural steel rolled sections.

2. FIBRE REINFORCED POLYMER COMPOSITE JACKETING
The CFRP (Carbon Fibre Reinforced Polymer) Jacketing scheme will not be successful because of the following reasons.

2.1. Abnormal Strength Reduction
The strength reduction in characteristic compression strength due to poor grade of concrete is in the range of 19% to 86% and even with multiple plies of wrapping, the strength could not be achieved also the performance under service condition will be risk as the strength enhancement only depends upon the glue applied to paste the fibre mat.

2.2. Failure Mechanism of CFRP
The rupture strain of the CFRP is considered as equal to the rupture strain of the concrete in compression for the CFRP strengthening to be effective. In this case, the failure mode will be the concrete will crush before the CFRP undergoes peeling off or tearing off. So, the CFRP will not suit the situation.

And, the CFRP jacketing should not be damaged at future stage which cannot be assured during the entire life span of the building for various functional and environmental reasons.

2.3. Ineffective Development Length
The reinforcement fabrication might be done in accordance with the initial design (original design done by Principal Structural Consultant) in which the development length might be worked out and issued based on the bond stress applicable to M25 grade concrete whereas the core compression test shows far lesser grade and the deficient varies from 22% to 86% (% of strength reduction).

As the grade of concrete in the deficit column varies from 22% to 86% (% of strength reduction), enhancing bond between the reinforcement provided with casted low-grade concrete at the present scenario is not practical.
3. EXTERNAL JACKETING WITH HIGH STRENGTH CONCRETE OR STEEL

As the external jacketing with concrete or steel will result in increased size of columns, external jacketing was not preferred. Also, steel jacketing is susceptible to corrosion in the region and concrete jacketing will result in projection inside the walls and hence the space could not be utilised effectively. Also the external jacketing with steel need be anchored with the existing low strength concrete, which will not exert required bond between the anchors. Hence the external jacketing with steel is also not preferred.

4. DEMOLITION AND RE-CASTING OF COLUMNS

As the general options of strengthening are found to be unviable for this particular case of failure, the demolition and re-casting of column is opted and found to be structurally viable without altering the structural configuration of the building.

5. PARTIAL DESTRUCTIVE TESTING

Core samples were extracted from the deficit column and tested and the results are tabulated as below:

![Figure 1 Extraction of Core Sample](image)

### Table 1 Core compression strength.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Core strength in M.Pa</th>
<th>Strength reduction in %</th>
<th>Sample ID</th>
<th>Core strength in M.Pa</th>
<th>Strength reduction in %</th>
<th>Sample ID</th>
<th>Core strength in M.Pa</th>
<th>Strength reduction in %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.54</td>
<td>22</td>
<td>8</td>
<td>11.97</td>
<td>52</td>
<td>15</td>
<td>3.57</td>
<td>86</td>
</tr>
<tr>
<td>2</td>
<td>16.44</td>
<td>34</td>
<td>9</td>
<td>11.81</td>
<td>53</td>
<td>16</td>
<td>15.71</td>
<td>37</td>
</tr>
<tr>
<td>3</td>
<td>16.15</td>
<td>35</td>
<td>10</td>
<td>9.40</td>
<td>62</td>
<td>17</td>
<td>14.59</td>
<td>42</td>
</tr>
<tr>
<td>4</td>
<td>12.99</td>
<td>48</td>
<td>11</td>
<td>9.18</td>
<td>63</td>
<td>18</td>
<td>14.41</td>
<td>42</td>
</tr>
<tr>
<td>5</td>
<td>12.82</td>
<td>49</td>
<td>12</td>
<td>20.16</td>
<td>19</td>
<td>19</td>
<td>13.69</td>
<td>45</td>
</tr>
<tr>
<td>6</td>
<td>12.43</td>
<td>50</td>
<td>13</td>
<td>17.27</td>
<td>31</td>
<td>20</td>
<td>13.55</td>
<td>46</td>
</tr>
<tr>
<td>7</td>
<td>12.32</td>
<td>51</td>
<td>14</td>
<td>13.55</td>
<td>46</td>
<td>21</td>
<td>10.69</td>
<td>57</td>
</tr>
</tbody>
</table>
6. DESIGN OF SUPPORT SYSTEM
The propping system is designed based on the following assumption generally followed in structural analysis and design of framed structures i.e. the floors are loaded and the loads on the floors are supported by the beams and the loads of the beams are transferred to the columns and ultimately to the foundation system.

6.1. Design Procedure

6.1.1. The structure is analysed using Staad Pro software.

6.1.2. The member end forces are obtained from all the beams connection at the particular column top junction and the maximum forces are taken for design of prop.

6.1.3. The additional moment due to the moment induced due to the axial load from the floor above and the eccentricity of the support with respect to the present column location also calculated and added to the design moments.

6.1.4. To resist the resulting forces, structural steel columns (to act as prop) are designed as per limit state method and appropriate connections are also designed.

6.2. Input Parameters
The loads considered are as below:

<table>
<thead>
<tr>
<th></th>
<th>Stilt</th>
<th>First floor</th>
<th>Second floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Self-weight of slab</td>
<td>Construction load (any construction materials lying in the floor- assumed @50Kgs per Sq.mtr)</td>
<td>Load due to form work (as per clause 7.3.1.2 of IS: 14687)</td>
</tr>
<tr>
<td></td>
<td>= 1 × 1 × 0.15 × 25 = 3.75 KN/m²</td>
<td>Live load- as per IS 875,Part-II (though the building is not occupied, this load is assumed for people movement and also, due to the construction crew movement, the slab of the first floor might have attained its strain)</td>
<td>= 0.50 KN/m²</td>
</tr>
<tr>
<td></td>
<td>= 0.50 KN/m²</td>
<td></td>
<td>= 0.50 KN/m²</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Weight of Steel reinforcement (as per clause 7.3.1.1-b of IS: 14687 and IS: 875 (Part-I), Table-1 (Srl no. 20 &amp; 22))</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>= 4.50 × 0.15 = 0.675 KN/m²</td>
<td></td>
</tr>
</tbody>
</table>

7. ANALYSIS
The static analysis is performed and the required beam end forces for the required structural members under which the supports are to be erected is obtained as below.
The maximum load on the support is found to be 234 KN-m and the support system is designed for an axial load of 150 KN and a moment of 300 KN-m. The design output from the structural analysis software Staadpro is reproduced in the Table.2 below.

Table 2 Output from structural analysis. (Software output is re-produced)

<table>
<thead>
<tr>
<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Axial (kN)</th>
<th>Shear (kN)</th>
<th>Torsion (kNm)</th>
<th>Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fx</td>
<td>Fy</td>
<td>Fz</td>
<td>Mx</td>
</tr>
<tr>
<td>Max Fx</td>
<td>696</td>
<td>4</td>
<td>3:1.50 (DL+LL)</td>
<td>1.05E+3</td>
<td>-9.119</td>
<td>-5.163</td>
</tr>
<tr>
<td>Min Fx</td>
<td>124</td>
<td>153</td>
<td>3:1.50 (DL+LL)</td>
<td>-42.419</td>
<td>145.187</td>
<td>0.993</td>
</tr>
<tr>
<td>Min Fy</td>
<td>374</td>
<td>338</td>
<td>3:1.50 (DL+LL)</td>
<td>93.412</td>
<td>-149.764</td>
<td>12.710</td>
</tr>
<tr>
<td>Max Fz</td>
<td>370</td>
<td>328</td>
<td>3:1.50 (DL+LL)</td>
<td>0.396</td>
<td>-61.864</td>
<td>38.207</td>
</tr>
<tr>
<td>Max Mx</td>
<td>361</td>
<td>339</td>
<td>3:1.50 (DL+LL)</td>
<td>29.818</td>
<td>-26.672</td>
<td>-7.199</td>
</tr>
<tr>
<td>Min Mx</td>
<td>370</td>
<td>328</td>
<td>3:1.50 (DL+LL)</td>
<td>0.396</td>
<td>-61.864</td>
<td>38.207</td>
</tr>
<tr>
<td>Max My</td>
<td>623</td>
<td>416</td>
<td>3:1.50 (DL+LL)</td>
<td>127.354</td>
<td>12.532</td>
<td>28.032</td>
</tr>
<tr>
<td>Max Mz</td>
<td>374</td>
<td>338</td>
<td>3:1.50 (DL+LL)</td>
<td>93.412</td>
<td>-149.764</td>
<td>12.710</td>
</tr>
</tbody>
</table>
7.1. Design of Structural Column

Loads considered for design:
Axial load \( P_u \): 150 KN.
Moment \( M_x \): 240 KN-m
Moment \( M_y \): 22 KN-m
Section considered: ISMC 400-2 no’s made to box
Effective axial load \( P_{eff} \):

\[
P_{eff} = P + \left( 2 \times \frac{M_x}{d} \right) + \left( 7.50 \times \frac{M_y}{b} \right)
\]

Substituting values, we get:

\[
P_{eff} = 150 + \left( 2 \times \frac{240}{0.30} \right) + \left( 7.50 \times \frac{22}{0.20} \right) = 2175 \text{ KN}
\]

\( P_{eff} \): 2175 KN.

Properties of Section:
(taken from SP:6-1, Structural Steel Hand Book)
Width of flange \( b_f \): 100 mm
Maximum radius of Gyration about XX axis \( r_{xx} \): 154.80 mm
Plastic Modulus of Section about XX axis, \( Z_{px} \):

\[
= 2 \times 891.03 \times 10^3 \text{ mm}^3
\]

Elastic Modulus of section about XX axis, \( Z_{ex} \):

\[
= 2 \times 754.10 \times 10^3 \text{ mm}^3
\]

Area of the section, \( A_g \):

\[
= 2 \times 6293 \text{ mm}^2
\]

Effective length of column, \( l_{eff} \):

\( K \) is taken as 1.0, assuming conservatively, and ‘L’ is unsupported length of column

Check for the buckling class of the section:
As per table-10 of IS: 800-2007, the built up section chosen comes under buckling class “c”

\[
\frac{KL}{r_{xx}} = \frac{2550}{154.80} = 16.47
\]

From the table 9-c, for \( KL/r_{xx} = 16.47 \), and \( f_y = 250 \) MPa, the value of compressive stress is, \( f_{cd} \)

Axial load carrying capacity of the section selected is

\[
= A_g \times f_{cd}
= 2 \times 6293 \times 224 = 2819 \text{ KN},
= 2819 \text{ KN} > 2175 \text{ KN} \text{ (required), hence the section chosen is safe.}
\]

Cross section classification:

\[
\varepsilon = \frac{250}{f_y}
\]
Design Compression
Strength of the Section:
As per clause 9.3.1.1 of IS: 800-2007,

Design Compression
Strength of the Section:
As per clause 9.3.1.1 of IS: 800-2007,

\[ N_d = \frac{A_y \times f_y}{Y_{mo}} \]

\[ N_d = \frac{2 \times 6293 \times 250}{1.10} = 2860.45 \text{ KN} > 150 \text{ KN} \]

Bending Resistance of the Cross-Section:
Bending resistance about ‘YY’ axis \( (M_{dy}) \):

\[ M_{dy} = \frac{Z_e \times Z_{py} \times f_y}{Y_{mo}} = \frac{Z_{ey} \times f_x}{Y_{mo}} \]

\[ Z_e = \beta_b \times Z_p \]

\[ = \frac{824.10 \times 10^3 \times 250}{1.10} = 187.29 \text{ KNm} \]

Which is greater than the applied moment of 22 KN-m, hence the section selected is safe.

Bending resistance about ‘XX’ axis \( (M_{dx}) \):

\[ M_{dx} = \frac{\beta_b \times Z_{px} \times f_y}{Y_{mo}} = \frac{Z_{ex} \times f_x}{Y_{mo}} \]

\[ = \frac{2 \times 754.10 \times 10^3 \times 250}{1.10} = 342.77 \text{ KNm} \]

Which is greater than applied moment of 240 KN-m, hence section selected is safe.

Check for Interaction Equation:

\[ \frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_x}{M_{dx}} \leq 1.0 \]

\[ = \frac{150}{2860.45} + \frac{22.00}{187.29} + \frac{240}{342.77} = 0.87 < 1.0 \]

So, the section selected is safe to resist the induced forces.
8. DESIGN OF CONNECTIONS

1. Type of Connection : Bolted connection using 4.6 grade.
2. Diameter of bolt considered : 20mm (M20)
3. Shear capacity in single shear : 45.30 KN in single shear.
4. Pitch : 150mm
5. No’s of rows : 2
6. No of bolts required in double shear (both sides of the beams to be connected) :
   \[ n = \left( \frac{6 \times M}{n \times p \times V_{sd}} \right)^{1/3} \]
   
   \( n \) : No of bolts
   \( M \) : Maximum moment applied at the connection
   \( = 240 \text{ KN-m for type –II columns} \)
   \( n \) : No of rows
   \( p \) : Pitch of bolts (=275mm)
   \( V_{sd} \) : Shear capacity of bolt in single shear (=45.30KN for 4.6 grade bolt)

   As the beam is connected with plates on both the sides, no of shear = 2
   So, shear strength = 2\times45.30

   Bolted connection using 8.8 grade high strength grip bolts.
   Bolted connection using 8.8 grade high strength grip bolts.

   \[ n = \left( \frac{6 \times 240}{2 \times 0.275 \times 45.30 \times 2} \right)^{1/3} \]
   
   So, no of bolts required : = 5.40 say 6 nos.

   So, provide M20 bolts 6 no’s on each face of the plate.

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**Figure 3** Structural steel columns placed in position.  
**Figure 4** RCC column demolished below beam-column junction.
9. DEMOLITION AND RE-CASTING THE COLUMNS WITH HIGH-STRENGTH CONCRETE (≥ M30):

Once the structural supports are erected, the deficit column is demolished using electrical breakers and the column is re-casted in usual conventional way. To ensure the proper bonding between the beam-column junctions, epoxy chemical Nitofil-EPLV is grouted.

![Figure 5 Re-casting of column](image1)
![Figure 6 Grouting of epoxy grout](image2)

10. CONCLUSIONS

- The lapses in quality control measures result in massive failure of structural members which involves a complex remedial procure and the stability of structure is put under risk.
- The demolition and re-casting is crucial task to carryout but it is feasible to recast the columns with an effective retrofitting design.
- The advances in material science and technology enable to cast the concrete and achieve strength of about M30 in less than 3 days against the conventional period of 28 days.
- The structural steel rolled sections are very compatible with any situation to be used as composite structure.

REFERENCES

Design of Support System For Replacement of A Deficit Column at Internal Beam-Column Junction


