Project ID: 3751
Project Type: Engineering
Project Title: USE OF COLD FORMED STEEL IN RESIDENTIAL HOUSING
Principal Investigator:
EGYPT: Dr Metwally Abu-Hamd,
        Professor of Steel Structures, Faculty of Engineering, Cairo University.
USA:    Dr Benjamin Schafer,
        Chairman, Civil Engineering Dept., Johns Hopkins University

Project Start Date: October 16, 2011
Project End Date: October 15, 2013
Project Duration: Two years

Reporting period: From: October 16, 2012
                 To: April 15, 2013

Date of submission: April 10, 2013

Signature of Principal Investigators:

Egypt P.I.                                                    U.S. P.I.

Prof Dr Metwally Abu-Hamd                                   Prof Dr Ben Schaffer
1. Objectives of the reporting period, as given in the submitted grant application:

1-1) Research Activity No.1:

Development of a novel non-proprietary cold-formed steel framing system

Cold-formed steel framing has not yet realized its potential. Currently, the framing systems employed are largely direct translations of timber framing and the designs are component-based using only commodity sections (i.e. the lipped channel or “stud” and the un-lipped channel or “track”). Over the last 10 years two new technologies have been developed and introduced into U.S. design specifications that allow for radical departure from this thinking: the Direct Strength Method (e.g. AISI-S100 Appendix 1) which allows for shape optimization in a far more robust way than previously possible, and Advanced (or second-order) Analysis Methods (AISI-S100 Appendix 2), which allows for frame optimization in a far more robust way than previously possible.

A framing system will be developed with the full implementation of two key features: (1) the use of optimized shapes, and (2) the implementation of a ‘dual’ system whereby a primary cold-formed steel structural system and a secondary cold-formed steel structural system both exist blending the advantages of sparsely spaced primary frames conventionally used in hot-rolled steel construction with closely spaced stick framing methods conventionally used in cfs construction. The key steps in the development of the novel framing system include:
(Numbers refer to Gantt chart in original proposal)

1-1 Develop a library of optimal shapes (covered in this report)
1-2 Develop ‘dual’ system for walls and floors (covered in this report)
1-3 Develop home archetype (covered in first report)
1-4 Develop full framing solution for archetypical home (covered in first and second reports)
1-5 Demonstrate flexibility of ‘dual’ framing system (scheduled to be covered in the final report)
1-6 Develop price estimates building archetypes studies (scheduled to be covered in the final report)

1-2) Research Activity No.2:

Building Archetypes Study
A building archetype study is being executed considering the following key steps:
(Numbers refer to Gantt chart in original proposal):
2.1 Selection of rural and urban locations for archetype homes (Already covered in first progress report)

2.2 Design and cost analysis of traditional concrete framing in Egypt (covered in second report)

2.3 Design and cost analysis of conventional cold formed steel framing (covered in second report)

2.4 Design and cost analysis of dual system cold formed steel framing (covered in this report and to be continued in final report).

2.5 Environmental impact and sustainability assessment (covered in second report and continued in this report)

2.6 Sensitivity analysis (covered in this report and to be continued in final report)

2. Former achievements through this contract:

The following research activities have been executed as reported previously in the first progress report submitted on 16 January 2012 (covering the first three months of the project) and the second progress report submitted on 16 October 2012 (covering the first year of the project):
(Numbers refer to Gantt chart in original proposal):

2.1 **Research Activity No.1: Development of a novel non-proprietary cold-formed steel framing system**

1-3 Develop home archetype (covered in first report)
1-4 Develop full framing solution for archetypical home (covered in first and second reports)

2.2 **Research Activity No.2: Building Archetypes Study**

2.1 Selection of rural and urban locations for archetype homes (Already covered in first progress report)

2.2 Design and cost analysis of traditional concrete framing in Egypt (covered in second report)

2.3 Design and cost analysis of conventional cold formed steel framing (covered in second report)
2.5 Environmental impact and sustainability assessment (covered in second report and continued in this report)

In addition to the a.m. research activities, a Two-day Workshop titled 'Use of Light Steel Framing in Residential Buildings' was organized and held in the Faculty of Engineering of Cairo University on 9, 10 December 2012. The workshop was opened in the presence of the Minister of Higher Education and representatives from the Ministry of Scientific Research, Minister of Housing, Science and Technology Development Fund (STDF), Governors of Cairo and Giza. Approximately 200 engineers, architects, builders, and manufacturers attended the workshop.

The workshop was led by the United States and Egyptian project teams and included key participation from the United States Industrial Advisory Board. Four members of that board: George Richards (BORM), Don Allen (DSi), Nader ElHajj (Framecad), and Nabil Rahman (The Steel Network), came to Cairo to share their experiences with making light steel framing a reality not only in the United States, but around the world. Their talks were augmented by research talks from the project team (Egypt: Metwally Abu-Hamd, Maged Hanna, Mohammed Badr; United States: Benjamin Schafer, Zhanjie Li). An industrial exhibition also complemented the workshop and demonstrated to the Egyptian engineers that the manufacturing base was already in place to make light steel framing happen in Egypt.

The workshop included twenty lectures in six sessions covering all design, fabrication and erection aspects of the subject. A 700-pages two-volume proceedings hard copy was given to each participant in addition to the lecture handouts. All technical material of the workshop was uploaded to the web site of the Egyptian Society of Steel Structures: www.esss-eg.org where all participants can further download a soft copy of the workshop material.

Appendix 1 contains some introductory material from the workshop documents. A CD containing the proceedings is also enclosed.
3. Technical/Scientific Accomplishment/Activities

3.1.a) Task no. 1-1
3.1.b) Task Title: Develop a library of optimal shapes.
3.1.c) Duration: Twelve Month (M1 to M12)
3.1.d) Objective(s): Develop design data related to more efficient cold formed sections that can be used instead of traditional sections.

3.1.e) Narrative Description of actual accomplishments:

Cold-formed steel shapes in current use for light steel framing include the lipped channel (such as that of Figure 1a) that is employed for studs, joists, headers, jambs, etc. and the un-lipped or plain channel that is employed as track. The shapes are designed essentially to mimic timber construction, at least in their common outer dimension, but are not optimal in any structural sense. Today’s cold-formed steel is largely the same as George Winter envisioned in the 1940’s. Given the advancement in roll-forming technology, and the recent advancement in predictive methods formally adopted in design specifications the potential for a new generation of shapes exists. The potential payoff can readily be observed even in a simple ad hoc optimization as depicted in Figure 1, where options (b) and (c) provide a 270% and 370% (respectively) improvement in local buckling over a standard lipped channel stud. The example I section of Figure 2 (b) and (c) meets important constraints of current design: providing parallel surfaces to attach sheathing and a flat web region that could easily have perforations for services, while at the same time the new section vastly improves the local buckling performance, and also the torsional performance as in the example section the shear center and centroid coincide and the principal axes are aligned with the geometric axes removing a complication with the use of singly symmetric sections such as channels.

Figure 1 Finite strip local buckling analysis of (a) conventional and (b),(c) example cold-formed steel sections

(a) 365S100-63 (50ksi) \(P_{cr}/P_f = 1.03\)  (b) example novel \(P_{cr}/P_f = 2.77\)  (c) example novel \(P_{cr}/P_f = 3.84\)
Similarly, the use of Sigma Section shaped cold formed sections proved to have advantages over traditional channel shaped sections. This is attributed to the much higher local buckling resistance provided by the stiffened web of the Sigma Section section. In order to explore this point for possible application in residential cold formed steel buildings, a member optimization study was performed on the entire range of production locally available in Egypt by AlexForm Company, see Tables in first report. In that study, the carrying capacity of several section alternatives was compared to that of conventional hot rolled steel shapes. Table 1 shows the comparison matrix used in the member optimization study:

<table>
<thead>
<tr>
<th>Rolled IPE</th>
<th>Rolled HEA</th>
<th>Single C</th>
<th>Double C</th>
<th>Single Sigma Section</th>
<th>Double Sigma Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>IPE 200</td>
<td>HEA 160</td>
<td>C 300</td>
<td>2C 300</td>
<td>S 300</td>
<td>2S 300</td>
</tr>
<tr>
<td>IPE 220</td>
<td>HEA 180</td>
<td>C 360</td>
<td>2C 360</td>
<td>S 360</td>
<td>2S 360</td>
</tr>
<tr>
<td>IPE 240</td>
<td>HEA 200</td>
<td>C 400</td>
<td>2C 400</td>
<td>S 400</td>
<td>2S 400</td>
</tr>
<tr>
<td>IPE 270</td>
<td>HEA 220</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

The results of comparisons are shown on the next pages:
Minimum Weight Cold Formed Channel Sections versus Rolled Sections

WEIGHT SAVING CEE PROFILE VS. IPE/HEA PROFILE

Profile Weight G [kg/m]

WEIGHT SAVING I PROFILE VS. IPE/HEA PROFILE

Profile Weight G [kg/m]
### Example 1

<table>
<thead>
<tr>
<th>HEA 240</th>
<th>IPE 300</th>
<th>360°×4</th>
</tr>
</thead>
<tbody>
<tr>
<td>60.3 Kg/m</td>
<td>42.2 Kg/m</td>
<td>25 Kg/m</td>
</tr>
</tbody>
</table>

Weight Saving: 42%

### Example 2

<table>
<thead>
<tr>
<th>HEA 220</th>
<th>IPE 270</th>
<th>360°×3</th>
</tr>
</thead>
<tbody>
<tr>
<td>52.5 Kg/m</td>
<td>36.1 Kg/m</td>
<td>26.8 Kg/m</td>
</tr>
</tbody>
</table>

Weight Saving: 52%

### Example 3

<table>
<thead>
<tr>
<th>HEA 200</th>
<th>IPE 240</th>
<th>360°×3</th>
</tr>
</thead>
<tbody>
<tr>
<td>42.3 Kg/m</td>
<td>30.7 Kg/m</td>
<td>23.8 Kg/m</td>
</tr>
</tbody>
</table>

Weight Saving: 65%

### Example 4

<table>
<thead>
<tr>
<th>HEA 180</th>
<th>IPE 220</th>
<th>360°×2.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>35.5 Kg/m</td>
<td>26.2 Kg/m</td>
<td>20 Kg/m</td>
</tr>
</tbody>
</table>

Weight Saving: 72%
3.1.f) Deliverables

The results obtained from the member optimization study shall be combined next with the system optimization study.

3.2.a) Tasks no. 1-2 and 2-4

3.2.b) Task Title: Develop and Design ‘dual’ system for walls and floors

3.2.c) Duration: 15 Months (M4 to M18)

3.2.d) Objective(s): Develop and design a dual structural system for cold formed steel buildings where primary systems, e.g., main framing members, are combined with secondary systems, e.g., wall studs, in carrying the loads.

3.2.e) Narrative Description of actual accomplishments:

As Figure 2(a) shows, conventional cold-formed steel framing is essentially mimicry of timber framing. This, in part, reflects a desire to use the dominant low-rise building process and was intended to allow for wide adoption by builders. Light framed construction (timber or current steel) may be understood as a dense network of slight members that is occasionally perforated to create openings for doors, windows, etc., and additional stiffening is required at the perforations. Heavy framed construction (reinforced concrete or hot-rolled steel) may be understood as a sparse network of heavy members that essentially utilizes a non-structural skin between the main structural members and thus has great flexibility with respect to interior layout and exterior openings. What is proposed here is the exploration of a hybrid, or ‘dual’ system where the entire structure is framed from cold-formed steel but the same members are not used throughout.
Consider Figure 2, in (b) a primary framing system is mapped onto the existing system (a) and in (c) an example of a ‘dual’ system is provided for the wall. The ‘dual’ system utilizes primary cold-formed steel members that are of greater stiffness and strength than secondary members (but that still share the same section depth so the wall line is not disrupted). The actual member shapes will be based on the optimization results of research activity 1-1. Key features of the primary frame include semi-rigid connections between primary frame columns and beams, and beams that serve as track for the secondary frames as well as allow load redistribution around missing secondary frame members. The secondary frame is structural (i.e. it is not isolated from the primary frame) but the members will be lighter than conventional owing to the presence of the primary frame – greater flexibility in openings is an obvious advantage of such a system.

The above approach was applied to the home archetypes previously studied, see progress reports 1 and 2. The previous structural systems, i.e., rigid frames or wall panels, were augmented by a series of new design incorporating the ‘dual’ system as explained above. Furthermore, the more efficient Sigma Section sections characteristics developed under research activity 1-1 were also employed in the new study. The following additional design matrix was thus developed:
1- Rigid Framing using Sigma Section sections
2- Wall Panels using Sigma Section sections
3- Dual system using Channel sections
4- Dual system using Sigma Section sections.

The a.m. models were performed for two home archetypes having both six floors with four either 63 m² or 80 m² flats in each floor. Detailed design calculations of the eight models are give in appendix 1.

The results of all designs can be summarized in the following Table:

<table>
<thead>
<tr>
<th>System</th>
<th>Wall Panels</th>
<th>Dual System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td>Channel</td>
<td>Sigma Section</td>
</tr>
<tr>
<td>63 m²</td>
<td>36.17</td>
<td>36.03</td>
</tr>
<tr>
<td>80 m²</td>
<td>45.27</td>
<td>41</td>
</tr>
</tbody>
</table>

The following conclusions may be drawn from the a.m. results:

1- The use of Dual System gives less steel weight than Wall Panels Systems. The percentage of savings equals 9.1 % for the 63 m² model and 5.7 % for the 80 m² model.

2- The use of Sigma Section sections generally gives less steel weight than Wall panel systems. The savings in weight for the smaller model is not considerable due to the limited available selection currently produced in Egypt by ALexForm. For the bigger model, the percentage of weight savings equals 9.4 % for the Wall Panel system and 4.65 % for the dual system.

3- The combined use of Sigma Section sections and dual system is very advantageous leading to a percentage of weight savings of 9.1 % for the 63 m² model and 10 % for the 80 m² model.

3.3.f) Deliverables

The work executed under research activities 1-1 and 1-2 was the subject of a research paper Titled 'Alternative Structural System for CFS Buildings'. This paper has been accepted for Publication in the International Conference on Civil, Structural, and Earthquake Engineering (ICCSEE 2013) to be held on June 20-21, 2013 in Toronto, Canada. Acceptance letter and copy of the submitted paper are given in Appendix B.
3.3.a) Task no. 2-5

3.3.b) Task Title: Environmental Impact and Sustainability Assessment
3.3.c) Duration: Three Month (M13 to M15)
3.3.d) Objective(s): Existing sustainability tools will be utilized to assess the environmental impact of various home archetypes.

3.3.e) Narrative Description of actual accomplishments:

This work builds on the previous work reported in Progress Report no.2 on Environmental Impact and Sustainability issues. Having calculated the various Life Cycle Assessment (LCA) indicators for the two residential building systems using Athena software, the study is extended to evaluate other aspects related to sustainability with respect to some applicable credits from LEED (Leadership in Energy and Environmental Design). LEED is an internationally recognized green building program that provides building owners and operators with a framework for identifying and implementing practical and measurable green building design, construction, operations and maintenance solutions. For commercial buildings a project must satisfy all LEED prerequisites and earn a minimum of 40 points on a 110-point LEED rating system scale. Homes must earn a minimum of 45 points on a 136-point scale

*Evaluation of Building Sustainability*

*Assessment Approach*

- Review of LEED 2009 for New Construction & Major Renovations
- Identification of LEED credits that are directly impacted by the choice of the building structural system (RC versus CFS)
- Undertake a quantitative and qualitative comparison of relevant credits based on common building practices in Egypt
- Analysis based on the ‘intent’ of the credit rather than the suggested LEED calculation procedure
- Based on LEED NC 2009: up to 3 credits can be awarded to promote our Evaluation of Buildings Sustainability
  
  a. Materials & Resources: Material Reuse
  b. Materials & Resources: Recycled Content
c. Materials & Resources: Regional Materials

a. **Materials & Resources: Material Reuse:**

   ➜ **Intent:** To reuse building materials and products to reduce demand for virgin materials and reduce waste, thereby lessening impacts associated with the extraction and processing of virgin resources.

   According to LEED; these are the awarded points for materials reuses:

<table>
<thead>
<tr>
<th>Reused Materials</th>
<th>Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1</td>
</tr>
<tr>
<td>10%</td>
<td>2</td>
</tr>
</tbody>
</table>

   Application for the idea of this LEED credit in our project:
   ➢ Main building materials for a traditional RC and the proposed CFS design were assessed based on their potential for reuse based on existing common building practices in Egypt.
   ➢ A weighted average based on % cost was calculated for each building

**RC Building**

**Comment:**

<table>
<thead>
<tr>
<th>Components</th>
<th>% Cost</th>
<th>% reuse</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>6.06%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>RC base Concrete</td>
<td>5.35%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Skeleton Concrete</td>
<td>41.34%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>RFT</td>
<td>20.49%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Brick walls</td>
<td>13.81%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Ceramic*</td>
<td>12.95%</td>
<td>10.00%</td>
<td><strong>1.30%</strong></td>
</tr>
</tbody>
</table>

The Percentage of materials reuse was found only 1.3% which is significantly lower than LEED’s threshold of 5% for 1 credit.
Progress Report No. 3

CFS Building

<table>
<thead>
<tr>
<th>Components</th>
<th>% Cost</th>
<th>% reuse</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>1.94%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>RC base Concrete</td>
<td>2.08%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>RFT</td>
<td>3.75%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Steel framing*</td>
<td>35.08%</td>
<td>85.00%</td>
<td>29.82%</td>
</tr>
<tr>
<td>GRC floor panels*</td>
<td>20.41%</td>
<td>90.00%</td>
<td>18.37%</td>
</tr>
<tr>
<td>Cement board walls*</td>
<td>20.41%</td>
<td>90.00%</td>
<td>18.37%</td>
</tr>
<tr>
<td>False ceiling*</td>
<td>10.20%</td>
<td>95.00%</td>
<td>9.69%</td>
</tr>
<tr>
<td>Mortar screed flooring</td>
<td>2.04%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Fire proofing</td>
<td>4.08%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>76.25%</strong></td>
</tr>
</tbody>
</table>

Comment:
The Percentage of materials reuse was found 76.25% surpassing the LEED’s threshold of 10% for 2 credits. This is due to the modular nature of the product that will allow the building element to be disassembled and reused on a new building or in case of needed building relocation.

b- Recycled Content:

➤ **Intent:** To increase demand for building products that incorporate recycled content materials, thereby reducing impacts resulting from extraction and processing of virgin materials. This table also represents the awarded points by LEED for the recycled content.

<table>
<thead>
<tr>
<th>Recycled Content</th>
<th>Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>1</td>
</tr>
<tr>
<td>20%</td>
<td>2</td>
</tr>
</tbody>
</table>

Another application for a LEED credit in our project:
Main building materials for a traditional RC and the proposed CFS design were assessed based on the values of their recycled content according to common manufacturing practices in Egypt

➤ A weighted average based on % cost was calculated for each building.

RC Building

<table>
<thead>
<tr>
<th>Components</th>
<th>% Cost</th>
<th>% Recycled</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>6.06%</td>
<td>5.00%</td>
<td>0.30%</td>
</tr>
<tr>
<td>RC base Concrete</td>
<td>5.35%</td>
<td>5.00%</td>
<td>0.27%</td>
</tr>
<tr>
<td>Skeleton Concrete</td>
<td>41.34%</td>
<td>5.00%</td>
<td>2.07%</td>
</tr>
<tr>
<td>RFT</td>
<td>20.49%</td>
<td>90.00%</td>
<td>18.44%</td>
</tr>
<tr>
<td>Brick walls</td>
<td>13.81%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Ceramic</td>
<td>12.95%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>21.08%</strong></td>
</tr>
</tbody>
</table>
**Comment:**
The estimated recycled content surpasses the LEED’s threshold of 10% for 2 credits.

**CFS Building**

<table>
<thead>
<tr>
<th>Components</th>
<th>% Cost</th>
<th>% Recycled</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC</td>
<td>1.94%</td>
<td>5.00%</td>
<td>0.10%</td>
</tr>
<tr>
<td>RC base Concrete</td>
<td>2.08%</td>
<td>5.00%</td>
<td>0.10%</td>
</tr>
<tr>
<td>RFT</td>
<td>3.75%</td>
<td>90.00%</td>
<td>3.37%</td>
</tr>
<tr>
<td>Steel framing</td>
<td>35.08%</td>
<td>90.00%</td>
<td>31.57%</td>
</tr>
<tr>
<td>GRC floor panels</td>
<td>20.41%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Cement board walls</td>
<td>20.41%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>False ceiling</td>
<td>10.20%</td>
<td>15.00%</td>
<td>1.53%</td>
</tr>
<tr>
<td>Mortar screed flooring</td>
<td>2.04%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Fire proofing</td>
<td>4.08%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
</tbody>
</table>

**Comment:**
The Recycled content’s percentage is almost double that of the RC Building. We can illustrate that False Ceiling has a percentage of recycling due to the presence of Aluminum framing as a part of its components.

**c. Regional materials**

➤ **Intent:** To increase demand for building materials and products that are extracted and manufactured within the region, thereby supporting the use of indigenous resources and reducing the environmental impacts resulting from transportation

**Third LEED credit application in our project:**

- Thresholds used in LEED were quite high (500 miles) and inapplicable to Egypt
- A weighted average transportation distance from the nearest manufacturer based on % cost was calculated for each building for 6 main geographical locations in Egypt
Governorates' location on Egypt's map

- Cairo
- Alexandria
- Ismailia
- Asyut
- Aswan
- Hurghada

RC Building

Expected transportation distance from nearest manufacturer (km)

<table>
<thead>
<tr>
<th></th>
<th>PC</th>
<th>RC base</th>
<th>RFT</th>
<th>Skeleton</th>
<th>Brick walls</th>
<th>ceramic</th>
<th>Av. Dist.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cairo</td>
<td>10</td>
<td>10</td>
<td>180.18</td>
<td>10</td>
<td>15</td>
<td>10</td>
<td>81</td>
</tr>
<tr>
<td>Alexandria</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>135</td>
<td>30</td>
<td>26</td>
</tr>
<tr>
<td>Aswan</td>
<td>300</td>
<td>300</td>
<td>1168</td>
<td>300</td>
<td>640</td>
<td>795</td>
<td>770</td>
</tr>
<tr>
<td>Ismailia</td>
<td>10</td>
<td>10</td>
<td>288</td>
<td>10</td>
<td>140</td>
<td>140</td>
<td>160</td>
</tr>
<tr>
<td>Assyut</td>
<td>240</td>
<td>240</td>
<td>571</td>
<td>240</td>
<td>130</td>
<td>300</td>
<td>369</td>
</tr>
<tr>
<td>Hurghada</td>
<td>7</td>
<td>7</td>
<td>680</td>
<td>7</td>
<td>458</td>
<td>345</td>
<td>391</td>
</tr>
<tr>
<td>% Cost</td>
<td>6.06%</td>
<td>5.35%</td>
<td>41.34%</td>
<td>20.49%</td>
<td>13.81%</td>
<td>12.95%</td>
<td></td>
</tr>
</tbody>
</table>
CFS Design
Weighted average transportation distances (km)

<table>
<thead>
<tr>
<th>Place</th>
<th>PC</th>
<th>RC</th>
<th>RFT</th>
<th>Steel framing</th>
<th>GRC floor panels</th>
<th>Cement walls</th>
<th>False ceiling</th>
<th>Mortar flooring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cairo</td>
<td>10</td>
<td>10</td>
<td>180.18</td>
<td>180.18</td>
<td>68</td>
<td>68</td>
<td>30</td>
<td>10</td>
</tr>
<tr>
<td>Alexandria</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>154</td>
<td>154</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>Aswan</td>
<td>300</td>
<td>300</td>
<td>1168</td>
<td>1168</td>
<td>930</td>
<td>930</td>
<td>870</td>
<td>300</td>
</tr>
<tr>
<td>Ismailia</td>
<td>10</td>
<td>10</td>
<td>288</td>
<td>288</td>
<td>80</td>
<td>80</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Assyut</td>
<td>240</td>
<td>240</td>
<td>571</td>
<td>571</td>
<td>435</td>
<td>435</td>
<td>390</td>
<td>240</td>
</tr>
<tr>
<td>Hurghada</td>
<td>7</td>
<td>7</td>
<td>680</td>
<td>680</td>
<td>534</td>
<td>534</td>
<td>480</td>
<td>7</td>
</tr>
<tr>
<td>%Cost</td>
<td>1.94%</td>
<td>2.08%</td>
<td>3.75%</td>
<td>35.08%</td>
<td>20.41%</td>
<td>20.41%</td>
<td>10.20%</td>
<td>2.04%</td>
</tr>
</tbody>
</table>

Comparison
1- In most Egyptian cities RC buildings outperform CFS buildings in terms of average distances for material transportation (only exception is Ismailia City)
2- Cities in Upper Egypt have relatively high average material transportation distances for both designs due to their large proximity from major steel manufacturing facilities in the north.

<table>
<thead>
<tr>
<th>Place</th>
<th>CFS Design</th>
<th>RC Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alexandria</td>
<td>71</td>
<td>26</td>
</tr>
<tr>
<td>Cairo</td>
<td>102</td>
<td>81</td>
</tr>
<tr>
<td>Ismailia</td>
<td>147</td>
<td>160</td>
</tr>
<tr>
<td>Assyut</td>
<td>454</td>
<td>369</td>
</tr>
<tr>
<td>Hurghada</td>
<td>532</td>
<td>391</td>
</tr>
<tr>
<td>Aswan</td>
<td>940</td>
<td>770</td>
</tr>
</tbody>
</table>

Summary
- After applying the credit of material reuse the CFS building has a significant high percentage of material reuse (76.25%) where the RC Building has only 1.3% of materials reuse.
- The second credit of Recycled content showed that the CFS Building is having approximately about double percentage of the RC Building.
- The results of the third credit of Regional Materials were better at the RC building Except at Ismailia

3.3.f) Deliverables
The work executed under research activities 1-1 and 1-2 was the subject of a research paper titled 'Cost and Sustainability Analysis of Cold Formed Steel Residential Buildings'. This paper has been accepted for Publication in the International Conference on Civil, Structural, and Earthquake Engineering (ICCSEE 2013) to be held on June 20-21, 2013 in Toronto, Canada. Copy of Acceptance letter and copy of the submitted paper are given in Appendix B.
3.4.a) Task no. 2-5

3.4.b) Task Title: Sensitivity Analysis
3.4.c) Duration: Twelve Month (M13 to M24)
3.4.d) Objective(s): Perform sensitivity analysis to study effect of variations of design parameters on the results.

3.4.e) Narrative Description of actual accomplishments:

**Sensitivity analysis:**

It is the study of how the uncertainty in the output of a mathematical model or system (numerical or otherwise) can be apportioned to different sources of uncertainty in its inputs. A related practice is uncertainty analysis, which has a greater focus on uncertainty quantification and propagation of uncertainty. Ideally, uncertainty and sensitivity analysis should be run in tandem.

Sensitivity analysis consists in computing derivatives of one or more quantities (outputs) with respect to one or several independent variables (inputs). Although there are various uses for sensitivity information, our main motivation is the use of this information in gradient-based optimization. Since the calculation of gradients is often the most costly step in the optimization cycle, using efficient methods that accurately calculate sensitivities is extremely important.

Good modeling practice requires that the modeler provides an evaluation of the confidence in the model. This requires, first, a *quantification* of the uncertainty in any model results (*uncertainty analysis*); and second, an evaluation of how much each input is contributing to the output uncertainty. Sensitivity analysis addresses the second of these issues (although uncertainty analysis is usually a necessary precursor), performing the role of ordering by importance the strength and relevance of the inputs in determining the variation in the output.

**Sensitivity analysis can be useful for a range of purposes, including:**

- Testing the robustness of the results of a model or system in the presence of uncertainty.
- Increased understanding of the relationships between input and output variables in a system or model.
- Uncertainty reduction: identifying model inputs that cause significant uncertainty in the output and should therefore is the focus of attention if the robustness is to be increased (perhaps by further research).
- Searching for errors in the model (by encountering unexpected relationships between inputs and outputs).
- Model simplification - fixing model inputs that have no effect on the output, or identifying and removing redundant parts of the model structure.
- Enhancing communication from modelers to decision makers (e.g. by making recommendations more credible, understandable, compelling or persuasive).
Settings and constraints:
The choice of method of sensitivity analysis is typically dictated by a number of problem constraints or settings. Some of the most common are:

- **Computational expense**: Sensitivity analysis is almost always performed by running the model a (possibly large) number of times, i.e. a sampling-based approach. This can be a significant problem when,
  - A single run of the model takes a significant amount of time (minutes, hours or longer). This is not unusual with very complex models.
  - The model has a large number of uncertain inputs. Sensitivity analysis is essentially the exploration of the multidimensional input space, which grows exponentially in size with the number of inputs. See the curse of dimensionality.
Computational expense is a problem in many practical sensitivity analyses. Some methods of reducing computational expense include the use of emulators (for large models), and screening methods (for reducing the dimensionality of the problem).

- **Correlated inputs**: Most common sensitivity analysis methods assume independence between model inputs, but sometimes inputs can be strongly correlated. This is still an immature field of research and definitive methods have yet to be established.

- **Nonlinearity**: Some sensitivity analysis approaches, such as those based on linear regression, can inaccurately measure sensitivity when the model response is nonlinear with respect to its inputs. In such cases, variance-based measures are more appropriate.

- **Model interactions**: Interactions occur when the perturbation of two or more inputs simultaneously causes variation in the output greater than that of varying each of the inputs alone. Such interactions are present in any model that is non-additive, but will be neglected by methods such as scatterplots and one-at-a-time perturbations. The effect of interactions can be measured by the total-order sensitivity index.

- **Multiple outputs**: Virtually all sensitivity analysis methods consider a single univariate model output, yet many models output a large number of possibly spatially or time-dependent data. Note that this does not preclude the possibility of performing different sensitivity analyses for each output of interest. However, for models in which the outputs are correlated, the sensitivity measures can be hard to interpret.

- **Given data**: While in many cases the practitioner has access to the model, in some instances a sensitivity analysis must be performed with "given data", i.e. where the sample points (the values of the model inputs for each run) cannot be chosen by the analyst. This may occur when a sensitivity analysis has to performed retrospectively, perhaps using data from an
optimization or uncertainty analysis, or when data comes from a discrete source

**One-at-a-time (OAT/OFAT)**

One of the simplest and most common approaches is that of changing one-factor-at-a-time (OFAT or OAT), to see what effect this produces on the output. OAT customarily involves:

- Moving one input variable, keeping others at their baseline (nominal) values, then,

- Returning the variable to its nominal value, and then repeating for each of the other inputs in the same way.

Sensitivity may then be measured by monitoring changes in the output, e.g. by partial derivatives or linear regression. This appears a logical approach as any change observed in the output will unambiguously be due to the single variable changed. Furthermore, by changing one variable at a time, one can keep all other variables fixed to their central or baseline values. This increases the comparability of the results (all ‘effects’ are computed with reference to the same central point in space) and minimizes the chances of computer program crashes, more likely when several input factors are changed simultaneously. OAT is frequently preferred by modelers because of practical reasons. In case of model failure under OAT analysis the modeler immediately knows which the input factor responsible for the failure is.

Despite its simplicity however, this approach does not fully explore the input space, since it does not take into account the simultaneous variation of input variables. This means that the OAT approach cannot detect the presence of interactions between input variables.

**Short comings of Sensitivity analysis:**

- Unrealistic to assume variations to occur one at a time.
- It doesn’t give an overall indication for the project.

**Simulation method:**

It’s developed to better understand a project under certain circumstances. It creates imaginary scenarios for the project duration or/and cost to reflect different results of impact.

It’s called **Monte Carlo simulation**

1. It creates an imaginary scenario for the project.
2. Project scheduling and/or costing is performed.
3. Results are recorded.
4. The process is repeated many times.
5. A probability distribution of the results is performed.

Data analysis tools like Scenario analysis, Probability analysis and PERT, but Monte Carlo simulation is better to be used as it can take make a mixture for all of these analysis tools.
Assessment Approach: (For Applying Monte Carlo in the project)

We have performed two Monte Carlo simulations by using two different historical data; the first is quarterly material costs (2003-2008) and the other is annual material costs (2003-2012). Here are the steps that were done to complete the simulation.

1. A capture for the materials historical costs got from the C.A.P.M.A.S
2. Performing a time series analysis using these data, by getting their statistical data and making a best fitting for these data using Exponential smoothing technique
3. Prediction of the future costs (5 years predictions); the prediction is performed by ARIMA method (ARIMA forecasting, Exponential Smoothing or moving averages)
4. Fitting the results to the best probability distribution and recording their statistical data
5. Performing a 5000 trials Monte Carlo simulation using a combination of these recorded materials’ statistical data
6. This methodology was applied similarly in both simulations
## Time Series Cost Analysis: (Quarters Forecasting)

<table>
<thead>
<tr>
<th>Year</th>
<th>Month</th>
<th>Reinforcement (Historical Prices)</th>
<th>Fit &amp; Forecast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>January</td>
<td>1465.1</td>
<td><strong>Arima</strong> 1,803.08</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>1831.38</td>
<td>1,767.80</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>1831.38</td>
<td>1,575.03</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>2084.95</td>
<td>1,803.08</td>
</tr>
<tr>
<td>2004</td>
<td>January</td>
<td>2774</td>
<td>2,591.74</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>3027</td>
<td>2,908.96</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>3016</td>
<td>2,614.81</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>3103</td>
<td>3,140.78</td>
</tr>
<tr>
<td>2005</td>
<td>January</td>
<td>3033</td>
<td>3,104.63</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>2914</td>
<td>2,878.34</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>2610</td>
<td>2,970.23</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>2655</td>
<td>2,317.67</td>
</tr>
<tr>
<td>2006</td>
<td>January</td>
<td>2590</td>
<td>3,213.43</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>2590.6</td>
<td>2,001.37</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>3180</td>
<td>3,148.83</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>3180</td>
<td>3,372.39</td>
</tr>
<tr>
<td>2007</td>
<td>January</td>
<td>3180</td>
<td>2,515.96</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>3490</td>
<td>3,746.80</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>3530</td>
<td>3,358.28</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>3530</td>
<td>3,424.98</td>
</tr>
<tr>
<td>2008</td>
<td>January</td>
<td>4153</td>
<td>3,585.72</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>5766</td>
<td>4,812.98</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>6453</td>
<td>6,506.25</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>4457</td>
<td>5,233.53</td>
</tr>
</tbody>
</table>

**Historical Data statistics:**

- 2,648.42
- 3,830.77
- 5,698.08
- 5,222.29
- 3,504.54
- 3,423.34
- 4,857.12
- 5,330.52
- 4,248.22
- 3,541.40
- 4,259.77
- 5,061.86
- 4,678.98
- 3,890.76
- 3,993.05
- 4,690.34
- 4,800.33
- 4,240.06
- 3,988.70
- 4,392.90
### Forecasted data statistics:

**Probability Distribution**

**Normal**  
Mean = 4,315.07, Std. Dev. = 746.91

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Historical data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Values</td>
<td>24</td>
</tr>
<tr>
<td>Minimum</td>
<td>1,465.10</td>
</tr>
<tr>
<td>Mean</td>
<td>3,185.18</td>
</tr>
<tr>
<td>Maximum</td>
<td>6,453.00</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>1,136.45</td>
</tr>
<tr>
<td>Ljung-Box</td>
<td>28.04</td>
</tr>
<tr>
<td>Seasonality</td>
<td>Non-seasonal</td>
</tr>
<tr>
<td>Year</td>
<td>Month</td>
</tr>
<tr>
<td>------</td>
<td>-------</td>
</tr>
<tr>
<td>2003</td>
<td>January</td>
</tr>
<tr>
<td></td>
<td>April</td>
</tr>
<tr>
<td></td>
<td>July</td>
</tr>
<tr>
<td></td>
<td>October</td>
</tr>
<tr>
<td>2004</td>
<td>January</td>
</tr>
<tr>
<td></td>
<td>April</td>
</tr>
<tr>
<td></td>
<td>July</td>
</tr>
<tr>
<td></td>
<td>October</td>
</tr>
<tr>
<td>2005</td>
<td>January</td>
</tr>
<tr>
<td></td>
<td>April</td>
</tr>
<tr>
<td></td>
<td>July</td>
</tr>
<tr>
<td></td>
<td>October</td>
</tr>
<tr>
<td>2006</td>
<td>January</td>
</tr>
<tr>
<td></td>
<td>April</td>
</tr>
<tr>
<td></td>
<td>July</td>
</tr>
<tr>
<td></td>
<td>October</td>
</tr>
<tr>
<td>2007</td>
<td>January</td>
</tr>
<tr>
<td></td>
<td>April</td>
</tr>
<tr>
<td></td>
<td>July</td>
</tr>
<tr>
<td></td>
<td>October</td>
</tr>
<tr>
<td>2008</td>
<td>January</td>
</tr>
<tr>
<td></td>
<td>April</td>
</tr>
<tr>
<td></td>
<td>July</td>
</tr>
<tr>
<td></td>
<td>October</td>
</tr>
</tbody>
</table>

24.13
24.84
25.55
26.24
26.92
27.60
28.26
28.91
29.55
30.19
30.81
31.42
32.02
32.62
33.20
33.78
34.35
34.90
35.45
36.00
Historical Statistics:

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Historical data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Values</td>
<td>24</td>
</tr>
<tr>
<td>Minimum</td>
<td>6.50</td>
</tr>
<tr>
<td>Mean</td>
<td>15.06</td>
</tr>
<tr>
<td>Maximum</td>
<td>23.40</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>4.48</td>
</tr>
<tr>
<td>Ljung-Box</td>
<td>25.17</td>
</tr>
</tbody>
</table>
| Seasonality         | Non-seasonal    

Forecasted Probability Distributions:

Probability Distribution

Beta, Minimum=23.74, Maximum=36.17, Alpha=1.04511, Beta=0.92449
<table>
<thead>
<tr>
<th>Year</th>
<th>Month</th>
<th>Sand (m$^3$)</th>
<th>Fit &amp; Forecast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>January</td>
<td>11</td>
<td>Double exp.</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>2004</td>
<td>January</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>14</td>
<td>13.33</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>14</td>
<td>15.00</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>14</td>
<td>16.00</td>
</tr>
<tr>
<td>2005</td>
<td>January</td>
<td>14</td>
<td>14.67</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>14.6</td>
<td>14.00</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>14.6</td>
<td>14.47</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>14.6</td>
<td>14.80</td>
</tr>
<tr>
<td>2006</td>
<td>January</td>
<td>14</td>
<td>15.00</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>14.6</td>
<td>14.27</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>14.6</td>
<td>14.27</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>14.6</td>
<td>14.40</td>
</tr>
<tr>
<td>2007</td>
<td>January</td>
<td>14.6</td>
<td>14.87</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>14.6</td>
<td>14.73</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>15.5</td>
<td>14.60</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>16.75</td>
<td>15.30</td>
</tr>
<tr>
<td>2008</td>
<td>January</td>
<td>18</td>
<td>16.77</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>18.5</td>
<td>18.74</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>21</td>
<td>19.84</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>22</td>
<td>21.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>23.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>24.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>25.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>27.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>28.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>30.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>31.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>32.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>34.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>35.47</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>36.83</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>38.19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>39.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>40.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>42.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>43.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>46.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>47.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>49.08</td>
</tr>
</tbody>
</table>
Progress Report No. 3

Historical Statistics:

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Historical data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Values</td>
<td>24</td>
</tr>
<tr>
<td>Minimum</td>
<td>11.00</td>
</tr>
<tr>
<td>Mean</td>
<td>14.86</td>
</tr>
<tr>
<td>Maximum</td>
<td>22.00</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>2.80</td>
</tr>
<tr>
<td>Ljung-Box</td>
<td>44.67</td>
</tr>
<tr>
<td>Seasonality</td>
<td>Non-seasonal</td>
</tr>
</tbody>
</table>

Forecasted Probability Distributions:

| Probability Distribution | Minimum=22.62, Maximum=49.69, Alpha=0.98668, Beta=0.98668 |
## Progress Report No. 3

<table>
<thead>
<tr>
<th>Year</th>
<th>Month</th>
<th>Aggregate (m³)</th>
<th>Fit&amp; Forecast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>January</td>
<td>24</td>
<td><em>Arima</em></td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>24</td>
<td>23.86</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>24</td>
<td>24.31</td>
</tr>
<tr>
<td>2004</td>
<td>January</td>
<td>24.25</td>
<td>23.91</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>24.25</td>
<td>24.21</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>24.25</td>
<td>24.79</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>24.25</td>
<td>23.94</td>
</tr>
<tr>
<td>2005</td>
<td>January</td>
<td>24</td>
<td>24.03</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>24.4</td>
<td>24.23</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>24.4</td>
<td>24.20</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>24.4</td>
<td>25.08</td>
</tr>
<tr>
<td>2006</td>
<td>January</td>
<td>24.3</td>
<td>24.13</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>24.3</td>
<td>23.90</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>24.3</td>
<td>24.57</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>24.3</td>
<td>24.44</td>
</tr>
<tr>
<td>2007</td>
<td>January</td>
<td>24.3</td>
<td>24.01</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>24.3</td>
<td>24.37</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>27.5</td>
<td>24.47</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>29.5</td>
<td>29.13</td>
</tr>
<tr>
<td>2008</td>
<td>January</td>
<td>34.6</td>
<td>35.28</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>35.25</td>
<td>36.28</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>40.8</td>
<td>39.46</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>42</td>
<td>41.22</td>
</tr>
</tbody>
</table>

### Graph

The graph shows the fitted, historical, and forecasted values for the given years. The forecast values are indicated by the blue line, with the historical data shown in red dots. The lower and upper limits for the forecast are marked by dotted lines.

- **Fitted**: The blue line represents the fitted values.
- **Historical**: The red dots represent the historical data points.
- **Forecast**: The blue line with the dotted lines represents the forecasted values.
### Historical Statistics:

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Historical data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Values</td>
<td>24</td>
</tr>
<tr>
<td>Minimum</td>
<td>24.00</td>
</tr>
<tr>
<td>Mean</td>
<td>26.90</td>
</tr>
<tr>
<td>Maximum</td>
<td>42.00</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>5.46</td>
</tr>
<tr>
<td>Ljung-Box</td>
<td>50.61</td>
</tr>
<tr>
<td>Seasonality</td>
<td>Non-seasonal</td>
</tr>
</tbody>
</table>

### Forecasted Statistics:

**Probability Distribution**

**Beta**  Minimum=46.32, Maximum=133.81, Alpha=0.96632, Beta=0.97008
<table>
<thead>
<tr>
<th>Year</th>
<th>Month</th>
<th>Brick (1000)</th>
<th>Fit&amp; Forecast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>January</td>
<td>130</td>
<td>Double exp.</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>130</td>
<td>130.00</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>130</td>
<td>130.00</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>132.5</td>
<td>130.00</td>
</tr>
<tr>
<td>2004</td>
<td>January</td>
<td>132.5</td>
<td>132.64</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>132.5</td>
<td>132.64</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>132.7</td>
<td>132.63</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>132.7</td>
<td>132.84</td>
</tr>
<tr>
<td>2005</td>
<td>January</td>
<td>132.7</td>
<td>132.83</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>132.7</td>
<td>132.82</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>132.7</td>
<td>132.81</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>132.7</td>
<td>132.81</td>
</tr>
<tr>
<td>2006</td>
<td>January</td>
<td>154.7</td>
<td>132.80</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>154.7</td>
<td>156.04</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>154.7</td>
<td>156.04</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>154.7</td>
<td>155.96</td>
</tr>
<tr>
<td>2007</td>
<td>January</td>
<td>166.3</td>
<td>155.88</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>166.3</td>
<td>168.07</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>166.3</td>
<td>168.00</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>166.3</td>
<td>167.90</td>
</tr>
<tr>
<td>2008</td>
<td>January</td>
<td>290</td>
<td>167.81</td>
</tr>
<tr>
<td></td>
<td>April</td>
<td>300</td>
<td>298.41</td>
</tr>
<tr>
<td></td>
<td>July</td>
<td>300</td>
<td>308.93</td>
</tr>
<tr>
<td></td>
<td>October</td>
<td>300</td>
<td>308.43</td>
</tr>
</tbody>
</table>

![Graph showing fitted, historical, and forecast values with upper and lower confidence bounds.](chart.png)
Historical Statistics:

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Historical data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data Values</td>
<td>24</td>
</tr>
<tr>
<td>Minimum</td>
<td>130.00</td>
</tr>
<tr>
<td>Mean</td>
<td>169.07</td>
</tr>
<tr>
<td>Maximum</td>
<td>300.00</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>60.23</td>
</tr>
<tr>
<td>Ljung-Box</td>
<td>33.97</td>
</tr>
<tr>
<td>Seasonality</td>
<td>Non-seasonal</td>
</tr>
</tbody>
</table>

Forecasted Statistics:

| Probability distribution | Beta Minimum=304.42, Maximum=461.34, Alpha=0.98668, Beta=0.98668 |

Linking the probability distributions (for costs) to the project quantities in both designs:

**RC Building:**

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC for Footings (m^3)</td>
<td>147.39</td>
<td></td>
</tr>
<tr>
<td>RC for Footings (m^3)</td>
<td>112.68</td>
<td></td>
</tr>
<tr>
<td>Skeleton (m^3)</td>
<td>370.23</td>
<td>Forecasts of Probability Distributions</td>
</tr>
<tr>
<td>Brick Walls (m^2/H.P.)</td>
<td>1680</td>
<td></td>
</tr>
<tr>
<td>Ceramic Flooring (m^2)</td>
<td>1680</td>
<td></td>
</tr>
</tbody>
</table>

**CFS Building:**

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC Footing (m^3)</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>RC Footing (m^3)</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Steel Framing (ton)</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>GRC Floor Panels (m^2)</td>
<td>1680</td>
<td></td>
</tr>
<tr>
<td>Cement Board Walls (m^2/H.P.)</td>
<td>1680</td>
<td></td>
</tr>
<tr>
<td>False Ceiling (m^2)</td>
<td>1680</td>
<td></td>
</tr>
<tr>
<td>Mortar Screed Flooring m^2</td>
<td>1680</td>
<td></td>
</tr>
<tr>
<td>Fire Proofing m^2</td>
<td>1680</td>
<td></td>
</tr>
</tbody>
</table>
Running a Monte Carlo Simulation: (Quarter Forecasting)  
(Quarters Historical Costs 2003-2008, Forecasted 2009-2013)

Results Analysis

- CFS buildings have a lower expected construction cost compared to RC buildings
- Uncertainty (risk) of cost deviations between CFS and RC is comparable based on historical material cost fluctuations in Egypt
Progress Report No. 3

Annual Forecasting (same steps for the second simulation):

<table>
<thead>
<tr>
<th>Year</th>
<th>RFT Hist. Price</th>
<th>Fit&amp; Forc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>1803.2025</td>
<td>Arima</td>
</tr>
<tr>
<td>2004</td>
<td>2980</td>
<td>2,306.92</td>
</tr>
<tr>
<td>2005</td>
<td>2803</td>
<td>2,428.00</td>
</tr>
<tr>
<td>2006</td>
<td>2885.15</td>
<td>2,901.11</td>
</tr>
<tr>
<td>2007</td>
<td>3432.5</td>
<td>3,373.16</td>
</tr>
<tr>
<td>2008</td>
<td>4985.8</td>
<td>3,564.05</td>
</tr>
<tr>
<td>2009</td>
<td>3000</td>
<td>3,509.97</td>
</tr>
<tr>
<td>2010</td>
<td>4042.5</td>
<td>4,536.63</td>
</tr>
<tr>
<td>2011</td>
<td>4686.5</td>
<td>4,701.03</td>
</tr>
<tr>
<td>2012</td>
<td>4324.5</td>
<td>4,433.11</td>
</tr>
<tr>
<td>2013</td>
<td>4,317.88</td>
<td></td>
</tr>
<tr>
<td>2014</td>
<td>4,191.42</td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>4,135.31</td>
<td></td>
</tr>
<tr>
<td>2016</td>
<td>4,110.40</td>
<td></td>
</tr>
<tr>
<td>2017</td>
<td>4,099.36</td>
<td></td>
</tr>
<tr>
<td>2018</td>
<td>4,094.45</td>
<td></td>
</tr>
<tr>
<td>2019</td>
<td>4,092.28</td>
<td></td>
</tr>
</tbody>
</table>

Statistic | Historical data
----------|-------------------
Data Values | 10
Minimum | 1,803.20
Mean | 3,494.32
Maximum | 4,985.80
Standard Deviation | 992.03
Ljung-Box | 7.40
Seasonality | Non-seasonal

Probability Distribution

Normal

Mean = 4170.87, Standard deviation = 80.0868
### Probability Distribution

<table>
<thead>
<tr>
<th>Year</th>
<th>Cement Hist. Price</th>
<th>Fit&amp; Forc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>8.8875</td>
<td>DES</td>
</tr>
<tr>
<td>2004</td>
<td>12.5</td>
<td>8.89</td>
</tr>
<tr>
<td>2005</td>
<td>13.0625</td>
<td>13.11</td>
</tr>
<tr>
<td>2006</td>
<td>15.425</td>
<td>13.67</td>
</tr>
<tr>
<td>2007</td>
<td>18.23</td>
<td>16.32</td>
</tr>
<tr>
<td>2008</td>
<td>22.73</td>
<td>19.45</td>
</tr>
<tr>
<td>2009</td>
<td>25.525</td>
<td>24.50</td>
</tr>
<tr>
<td>2010</td>
<td>27.15</td>
<td>27.47</td>
</tr>
<tr>
<td>2011</td>
<td>23.235</td>
<td>29.05</td>
</tr>
<tr>
<td>2012</td>
<td>26.455</td>
<td>24.15</td>
</tr>
<tr>
<td>2013</td>
<td>27.75</td>
<td></td>
</tr>
<tr>
<td>2014</td>
<td>29.06</td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>30.36</td>
<td></td>
</tr>
<tr>
<td>2016</td>
<td>31.66</td>
<td></td>
</tr>
<tr>
<td>2017</td>
<td>32.96</td>
<td></td>
</tr>
<tr>
<td>2018</td>
<td>34.26</td>
<td></td>
</tr>
<tr>
<td>2019</td>
<td>35.57</td>
<td></td>
</tr>
</tbody>
</table>

Normal

Mean = 30.358, Standard deviation = 1.841
### Probability Distribution

**Normal**

Mean = 82.8248, Standard deviation = 5.327

<table>
<thead>
<tr>
<th>Year</th>
<th>Aggregate Price</th>
<th>Fit &amp; Forecast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>24</td>
<td>DES</td>
</tr>
<tr>
<td>2004</td>
<td>24.25</td>
<td>24.00</td>
</tr>
<tr>
<td>2005</td>
<td>24.3</td>
<td>24.50</td>
</tr>
<tr>
<td>2006</td>
<td>24.3</td>
<td>24.35</td>
</tr>
<tr>
<td>2007</td>
<td>26.4</td>
<td>24.30</td>
</tr>
<tr>
<td>2008</td>
<td>40.042</td>
<td>28.49</td>
</tr>
<tr>
<td>2009</td>
<td>59.34</td>
<td>53.65</td>
</tr>
<tr>
<td>2010</td>
<td>65.7</td>
<td>78.63</td>
</tr>
<tr>
<td>2011</td>
<td>67.75</td>
<td>72.10</td>
</tr>
<tr>
<td>2012</td>
<td>71.525</td>
<td>69.80</td>
</tr>
<tr>
<td>2013</td>
<td>75.29</td>
<td></td>
</tr>
<tr>
<td>2014</td>
<td>79.06</td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>82.82</td>
<td></td>
</tr>
<tr>
<td>2016</td>
<td>86.59</td>
<td></td>
</tr>
<tr>
<td>2017</td>
<td>90.36</td>
<td></td>
</tr>
<tr>
<td>2018</td>
<td>94.13</td>
<td></td>
</tr>
<tr>
<td>2019</td>
<td>97.89</td>
<td></td>
</tr>
</tbody>
</table>
Progress Report No. 3

<table>
<thead>
<tr>
<th>Year</th>
<th>Brick Prices</th>
<th>Fit&amp; Forc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2003</td>
<td>130.625</td>
<td>DES</td>
</tr>
<tr>
<td>2004</td>
<td>132.6</td>
<td>130.63</td>
</tr>
<tr>
<td>2005</td>
<td>132.7</td>
<td>132.71</td>
</tr>
<tr>
<td>2006</td>
<td>154.7</td>
<td>132.81</td>
</tr>
<tr>
<td>2007</td>
<td>166.3</td>
<td>156.01</td>
</tr>
<tr>
<td>2008</td>
<td>302</td>
<td>168.19</td>
</tr>
<tr>
<td>2009</td>
<td>310</td>
<td>311.22</td>
</tr>
<tr>
<td>2010</td>
<td>325</td>
<td>319.29</td>
</tr>
<tr>
<td>2011</td>
<td>307.75</td>
<td>334.60</td>
</tr>
<tr>
<td>2012</td>
<td>305</td>
<td>315.89</td>
</tr>
<tr>
<td>2013</td>
<td>312.51</td>
<td></td>
</tr>
<tr>
<td>2014</td>
<td>320.02</td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>327.52</td>
<td></td>
</tr>
<tr>
<td>2016</td>
<td>335.03</td>
<td></td>
</tr>
<tr>
<td>2017</td>
<td>342.53</td>
<td></td>
</tr>
<tr>
<td>2018</td>
<td>350.04</td>
<td></td>
</tr>
<tr>
<td>2019</td>
<td>357.54</td>
<td></td>
</tr>
</tbody>
</table>

Probability Distribution

Normal

Mean= 327.52, Standard deviation= 10.612
Running the second Monte Carlo simulation (Annual Records)

Based on the same Quantities but with the newer forecasted data from 2013 to 2018 based on Historical Annual records from 2003 to 2012

Results Analysis

- CFS buildings have a lower expected construction cost compared to RC buildings
- Uncertainty (risk) of cost deviations between CFS and RC is significantly higher in the CFS Design

RC Design (Beta): Mean = 1,099,377.79, Standard Deviation = 86253.95
CFS Design (Beta): Mean = 832536.06, Standard Deviation = 74195.03
Summary

Quarter Predictions:
- CFS buildings have a lower expected construction cost compared to RC buildings
- Uncertainty (risk) of cost deviations between CFS and RC is comparable based on historical material cost fluctuations in Egypt

Annual Predictions:
- CFS buildings have a lower expected construction cost compared to RC buildings
- Uncertainty (risk) of cost deviations between CFS and RC is significantly higher in the CFS Design

3.5.f) Deliverables

The work executed under research activities 2-5 and 2-6 was the subject of a research paper Titled ‘Cost and Sustainability Analysis of Cold Formed Steel Residential Buildings’. This paper has been accepted for Publication in the International Conference on Civil, Structural, and Earthquake Engineering (ICCSEE 2013) to be held on June 20-21, 2013 in Toronto, Canada. Copy of Acceptance letter and copy of the submitted paper are given in Appendix B.
3.6 The Gantt Chart for the reporting period:
All the scheduled tasks stated in the Gantt Chart have been executed successfully. These tasks are:

**1-2) Research Activity No.1:**

1-1 Develop a library of optimal shapes (covered in this report)
1-2 Develop 'dual' system for walls and floors (covered in this report)

**1-2) Research Activity No.2:**

2.4 Design and cost analysis of dual system cold formed steel framing (covered in this report and to be continued in final report).
2.5 Environmental impact and sustainability assessment (covered in second report and continued in this report)
2.6 Sensitivity analysis (covered in this report and to be continued in final report)

A revised version of the Gantt Chart is shown next.
Established by the presidential decree number 218 for the year 2007

Science and Technology Development Fund

GANTT Chart

Project Title: Use of Cold Formed Steel in Residential Housing
Project ID: 3751
Principle Investigator: Dr Metwally Abu-Hamd (Cairo University, Egypt) & Dr Benjamin Schafer (Johns Hopkins University, USA)

Start Date: 16/10/2011
Activity primarily in the U.S. (JHU)
Expected Duration: Two Years
Activity primarily in Egypt (Cairo)

<table>
<thead>
<tr>
<th>Tasks/Activities</th>
<th>Start</th>
<th>End</th>
<th>Duration (Days)</th>
<th>% Completed</th>
<th>Days Complete</th>
<th>Remaining Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Research Activity 1: Development of a novel non-proprietary cold-formed steel framing system</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 1-1 Develop Library of Optimal Shapes</td>
<td>M1</td>
<td>M12</td>
<td>225+135</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.2 1-2 Develop 'dual' system for walls and floors</td>
<td>M4</td>
<td>M24</td>
<td>360+270</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.3 1-3 Develop home archetype</td>
<td>M4</td>
<td>M6</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.4 1-4 Develop full framing solution for archetype home</td>
<td>M10</td>
<td>M18</td>
<td>270</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5 1-5 Demonstrate flexibility of 'dual' framing system</td>
<td>M19</td>
<td>M23</td>
<td>135</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.6 1-6 Price estimates for building archetypes study</td>
<td>M10</td>
<td>M15</td>
<td>180</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Research Activity 2: Building Archetypes Study</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 2-1 Selection of rural and urban locations in U.S. and Egypt</td>
<td>M1</td>
<td>M3</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.2 2-2 traditional framing (U.S. timber, Egypt concrete)</td>
<td>M4</td>
<td>M6</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.3 2-3 conventional cold-formed steel framing</td>
<td>M7</td>
<td>M12</td>
<td>180</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.4 2-4 novel 'dual' system cold-formed steel framing</td>
<td>M13</td>
<td>M18</td>
<td>180</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5 2-5 Environmental Impact and Sustainability assessment</td>
<td>M4</td>
<td>M21</td>
<td>360</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.6 2-6 Sensitivity Analysis</td>
<td>M13</td>
<td>M24</td>
<td>360</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Final Report</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: M1 to M24 represent months from January to December.
3.7 The Logical Framework Matrix (LFM):

3.7.1 At the project objective level, a new non-proprietary steel framed systems using novel-optimized cross-section shapes and new dual system for load bearing was developed.

3.7.2 At the research activities level, the following LFM activities have been performed:
   4.5 Assess sustainability and environmental impact of developed designs.
   4.6 Perform detailed comparisons among developed building designs.
   4.7 Analyze results to arrive at appropriate recommendations for different design situations

3.7.3 At the performance indicators levels, the following have been achieved:
   1- Sustainability of structural systems and construction methods to be executed at the specified urban/rural location.
   2- At the implementation stage, training courses and workshops shall be arranged to familiarize practicing engineers and building contractors with the developed systems.

3.7.4 At the output (results) level, a comparative study between reinforced concrete houses and newly developed cold formed steel houses in Egypt was executed.

A revised version of the LFM is presented next.
(Affected Items on this period are shown in Bold)

Project Title: Use of Cold Formed Steel in Residential Housing

Project ID: 3751

Principle Investigator: Prof Dr Metwally Abu-Hamd (Egypt) & Prof Dr Benjamin Schafer (USA)

<table>
<thead>
<tr>
<th>Activity description</th>
<th>Performance Indicators</th>
<th>Means of Verification</th>
<th>Assumptions</th>
</tr>
</thead>
</table>
| 1- Goal (Overall Objective) Increasing residential housing building capacity by using cold formed steel framing. | Increase of the share of steel framed buildings in newly built homes up to 30% of the total building market. | 1- Analysis of relevant governmental and private sector statistics.  
2- Market survey of building contractors.  
3- Monitoring of building sector activities | 1- Continued market demand for more residential housing.  
2- Newly developed cold formed steel framing systems shall be more affordable to people and financially profitable to building contractors.  
1. People and building contractors have the ability to use the developed building designs once |
<table>
<thead>
<tr>
<th>Activity Description</th>
<th>Performance Indicators</th>
<th>Means of Verification</th>
<th>Assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2- Project Objectives</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1. Development of typical steel framing building systems using locally available cross-sections</td>
<td>1- Newly built houses implement developed systems. 2- Construction times are reduced considerably. 3- Building contractors strongly support the developed systems.</td>
<td>1- Monitoring of building sector activities. 2- Market survey of newly built houses. 3- Questionnaire in building fairs and workshops.</td>
<td>1- Locally available materials and construction methods produce affordable designs in terms of economic, environmental and sustainability aspects. 2- Society is made aware of the benefits of the developed systems through successful marketing.</td>
</tr>
<tr>
<td>2.2. Developing new non-proprietary steel framed systems using novel-optimized cross-section shapes and new dual system for load bearing and lateral resistance.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Activity Description</strong></td>
<td><strong>Performance Indicators</strong></td>
<td><strong>Means of Verification</strong></td>
<td><strong>Assumptions</strong></td>
</tr>
<tr>
<td>------------------------</td>
<td>---------------------------</td>
<td>--------------------------</td>
<td>------------------</td>
</tr>
</tbody>
</table>
| **3- Outputs (Results)** | 1- Newly built houses implement developed systems.  
2- Construction times are reduced considerably.  
3- Building contractors strongly support the developed systems. | 1- Monitoring of building sector activities.  
2- Market survey of newly built houses.  
3- Questionnaire in building fairs and workshops. | 1- Newly developed cold formed steel framing systems shall be more affordable to people and financially profitable to building contractors.  
2. Building of the demonstration model shall be financed totally by private sector building contractors (see annex 6). |
<p>| 3.1. Comparative study between reinforced concrete houses and developed typical cold formed steel houses in Egypt | | | |
| 3.2. Comparative study between wood houses and developed typical cold formed steel houses in USA | | | |
| <strong>3.3. Comparative study between reinforced concrete houses and newly developed cold formed steel houses in Egypt</strong> | | | |
| 3.4. Comparative study between wood houses and newly developed cold formed steel houses in USA. | | | |
| 3.5. Building a demonstration model of one of the developed designs | | | |</p>
<table>
<thead>
<tr>
<th><strong>Activity Description</strong></th>
<th><strong>Performance Indicators</strong></th>
<th><strong>Means of Verification</strong></th>
<th><strong>Assumptions</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4- Activities</strong></td>
<td><strong>I. Indicators:</strong></td>
<td>1- Survey of present residential building trends from building contractor data.</td>
<td>1- Availability of typical layout designs presently used in residential housing.</td>
</tr>
<tr>
<td></td>
<td>i. Satisfaction of housing needs</td>
<td>2- Survey of social housing needs in selected urban and rural locations.</td>
<td>3. Availability of data related to construction material resources and present construction methods.</td>
</tr>
<tr>
<td></td>
<td>ii. Compliance with local building codes and regulations.</td>
<td>3- Survey of available construction material resources</td>
<td>4. Existing facilities at Cairo University and Johns Hopkins University are sufficient to perform the needed design work.</td>
</tr>
<tr>
<td></td>
<td><strong>iii. Sustainability of structural systems and construction methods to be executed at the specified urban/rural location.</strong></td>
<td>4- Review of developed design against design codes.</td>
<td></td>
</tr>
</tbody>
</table>
3.9 Planning for the next reporting period:

The following research activities shall be continued:

1-1 Develop a library of optimal shapes (covered in this report and to be continued in final report)
1-2 Develop ‘dual’ system for walls and floors (covered in this report and to be continued in final report)

2.4 Design and cost analysis of dual system cold formed steel framing (covered in this report and to be continued in final report).
2.5 Environmental impact and sustainability assessment (covered in second report and continued in this report and in the final report)
2.6 Sensitivity analysis (covered in this report and to be continued in final report).
4. The PI evaluation of the progress of the project:

1- The work executed in the reporting period went exactly according to the planned activities.
2- All the scheduled tasks have been completed.
3- The obtained results are very encouraging.

5. Actual or Expected Problems Encountered and Resolutions

Description of problems encountered: No technical problems were encountered. However, financial problems exist due to unavailability of STDF funds. Up till now, the second installment that was due on October 2012 have not been paid. Project team had to finance all research activities for the six months period out from their personal resources.

Description of actions taken to resolve the problems: STDF should find a quick solution to this problem

Description of problems expected in the future: None at the moment

Description of actions proposed to resolve the problems: NA

6. Implementing Teams:

6.1 Egypt Team
1- Prof Dr Metwally Abu-Hamd
2- Prof Dr Mohammed Ragaee badr
3- Dr Maged Tawfick Hanna

6.2 U.S. Team
1- Prof Dr Ben Schaffer
2- Dr. Li Zhanjie
Appendix A:

Design of Novel System Archetypes

Appendix A1_63m2-4F -GRC-Rigid Frame-Sigma Section
Appendix A2_63m2-4F -GRC-Wall Panels-Sigma Section
Appendix A3_63m2-4F -GRC-Dual System-Lipped Channel
Appendix A4_63m2-4F -GRC-Dual System-Sigma Section
Appendix A5_80m2 -GRC-Rigid Frame – Sigma Section
Appendix A6_80m2 -GRC - Wall Panel – Sigma Section
Appendix A7_80m2 -GRC-Dual System - Lipped Channel
Appendix A8_80m2 -GRC - Dual System-Sigma Section
Appendix A1

Design Report for a Residential Building
Rigid Frame
Decking: GRC panels
63 m²- 4 flats/floor
Section: Sigma

1. Introduction

In this report the detailed design of 6 story residential building is presented. The building covers an area of 315 m² (including voids), each floor is divided into 4 flats each of which is 63 m². Fig.1 shows the typical architectural floor plan of the building. The primary vertical loads are carried by rigid frame, and the lateral loads are resisted by vertical bracing elements.

Fig.1 : Architectural typical floor plan
2. Design Criteria
- Egyptian Code of Practice for Calculation of Loads and Forces on Structural Works and Building Works, ECP 201-2011 edition was used for determining gravity and lateral loads.
- Egyptian Code of Practice for Steel Construction and Bridges (Allowable Stress Design), code No. ECP 205-2001, 2011 edition was used for member seizing
- Alexform section tables was used for member callout
- Allowable strength design (ASD) was used for members and connections design.
- Deflection of floor joists was limited to L/200 due to live load, where L is the joist span

3. Loads
3.1 DEAD LOADS, DL
- Self weight of steel per unit volume = 78.5 KN/m³ (7850 kg/m³)
- Weight GRC panels = 0.5 KN/m² (50 kg/m²)
- Weight of 1 cm screed = 0.25 KN/m² (25 kg/m²).
- Partition (GRC panels) weight = 0.5 KN/m² (50 kg/m²). Partition load was included to account for partitions that may be moved at various times during the structure’s life span.

3.2 LIVE LOAD, LL
- On floor areas = 2 KN/m² (200kg/m²)
- On stairs, corridors, kitchens and bathrooms = 3 KN/m² (300 kg/m²)

3.3 WIND LOAD, WL
Wind loads, P, are calculated from the following equation

\[ P = C_e k q \]

- \( C_e \) = Shape factor = 0.8 for inward side, = 0.5 for leeward side
- \( k \) = Height factor = 1 for height = 0 -10 m
  = 1.15 for height = 10-20 m
Basic wind pressure = 0.68 KN/m² corresponds to basic wind speed = 33 m/sec

3.4 SEISMIC LOAD, S
Total base force = \( F_b = S_d(T_I) \cdot \lambda \cdot W/g \)
\( S_d(T_I) = \text{Design response spectrum at fundamental period of vibration } T_I \)
\( = a_g \cdot \gamma_I \cdot S \cdot (2.5/R) \cdot (T_C/T_I) \cdot \eta \)
\( a_g = \text{design acceleration} = 0.15 \text{ g (Zone (3) acc. to Egypt zoning)} \)
\( S = \text{Soil class factor} = 1.5 \text{ for soil class } C \)
\( T_c = \text{constant response spectrum period} = 0.25 \text{ sec for soil class } C \)
\( T_I = \text{structure period} = C_t (H)^{3/4}, \ C_t = 0.085 \text{ for steel frames,} \)
\( = 0.075 \text{ for RC frames.} \)
\( H = \text{building height in meters} \)
\( W = \text{Total dead load plus 25 \% of live loads} \)
\( \lambda = 1 \text{ for } T_I > 2 \text{ } T_C, \text{ otherwise } = 0.85 \)
\( R = \text{Response modification factor} = 5 \text{ for Moment Resisting Frames} \)

Important Note: \( F_b \) represents the FACTORED load in LRFD and to be divided by 1.4 in ASD.

The calculated base shear factor = 0.037

4. Load Combinations
Load combinations considered are according to that defined in the ECP 201-2011 edition and ECP 205-2001, 2011 edition. They are as follows
- DL
- DL+LL
- DL+WL
- DL+LL+WL
- DL+S
- DL+LL+S

Note: The ECP 205-2001 allows increasing the allowable stresses by 20\% when WL or S is considered in the design

5. Software
The following software was used in the development of the calculations:
- SAP2000: Used in developing the 3D model analysis model
- Microsoft Excel: Used to develop spreadsheets for design of sections
6. Statical System and structural analysis
The statical system that carries the vertical loads (dead and live loads) consists of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to rigid frames. The frames are arranged along the vertical axis 1 through 13. Lateral loads in Y-direction are carried by the rigid frames, while lateral loads in X-direction are carried by group of vertical bracing systems. Vertical bracing systems along Y-direction are arranged on axis “1”, “7”, and “13”. Axis “1” and “7” have 2 bracing bays.

Fig. 2: Structural plan showing the arrangement of the joists, Dual System and the vertical bracing bays.

3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:

- Beams (joists) are hinged connected to the vertical columns.
- Vertical bracing members are pinned connected to the vertical columns.
- The floor slab moved horizontally in the principle directions as rigid diaphragm.
7. Member Seizing
The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

7.1 Joists
Joists are designed as simple beams with variable spans. The spacing between joists ranges from 65 cm to 75 cm. The later spacing lays between axe “E” and axe “G”. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected for all joists, however, 200SG65-200 section was selected for joists that have spans of 5.25m. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking.

7.2 Columns
The rigid frame columns are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 200 SG65-170 back to back, 200SG65-225 back to back, and 300SG80-225 back to back sections were selected.

7.3 Beams
The rigid frame columns are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 200SG65-200 back to back and 300SG80-200 back to back sections were selected.

7.4 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

<table>
<thead>
<tr>
<th>Applied Load</th>
<th>Base Reaction (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind in X-direction</td>
<td>+/- 22.403</td>
</tr>
<tr>
<td>Wind in Y-direction</td>
<td>-32.44 / + 46.94</td>
</tr>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 10.821</td>
</tr>
</tbody>
</table>
Axial forces in the vertical members range from 9.8 ton to -15.05 ton. Also, axial forces in the diagonal members range from 3.5 ton to -5.9 ton. Based on this along with the limits that are stated in the design criteria square hollow section 140x140x4 section was selected for the vertical members, and 140x140x2 was selected for the diagonals.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.6 t.m, and maximum shear force of 2.4 ton. Based on this along with the limits that are stated in the design criteria hollow section 250x140x3 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 160C60-170 section was selected.

7.6 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are .42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

9. Material Quantities
From the above design the following quantities are calculated

- Own weight of steel elements = 33.5 x 1.1 = 36.85 ton
Fig. 3: Plan of one floor

Horizontal beams (joists) along X-axis, pin connected to the Dual System.
Dual Systems are arranged on axis 1 to 13
Fig. 4: Cross section along axe "B"

*Horizontal joists are pin connected to the vertical Dual System*
Fig. 5: Cross section along axe "7"

Rigid Frame
Fig. 6: Cross section along axis "I"

Vertical bracing resists lateral loads in the X-direction, and provides lateral stability.
Design of Sections

JOIST DESIGN

Section Name 170SG60-150

Section Dimensions

\[
\begin{align*}
H &= 170 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A &= 4.97 \text{ cm}^2 \\
I_x &= 206.1 \text{ cm}^4 \\
S_x &= 24.4 \text{ cm}^3 \\
S_{xy} &= 24.4 \text{ cm}^3 \\
\text{Weight} &= 3.9 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
M_x &= 0.48 \text{ t.m.} \\
Q &= 0.49 \text{ ton}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_b &= 1.9672 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.188 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Maximum deflection due to LL = 8.3 mm

L/200 = 19 mm
JOIST DESIGN

Section Name 200SG60-200

Section Dimensions

- $H = 200$ mm
- $B = 60$ mm
- $D = 20$ mm
- $t = 2$ mm
- $r = 4$ mm

Steel Properties

Steel Type

- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A = 7.134$ cm$^2$
- $I_x = 421.86$ cm$^4$
- $S_x = 42.18$ cm$^3$
- $S_{xx} = 42.18$ cm$^3$
- Weight $= 5.6$ kg/m

Applied Straining Actions

- $M_x = 0.8$ t.m.
- $Q = 0.61$ ton
- Span $= 5.25$ m

Check of Stresses

- $f_b = 1.8966$ t/cm$^2 < F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0.137$ t/cm$^2 < \tau_a = 0.73$ t/cm$^2$ Safe

Maximum deflection due to LL $= 15.2$ mm

$L/200 = 26$ mm
Beam 1

Section Name 2 x (300SG80-200)

Section Dimensions

- $H = 300$ mm
- $B = 80$ mm
- $D = 25$ mm
- $t = 2$ mm
- $r = 4$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_e = 16.282$ cm$^2$
- $I_x = 2554.6$ cm$^4$
- $i_x = 11.339$ cm
- $S_x = 170.31$ cm$^3$
- $S_{ax} = 170.31$ cm$^3$
- $I_y = 233.3$ cm$^4$
- $i_y = 3.427$ cm
- Weight = 15.59 kg/m

Applied Straining Actions

- $N = 0$ ton
- $M_x = 2.86$ t.m.
- $Q = 0$ ton
- Height = 3 m

Check of Stresses

- $f_c = 0$ t/cm$^2$ < $F_c = 2.1$ t/cm$^2$ Safe
- $f_y = 1.67$ t/cm$^2$ < $F_y = 2.1$ t/cm$^2$ Safe
- $\tau = 0$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$ Safe

Interaction Check

$$\frac{f_{cw}}{F_c} + \frac{f_{bcy}}{F_{bcy}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.75$$
Beam 2

Section Name 2 (200SG65-200)

Section Dimensions
- \( H = 200 \) mm
- \( B = 65 \) mm
- \( D = 20 \) mm
- \( t = 2 \) mm
- \( r = 4 \) mm

Steel Properties
- Steel Type 52
- \( F_{y} = 3.6 \) t/cm²
- \( F_{u} = 5.2 \) t/cm²

Section Properties
- \( A_{e} = 12.97 \) cm²
- \( I_{x} = 843.72 \) cm⁴
- \( i_{x} = 7.69 \) cm
- \( S_{x} = 84.37 \) cm³
- \( S_{x} = 84.37 \) cm³
- \( I_{y} = 120.62 \) cm⁴
- \( i_{y} = 2.9 \) cm
- Weight = 11.2 kg/m

Applied Straining Actions
- \( N = 0 \) ton
- \( M_{x} = 1 \) t.m.
- \( Q = 0 \) ton
- height = 3 m

Check of Stresses
- \( f_{c} = 0 \) t/cm²
- \( f_{b} = 1.1853 \) t/cm²
- \( \tau = 0 \) t/cm²
- \( F_{c} = 2.1 \) t/cm²
- \( F_{b} = 2.1 \) t/cm²
- \( \tau_{a} = 0.73 \) t/cm²

Safe

Interaction Check
\[
\frac{f_{c} A_{1}}{F_{c} F_{bcx}} + \frac{f_{b} A_{1}}{F_{bcx} F_{bce}} + \frac{f_{bcy} A_{2}}{F_{bcy} F_{bcy}} = 0.67
\]
Column 1

Section Name 2 x (200SG65-170)

Section Dimensions

- H = 200 mm
- B = 65 mm
- D = 20 mm
- t = 2 mm
- r = 4 mm

Steel Properties

- Steel Type 52
- F_y = 3.6 t/cm²
- F_u = 5.2 t/cm²

Section Properties

- A_e = 10.34 cm²
- I_x = 726.43 cm⁴
- i_x = 7.71 cm
- S_y = 72.64 cm³
- S_{ax} = 72.64 cm³
- I_y = 104.96 cm⁴
- i_y = 2.93 cm

Weight = 9.57 kg/m

Applied Straining Actions

- N = 6.18 ton
- M_x = 1 t.m. height = 3 m
- Q = 0 ton

Check of Stresses

\[
\frac{f_c}{F_c} < F_c = 2.1 \quad \text{t/cm}^2 \quad \text{Safe}
\]

\[
\frac{f_b}{F_b} < F_b = 2.1 \quad \text{t/cm}^2 \quad \text{Safe}
\]

\[
\tau = 0 \quad \text{t/cm}^2 \quad \tau_b = 0.73 \quad \text{t/cm}^2 \quad \text{Safe}
\]

Interaction Check

\[
\frac{f_{ex}}{F_{ex}} + \frac{f_{ex}}{F_{ex}} A_x + \frac{f_{ex}}{F_{ex}} A_z = 0.94
\]
Section 2

Section Name 2 x (300SG80-225)

Section Dimensions

- \( H = 300 \text{ mm} \)
- \( B = 80 \text{ mm} \)
- \( D = 25 \text{ mm} \)
- \( t = 2.25 \text{ mm} \)
- \( r = 5.3 \text{ mm} \)

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

- \( A_e = 19.33 \text{ cm}^2 \)
- \( I_x = 2850.9 \text{ cm}^4 \)
- \( i_x = 11.31 \text{ cm} \)
- \( S_x = 190.06 \text{ cm}^3 \)
- \( S_{xe} = 190.06 \text{ cm}^3 \)
- \( I_y = 258.3 \text{ cm}^4 \)
- \( i_y = 3.4 \text{ cm} \)
- Weight = 17.48 \text{ kg/m} \)

Applied Straining Actions

- \( N = 20 \text{ ton} \)
- \( M_x = 20 \text{ t.m.} \)
- \( Q = 0 \text{ ton} \)
- Height = 3 \text{ m} \)

Check of Stresses

- \( f_c = 1.03 \text{ t/cm}^2 \) < \( F_c = 2.1 \text{ t/cm}^2 \) Safe
- \( f_b = 1.05 \text{ t/cm}^2 \) < \( F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0 \text{ t/cm}^2 \) < \( \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bca}}{F_{bca}} A_1 + \frac{f_{bey}}{F_{bey}} A_2 = 0.99
\]
Column 3

Section Name 2 x (200SG65-225)

Section Dimensions

- $H = 200$ mm
- $B = 65$ mm
- $D = 20$ mm
- $t = 2.25$ mm
- $r = 4.5$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_e = 15.04$ cm$^2$
- $I_x = 938.97$ cm$^4$
- $i_x = 7.66$ cm
- $S_x = 93.89$ cm$^3$
- $I_y = 133.07$ cm$^4$
- $i_y = 2.88$ cm
- Weight = 12.54 kg/m

Applied Straining Actions

- $N = 5.8$ ton
- $M_x = 1.6$ t.m. height = 3 m
- $Q = 0$ ton

Check of Stresses

- $f_c = 0.38$ t/cm$^2$ < $F_c = 2.1$ t/cm$^2$ Safe
- $f_b = 1.7041$ t/cm$^2$ < $F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$ Safe

Interaction Check

$$\frac{f_{cx}}{F_c} + \frac{f_{bxy}}{F_{bxy}} A_x + \frac{f_{bxy}}{F_{bxy}} A_y = 0.99$$
Column 4

Section Name 2 x (200SG65-170)

Section Dimensions

- \( H = 200 \text{ mm} \)
- \( B = 65 \text{ mm} \)
- \( D = 20 \text{ mm} \)
- \( t = 1.7 \text{ mm} \)
- \( r = 3.4 \text{ mm} \)

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

- \( A_e = 10.34 \text{ cm}^2 \)
- \( I_x = 726.43 \text{ cm}^4 \)
- \( i_x = 7.71 \text{ cm} \)
- \( S_x = 72.64 \text{ cm}^3 \)
- \( S_{ex} = 72.64 \text{ cm}^3 \)
- \( I_y = 104.96 \text{ cm}^4 \)
- \( i_y = 2.93 \text{ cm} \)

Weight \( = 9.57 \text{ kg/m} \)

Applied Straining Actions

- \( N = 3.71 \text{ ton} \)
- \( M_x = 1 \text{ t.m.} \)
- \( Q = 0 \text{ ton} \)

Check of Stresses

- \( f_c = 0.358 \text{ t/cm}^2 \) < \( F_c = 2.1 \text{ t/cm}^2 \) Safe
- \( f_b = 1.3767 \text{ t/cm}^2 \) < \( F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0 \text{ t/cm}^2 \) < \( \tau_s = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check

\[
\frac{f_{cx}}{F_c} + \frac{f_{bex} A_1}{F_{bex}} + \frac{f_{bcy} A_2}{F_{bcy}} = 0.82
\]
Vertical and Diagonal Bracing elements

Section Name 140x140x4

Section Dimensions

- $H = 140$ mm
- $B = 140$ mm
- $t = 4$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_e = 21.76$ cm$^2$
- $I_x = 671.37$ cm$^4$
- $i_x = 5.55$ cm
- $S_x = 95.91$ cm$^3$
- $I_y = 671.37$ cm$^4$
- $i_y = 5.55$ cm
- Weight = 17.08 kg/m

Applied Straining Actions

- $N = -25$ ton
- $M_x = 0.14$ t.m., height = 3 m
- $Q = 0.18$ ton
- $k_x = 1$, $k_y = 1$
- $L_x = 300$ cm, $L_y = 300$ cm
- $k_x L_x / i_x = 54.054$, $k_y L_y / i_y = 54.054$

Check of Stresses

- $f_c = 1.14$ t/cm$^2$ < $F_c = 1.7$ t/cm$^2$, Safe
- $f_b = 0.145$ t/cm$^2$ < $F_b = 2.1$ t/cm$^2$, Safe
- $\tau = 0.017$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$, Safe

Interaction Check

\[ \frac{f_{ca}}{F_c} + \frac{f_{bex}}{F_{bex}} A_1 + \frac{f_{bey}}{F_{bey}} A_2 = 0.74 \]
Digonal Bracing elements

Section Name 140x140x2

Section Dimensions
- \( H = 140 \text{ mm} \)
- \( B = 140 \text{ mm} \)
- \( t = 2 \text{ mm} \)

Steel Properties
- Steel Type 52
- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties
- \( A_e = 11.04 \text{ cm}^2 \)
- \( I_x = 350.48 \text{ cm}^4 \)
- \( i_x = 5.63 \text{ cm} \)
- \( S_x = 50.069 \text{ cm}^3 \)
- \( I_y = 350.48 \text{ cm}^4 \)
- \( i_y = 5.63 \text{ cm} \)
- Weight = 17.08 kg/m

Applied Straining Actions
- \( N = -10 \text{ ton} \)
- \( M_x = 0 \text{ t.m.} \)
- \( Q = 0 \text{ ton} \)
- \( k_x = 0.5 \)
- \( L_x = 490 \text{ cm} \)
- \( k_y = 0.8 \)
- \( L_y = 490 \text{ cm} \)
- \( k_x L_x / i_x = 43.51 \)
- \( k_y L_y / i_y = 69.63 \)

Check of Stresses
- \( f_{C} = 0.9 \text{ t/cm}^2 < F_C = 1.44 \text{ t/cm}^2 \) Safe
- \( f_{b} = 0 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check
\[
\frac{f_{c,a}}{F_c} + \frac{f_{b,ex}}{F_{b,ex}} A_1 + \frac{f_{b,ex}}{F_{b,ex}} A_2 = 0.63
\]
Horizontal bracing member (axe-7)

Section Name 250x140x3

Section Dimensions
\[ H = 250 \text{ mm} \]
\[ B = 140 \text{ mm} \]
\[ t = 3 \text{ mm} \]

Steel Properties
Steel Type 52
\[ F_y = 3.6 \text{ t/cm}^2 \]
\[ F_u = 5.2 \text{ t/cm}^2 \]

Section Properties
\[ A_e = 23.04 \text{ cm}^2 \]
\[ I_x = 2007.6 \text{ cm}^4 \]
\[ i_x = 9.33 \text{ cm} \]
\[ S_x = 160.61 \text{ cm}^3 \]
\[ I_y = 824.25 \text{ cm}^4 \]
\[ i_y = 5.98 \text{ cm} \]
Weight = 18.08 kg/m

Applied Straining Actions
\[ N = 0 \text{ ton} \]
\[ M_x = 2.6 \text{ t.m.} \]
\[ Q = 2.4 \text{ ton} \]

Span = 5 m

Check of Stresses
\[ f_c = 0 \text{ t/cm}^2 < F_c = \text{ t/cm}^2 \quad \text{Safe} \]
\[ f_b = 1.61 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \]
\[ \tau = 0.172 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe} \]

Maximum deflection due to LL = 6 mm
\[ L/200 = 25 \text{ mm} \]
Horizontal bracing member (axe-1 & 13)

Section Name 200x140x3

Section Dimensions

- \( H = 200 \) mm
- \( B = 140 \) mm
- \( t = 3 \) mm

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \) t/cm²
- \( F_u = 5.2 \) t/cm²

Section Properties

- \( A_e = 20.04 \) cm²
- \( I_x = 1180.1 \) cm⁴
- \( i_x = 7.67 \) cm
- \( S_x = 118.01 \) cm³
- \( l_y = 683.47 \) cm³
- \( i_y = 5.84 \) cm
- Weight = 15.73 kg/m

Applied Straining Actions

- \( N = 0 \) ton
- \( M_x = 1.1 \) t.m.
- \( Q = 0.95 \) ton
- Span = 3.87 m

Check of Stresses

- \( f_c = 0 \) t/cm² < \( F_c = \) t/cm² Safe
- \( f_b = 0.932 \) t/cm² < \( F_b = 2.1 \) t/cm² Safe
- \( \tau = 0.086 \) t/cm² < \( \tau_a = 0.73 \) t/cm² Safe

Maximum deflection due to LL = 4.25 mm
- \( L/200 = 19 \) mm
Stair Beam

Section Name  200C75-200

Section Dimensions

- $H = 200 \text{ mm}$
- $B = 75 \text{ mm}$
- $D = 25 \text{ mm}$
- $t = 2 \text{ mm}$
- $r = 4 \text{ mm}$

Steel Properties

Steel Type
- $F_y = 3.6 \text{ t/cm}^2$
- $F_u = 5.2 \text{ t/cm}^2$

Section Properties

- $A = 7.66 \text{ cm}^2$
- $I_x = 471.64 \text{ cm}^4$
- $S_x = 47.16 \text{ cm}^3$
- $S_{xe} = 47.155 \text{ cm}^3$
- Weight = 6.019 kg/m

Applied Straining Actions

- $M_x = 0.42 \text{ t.m.}$
- $Q = 0.64 \text{ ton}$
- Span = 2.62 m

Check of Stresses

- $f_b = 0.89068 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2$ Safe
- $\tau = 0.16 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2$ Safe

Maximum deflection due to LL = 10 mm
L/200 = 26 mm
Appendix A2

Design Report for a Residential Building
Load Bearing wall panel system
Decking: GRC panels
63 m² - 4 flats/floor
Section: Sigma

1. Introduction
This report presents the structural analysis and design of a residential building where
the primary vertical loads are carried by wall bearing formed of cold-formed steel “C”
sections, and the primary lateral loads are resisted by vertical bracing elements. The
considered building consists of 6 floors, and covers an area of 315 m² (including
voids), each floor is divided into 4 flats each of which is 63 m². Fig.1 shows the
typical architectural floor plan of the building.

Fig.1 : Architectural typical floor plan
2. Statical System and structural analysis

The statical system that carries the vertical loads (dead and live loads) consists of concrete slab plus corrugated steel deck supported on series of horizontal beams (joists). The beams transmit their loads directly to vertical columns (studs). The spacing between the beams and the columns are nearly the same. So, the vertical loads transmitted from the slab to the beams, and then to the vertical columns. However, lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The load bearing walls are arranged along axis 1 to 13, while the vertical bracing systems along X-direction are arranged on axis “A”, “D”, “H”, and “I”. Each axe receives 2 bracing bays. Inaddition, the vertical bracing systems along Y-direction are arranged on axis “1”, “7”, and “13”. Axis “1” and “7” have 2 bracing bays.

![Structural plan showing the arrangement of the joists, load bearing walls and the vertical bracing bays.](image)

Fig. 2: Structural plan showing the arrangement of the joists, load bearing walls and the vertical bracing bays.

3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:
• Beams (joists) are hinged connected to the vertical columns.
• Places where opening like doors or windows are placed, lintel beam carry the reaction from the joist to two vertical columns adjacent to the opening.
• Vertical bracing members are pinned connected to the vertical columns.
• The floor slab moved horizontally in the principle directions as rigid diaphragm.

Note: SAP output is provided in appendix A

3. Member Seizing
The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

3.1 JOISTS
Joists are designed as simple beams with variable spans. The spacing between joists ranges from 65 cm to 75 cm. The later spacing lays between axe “E” and axe “G”. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected for all joists, however, 200SG60-200 section was selected for joists that have spans of 5.25m. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking.

3.2 Beams bridging openings (Header Beams)
These beams are designed as simple beams. The maximum span is 1.5 m The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 200T85-225 section was selected. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking.

3.3 External load bearing walls
The external vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. In addition to the vertical loads, lateral wind loads are considered. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected.
3.4 **Internal load bearing walls**

The internal vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected.

3.5 **Bracing elements**

Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

<table>
<thead>
<tr>
<th>Applied Load</th>
<th>Base Reaction (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind in X-direction</td>
<td>+/- 22.403</td>
</tr>
<tr>
<td>Wind in Y-direction</td>
<td>-32.44 / + 46.94</td>
</tr>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 11.132</td>
</tr>
<tr>
<td>Seismic in Y-direction</td>
<td>+/- 11.132</td>
</tr>
</tbody>
</table>

Axial forces in the vertical members range from 9.8 ton to -15.05 ton. Also, axial forces in the diagonal members range from 3.5 ton to -5.9 ton. Based on this along with the limits that are stated in the design criteria square hollow section 140x140x4 section was selected for the vertical members, and 140x140x2 was selected for the diagonals.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.6 t.m, and maximum shear force of 2.4 ton. Based on this along with the limits that are stated in the design criteria hollow section 250x140x3 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 160C60-170 section was selected.
3.6 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are .42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

4. Material Quantities
From the above design the following quantities are calculated
- Own weight of steel elements = 36.03 x 1.1 = 39.63 ton
Horizontal beams (joists) along X-axis, pin connected to the load bearing walls.
Load bearing walls are arranged on axis 1 to 13
Fig. 4: Cross section along axe "B"

Horizontal joists are pin connected to the vertical load bearing walls
Vertical bracing resists lateral loads in the X-direction, and provides lateral stability.
Fig. 6: Cross section along axe "13"
*Vertical bracing resists lateral loads in the Y-direction, and provides lateral stability*
JOIST DESIGN

Section Name 170SG60-150

Section Dimensions

\[ \begin{align*}
H &= 170 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*} \]

Steel Properties

Steel Type

\[ \begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*} \]

Section Properties

\[ \begin{align*}
A &= 4.97 \text{ cm}^2 \\
I_x &= 206.1 \text{ cm}^4 \\
S_x &= 24.4 \text{ cm}^3 \\
S_{xe} &= 24.4 \text{ cm}^3 \\
\text{Weight} &= 3.9 \text{ kg/m}
\end{align*} \]

Applied Straining Actions

\[ \begin{align*}
M_x &= 0.48 \text{ t.m.} \\
Q &= 0.49 \text{ ton} \\
\text{Span} &= 3.94 \text{ m}
\end{align*} \]

Check of Stresses

\[ \begin{align*}
f_b &= 1.9672 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \text{ Safe} \\
\tau &= 0.188 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \text{ Safe}
\end{align*} \]

Maximum deflection due to LL = 8.3 mm

L/200 = 19 mm
JOIST DESIGN

Section Name 200SG60-200

Section Dimensions

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>200 mm</td>
</tr>
<tr>
<td>B</td>
<td>60 mm</td>
</tr>
<tr>
<td>D</td>
<td>20 mm</td>
</tr>
<tr>
<td>t</td>
<td>2 mm</td>
</tr>
<tr>
<td>r</td>
<td>4 mm</td>
</tr>
</tbody>
</table>

Steel Properties

Steel Type

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fy</td>
<td>3.6 t/cm²</td>
</tr>
<tr>
<td>Fu</td>
<td>5.2 t/cm²</td>
</tr>
</tbody>
</table>

Section Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>7.134 cm²</td>
</tr>
<tr>
<td>Iₓ</td>
<td>421.86 cm⁴</td>
</tr>
<tr>
<td>Sₓ</td>
<td>42.18 cm³</td>
</tr>
<tr>
<td>Sₓₑ</td>
<td>42.18 cm³</td>
</tr>
<tr>
<td>Weight</td>
<td>5.6 kg/m</td>
</tr>
</tbody>
</table>

Applied Straining Actions

<table>
<thead>
<tr>
<th>Load</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mₓ</td>
<td>0.8 t.m.</td>
</tr>
<tr>
<td>Q</td>
<td>0.61 ton</td>
</tr>
<tr>
<td>Span</td>
<td>5.25 m</td>
</tr>
</tbody>
</table>

Check of Stresses

<table>
<thead>
<tr>
<th>Stress</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>fᵇ</td>
<td>1.8966 t/cm² &lt; 2.1 t/cm² Safe</td>
</tr>
<tr>
<td>τ</td>
<td>0.137 t/cm² &lt; 0.73 t/cm² Safe</td>
</tr>
</tbody>
</table>

Maximum deflection due to LL = 15.2 mm

L/200 = 26 mm
Header Beam

Section Name 160C60-150

Section Dimensions

- H = 160 mm
- B = 60 mm
- D = 20 mm
- t = 1.5 mm
- r = 3 mm

Steel Properties

- Steel Type
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties

- A = 4.613 cm²
- Ix = 182.04 cm⁴
- Sx = 22.754 cm³
- Sxe = 21.084 cm³
- Weight = 3.621 kg/m

Applied Straining Actions

- Mx = 0.24 t.m.
- Q = 0.33 ton
- Span = 1.5 m

Check of Stresses

- fb = 1.13 t/cm² < Fb = 2.1 t/cm² Safe
- τ = 0.146 t/cm² < τa = 0.73 t/cm² Safe

Maximum deflection due to LL = 2 mm
- L/200 = 7.5 mm
External Stud

Section Name 170SG60-150

Section Dimensions

\[
\begin{align*}
H &= 170 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_f &= 4.97 \text{ cm}^2 \\
A_e &= 4.32 \text{ cm}^2 \\
I_x &= 206.1 \text{ cm}^4 \\
i_x &= 6.43 \text{ cm} \\
S_x &= 24.4 \text{ cm}^3 \\
S_{xy} &= 24.4 \text{ cm}^3 \\
I_y &= 24.7 \text{ cm}^4 \\
i_y &= 2.21 \text{ cm} \\
\text{Weight} &= 3.9 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
N &= 4.5 \text{ ton} \\
M_x &= 4.5 \text{ t.cm.} \\
Q &= 0.67 \text{ ton}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_C &= 1.04 \text{ t/cm}^2 < F_C = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 0.18 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.26 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.58
\]
Internal Stud

Section Name 170SG60-150

Section Dimensions

- \( H = 170 \text{ mm} \)
- \( B = 60 \text{ mm} \)
- \( D = 20 \text{ mm} \)
- \( t = 1.5 \text{ mm} \)
- \( r = 3 \text{ mm} \)

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

- \( A_f = 4.97 \text{ cm}^2 \)
- \( A_e = 4.32 \text{ cm}^2 \)
- \( I_x = 206.1 \text{ cm}^4 \)
- \( i_x = 6.43 \text{ cm} \)
- \( S_x = 24.4 \text{ cm}^3 \)
- \( S_{xe} = 24.4 \text{ cm}^3 \)
- \( I_y = 24.27 \text{ cm}^4 \)
- \( i_y = 2.21 \text{ cm} \)
- Weight = 3.9 kg/m

Applied Straining Actions

- \( N = 6 \text{ ton} \)
- \( M_x = 0 \text{ t.cm.} \)
- \( Q = 0 \text{ ton} \)
- height = 3 m

Check of Stresses

- \( f_c = 1.38 \text{ t/cm}^2 \) < \( F_c = 2.1 \text{ t/cm}^2 \) Safe
- \( f_b = 0.18 \text{ t/cm}^2 \) < \( F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0.18 \text{ t/cm}^2 \) < \( \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{b,cx}}{F_{b,cx}} A_1 + \frac{f_{b,cx}}{F_{b,cy}} A_2 = 0.66
\]
Vertical and Diagonal Bracing Elements

Section Name 140x140x4

Section Dimensions

\[
\begin{align*}
H &= 140 \text{ mm} \\
B &= 140 \text{ mm} \\
t &= 4 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_e &= 21.76 \text{ cm}^2 \\
I_x &= 671.37 \text{ cm}^4 \\
i_x &= 5.55 \text{ cm} \\
S_x &= 95.91 \text{ cm}^3 \\
I_y &= 671.37 \text{ cm}^4 \\
i_y &= 5.55 \text{ cm}
\end{align*}
\]

Weight = 17.08 kg/m

Applied Straining Actions

\[
\begin{align*}
N &= -25 \text{ ton} \\
M_x &= 0.14 \text{ t.m.} \\
Q &= 0.18 \text{ ton} \\
k_x &= 1 \\
k_y &= 1 \\
L_x &= 300 \text{ cm} \\
L_y &= 300 \text{ cm} \\
k_xL_x/i_x &= 54.054 \\
k_yL_y/i_y &= 54.054
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_C &= 1.14 \text{ t/cm}^2 < F_C = 1.7 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 0.145 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.017 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}}A_1 + \frac{f_{bcy}}{F_{bcy}}A_2 = 0.74
\]
Digonal Bracing elements

Section Name 140x140x2

Section Dimensions

- $H = 140$ mm
- $B = 140$ mm
- $t = 2$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_e = 11.04$ cm$^2$
- $I_x = 350.48$ cm$^4$
- $i_x = 5.63$ cm
- $S_x = 50.069$ cm$^3$
- $I_y = 350.48$ cm$^4$
- $i_y = 5.63$ cm
- Weight = 17.08 kg/m

Applied Straining Actions

- $N = -10$ ton
- $M_x = 0$ t.m. height = 4.9 m
- $Q = 0$ ton
- $k_x = 0.5$
- $k_y = 0.8$
- $L_x = 490$ cm
- $L_y = 490$ cm
- $k_xL_x/i_x = 43.51$
- $k_yL_y/i_y = 69.63$

Check of Stresses

- $f_C = 0.9$ t/cm$^2$ < $F_C = 1.44$ t/cm$^2$ Safe
- $f_b = 0$ t/cm$^2$ < $F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$ Safe

Interaction Check

$$\frac{f_{ex}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.63$$
Horizontal bracing member (axe-7)

Section Name  250x140x3

Section Dimensions

\[ H = 250 \text{ mm} \]
\[ B = 140 \text{ mm} \]
\[ t = 3 \text{ mm} \]

Steel Properties

Steel Type  52
\[ F_y = 3.6 \text{ t/cm}^2 \]
\[ F_u = 5.2 \text{ t/cm}^2 \]

Section Properties

\[ A_e = 23.04 \text{ cm}^2 \]
\[ I_x = 2007.6 \text{ cm}^4 \]
\[ i_x = 9.33 \text{ cm} \]
\[ S_x = 160.61 \text{ cm}^3 \]
\[ I_y = 824.25 \text{ cm}^4 \]
\[ i_y = 5.98 \text{ cm} \]

Weight = 18.08 kg/m

Applied Straining Actions

\[ N = 0 \text{ ton} \]
\[ M_x = 2.6 \text{ t.m.} \]
\[ Q = 2.4 \text{ ton} \]

Span = 5 m

Check of Stresses

\[ f_C = 0 \text{ t/cm}^2 < F_C = \text{ t/cm}^2 \] Safe
\[ f_b = 1.61 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \] Safe
\[ \tau = 0.172 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \] Safe

Maximum deflection due to LL = 6 mm

\[ L/200 = 25 \text{ mm} \]
Horizontal bracing member (axe-1 & 13)

Section Name 200x140x3

Section Dimensions

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>200 mm</td>
</tr>
<tr>
<td>B</td>
<td>140 mm</td>
</tr>
<tr>
<td>t</td>
<td>3 mm</td>
</tr>
</tbody>
</table>

Steel Properties

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>Fy (t/cm²)</th>
<th>Fu (t/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>52</td>
<td>3.6</td>
<td>5.2</td>
</tr>
</tbody>
</table>

Section Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A_e</td>
<td>20.04 cm²</td>
</tr>
<tr>
<td>I_x</td>
<td>1180.1 cm⁴</td>
</tr>
<tr>
<td>i_x</td>
<td>7.67 cm</td>
</tr>
<tr>
<td>S_x</td>
<td>118.01 cm³</td>
</tr>
<tr>
<td>I_y</td>
<td>683.47 cm⁴</td>
</tr>
<tr>
<td>i_y</td>
<td>5.84 cm</td>
</tr>
</tbody>
</table>

Weight = 15.73 kg/m

Applied Straining Actions

<table>
<thead>
<tr>
<th>Action</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>0 ton</td>
</tr>
<tr>
<td>M_x</td>
<td>1.1 t.m.</td>
</tr>
<tr>
<td>Q</td>
<td>0.95 ton</td>
</tr>
</tbody>
</table>

Span = 3.87 m

Check of Stresses

<table>
<thead>
<tr>
<th>Stress</th>
<th>Value</th>
<th>Comparison</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_c</td>
<td>0 t/cm²</td>
<td>&lt; F_c = 0 t/cm²</td>
<td>Safe</td>
</tr>
<tr>
<td>f_b</td>
<td>0.932 t/cm²</td>
<td>&lt; F_b = 2.1 t/cm²</td>
<td>Safe</td>
</tr>
<tr>
<td>τ</td>
<td>0.086 t/cm²</td>
<td>&lt; τ_a = 0.73 t/cm²</td>
<td>Safe</td>
</tr>
</tbody>
</table>

Maximum deflection due to LL = 4.25 mm

L/200 = 19 mm
Stair Beam

Section Name 200C75-200

Section Dimensions
\[ H = 200 \text{ mm} \]
\[ B = 75 \text{ mm} \]
\[ D = 25 \text{ mm} \]
\[ t = 2 \text{ mm} \]
\[ r = 4 \text{ mm} \]

Steel Properties

Steel Type
\[ F_y = 3.6 \text{ t/cm}^2 \]
\[ F_u = 5.2 \text{ t/cm}^2 \]

Section Properties
\[ A = 7.66 \text{ cm}^2 \]
\[ I_x = 471.64 \text{ cm}^4 \]
\[ S_x = 47.16 \text{ cm}^3 \]
\[ S_{xe} = 47.155 \text{ cm}^3 \]
\[ W = 6.019 \text{ kg/m} \]

Applied Straining Actions
\[ M_x = 0.42 \text{ t.m.} \]
\[ Q = 0.64 \text{ ton} \]

Check of Stresses
\[ f_s = 0.89068 \text{ t/cm}^2 < F_s = 2.1 \text{ t/cm}^2 \text{ Safe} \]
\[ \tau = 0.16 \text{ t/cm}^2 < \tau_s = 0.73 \text{ t/cm}^2 \text{ Safe} \]

Maximum deflection due to LL = 10 mm
\[ L/200 = 26 \text{ mm} \]
Appendix A3

Design Report for a Residential Building
Dual System (Rigid Frame + Vertical Studs)
Decking: GRC panels
63 m²- 4 flats/floor
Section: Lipped Channel

1. Introduction

In this report the detailed design of 6 story residential building is presented. The building covers an area of 315 m² (including voids), each floor is divided into 4 flats each of which is 63 m². Fig.1 shows the typical architectural floor plan of the building. The primary vertical loads are carried by dual system, and the lateral loads are resisted by vertical bracing elements. Dual system composed of rigid frame and vertical studs, Fig. 2, the axial stiffness of the studs can interacts with the bending stiffness of the rigid frame in a manner that minimizing the total weight of steel used. Moreover, dual system provides flexibility in the size and location of any opening.
Fig. 1: Architectural typical floor plan
2. Statical System and structural analysis

The statical system that carries the vertical loads (dead and live loads) consists of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to the dual system. The dual systems are arranged along the vertical axis 1 through 13. Lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The load bearing walls are arranged along axis 1 to 13, while the vertical bracing systems along X-direction are arranged on axis “A”, “D”, “H”, and “I”. Each axe receives 2 bracing bays. In addition, the vertical bracing systems along Y-direction are arranged on axis “1”, “7”, and “13”. Axis “1” and “7” have 2 bracing bays.
Fig. 3: Structural plan showing the arrangement of the joists, Dual System and the vertical bracing bays.

3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:

- Beams (joists) are hinged connected to the vertical columns.
- Vertical bracing members are pinned connected to the vertical columns.
- The floor slab moved horizontally in the principle directions as rigid diaphragm.

Note: SAP output is provided in appendix A

3. Member Sizing
The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

3.1 JOISTS
Joists are designed as simple beams with variable spans. The spacing between joists ranges from 65 cm to 75 cm. The later spacing lays between axe “E” and axe “G”. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 160C60-170 section was selected for all joists, however,
160C60-300 section was selected for joists that have spans of 5.25m. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking.

3.2 Rigid Frame (Columns / Beams)
The rigid frame members are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 140C60-150 section was selected.

3.3 External load bearing walls
The external vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. In addition to the vertical loads, lateral wind loads are considered. Based on this along with the limits that are stated in the design criteria 140C60-150 section was selected.

3.4 Internal load bearing walls
The internal vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. Based on this along with the limits that are stated in the design criteria 140C60-200 section was selected.

3.5 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

<table>
<thead>
<tr>
<th>Applied Load</th>
<th>Base Reaction (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind in X-direction</td>
<td>+/- 22.403</td>
</tr>
<tr>
<td>Wind in Y-direction</td>
<td>-32.44 / + 46.94</td>
</tr>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 10.821</td>
</tr>
<tr>
<td>Seismic in Y-direction</td>
<td>+/- 10.821</td>
</tr>
</tbody>
</table>
Axial forces in the vertical members range from 9.8 ton to -15.05 ton. Also, axial forces in the diagonal members range from 3.5 ton to -5.9 ton. Based on this along with the limits that are stated in the design criteria square hollow section 140x140x4 section was selected for the vertical members, and 140x140x2 was selected for the diagonals.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.6 t.m, and maximum shear force of 2.4 ton. Based on this along with the limits that are stated in the design criteria hollow section 250x140x3 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 160C60-170 section was selected.

3.6 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are .42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

4. Material Quantities

From the above design the following quantities are calculated

- Own weight of steel elements = 32.87 x 1.1 = 36.15 ton
Fig. 3: Plan of one floor

Horizontal beams (joists) along X-axis, pin connected to the Dual System.

Dual Systems are arranged on axis 1 to 13
Fig. 4: Cross section along axe "B"

*Horizontal joists are pin connected to the vertical Dual System*
Fig. 5: Cross section along axe "10"
Dual System (Rigid Frame + Studs)
Fig. 6: Cross section along axe "I"

Vertical bracing resists lateral loads in the X-direction, and provides lateral stability
Fig. 7: Cross section along axe "13"

Vertical bracing resists lateral loads in the Y-direction, and provides lateral stability
Design of Sections

JOIST DESIGN

Section Name 160C60-170

Section Dimensions
- H = 160 mm
- B = 60 mm
- D = 20 mm
- t = 1.7 mm
- r = 3.4 mm

Steel Properties

Steel Type
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties
- A = 5.2 cm²
- Ix = 204.21 cm⁴
- Sx = 25.526 cm³
- Sxe = 24.087 cm³
- Weight = 4.082 kg/m

Applied Straining Actions
- Mx = 0.48 t.m.
- Q = 0.49 ton
- Span = 3.94 m

Check of Stresses
- f_y = 1.9928 t/cm² < F_y = 2.1 t/cm² Safe
- τ = 0.188 t/cm² < τ_a = 0.73 t/cm² Safe

Maximum deflection due to LL = 11 mm
L/200 = 19 mm
JOIST DESIGN

Section Name 160C60-300

Section Dimensions

- \( H = 160 \text{ mm} \)
- \( B = 60 \text{ mm} \)
- \( D = 20 \text{ mm} \)
- \( t = 3 \text{ mm} \)
- \( r = 6 \text{ mm} \)

Steel Properties

Steel Type

- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

- \( A = 8.853 \text{ cm}^2 \)
- \( I_x = 336.52 \text{ cm}^4 \)
- \( S_x = 42.065 \text{ cm}^3 \)
- \( S_{xe} = 41.244 \text{ cm}^3 \)
- \( \text{Weight} = 6.95 \text{ kg/m} \)

Applied Straining Actions

- \( M_x = 0.8 \text{ t.m.} \)
- \( Q = 0.61 \text{ ton} \)
- \( \text{Span} = 5.25 \text{ m} \)

Check of Stresses

- \( f_b = 1.9397 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \text{ Safe} \)
- \( \tau = 0.137 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \text{ Safe} \)

Maximum deflection due to LL = 19.6 mm
- \( L/200 = 26 \text{ mm} \)
External load bearing walls

Section Name: 100C50-150

Section Dimensions

- **H**: 100 mm
- **B**: 50 mm
- **D**: 17 mm
- **t**: 1.5 mm
- **r**: 3 mm

Steel Properties

- **Steel Type**: 52
- **Fy**: 3.6 t/cm²
- **Fu**: 5.2 t/cm²

Section Properties

- **Af**: 3.32 cm²
- **Ae**: 2.73 cm²
- **Ix**: 53.19 cm⁴
- **ix**: 4.001 cm
- **Sx**: 10.63 cm³
- **Sxy**: 10.63 cm³
- **Iy**: 12.07 cm⁴
- **iy**: 1.92 cm
- **Weight**: 2.609 kg/m

Applied Straining Actions

- **N**: 4.3 ton
- **Mx**: 0.045 t.m. (height = 3 m)
- **Q**: 0 ton

Check of Stresses

- **f_C**: 1.57 t/cm² < **F_C**: 2.1 t/cm² Safe
- **f_b**: 0.4233 t/cm² < **F_b**: 2.1 t/cm² Safe
- **τ**: 0 t/cm² < **τ_a**: 0.73 t/cm² Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.95
\]
Internal load bearing walls

Section Name 100C50-200

Section Dimensions

- $H = 100$ mm
- $B = 50$ mm
- $D = 17$ mm
- $t = 2$ mm
- $r = 4$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm²
- $F_u = 5.2$ t/cm²

Section Properties

- $A_f = 4.34$ cm²
- $A_e = 4.05$ cm²
- $I_x = 68.42$ cm⁴
- $I_y = 3.967$ cm
- $S_x = 13.685$ cm³
- $S_{xy} = 13.68$ cm³
- $I_y = 15.29$ cm⁴
- $i_y = 1.87$ cm
- Weight = 3.41 kg/m

Applied Straining Actions

- $N = 8.3$ ton
- $M_x = 0$ t.m.
- $Q = 0$ ton
- height = 3 m

Check of Stresses

- $f_C = 2.04$ t/cm² < $F_C = 2.1$ t/cm² Safe
- $f_b = 0$ t/cm² < $F_b = 2.1$ t/cm² Safe
- $\tau = 0$ t/cm² < $\tau_a = 0.73$ t/cm² Safe

Interaction Check

$$\frac{f_{cx}}{F_c} + \frac{f_{cy}}{F_{bcy}} A_1 + \frac{f_{bxy}}{F_{bcy}} A_2 = 0.97$$
Rigid Frame Columns

Section Name 140C60-150

Section Dimensions

\[
\begin{align*}
H &= 140 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_e &= 4.313 \text{ cm}^2 \\
A_f &= 2.98 \text{ cm}^2 \\
I_x &= 133.38 \text{ cm}^4 \\
I_y &= 22.396 \text{ cm}^4 \\
S_x &= 19.05 \text{ cm}^3 \\
S_{xy} &= 17.674 \text{ cm}^3 \\
S_{x0} &= 13.239 \text{ cm}^3 \\
l_x &= 5.56 \text{ cm} \\
l_y &= 2.279 \text{ cm}
\end{align*}
\]

Weight = 3.386 kg/m

Applied Straining Actions

\[
\begin{align*}
N &= 6.09 \text{ ton} \\
M &= 4.5 \text{ t.cm.} \\
Q &= 0.42 \text{ ton}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_c &= 2.04 \text{ t/cm}^2 < F_c = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 0.254 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.2 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{cy}}{F_{cy}} + \frac{f_{bcy}}{F_{bcy}} A_1 + \frac{f_{bcx}}{F_{bcx}} A_2 = 1
\]
Rigid Frame Beams

Section Name 140C60-150

Section Dimensions
- \( H = 140 \text{ mm} \)
- \( B = 60 \text{ mm} \)
- \( D = 20 \text{ mm} \)
- \( t = 1.5 \text{ mm} \)
- \( r = 3 \text{ mm} \)

Steel Properties
- Steel Type 52
  - \( F_y = 3.6 \text{ t/cm}^2 \)
  - \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties
- \( A_f = 4.313 \text{ cm}^2 \)
- \( A_e = 2.98 \text{ cm}^2 \)
- \( I_x = 133.38 \text{ cm}^4 \)
- \( I_y = 22.396 \text{ cm}^4 \)
- \( S_x = 19.05 \text{ cm}^3 \)
- \( S_y = 17.674 \text{ cm}^3 \)
- \( W = 3.386 \text{ kg/m} \)

Applied Straining Actions
- \( N = 0 \text{ ton} \)
- \( M_x = 31 \text{ t.cm.} \)
- \( Q = 0.61 \text{ ton} \)

Check of Stresses
- \( f_C = 0 \text{ t/cm}^2 \)
- \( f_B = 1.75 \text{ t/cm}^2 \)
- \( \tau = 0.29 \text{ t/cm}^2 \)

Interaction Check
- \( \frac{f_{cu}}{F_c} + \frac{f_{b,cx}}{F_{bcx}} A_1 + \frac{f_{b,cy}}{F_{bcy}} A_2 = 0.83 \)
Vertical and Diagonal Bracing elements

Section Name 140x140x4

Section Dimensions

\[
\begin{align*}
H & = 140 \text{ mm} \\
B & = 140 \text{ mm} \\
t & = 4 \text{ mm}
\end{align*}
\]

Steel Properties

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>52</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_y )</td>
<td>3.6 t/cm²</td>
</tr>
<tr>
<td>( F_u )</td>
<td>5.2 t/cm²</td>
</tr>
</tbody>
</table>

Section Properties

| \( A_e \) | 21.76 cm² |
| \( I_x \) | 671.37 cm⁴ |
| \( i_x \) | 5.55 cm |
| \( S_x \) | 95.91 cm³ |
| \( I_y \) | 671.37 cm⁴ |
| \( i_y \) | 5.55 cm |

Weight = 17.08 kg/m

Applied Straining Actions

\[
\begin{align*}
N & = -25 \text{ ton} \\
M_x & = 0.14 \text{ t.m.} \\
Q & = 0.18 \text{ ton} \\
k_x & = 1 \\
k_y & = 1 \\
L_x & = 300 \text{ cm} \\
L_y & = 300 \text{ cm} \\
k_x L_x i_x & = 54.054 \\
k_y L_y i_y & = 54.054
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_C & = 1.14 \text{ t/cm}^2 < F_C = 1.7 \text{ t/cm}^2 \quad \text{Safe} \\
f_b & = 0.145 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau & = 0.017 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{b,ex}}{F_{b,ex}} A_1 + \frac{f_{b,ex}}{F_{b,ex}} A_2 = 0.74
\]
Digonal Bracing elements

Section Name 140x140x2

Section Dimensions

- H = 140 mm
- B = 140 mm
- t = 2 mm

Steel Properties

Steel Type 52
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties

- A_e = 11.04 cm²
- I_x = 350.48 cm⁴
- i_x = 5.63 cm
- S_y = 50.069 cm³
- I_y = 350.48 cm⁴
- i_y = 5.63 cm
Weight = 17.08 kg/m

Applied Straining Actions

- N = -10 ton
- M_x = 0 t.m.
- Q = 0 ton
- k_x = 0.5
- L_x = 490 cm
- k_y = 0.8
- L_y = 490 cm
k_x L_x / i_x = 43.51
k_y L_y / i_y = 69.63

Check of Stresses

- f_c = 0.9 t/cm² < F_c = 1.44 t/cm² Safe
- f_y = 0 t/cm² < F_y = 2.1 t/cm² Safe
- τ = 0 t/cm² < τ_a = 0.73 t/cm² Safe

Interaction Check

\[ \frac{f_{ex}}{F_e} + \frac{f_{exx}}{F_{exx}} A_1 + \frac{f_{exy}}{F_{exy}} A_2 = 0.63 \]
Horizontal bracing member (axe-7)

Section Name 250x140x3

Section Dimensions

H = 250 mm  
B = 140 mm  
t = 3 mm

Steel Properties

Steel Type 52

Fy = 3.6 t/cm²  
Fu = 5.2 t/cm²

Section Properties

Ae = 23.04 cm²  
Iₓ = 2007.6 cm⁴  
iₓ = 9.33 cm  
Sₓ = 160.61 cm³  
lₓ = 824.25 cm³  
iᵧ = 5.98 cm  
Weight = 18.08 kg/m

Applied Straining Actions

N = 0 ton  
Mₓ = 2.6 t.m.  
Q = 2.4 ton  
Span = 5 m

Check of Stresses

fC = 0 t/cm² < FC = t/cm² Safe
fb = 1.61 t/cm² < Fb = 2.1 t/cm² Safe

Maximum deflection due to LL = 6 mm  
L/200 = 25 mm
Horizontal bracing member (axe-1 & 13)

Section Name 200x140x3

Section Dimensions
- H = 200 mm
- B = 140 mm
- t = 3 mm

Steel Properties
- Steel Type 52
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties
- \( A_e = 20.04 \text{ cm}^2 \)
- \( I_x = 1180.1 \text{ cm}^4 \)
- \( i_x = 7.67 \text{ cm} \)
- \( S_x = 118.01 \text{ cm}^3 \)
- \( I_y = 683.47 \text{ cm}^4 \)
- \( i_y = 5.84 \text{ cm} \)
- Weight = 15.73 kg/m

Applied Straining Actions
- N = 0 ton
- \( M_x = 1.1 \text{ t.m.} \)
- Q = 0.95 ton
- Span = 3.87 m

Check of Stresses
- \( f_C = 0 \text{ t/cm}^2 \) < \( F_C = \text{t/cm}^2 \) Safe
- \( f_b = 0.932 \text{ t/cm}^2 \) < \( F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0.086 \text{ t/cm}^2 \) < \( \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Maximum deflection due to LL = 4.25 mm
- L/200 = 19 mm
Stair Beam

Section Name 200C75-200

Section Dimensions
- H = 200 mm
- B = 75 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties

- Steel Type
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties
- A = 7.66 cm²
- Ix = 471.64 cm⁴
- Sx = 47.16 cm³
- Sxe = 47.155 cm³
- Weight = 6.019 kg/m

Applied Straining Actions
- Mx = 0.42 t.m.
- Q = 0.64 ton
- Span = 2.62 m

Check of Stresses
- fb = 0.89068 t/cm² < Fb = 2.1 t/cm² Safe
- τ = 0.16 t/cm² < τa = 0.73 t/cm² Safe

Maximum deflection due to LL = 10 mm
- L/200 = 26 mm
1. Introduction
In this report the detailed design of 6 story residential building is presented. The building covers an area of 315 m$^2$ (including voids), each floor is divided into 4 flats each of which is 63 m$^2$. Fig.1 shows the typical architectural floor plan of the building. The primary vertical loads are carried by dual system, and the lateral loads are resisted by vertical bracing elements. Dual system composed of rigid frame and vertical studs, Fig. 2, the axial stiffness of the studs can interacts with the bending stiffness of the rigid frame in a manner that minimizing the total weight of steel used. Moreover, dual system provides flexibility in the size and location of any opening.

Fig.1 : Architectural typical floor plan
2. **Statitical System and structural analysis**

The statical system that carries the vertical loads (dead and live loads) consists of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to the dual system. The dual systems are arranged along the vertical axis 1 through 13. Lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The load bearing walls are arranged along axis 1 to 13, while the vertical bracing systems along X-direction are arranged on axis “A”, “D”, “H”, and “I”. Each axe receives 2 bracing bays. In addition, the vertical bracing systems along Y-direction are arranged on axis “1”, “7”, and “13”. Axis “1” and “7” have 2 bracing bays.
3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:

- Beams (joists) are hinged connected to the vertical columns.
- Vertical bracing members are pinned connected to the vertical columns.
- The floor slab moved horizontally in the principle directions as rigid diaphragm.

*Note: SAP output is provided in appendix A*

### 3. Member Seizing

The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

#### 3.1 JOISTS

Joists are designed as simple beams with variable spans. The spacing between joists ranges from 65 cm to 75 cm. The later spacing lays between axe “E” and axe “G”. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected for all joists, however, 200SG65-200 section was selected for joists that have spans of 5.25m. The
compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking.

3.2 Rigid Frame (Columns / Beams)
The rigid frame members are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected.

3.3 External load bearing walls
The external vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. In addition to the vertical loads, lateral wind loads are considered. Based on this along with the limits that are stated in the design criteria 140C60-150 section was selected.

3.4 Internal load bearing walls
The internal vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. Based on this along with the limits that are stated in the design criteria 140C60-200 section was selected.

3.5 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

<table>
<thead>
<tr>
<th>Applied Load</th>
<th>Base Reaction (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind in X-direction</td>
<td>+/- 22.403</td>
</tr>
<tr>
<td>Wind in Y-direction</td>
<td>-32.44 / + 46.94</td>
</tr>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 10.821</td>
</tr>
<tr>
<td>Seismic in Y-direction</td>
<td>+/- 10.821</td>
</tr>
</tbody>
</table>
Axial forces in the vertical members range from 9.8 ton to -15.05 ton. Also, axial forces in the diagonal members range from 3.5 ton to -5.9 ton. Based on this along with the limits that are stated in the design criteria square hollow section 140x140x4 section was selected for the vertical members, and 140x140x2 was selected for the diagonals. Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.6 t.m, and maximum shear force of 2.4 ton. Based on this along with the limits that are stated in the design criteria hollow section 250x140x3 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 160C60-170 section was selected.

3.6 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are 0.42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

4. Material Quantities
From the above design the following quantities are calculated

- Own weight of steel elements = 33.2 x 1.1 = 36.52 ton
Fig. 3: Plan of one floor

Horizontal beams (joists) along X-axis, pin connected to the Dual System.

Dual Systems are arranged on axis 1 to 13
Fig. 4: Cross section along axe "B"

*Horizontal joists are pin connected to the vertical Dual System*
Fig. 5: Cross section along axe "10"
Dual System (Rigid Frame + Studs)
Fig. 6: Cross section along axe "I"

Vertical bracing resists lateral loads in the X-direction, and provides lateral stability
Vertical bracing resists lateral loads in the Y-direction, and provides lateral stability.
Design of Sections

JOIST DESIGN

Section Name 170SG60-150

Section Dimensions

- $H = 170$ mm
- $B = 60$ mm
- $D = 20$ mm
- $t = 1.5$ mm
- $r = 3$ mm

Steel Properties

Steel Type

- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A = 4.97$ cm$^2$
- $I_x = 206.1$ cm$^4$
- $S_x = 24.4$ cm$^3$
- $S_{xe} = 24.4$ cm$^3$
- Weight = 3.9 kg/m

Applied Straining Actions

- $M_x = 0.48$ t.m.
- $Q = 0.49$ ton
- Span = 3.94 m

Check of Stresses

- $f_b = 1.9672$ t/cm$^2$ $< F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0.188$ t/cm$^2$ $< \tau_a = 0.73$ t/cm$^2$ Safe

Maximum deflection due to LL = 8.3 mm
- $L/200 = 19$ mm
JOIST DESIGN

Section Name 200SG60-200

Section Dimensions

- H = 200 mm
- B = 60 mm
- D = 20 mm
- t = 2 mm
- r = 4 mm

Steel Properties

- Steel Type
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties

- A = 7.134 cm²
- Iₓ = 421.86 cm⁴
- Sₓ = 42.18 cm³
- Sₓe = 42.18 cm³
- Weight = 5.6 kg/m

Applied Straining Actions

- Mₓ = 0.8 t.m.
- Q = 0.61 ton
- Span = 5.25 m

Check of Stresses

- fₓ = 1.8966 t/cm² < Fₓ = 2.1 t/cm² Safe
- τ = 0.137 t/cm² < τa = 0.73 t/cm² Safe

Maximum deflection due to LL = 15.2 mm
- L/200 = 26 mm
External Stud

Section Name  170SG60-150

Section Dimensions

- $H = 170$ mm
- $B = 60$ mm
- $D = 20$ mm
- $t = 1.5$ mm
- $r = 3$ mm

Steel Properties

- Steel Type 52
  - $F_y = 3.6$ t/cm²
  - $F_u = 5.2$ t/cm²

Section Properties

- $A_f = 4.97$ cm²
- $A_e = 4.32$ cm²
- $I_x = 206.1$ cm⁴
- $I_y = 24.4$ cm³
- $S_x = 24.4$ cm³
- $S_{xy} = 24.4$ cm³
- $I_y = 24.27$ cm⁴
- $i_y = 2.21$ cm
- Weight = 3.9 kg/m

Applied Straining Actions

- $N = 4.5$ ton
- $M_x = 4.5$ t.cm.
- $Q = 0.2$ ton

height = 3 m

Check of Stresses

- $f_C = 1.04$ t/cm² $< F_C = 2.1$ t/cm² Safe
- $f_b = 0.18$ t/cm² $< F_b = 2.1$ t/cm² Safe
- $\tau = 0.18$ t/cm² $< \tau_a = 0.73$ t/cm² Safe

Interaction Check

$$\frac{f_{ec}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.58$$
Internal Stud

Section Name 170SG60-150

Section Dimensions

\[ \begin{align*}
H &= 170 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*} \]

Steel Properties

Steel Type 52

\[ \begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*} \]

Section Properties

\[ \begin{align*}
A_f &= 4.97 \text{ cm}^2 \\
A_e &= 4.32 \text{ cm}^2 \\
I_x &= 206.1 \text{ cm}^4 \\
i_x &= 6.43 \text{ cm} \\
S_x &= 24.4 \text{ cm}^3 \\
S_{xe} &= 24.4 \text{ cm}^3 \\
I_y &= 24.27 \text{ cm}^4 \\
i_y &= 2.21 \text{ cm} \\
\text{Weight} &= 3.9 \text{ kg/m}
\end{align*} \]

Applied Straining Actions

\[ \begin{align*}
N &= 9.5 \text{ ton} \\
M_x &= 0 \text{ t.cm.} \\
Q &= 0 \text{ ton} \\
\text{height} &= 3 \text{ m}
\end{align*} \]

Check of Stresses

\[ \begin{align*}
f_c &= 2.19 \text{ t/cm}^2 < F_c = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 0 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*} \]

Interaction Check

\[ \frac{f_{cu}}{F_c} + \frac{f_{b,ex}}{F_{b,ex}} A_1 + \frac{f_{b,ey}}{F_{b,ey}} A_2 = 1 \]
Rigid Frame Columns

Section Name 170SG60-150

Section Dimensions

- $H = 170$ mm
- $B = 60$ mm
- $D = 20$ mm
- $t = 1.5$ mm
- $r = 3$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_f = 4.97$ cm$^2$
- $A_e = 4.32$ cm$^2$
- $I_x = 206.1$ cm$^4$
- $i_x = 6.43$ cm
- $S_x = 24.4$ cm$^3$
- $S_{xe} = 24.4$ cm$^3$
- $I_y = 24.27$ cm$^4$
- $i_y = 2.21$ cm
- Weight = 3.9 kg/m

Applied Straining Actions

- $N = 6.5$ ton
- $M_x = 3.9$ t.cm.
- $Q = 0.35$ ton
- height = 3 m

Check of Stresses

- $f_c = 1.5$ t/cm$^2$ < $F_c = 2.1$ t/cm$^2$ Safe
- $f_b = 0.15$ t/cm$^2$ < $F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0.18$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$ Safe

Interaction Check

$$\frac{f_{ca}}{F_c} + \frac{f_{b,cx}}{F_{b,ex}} A_i + \frac{f_{b,cy}}{F_{b,ey}} A_2 = 0.78$$
Rigid Frame Beam

Section Name 170SG60-150

Section Dimensions

- \( H = 170 \text{ mm} \)
- \( B = 60 \text{ mm} \)
- \( D = 20 \text{ mm} \)
- \( t = 1.5 \text{ mm} \)
- \( r = 3 \text{ mm} \)

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

- \( A_f = 4.97 \text{ cm}^2 \)
- \( A_e = 4.32 \text{ cm}^2 \)
- \( I_x = 206.1 \text{ cm}^4 \)
- \( i_x = 6.43 \text{ cm} \)
- \( S_x = 24.4 \text{ cm}^3 \)
- \( S_{xe} = 24.4 \text{ cm}^3 \)
- \( I_y = 24.27 \text{ cm}^4 \)
- \( i_y = 2.21 \text{ cm} \)
- Weight = 3.9 kg/m

Applied Straining Actions

- \( N = 0 \text{ ton} \)
- \( M_x = 30 \text{ t.cm.} \)
- \( Q = 0.91 \text{ ton} \)
- height = 3 m

Check of Stresses

- \( f_c = 0 \text{ t/cm}^2 < F_C = 2.1 \text{ t/cm}^2 \) Safe
- \( f_b = 1.22 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0.33 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{b,cx}}{F_{b,cx}} A_1 + \frac{f_{b,cy}}{F_{b,cy}} A_2 = 0.58
\]
Vertical and Diagonal Bracing elements

Section Name 140x140x4

Section Dimensions
- H = 140 mm
- B = 140 mm
- t = 4 mm

Steel Properties
- Steel Type 52
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties
- \( A_e = 21.76 \) cm²
- \( I_x = 671.37 \) cm⁴
- \( i_x = 5.55 \) cm
- \( S_x = 95.91 \) cm³
- \( I_y = 671.37 \) cm⁴
- \( i_y = 5.55 \) cm
- Weight = 17.08 kg/m

Applied Straining Actions
- N = -25 ton
- \( M_x = 0.14 \) t.m. height = 3 m
- \( Q = 0.18 \) ton
- \( k_x = 1 \)
- \( k_y = 1 \)
- \( L_x = 300 \) cm
- \( L_y = 300 \) cm
- \( k_xL_x/i_x = 54.054 \)
- \( k_yL_y/i_y = 54.054 \)

Check of Stresses
- \( f_C = 1.14 \) t/cm² < \( F_C = 1.7 \) t/cm² Safe
- \( f_b = 0.145 \) t/cm² < \( F_b = 2.1 \) t/cm² Safe
- \( \tau = 0.017 \) t/cm² < \( \tau_a = 0.73 \) t/cm² Safe

Interaction Check
\[
\frac{f_{ca}}{F_c} + \frac{f_{bex}}{F_{bex}} A_1 + \frac{f_{bey}}{F_{bey}} A_2 = 0.74
\]
Digonal Bracing elements

Section Name: 140x140x2

Section Dimensions:
- \( H = 140 \) mm
- \( B = 140 \) mm
- \( t = 2 \) mm

Steel Properties:
- Steel Type: 52
  - \( F_y = 3.6 \) t/cm\(^2\)
  - \( F_u = 5.2 \) t/cm\(^2\)

Section Properties:
- \( A_e = 11.04 \) cm\(^2\)
- \( I_x = 350.48 \) cm\(^4\)
- \( i_x = 5.63 \) cm
- \( S_x = 50.069 \) cm\(^3\)
- \( I_y = 350.48 \) cm\(^4\)
- \( i_y = 5.63 \) cm

Weight: 17.08 kg/m

Applied Straining Actions:
- \( N = -10 \) ton
- \( M_x = 0 \) t.m.
- \( Q = 0 \) ton
- \( k_x = 0.5 \)
- \( L_x = 490 \) cm
- \( k_y = 0.8 \)
- \( L_y = 490 \) cm

Check of Stresses:
- \( f_c = 0.9 \) t/cm\(^2\) < \( F_c = 1.44 \) t/cm\(^2\) Safe
- \( f_b = 0 \) t/cm\(^2\) < \( F_b = 2.1 \) t/cm\(^2\) Safe
- \( \tau = 0 \) t/cm\(^2\) < \( \tau_a = 0.73 \) t/cm\(^2\) Safe

Interaction Check:
\[
\frac{f_{c_a}}{F_c} + \frac{f_{b_{c_x}}}{F_{b_{c_x}}} A_1 + \frac{f_{b_{c_y}}}{F_{b_{c_y}}} A_2 = 0.63
\]
Horizontal bracing member (axe-7)

Section Name  250x140x3

Section Dimensions

\[ \begin{align*}
H &= 250 \text{ mm} \\
B &= 140 \text{ mm} \\
t &= 3 \text{ mm}
\end{align*} \]

Steel Properties

Steel Type 52

\[ \begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*} \]

Section Properties

\[ \begin{align*}
A_e &= 23.04 \text{ cm}^2 \\
I_x &= 2007.6 \text{ cm}^4 \\
i_x &= 9.33 \text{ cm} \\
S_x &= 160.61 \text{ cm}^3 \\
l_y &= 824.25 \text{ cm}^4 \\
i_y &= 5.98 \text{ cm} \\
\text{Weight} &= 18.08 \text{ kg/m}
\end{align*} \]

Applied Straining Actions

\[ \begin{align*}
N &= 0 \text{ ton} \\
M_x &= 2.6 \text{ t.m.} \\
Q &= 2.4 \text{ ton} \\
\text{Span} &= 5 \text{ m}
\end{align*} \]

Check of Stresses

\[ \begin{align*}
f_c &= 0 \text{ t/cm}^2 < F_c = \text{t/cm}^2 \quad \text{Safe} \\
f_b &= 1.61 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.172 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*} \]

Maximum deflection due to LL = 6 mm

L/200 = 25 mm
Horizontal bracing member (axe-1 & 13)

Section Name 200x140x3

Section Dimensions

\[
\begin{align*}
H &= 200 \text{ mm} \\
B &= 140 \text{ mm} \\
t &= 3 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_e &= 20.04 \text{ cm}^2 \\
I_x &= 1180.1 \text{ cm}^4 \\
i_x &= 7.67 \text{ cm} \\
S_x &= 118.01 \text{ cm}^3 \\
I_y &= 683.47 \text{ cm}^4 \\
i_y &= 5.84 \text{ cm} \\
\text{Weight} &= 15.73 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
N &= 0 \text{ ton} \\
M_x &= 1.1 \text{ t.m.} \\
Q &= 0.95 \text{ ton}
\end{align*}
\]

Span = 3.87 m

Check of Stresses

\[
\begin{align*}
f_C &= 0 \text{ t/cm}^2 < F_C = \text{t/cm}^2 \text{ Safe} \\
f_b &= 0.932 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \text{ Safe} \\
\tau &= 0.086 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \text{ Safe}
\end{align*}
\]

Maximum deflection due to LL = 4.25 mm

L/200 = 19 mm
Stair Beam

Section Name 200C75-200

Section Dimensions

- $H = 200$ mm
- $B = 75$ mm
- $D = 25$ mm
- $t = 2$ mm
- $r = 4$ mm

Steel Properties

- Steel Type
  - $F_y = 3.6$ t/cm$^2$
  - $F_u = 5.2$ t/cm$^2$

Section Properties

- $A = 7.66$ cm$^2$
- $I_x = 471.64$ cm$^4$
- $S_x = 47.16$ cm$^3$
- $S_{xe} = 47.155$ cm$^3$
- Weight = 6.019 kg/m

Applied Straining Actions

- $M_x = 0.42$ t.m.
- $Q = 0.64$ ton
- Span = 2.62 m

Check of Stresses

- $f_b = 0.89068$ t/cm$^2$ < $F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0.16$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$ Safe

Maximum deflection due to LL = 10 mm
- $L/200 = 26$ mm
1. Introduction
In this report the detailed design of 6 story residential building is presented. The building covers an area of 445 m² (including voids), each floor is divided into 4 flats each of which is 80 m². Fig.1 shows the typical architectural floor plan of the building. The primary vertical loads are carried by rigid frame, and the lateral loads are resisted by vertical bracing elements.

2. Design Criteria
2. Statical System and structural analysis

The statical system that carries the vertical loads (dead and live loads) consists of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to the rigid frame. The rigid frames are arranged along the vertical axis 1 through 13. Lateral loads in Y-direction are carried by the rigid frames, while lateral loads in X-direction are carried by group of vertical bracing systems. Vertical bracing systems along Y-direction are arranged on axis “2”, “7”, and “12”, also each axe receives 2 bracing bays.

Fig. 2 : Structural plan showing the arrangement of the joists, Dual System and the vertical bracing bays.

3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:
• Beams (joists) are hinged connected to the vertical columns.
• Vertical bracing members are pinned connected to the vertical columns.
• The floor slab moved horizontally in the principle directions as rigid diaphragm.

3. Member Seizing
The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

3.1 JOISTS
Joists are designed as simple beams with variable spans. The maximum span is 5.9 m (between axes 1 & 4). The spacing between joists 120 cm. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 300SG80-200 section was selected. The compression flange of the joists was considered to be continuously braced via attachment of GRC panels.

3.2 Columns
The rigid frame columns are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 200SG65-200, 200SG65-200 back to back, 300SG80-265 back to back, and 300SG80-300 back to back sections were selected.

3.3 Beams
The rigid frame columns are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 200SG65-200 and 300SG80-200 back to back sections were selected.

3.4 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

<table>
<thead>
<tr>
<th>Wind in X-direction</th>
<th>+36.94/- 29.89</th>
</tr>
</thead>
</table>

Table 1: Base reactions due to lateral loads
Axial forces in the vertical members range from 18 ton to -23 ton. Also, axial forces in the diagonal members range from 3.5 ton to -8.87 ton. Based on this along with the limits that are stated in the design criteria square hollow section 200x200x2 section was selected.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.26 t.m, and maximum shear force of 1.88 ton. Based on this along with the limits that are stated in the design criteria hollow section200x220x2 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

3.5 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are .42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

4. Material Quantities
From the above design the following quantities are calculated

- Own weight of steel elements = 40.732 x 1.1 = 42.79 ton

<table>
<thead>
<tr>
<th>Wind in Y-direction</th>
<th>+/- 28.391</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 12.55</td>
</tr>
<tr>
<td>Seismic in Y-direction</td>
<td>+/- 12.55</td>
</tr>
</tbody>
</table>
Fig. 3: Plan of one floor
Horizontal beams (joists) along X-axis, pin connected to the Rigid Frames.
Rigid Frames are arranged on axis 1 to 13
Fig. 4: Cross section along axe "D"

*Horizontal joists are pin connected to the vertical rigid frames*
Fig. 5: Cross section along axe "G"

*Vertical bracing resists lateral loads in the X-direction, and provides lateral stability*
Design of Sections

JOIST DESIGN

Section Name 300SG80-200

Section Dimensions

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>300 mm</td>
</tr>
<tr>
<td>B</td>
<td>80 mm</td>
</tr>
<tr>
<td>D</td>
<td>25 mm</td>
</tr>
<tr>
<td>t</td>
<td>2 mm</td>
</tr>
<tr>
<td>r</td>
<td>4 mm</td>
</tr>
</tbody>
</table>

Steel Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Type</td>
<td>Fy = 3.6 t/cm²</td>
</tr>
<tr>
<td></td>
<td>Fu = 5.2 t/cm²</td>
</tr>
</tbody>
</table>

Section Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>9.934 cm²</td>
</tr>
<tr>
<td>Iₓ</td>
<td>1277.3 cm⁴</td>
</tr>
<tr>
<td>Sₓ</td>
<td>85.153 cm³</td>
</tr>
<tr>
<td>Sₓₑ</td>
<td>85.153 cm³</td>
</tr>
<tr>
<td>Weight</td>
<td>7.798 kg/m</td>
</tr>
</tbody>
</table>

Applied Straining Actions

<table>
<thead>
<tr>
<th>Action</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mₓ</td>
<td>1.76 t.m.</td>
</tr>
<tr>
<td>Q</td>
<td>1.2 ton</td>
</tr>
<tr>
<td>Span</td>
<td>5.9 m</td>
</tr>
</tbody>
</table>

Check of Stresses

<table>
<thead>
<tr>
<th>Stress</th>
<th>Value</th>
<th>Comparison</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>fᵦ</td>
<td>2.0669 t/cm²</td>
<td>&lt; Fᵦ = 2.1 t/cm²</td>
<td>Safe</td>
</tr>
<tr>
<td>τ</td>
<td>0.6 t/cm²</td>
<td>&lt; τₐ = 0.73 t/cm²</td>
<td>Safe</td>
</tr>
</tbody>
</table>

Maximum deflection due to LL = 14.1 mm

L/200 = 29.5 mm
Beam 1

Section Name 2 x (300SG80-200)

Section Dimensions
- H = 300 mm
- B = 80 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties
- Steel Type 52
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties
- $A_e = 16.282$ cm²
- $I_x = 2554.6$ cm⁴
- $i_x = 11.339$ cm
- $S_x = 170.31$ cm³
- $S_{xe} = 170.31$ cm³
- $I_y = 233.3$ cm⁴
- $i_y = 3.427$ cm
- Weight = 15.59 kg/m

Applied Straining Actions
- $N = 0$ ton
- $M_x = 260$ t.m. height = 3 m
- $Q = 0$ ton

Check of Stresses
- $f_c = 0$ t/cm² < $F_c = 2.1$ t/cm² Safe
- $f_b = 1.52$ t/cm² < $F_b = 2.1$ t/cm² Safe
- $\tau = 0$ t/cm² < $\tau_a = 0.73$ t/cm² Safe

Interaction Check
\[
\frac{f_{ex}}{F_c} + \frac{f_{exx}}{F_{exx}} A_1 + \frac{f_{exy}}{F_{exy}} A_2 = 0.75
\]
Beam 2

Section Name 200SG65-200

Section Dimensions

- \( H = 200 \) mm
- \( B = 65 \) mm
- \( D = 20 \) mm
- \( t = 2 \) mm
- \( r = 4 \) mm

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \) t/cm²
- \( F_u = 5.2 \) t/cm²

Section Properties

- \( A_e = 6.48 \) cm²
- \( l_x = 421.86 \) cm⁴
- \( i_x = 7.69 \) cm
- \( S_x = 42.1 \) cm³
- \( S_{xe} = 42.1 \) cm³
- \( l_y = 36.7 \) cm⁴
- \( i_y = 2.26 \) cm
- Weight = 5.597 kg/m

Applied Straining Actions

- \( N = 0 \) ton
- \( M_x = 0.6 \) t.m.
- \( Q = 0 \) ton
- height = 3 m

Check of Stresses

- \( f_c = 0 \) t/cm²
- \( f_b = 1.4252 \) t/cm²
- \( \tau = 0 \) t/cm²
- \( F_C = 2.1 \) t/cm²
- \( F_b = 2.1 \) t/cm²
- \( \tau_a = 0.73 \) t/cm²

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.67
\]
Column 1

Section Name 200SG65-200

Section Dimensions

- H = 200 mm
- B = 65 mm
- D = 20 mm
- t = 2 mm
- r = 4 mm

Steel Properties

Steel Type 52

- $F_y = 3.6 \ t/cm^2$
- $F_u = 5.2 \ t/cm^2$

Section Properties

- $A_e = 6.48 \ cm^2$
- $I_x = 421.86 \ cm^3$
- $i_x = 7.69 \ cm$
- $S_x = 42.1 \ cm^3$
- $S_{xe} = 42.1 \ cm^3$
- $I_y = 36.7 \ cm^4$
- $i_y = 2.26 \ cm$

Weight = 5.597 kg/m

Applied Straining Actions

- $N = 10 \ t/m$
- $M_x = 0.2 \ t.m.$
- $Q = 0 \ t/m$
- height = 3 m

Check of Stresses

- $f_c = 1.54 \ t/cm^2 < F_c = 2.1 \ t/cm^2$ Safe
- $f_b = 0.4751 \ t/cm^2 < F_b = 2.1 \ t/cm^2$ Safe
- $\tau = 0 \ t/cm^2 < \tau_a = 0.73 \ t/cm^2$ Safe

Interaction Check

$$\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.959$$
Column 2

Section Name 2 x (200SG65-200)

Section Dimensions

- $H = 200$ mm
- $B = 65$ mm
- $D = 20$ mm
- $t = 2$ mm
- $r = 4$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_e = 12.96$ cm$^2$
- $I_x = 843.72$ cm$^4$
- $i_x = 7.69$ cm
- $S_a = 84.372$ cm$^3$
- $S_{ae} = 84.372$ cm$^3$
- $I_y = 120.62$ cm$^4$
- $i_y = 2.908$ cm
- Weight = 11.18 kg/m

Applied Straining Actions

- $N = 17$ ton
- $M_x = 0.6$ t.m.
- $Q = 0$ ton
- height = 3 m

Check of Stresses

- $f_c = 1.31$ t/cm$^2$ < $F_c = 2.1$ t/cm$^2$ Safe
- $f_b = 0.7111$ t/cm$^2$ < $F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0$ t/cm$^2$ < $\tau_a = 0.73$ t/cm$^2$ Safe

Interaction Check

$$\frac{f_{cx}}{F_c} + \frac{f_{hcx}}{F_{hcx}} A_1 + \frac{f_{hcy}}{F_{hcy}} A_2 = 0.96$$
Section Name: 2 x (300SG80-265)

Section Dimensions:
- H = 300 mm
- B = 80 mm
- D = 25 mm
- t = 2.65 mm
- r = 5.3 mm

Steel Properties:
- Steel Type: 52
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties:
- Ae = 25.152 cm²
- Iₓ = 3314.4 cm⁴
- iₓ = 11.27 cm
- Sₓ = 220.96 cm³
- Sₓₑ = 220.96 cm³
- Iᵧ = 296.51 cm⁴
- iᵧ = 3.371 cm

Weight = 20.487 kg/m

Applied Straining Actions:
- N = 30 ton
- Mₓ = 1.2 t.m.
- Q = 0 ton

height = 3 m

Check of Stresses:
- fₓ ≤ Fₓ = 2.1 t/cm² Safe
- fᵧ ≤ Fᵧ = 2.1 t/cm² Safe
- τ ≤ τₓ = 0.73 t/cm² Safe

Interaction Check:
\[ \frac{fₓ}{Fₓ} + \frac{fᵧ}{Fᵧ A₁} + \frac{τ}{τₓ A₂} = 0.82 \]
Column 4

Section Name 2 x (300SG80-300)

Section Dimensions
- \( H = 300 \) mm
- \( B = 80 \) mm
- \( D = 25 \) mm
- \( t = 3 \) mm
- \( r = 6 \) mm

Steel Properties
- Steel Type 52
  - \( F_y = 3.6 \) t/cm\(^2\)
  - \( F_u = 5.2 \) t/cm\(^2\)

Section Properties
- \( A_e = 29.053 \) cm\(^2\)
- \( I_x = 3709.3 \) cm\(^4\)
- \( i_x = 11.232 \) cm
- \( S_y = 247.29 \) cm\(^3\)
- \( S_{xy} = 247.29 \) cm\(^3\)
- \( I_y = 326.19 \) cm\(^4\)
- \( i_y = 3.341 \) cm

Weight = 23.082 kg/m

Applied Straining Actions
- \( N = 42 \) ton
- \( M_x = 0.3 \) t.m.
- \( Q = 0 \) ton

height = 3 m

Check of Stresses
- \( f_c = 1.44 \) t/cm\(^2\) < \( F_c = 2.1 \) t/cm\(^2\) Safe
- \( f_b = 0.1213 \) t/cm\(^2\) < \( F_b = 2.1 \) t/cm\(^2\) Safe
- \( \tau \) = 0 t/cm\(^2\) < \( \tau_a = 0.73 \) t/cm\(^2\) Safe

Interaction Check
\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{hcy}}{F_{hcy}} A_2 = 0.75
\]
Vertical and Digonal Bracing elements

Section Name 200x200x2

Section Dimensions

\[
\begin{align*}
H &= 200 \text{ mm} \\
B &= 200 \text{ mm} \\
t &= 2 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_e &= 12.9 \text{ cm}^2 \\
I_x &= 1035.1 \text{ cm}^4 \\
ix &= 8.08 \text{ cm} \\
S_x &= 103.5 \text{ cm}^3 \\
I_y &= 1035.1 \text{ cm}^4 \\
i_y &= 8.08 \text{ cm}
\end{align*}
\]

Weight = 12.43 kg/m

Applied Straining Actions

\[
\begin{align*}
N &= 23 \text{ ton} \\
M_x &= 0 \text{ t.m.} \\
Q &= 0 \text{ ton} \\
k_x &= 1 \\
k_y &= 1 \\
L_x &= 300 \text{ cm} \\
L_y &= 300 \text{ cm} \\
k_xL_x/i_x &= 37.128 \\
k_yL_y/i_y &= 37.128
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_c &= 1.78 \text{ t/cm}^2 < F_c = 1.91 \text{ t/cm}^2 \quad \text{Safe} \\
f_y &= 0 \text{ t/cm}^2 < F_y = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
t &= 0 \text{ t/cm}^2 < \tau_s = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_c}{F_c} + \frac{f_y}{F_y} A_1 + \frac{f_y}{F_y} A_2 = 0.933
\]
Horizontal bracing member

Section Name  200x220x2

Section Dimensions

\[ H = 200 \text{ mm} \]
\[ B = 220 \text{ mm} \]
\[ t = 2 \text{ mm} \]

Steel Properties

Steel Type  52

\[ F_y = 3.6 \text{ t/cm}^2 \]
\[ F_u = 5.2 \text{ t/cm}^2 \]

Section Properties

\[ A_e = 14.14 \text{ cm}^2 \]
\[ I_x = 1286.4 \text{ cm}^4 \]
\[ i_x = 8.79 \text{ cm} \]
\[ S_x = 116.94 \text{ cm}^3 \]
\[ I_y = 1113.5 \text{ cm}^4 \]
\[ i_y = 8.18 \text{ cm} \]

Weight = 13.062 kg/m

Applied Straining Actions

\[ N = 0 \text{ ton} \]
\[ M_x = 2.26 \text{ t.m.} \]
\[ Q = 1.88 \text{ ton} \]

Span = 3.6 m

Check of Stresses

\[ f_c = 0 \text{ t/cm}^2 < F_c = 1.29 \text{ t/cm}^2 \quad \text{Safe} \]
\[ f_b = 1.93 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \]
\[ \tau = 0.24 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe} \]

Maximum deflection due to LL = 13 mm

L/200 = 18 mm
Stair Beam

Section Name 200C75-200

Section Dimensions

- H = 200 mm
- B = 75 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties

- Steel Type
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties

- A = 7.66 cm²
- Iₓ = 471.64 cm⁴
- Sₓ = 47.16 cm³
- Sₓₑ = 47.155 cm³
- Weight = 6.019 kg/m

Applied Straining Actions

- Mₓ = 0.42 t.m.
- Q = 0.64 ton

Span = 2.62 m

Check of Stresses

- fₑ = 0.89068 t/cm² < Fₑ = 2.1 t/cm² Safe
- τ = 0.16 t/cm² < τₑ = 0.73 t/cm² Safe

Maximum deflection due to LL = 10 mm

L/200 = 26 mm
Appendix A6

Design Report for a Residential Building
Load Bearing wall panel system
Decking: GRC panels
80 m²- 4 flats/floor
Section : Sigma

1. Introduction
This report presents the structural analysis and design of a residential building where the primary vertical loads are carried by wall bearing formed of cold-formed steel “C” sections, and the primary lateral loads are resisted by vertical bracing elements. The considered building consists of 6 floors, and covers an area of 445 m² (including voids), each floor is divided into 4 flats each of which is 80 m². Fig.1 shows the typical architectural floor plan of the building.
2. Statical System and structural analysis
The statical system that carries the vertical loads (dead and live loads) consists of GRC panels supported on series of horizontal beams (joists). The beams transmit their loads directly to vertical columns (studs). The spacing between the beams and the columns are nearly the same. So, the vertical loads transmitted from the slab to the beams, and then to the vertical columns. However, lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The load bearing walls are arranged along axis 1 to 13, while the vertical bracing systems along X-direction are arranged on axis “B”, “G”, and “L”. Each axe receives 2 bracing bays. Inaddition, the vertical bracing systems along Y-direction are arranged on axis “2”, “7”, and “12”. Also, each axe has 2 bracing bays.

3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:
• Beams (joists) are hinged connected to the vertical columns.
• Places where opening like doors or windows are placed, lintel beam carry the reaction from the joist to two vertical columns adjacent to the opening.
• Vertical bracing members are pinned connected to the vertical columns.
• The floor slab moved horizontally in the principle directions as rigid diaphragm.

3. Member Seizing
The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

3.1 JOISTS
Joists are designed as simple beams with variable spans. The maximum span is 5.9 m (between axes 1 & 4). The spacing between joists 120 cm. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 300SG80-200 section was selected. The compression flange of the joists was considered to be continuously braced via attachment of GRC panels.

3.2 Beams bridging openings
These beams are designed as simple beams. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 200T85-225 section was selected. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking.

3.3 External load bearing walls
The external vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. In addition to the vertical loads, lateral wind loads are considered. Based on this along with the limits that are stated in the design criteria 200SG65-170 section was selected.

3.4 Internal load bearing walls
The internal vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the
presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. Based on this along with the limits that are stated in the design criteria 200SG65-200 section was selected for the ground and first floors while 200SG65-170 was selected for the rest floors.

### 3.5 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the Seismic.

<table>
<thead>
<tr>
<th>Applied Load</th>
<th>Base Reaction (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind in X-direction</td>
<td>+36.94/- 29.391</td>
</tr>
<tr>
<td>Wind in Y-direction</td>
<td>+/- 28.391</td>
</tr>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 12.56</td>
</tr>
<tr>
<td>Seismic in Y-direction</td>
<td>+/- 12.56</td>
</tr>
</tbody>
</table>

Axial forces in the vertical members range from 18 ton to -23 ton. Also, axial forces in the diagonal members range from 3.5 ton to -8.87 ton. Based on this along with the limits that are stated in the design criteria square hollow section 200x200x2 section was selected.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.26 t.m, and maximum shear force of 1.88 ton. Based on this along with the limits that are stated in the design criteria hollow section 200x220x2 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

### 3.6 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are .42 t.m and 0.64 ton, respectively. The compression flange
of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

4. Material Quantities
From the above design the following quantities are calculated

- Own weight of steel elements = 41 x 1.1 = 46 ton
Fig. 3: Plan of one floor

Horizontal beams (joists) along X-axis, pin connected to the load bearing walls.
Load bearing walls are arranged on axis 1 to 13
Fig. 4: Cross section along axis "D"

*Horizontal joists are pin connected to the vertical load bearing walls*
Fig. 5: Cross section along axe "G"

Vertical bracing resists lateral loads in the X-direction, and provides lateral stability
Fig. 6: Cross section along axe "7"
Vertical bracing resists lateral loads in the Y-direction, and provides lateral stability
Design of Sections

JOIST DESIGN

Section Name 300SG80-200

Section Dimensions

- H = 300 mm
- B = 80 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties

Steel Type
- Fy = 3.6 \(\text{t/cm}^2\)
- Fu = 5.2 \(\text{t/cm}^2\)

Section Properties

- A = 9.934 \(\text{cm}^2\)
- I_x = 1277.3 \(\text{cm}^4\)
- S_x = 85.153 \(\text{cm}^3\)
- S_{xe} = 85.153 \(\text{cm}^3\)
- Weight = 7.798 kg/m

Applied Straining Actions

- M_x = 1.76 \(\text{t.m.}\)
- Q = 1.2 \(\text{ton}\)
- Span = 5.9 m

Check of Stresses

- \(f_b = \frac{2.0669}{2.1} < 2.1 \text{ t/cm}^2\) Safe
- \(\tau = \frac{0.6}{0.73} < 0.73 \text{ t/cm}^2\) Safe

Maximum deflection due to LL = 14.1 mm
- L/200 = 29.5 mm
External load bearing walls

Section Name 200SG65-170

Section Diminsions

\[ H = 200 \text{ mm} \]
\[ B = 65 \text{ mm} \]
\[ D = 20 \text{ mm} \]
\[ t = 1.7 \text{ mm} \]
\[ r = 3.4 \text{ mm} \]

Steel Properties

Steel Type 52
\[ F_y = 3.6 \text{ t/cm}^2 \]
\[ F_u = 5.2 \text{ t/cm}^2 \]

Section Properties

\[ A_e = 5.1718 \text{ cm}^2 \]
\[ I_x = 363.22 \text{ cm}^4 \]
\[ i_x = 7.718 \text{ cm} \]
\[ S_x = 36.322 \text{ cm}^3 \]
\[ I_y = 32.253 \text{ cm}^4 \]
\[ i_y = 2.2998 \text{ cm} \]

Weight = 4.7869 kg/m

Applied Straining Actions

\[ N = 6.5 \text{ ton} \]
\[ M_x = 0.075 \text{ t.m.} \]
\[ Q = 0.1 \text{ ton} \]
\[ k_x = 1 \]
\[ k_y = 0.5 \]
\[ L_x = 300 \text{ cm} \]
\[ L_y = 150 \text{ cm} \]
\[ k_x L_y / i_x = 38.87 \]
\[ k_y L_y / i_y = 65.24 \]

Check of Stresses

\[ f_C = 1.257 \text{ t/cm}^2 \] < \[ F_C = 1.525 \text{ t/cm}^2 \] Safe
\[ f_b = 0.2065 \text{ t/cm}^2 \] < \[ F_b = 2.1 \text{ t/cm}^2 \] Safe
\[ \tau = 0.005 \text{ t/cm}^2 \] < \[ \tau_a = 0.73 \text{ t/cm}^2 \] Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcy}}{F_{bcy}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.919
\]
Internal load bearing walls (floors 0, 1)

Section Name 200SG65-200

Section Dimensions

- \( H = 200 \) mm
- \( B = 65 \) mm
- \( D = 20 \) mm
- \( t = 2 \) mm
- \( r = 4 \) mm

Steel Properties

- Steel Type: 52
- \( F_y = 3.6 \) t/cm\(^2\)
- \( F_u = 5.2 \) t/cm\(^2\)

Section Properties

- \( A_e = 6.48 \) cm\(^2\)
- \( I_x = 421.86 \) cm\(^4\)
- \( i_x = 7.69 \) cm
- \( S_x = 42.1 \) cm\(^3\)
- \( S_{ax} = 42.1 \) cm\(^3\)
- \( I_y = 36.7 \) cm\(^4\)
- \( i_y = 2.26 \) cm
- Weight = 5.597 kg/m

Applied Straining Actions

- \( N = 11.2 \) ton
- \( M_x = 0 \) t.m.
- \( Q = 0 \) ton
- \( k_x = 1 \)
- \( k_y = 0.333 \)
- \( L_x = 300 \) cm
- \( L_y = 100 \) cm
- \( k_x L_x / i_x = 38.87 \)
- \( k_y L_y / i_y = 44.24 \)

Check of Stresses

- \( f_c = 1.72 \) t/cm\(^2\) \(< F_c = 1.83 \) t/cm\(^2\) Safe
- \( f_b = 0 \) t/cm\(^2\) \(< F_b = 2.1 \) t/cm\(^2\) Safe
- \( \tau = 0 \) t/cm\(^2\) \(< \tau_a = 0.73 \) t/cm\(^2\) Safe

Interaction Check

\[
\frac{f_{cu}}{F_c} + \frac{f_{bex}}{F_{bex}} A_1 + \frac{f_{pex}}{F_{pex}} A_2 = 0.944
\]
Internal load bearing walls (floors: 2, 3, 4, 5, 6)

Section Name: 200SG65-170

Section Dimensions:
- \( H = 200 \) mm
- \( B = 65 \) mm
- \( D = 20 \) mm
- \( t = 1.7 \) mm
- \( r = 3.4 \) mm

Steel Properties:
- Steel Type: 52
  - \( F_y = 3.6 \) t/cm²
  - \( F_u = 5.2 \) t/cm²

Section Properties:
- \( A_e = 5.1718 \) cm²
- \( I_x = 363.22 \) cm⁴
- \( i_x = 7.718 \) cm
- \( S_x = 36.322 \) cm³
- \( S_{ae} = 36.322 \) cm³
- \( l_y = 32.253 \) cm
- \( l_y = 2.2998 \) cm
- Weight = 4.7869 kg/m

Applied Straining Actions:
- \( N = 10 \) ton
- \( M_x = 0 \) t.m.
- \( Q = 0 \) ton
- \( k_x = 1 \)
- \( L_x = 300 \) cm
- \( k_y = 0.25 \)
- \( L_y = 75 \) cm
- \( k_x L_x / i_x = 38.87 \)
- \( k_y L_y / l_y = 32.75 \)

Check of Stresses:
- \( f_c = 1.93 \) t/cm² < \( F_c = 1.95 \) t/cm² Safe
- \( f_o = 0 \) t/cm² < \( F_o = 2.1 \) t/cm² Safe
- \( t = 0 \) t/cm² < \( \tau_a = 0.73 \) t/cm² Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{pcy}}{F_{pcy}} A_2 = 0.98
\]
Vertical and Digonal Bracing elements

Section Name 200x200x2

Section Diminsions

- $H = 200$ mm
- $B = 200$ mm
- $t = 2$ mm

Steel Properties

Steel Type 52
- $F_y = 3.6$ t/cm$^2$
- $F_u = 5.2$ t/cm$^2$

Section Properties

- $A_e = 12.9$ cm$^2$
- $I_x = 1035.1$ cm$^4$
- $i_x = 8.08$ cm
- $S_x = 103.5$ cm$^3$
- $I_y = 1035.1$ cm$^4$
- $i_y = 8.08$ cm
- Weight = 12.43 kg/m

Applied Straining Actions

- $N = 23$ ton
- $M_x = 0$ t.m. $h = 3$ m
- $Q = 0$ ton
- $k_x = 1$ $k_y = 1$
- $L_x = 300$ cm $L_y = 300$ cm
- $k_xL_x/i_x = 37.128$ $k_yL_y/i_y = 37.128$

Check of Stresses

- $f_C = 1.78$ t/cm$^2$ $< F_C = 1.91$ t/cm$^2$ Safe
- $f_b = 0$ t/cm$^2$ $< F_b = 2.1$ t/cm$^2$ Safe
- $\tau = 0$ t/cm$^2$ $< \tau_a = 0.73$ t/cm$^2$ Safe

Interaction Check

$$\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.933$$
Horizontal bracing member

Section Name 200x220x2

Section Dimensions

- H = 200 mm
- B = 220 mm
- t = 2 mm

Steel Properties

- Steel Type 52
- $F_y = 3.6 \text{ t/cm}^2$
- $F_u = 5.2 \text{ t/cm}^2$

Section Properties

- $A_e = 14.14 \text{ cm}^2$
- $I_x = 1286.4 \text{ cm}^4$
- $i_x = 8.79 \text{ cm}$
- $S_x = 116.94 \text{ cm}^3$
- $I_y = 1113.5 \text{ cm}^4$
- $i_y = 8.18 \text{ cm}$
- Weight = 13.062 kg/m

Applied Straining Actions

- $N = 0 \text{ ton}$
- $M_x = 2.26 \text{ t.m.}$
- $Q = 1.88 \text{ ton}$
  
  Span = 3.6 m

Check of Stresses

\[
\begin{align*}
  f_C &= 0 \text{ t/cm}^2 < F_C = 1.29 \text{ t/cm}^2 \quad \text{Safe} \\
  f_b &= 1.93 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
  \tau &= 0.24 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Maximum deflection due to LL = 13 mm
  
  $L/200 = 18 \text{ mm}$
Stair Beam

Section Name 200C75-200

Section Dimensions

\[H = 200 \text{ mm}\]
\[B = 75 \text{ mm}\]
\[D = 25 \text{ mm}\]
\[t = 2 \text{ mm}\]
\[r = 4 \text{ mm}\]

Steel Properties

Steel Type
\[F_y = 3.6 \text{ t/cm}^2\]
\[F_u = 5.2 \text{ t/cm}^2\]

Section Properties

\[A = 7.66 \text{ cm}^2\]
\[I_x = 471.64 \text{ cm}^4\]
\[S_x = 47.16 \text{ cm}^3\]
\[S_{xe} = 47.155 \text{ cm}^3\]
Weight = 6.019 kg/m

Applied Straining Actions

\[M_x = 0.42 \text{ t.m.}\]
\[Q = 0.64 \text{ ton}\]
Span = 2.62 m

Check of Stresses

\[f_b = 0.89068 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe}\]
\[\tau = 0.16 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}\]

Maximum deflection due to LL = 10 mm
\[L/200 = 26 \text{ mm}\]
Design Report for a Residential Building
Dual System (Rigid Frame + Vertical Studs)
Decking: GRC panels
80 m²
Section: Lipped Channel

1. Introduction
In this report the detailed design of 6 story residential building is presented. The building covers an area of 445 m² (including voids), each floor is divided into 4 flats each of which is 80 m². Fig.1 shows the typical architectural floor plan of the building. The primary vertical loads are carried by dual system, and the lateral loads are resisted by vertical bracing elements. Dual system composed of rigid frame and vertical studs, Fig. 2, the axial stiffness of the studs can interacts with the bending stiffness of the rigid frame in a manner that minimizing the total weight of steel used. Moreover, dual system provides flexibility in the size and location of any opening.

Fig.1 : Architectural typical floor plan
2. Statical System and structural analysis

The statical system that carries the vertical loads (dead and live loads) consists of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to the dual system. The dual systems are arranged along the vertical axis 1 through 13. Lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The load bearing walls are arranged along axis 1 to 13, while the vertical bracing systems along X-direction are arranged on axis “B”, “G” and “L”. Each axe receives 2 bracing bays. In addition, the vertical bracing systems along Y-direction are arranged on axis “2”, “7”, and “12”, also each axe receives 2 bracing bays.
3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:

- Beams (joists) are hinged connected to the vertical columns.
- Vertical bracing members are pinned connected to the vertical columns.
- The floor slab moved horizontally in the principle directions as rigid diaphragm.

3. Member Seizing
The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

3.1 JOISTS
Joists are designed as simple beams with variable spans. The maximum span is 5.9 m (between axes 1 & 4). The spacing between joists 120 cm. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 260C85-265 section was selected. The compression flange of the joists was considered to be continuously braced via attachment of GRC panels.

3.2 Rigid Frame (Columns / Beams)
The rigid frame members are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 140C60-150 section was selected for the external frames, while 240C75-225 sections for the interior one.

3.3 External vertical Studs
The external vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. In addition to the vertical loads, lateral wind loads are considered. Based on this along with the limits that are stated in the design criteria 140C60-150 section was selected.

3.4 Internal vertical studs
The internal vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. Based on this along with the limits that are stated in the design criteria 140C60-200, and 240C75-225 sections were selected.

3.5 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

| Table 1: Base reactions due to lateral loads |
Axial forces in the vertical members range from 18 ton to -23 ton. Also, axial forces in the diagonal members range from 3.5 ton to -8.87 ton. Based on this along with the limits that are stated in the design criteria square hollow section 200x200x2 section was selected.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.26 t.m, and maximum shear force of 1.88 ton. Based on this along with the limits that are stated in the design criteria hollow section200x220x2 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

3.6 Stairs
The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are .42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

4. Material Quantities
From the above design the following quantities are calculated

- Own weight of steel elements = 42.71 x 1.1 = 46.981 ton
Fig. 3: Plan of one floor

Horizontal beams (joists) along X-axis, pin connected to the load bearing walls.

Load bearing walls are arranged on axis 1 to 13
Fig. 4: Cross section along axe "D"

*Horizontal joists are pin connected to the vertical load bearing walls*
Fig. 5: Cross section along axe "G"

Vertical bracing resists lateral loads in the X-direction, and provides lateral stability
Fig. 6: Cross section along axe "7"

*Vertical bracing resists lateral loads in the Y-direction, and provides lateral stability*
Design of Sections

JOIST DESIGN

Section Name 260C85-265

Section Dimensions

- \( H = 260 \) mm
- \( B = 85 \) mm
- \( D = 25 \) mm
- \( t = 2.65 \) mm
- \( r = 5.3 \) mm

Steel Properties

Steel Type
- \( F_y = 3.6 \) t/cm²
- \( F_u = 5.2 \) t/cm²

Section Properties

- \( A = 12.137 \) cm²
- \( I_x = 1220.3 \) cm⁴
- \( S_x = 93.873 \) cm³
- \( S_{xe} = 89.352 \) cm³
- Weight = 9.5 kg/m

Applied Straining Actions

- \( M_x = 1.76 \) t.m.
- \( Q = 1.2 \) ton
- Span = 5.9 m

Check of Stresses

- \( f_b = 1.9697 \) t/cm² < \( F_b = 2.1 \) t/cm² Safe
- \( \tau = 0.6 \) t/cm² < \( \tau_a = 0.73 \) t/cm² Safe

Maximum deflection due to LL = 16 mm

L/200 = 29.5 mm
Internal Rigid Frame Columns

Section Name 240C75-225

Section Dimensions

\[ H = 240 \text{ mm} \]
\[ B = 75 \text{ mm} \]
\[ D = 25 \text{ mm} \]
\[ t = 2.25 \text{ mm} \]
\[ r = 4.5 \text{ mm} \]

Steel Properties

Steel Type 52
\[ F_y = 3.6 \text{ t/cm}^2 \]
\[ F_u = 5.2 \text{ t/cm}^2 \]

Section Properties

\[ A_f = 9.48 \text{ cm}^2 \]
\[ A_e = 6.934 \text{ cm}^2 \]
\[ I_x = 806.92 \text{ cm}^4 \]
\[ i_x = 9.226 \text{ cm} \]
\[ S_x = 67.243 \text{ cm}^3 \]
\[ S_{xe} = 65.263 \text{ cm}^3 \]
\[ I_y = 70.904 \text{ cm}^4 \]
\[ i_y = 2.735 \text{ cm} \]
Weight = 7.44 kg/m

Applied Straining Actions

\[ N = 15 \text{ ton} \]
\[ M_x = 10 \ttext{ t.cm.} \]
\[ Q = 0.35 \text{ ton} \]
height = 3 m

Check of Stresses

\[ f_c = 2.05 \text{ t/cm}^2 < F_C = 2.1 \text{ t/cm}^2 \text{ Safe} \]
\[ f_b = 0.15 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \text{ Safe} \]
\[ \tau = 0.18 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \text{ Safe} \]

Interaction Check

\[ \frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 1 \]
External Rigid Frame Columns

Section Name 140C60-150

Section Dimensions

\[
\begin{align*}
H &= 140 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_f &= 4.313 \text{ cm}^2 \\
A_e &= 2.98 \text{ cm}^2 \\
I_x &= 133.38 \text{ cm}^4 \\
i_x &= 5.56 \text{ cm} \\
S_x &= 19.05 \text{ cm}^3 \\
S_{xe} &= 17.674 \text{ cm}^3 \\
I_y &= 22.396 \text{ cm}^4 \\
i_y &= 2.279 \text{ cm} \\
\text{Weight} &= 3.386 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
N &= 6 \text{ ton} \\
M_x &= 5 \text{ t.cm.} \\
Q &= 0.35 \text{ ton} \\
\text{height} &= 3 \text{ m}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_c &= 2.01 \text{ t/cm}^2 < F_c = 2.1 \text{ t/cm}^2 \text{ Safe} \\
f_b &= 0.28 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \text{ Safe} \\
\tau &= 0.18 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \text{ Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bex}}{F_{bex}} A_1 + \frac{f_{bey}}{F_{bey}} A_2 = 1
\]
internal Rigid Frame Beam

Section Name 240C75-225

Section Dimensions

\[
\begin{align*}
H &= 240 \text{ mm} \\
B &= 75 \text{ mm} \\
D &= 25 \text{ mm} \\
t &= 2.25 \text{ mm} \\
r &= 4.5 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_f &= 9.48 \text{ cm}^2 \\
A_e &= 6.934 \text{ cm}^2 \\
I_x &= 806.92 \text{ cm}^4 \\
i_x &= 9.226 \text{ cm} \\
S_x &= 67.243 \text{ cm}^3 \\
S_{xe} &= 65.263 \text{ cm}^3 \\
I_y &= 70.904 \text{ cm}^4 \\
i_y &= 2.735 \text{ cm} \\
\text{Weight} &= 7.44 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
N &= 0 \text{ ton} \\
M_x &= 120 \text{ t.cm.} \quad \text{height} = 3 \text{ m} \\
Q &= 0.91 \text{ ton}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_c &= 0 \text{ t/cm}^2 < F_C = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 1.83 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.33 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{b,cx}}{F_{b,cx}} A_1 + \frac{f_{b,cy}}{F_{b,cy}} A_2 = 0.87
\]
External Rigid Frame Beam

Section Name 140C60-150

Section Dimensions

\[
\begin{align*}
H &= 140 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 1.5 \text{ mm} \\
r &= 3 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_f &= 4.313 \text{ cm}^2 \\
A_e &= 2.98 \text{ cm}^2 \\
I_x &= 133.38 \text{ cm}^4 \\
i_x &= 5.56 \text{ cm} \\
S_x &= 19.05 \text{ cm}^3 \\
S_{xe} &= 17.674 \text{ cm}^3 \\
I_y &= 22.396 \text{ cm}^4 \\
i_y &= 2.279 \text{ cm} \\
\text{Weight} &= 3.386 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
N &= 0 \text{ ton} \\
M_x &= 19.8 \text{ t.cm.} \\
Q &= 0.91 \text{ ton}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_c &= 0 \text{ t/cm}^2 < F_c = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 1.12 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.33 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{c,ax}}{F_c} + \frac{f_{b,cx}}{F_{b,ax}} A_1 + \frac{f_{b,cx}}{F_{b,cx}} A_2 = 0.53
\]
External Stud

Section Name 140C60-150

Section Dimensions
- \( H = 170 \) mm
- \( B = 60 \) mm
- \( D = 20 \) mm
- \( t = 1.5 \) mm
- \( r = 3 \) mm

Steel Properties
- Steel Type 52
- \( F_y = 3.6 \) t/cm\(^2\)
- \( F_u = 5.2 \) t/cm\(^2\)

Section Properties
- \( A_f = 4.313 \) cm\(^2\)
- \( A_e = 2.98 \) cm\(^2\)
- \( I_x = 133.38 \) cm\(^4\)
- \( i_x = 5.56 \) cm
- \( S_x = 19.05 \) cm\(^3\)
- \( S_{xe} = 17.674 \) cm\(^3\)
- \( I_y = 22.396 \) cm\(^4\)
- \( i_y = 2.279 \) cm
- Weight = 3.386 kg/m

Applied Straining Actions
- \( N = 5.84 \) ton
- \( M_x = 4.5 \) t.cm.
- \( Q = 0.2 \) ton

height = 3 m

Check of Stresses
- \( f_C = 1.95 \) t/cm\(^2\) \( < \) \( F_C = 2.1 \) t/cm\(^2\) Safe
- \( f_b = 0.254 \) t/cm\(^2\) \( < \) \( F_b = 2.1 \) t/cm\(^2\) Safe
- \( \tau = 0.18 \) t/cm\(^2\) \( < \) \( \tau_a = 0.73 \) t/cm\(^2\) Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bca}}{F_{bca}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 1
\]
Internal Stud 1

Section Name 140C60-200

Section Dimensions

\[
\begin{align*}
H &= 140 \text{ mm} \\
B &= 60 \text{ mm} \\
D &= 20 \text{ mm} \\
t &= 2 \text{ mm} \\
r &= 4 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_f &= 5.668 \text{ cm}^2 \\
A_e &= 4.586 \text{ cm}^2 \\
I_x &= 173.04 \text{ cm}^4 \\
i_x &= 5.525 \text{ cm} \\
S_x &= 24.72 \text{ cm}^3 \\
S_{xe} &= 24.33 \text{ cm}^3 \\
I_y &= 28.647 \text{ cm}^4 \\
i_y &= 2.248 \text{ cm} \\
\text{Weight} &= 4.449 \text{ kg/m}
\end{align*}
\]

Applied Straining Actions

\[
\begin{align*}
N &= 7.5 \text{ ton} \\
M_x &= 0 \text{ t.cm.} \\
Q &= 0 \text{ ton}
\end{align*}
\]

Check of Stresses

\[
\begin{align*}
f_c &= 1.67 \text{ t/cm}^2 < F_c = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 0 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{b,cx}}{F_{b,cx}} A_1 + \frac{f_{b,cy}}{F_{b,cy}} A_2 = 0.8
\]
Internal Stud 2

Section Name 240C75-225

Section Dimensions

- \( H = \) 240 mm
- \( B = \) 75 mm
- \( D = \) 25 mm
- \( t = \) 2.25 mm
- \( r = \) 4.5 mm

Steel Properties

- Steel Type 52
- \( F_y = \) 3.6 t/cm²
- \( F_u = \) 5.2 t/cm²

Section Properties

- \( A_f = \) 9.48 cm²
- \( A_e = \) 6.934 cm²
- \( I_x = \) 806.92 cm⁴
- \( i_x = \) 9.226 cm
- \( S_x = \) 67.243 cm³
- \( S_{xw} = \) 65.263 cm³
- \( I_y = \) 70.904 cm⁴
- \( i_y = \) 2.735 cm
- Weight = 7.44 kg/m

Applied Straining Actions

- \( N = \) 14.2 ton
- \( M_x = \) 0 t.cm.
- \( Q = \) 0 ton
- height = 3 m

Check of Stresses

- \( f_C = \) 2.04 t/cm² < \( F_C = \) 2.1 t/cm² Safe
- \( f_b = \) 0 t/cm² < \( F_b = \) 2.1 t/cm² Safe
- \( \tau = \) 0 t/cm² < \( \tau_a = \) 0.73 t/cm² Safe

Interaction Check

\[ \frac{f_{cu}}{F_c} + \frac{f_{bcy}}{F_{bcy}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.97 \]
Vertical and Digonal Bracing elements

Section Name 200x200x2

Section Dimensions

- H = 200 mm
- B = 200 mm
- t = 2 mm

Steel Properties

- Steel Type 52
- $F_y = 3.6 \text{ t/cm}^2$
- $F_u = 5.2 \text{ t/cm}^2$

Section Properties

- $A_e = 12.9 \text{ cm}^2$
- $I_x = 1035.1 \text{ cm}^4$
- $i_x = 8.08 \text{ cm}$
- $S_x = 103.5 \text{ cm}^3$
- $I_y = 1035.1 \text{ cm}^4$
- $i_y = 8.08 \text{ cm}$

Weight = 12.43 kg/m

Applied Straining Actions

- N = 23 ton
- $M_x = 0 \text{ t.m.}$
- $Q = 0 \text{ ton}$
- $k_x = 1$
- $L_x = 300 \text{ cm}$
- $k_y = 1$
- $L_y = 300 \text{ cm}$
- $k_x L_x / i_x = 37.128$
- $k_y L_y / i_y = 37.128$

Check of Stresses

- $f_C = 1.78 \text{ t/cm}^2 < F_C = 1.91 \text{ t/cm}^2$ Safe
- $f_y = 0 \text{ t/cm}^2 < F_y = 2.1 \text{ t/cm}^2$ Safe
- $\tau = 0 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2$ Safe

Interaction Check

$$\frac{f_{ex}}{F_c} + \frac{f_{ex}}{F_{b,ex}} A_1 + \frac{f_{ex}}{F_{b,c}} A_2 = 0.933$$
Horizontal bracing member

Section Name 200x220x2

Section Dimensions

- H = 200 mm
- B = 220 mm
- t = 2 mm

Steel Properties

Steel Type 52

- $F_y = 3.6\ t/cm^2$
- $F_u = 5.2\ t/cm^2$

Section Properties

- $A_e = 14.14\ cm^2$
- $I_x = 1286.4\ cm^4$
- $i_x = 8.79\ cm$
- $S_x = 116.94\ cm^3$
- $I_y = 1113.5\ cm^4$
- $i_y = 8.18\ cm$
- Weight = 13.062 kg/m

Applied Straining Actions

- $N = 0\ ton$
- $M_x = 2.26\ t.m.$
- $Q = 1.88\ ton$

Span = 3.6 m

Check of Stresses

- $f_C = 0\ t/cm^2 < F_C = 1.29\ t/cm^2$ Safe
- $f_b = 1.93\ t/cm^2 < F_b = 2.1\ t/cm^2$ Safe
- $\tau = 0.24\ t/cm^2 < \tau_a = 0.73\ t/cm^2$ Safe

Maximum deflection due to LL = 13 mm

$L/200 = 18\ mm$
Stair Beam

Section Name  200C75-200

Section Dimensions

- H = 200 mm
- B = 75 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties

Steel Type
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties

- A = 7.66 cm²
- Ix = 471.64 cm⁴
- Sx = 47.16 cm³
- Sxe = 47.155 cm³
- Weight = 6.019 kg/m

Applied Straining Actions

- Mx = 0.42 t.m.
- Q = 0.64 ton
- Span = 2.62 m

Check of Stresses

- \( f_b = 0.89068 \) t/cm² < \( F_b = 2.1 \) t/cm²  Safe
- \( \tau = 0.16 \) t/cm² < \( \tau_a = 0.73 \) t/cm²  Safe

Maximum deflection due to LL = 10 mm

L/200 = 26 mm
1. Introduction
In this report the detailed design of 6 story residential building is presented. The building covers an area of 445 m$^2$ (including voids), each floor is divided into 4 flats each of which is 80 m$^2$. Fig.1 shows the typical architectural floor plan of the building. The primary vertical loads are carried by dual system, and the lateral loads are resisted by vertical bracing elements. Dual system composed of rigid frame and vertical studs, Fig. 2, the axial stiffness of the studs can interacts with the bending stiffness of the rigid frame in a manner that minimizing the total weight of steel used. Moreover, dual system provides flexibility in the size and location of any opening.

Fig.1 : Architectural typical floor plan
2. Statical System and structural analysis
The statical system that carries the vertical loads (dead and live loads) consists of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to the dual system. The dual systems are arranged along the vertical axis 1 through 13. Lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The load bearing walls are arranged along axis 1 to 13, while the vertical bracing systems along X-direction are arranged on axis “B”, “G” and “L”. Each axe receives 2 bracing bays. In addition, the vertical bracing systems along Y-direction are arranged on axis “2”, “7”, and “12”, also each axe receives 2 bracing bays.
Fig. 3: Structural plan showing the arrangement of the joists, Dual System and the vertical bracing bays.

3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:

- Beams (joists) are hinged connected to the vertical columns.
- Vertical bracing members are pinned connected to the vertical columns.
- The floor slab moved horizontally in the principle directions as rigid diaphragm.

*Note: SAP output is provided in appendix A*

3. Member Sizing

The following section describes the design of each element. The spread excel sheets for the design of sections are given in appendix “B”.

3.1 JOISTS
Joists are designed as simple beams with variable spans. The maximum span is 5.9 m (between axes 1 & 4). The spacing between joists 120 cm. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 300SG80-200 section was selected. The compression flange of the joists was considered to be continuously braced via attachment of GRC panels.

3.2 Rigid Frame (Columns / Beams)
The rigid frame members are designed to satisfy the interaction equation of axial and bending moments. The critical case of loading was DL + LL. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected for the external frames, while 300SG80-200 sections for the interior one.

3.3 External vertical Studs
The external vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. In addition to the vertical loads, lateral wind loads are considered. Based on this along with the limits that are stated in the design criteria 170SG60-150 section was selected.

3.4 Internal vertical studs
The internal vertical studs are designed as hinged-hinged columns with clear height of 3m. The maximum gravity loads arise from the combination of DL+LL. Due to the presence of sheathing the overall out of plane buckling of the wall is prevented. A horizontal CFS member connecting the studs in the plane of the wall is placed at the mid height of the wall. Based on this along with the limits that are stated in the design criteria 170SG60-150, and 300SG80-200 sections were selected.

3.5 Bracing elements
Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Base reactions developed from the calculated wind load and seismic forces are listed in table 1. These reactions indicate that the wind load is critical than the seismic forces. Thus the bracing elements are designed according to the wind load.

<p>| Table 1: Base reactions due to lateral loads |</p>
<table>
<thead>
<tr>
<th>Applied Load</th>
<th>Base Reaction (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind in X-direction</td>
<td>+36.94/-29.89</td>
</tr>
<tr>
<td>Wind in Y-direction</td>
<td>-/+ 28.391</td>
</tr>
<tr>
<td>Seismic in X-direction</td>
<td>+/- 12.55</td>
</tr>
<tr>
<td>Seismic in Y-direction</td>
<td>+/- 12.55</td>
</tr>
</tbody>
</table>

Axial forces in the vertical members range from 18 ton to -23 ton. Also, axial forces in the diagonal members range from 3.5 ton to -8.87 ton. Based on this along with the limits that are stated in the design criteria square hollow section 200x200x2 section was selected.

Horizontal members in the bracing systems arranged in Y-direction carry mainly the reactions of the joists. Therefore, they are designed as beams with maximum bending moments of 2.26 t.m, and maximum shear force of 1.88 ton. Based on this along with the limits that are stated in the design criteria hollow section 200x220x2 section was selected. However, for members in the bracing systems arranged in Y-direction axial forces are zero, and the maximum bending moments equal to 0.055 t.m. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

### 3.6 Stairs

The statical system of the stairs consists of 4 inclined beams carrying the stairs. These beams are supported on another 2 transverse beams. One in the floor level while the other in the mid floor height level. The maximum bending moments and shear forces in the transverse beams are 0.42 t.m and 0.64 ton, respectively. The compression flange of the joists was considered to be continuously braced via attachment of corrugated sheet decking. Based on this along with the limits that are stated in the design criteria 240C75-225 section was selected.

### 4. Material Quantities

From the above design the following quantities are calculated:

- Own weight of steel elements = 40.732 x 1.1 = 44.803 ton
Fig. 3: Plan of one floor

Horizontal beams (joists) along X-axis, pin connected to the load bearing walls.
Load bearing walls are arranged on axis 1 to 13
Fig. 4: Cross section along axe "D"

Horizontal joists are pin connected to the vertical load bearing walls
Fig. 5: Cross section along axe "G"

Vertical bracing resists lateral loads in the X-direction, and provides lateral stability
Fig. 6: Cross section along axe "7"
Vertical bracing resists lateral loads in the Y-direction, and provides lateral stability
Design of Sections

JOIST DESIGN

Section Name 300SG80-200

Section Dimensions

- H = 300 mm
- B = 80 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties

- Steel Type
  - Fy = 3.6 t/cm²
  - Fu = 5.2 t/cm²

Section Properties

- A = 9.934 cm²
- Iₓ = 1277.3 cm⁴
- Sₓ = 85.153 cm³
- Sₓₑ = 85.153 cm³
- Weight = 7.798 kg/m

Applied Straining Actions

- Mₓ = 1.76 t.m.
- Q = 1.2 ton
- Span = 5.9 m

Check of Stresses

- fₒ = 2.0669 t/cm² < Fₒ = 2.1 t/cm² Safe
- τ = 0.6 t/cm² < τₐ = 0.73 t/cm² Safe

Maximum deflection due to LL = 14.1 mm
L/200 = 29.5 mm
Internal Rigid Frame Columns

Section Name 300SG80-200

Section Dimensions

- H = 300 mm
- B = 80 mm
- D = 25 mm
- t = 2 mm
- r = 4 mm

Steel Properties

- Steel Type: 52
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties

- \( A_f = 9.934 \, \text{cm}^2 \)
- \( A_e = 8.14 \, \text{cm}^2 \)
- \( I_x = 1277.3 \, \text{cm}^4 \)
- \( i_x = 11.339 \, \text{cm} \)
- \( S_x = 85.15 \, \text{cm}^3 \)
- \( S_{xe} = 82.5 \, \text{cm}^3 \)
- \( I_y = 77.318 \, \text{cm}^4 \)
- \( i_y = 2.79 \, \text{cm} \)
- Weight = 7.79 kg/m

Applied Straining Actions

- N = 15 ton
- \( M_x = 10 \, \text{t.cm.} \)
- Q = 0.35 ton
- height = 3 m

Check of Stresses

- \( f_c = 1.84 \, \text{t/cm}^2 \) < \( F_c = 2.1 \, \text{t/cm}^2 \) Safe
- \( f_b = 0.121 \, \text{t/cm}^2 \) < \( F_b = 2.1 \, \text{t/cm}^2 \) Safe
- \( \tau = 0.18 \, \text{t/cm}^2 \) < \( \tau_a = 0.73 \, \text{t/cm}^2 \) Safe

Interaction Check

\[
\frac{f_{ca}}{F_c} + \frac{f_{bex}}{F_{bex}} A_1 + \frac{f_{bey}}{F_{bey}} A_2 = 0.93
\]
External Rigid Frame Columns

Section Name 170SG60-150

Section Dimensions
- H = 170 mm
- B = 60 mm
- D = 20 mm
- t = 1.5 mm
- r = 3 mm

Steel Properties
- Steel Type 52
- Fy = 3.6 t/cm²
- Fu = 5.2 t/cm²

Section Properties
- A_f = 4.97 cm²
- A_e = 4.32 cm²
- I_x = 206.1 cm⁴
- i_x = 6.43 cm
- S_x = 24.4 cm³
- S_{xe} = 24.4 cm³
- I_y = 24.27 cm⁴
- i_y = 2.21 cm
- Weight = 3.9 kg/m

Applied Straining Actions
- N = 6 ton
- M_x = 5 t.cm.
- Q = 0.35 ton
- height = 3 m

Check of Stresses
- f_c = 1.38 t/cm² < F_C = 2.1 t/cm² Safe
- f_b = 0.2 t/cm² < F_b = 2.1 t/cm² Safe
- \tau = 0.18 t/cm² < \tau_a = 0.73 t/cm² Safe

Interaction Check
\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.75
**internal Rigid Frame Beam**

**Section Name** 300SG80-200

**Section Dimensions**

- $H = 300$ mm
- $B = 80$ mm
- $D = 25$ mm
- $t = 2$ mm
- $r = 4$ mm

**Steel Properties**

- Steel Type 52
- $F_y = 3.6$ t/cm²
- $F_u = 5.2$ t/cm²

**Section Properties**

- $A_f = 9.934$ cm²
- $A_e = 8.14$ cm²
- $I_x = 1277.3$ cm⁴
- $i_x = 11.339$ cm
- $S_x = 85.15$ cm³
- $S_{xe} = 82.5$ cm³
- $I_y = 77.318$ cm⁴
- $i_y = 2.79$ cm
- Weight = 7.79 kg/m

**Applied Straining Actions**

- $N = 0$ ton
- $M_x = 120$ t.cm.
- $Q = 0.91$ ton
- height = 3 m

**Check of Stresses**

- $f_c = 0$ t/cm²  $< F_c = 2.1$ t/cm²  Safe
- $f_b = 1.45$ t/cm²  $< F_b = 2.1$ t/cm²  Safe
- $\tau = 0.33$ t/cm²  $< \tau_a = 0.73$ t/cm²  Safe

**Interaction Check**

$$\frac{f_{ca}}{F_c} + \frac{f_{bx}}{F_{bx}} A_i + \frac{f_{by}}{F_{by}} A_2 = 0.69$$
External Rigid Frame Beam

Section Name 170SG60-150

Section Dimensions

- $H = 170$ mm
- $B = 60$ mm
- $D = 20$ mm
- $t = 1.5$ mm
- $r = 3$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm²
- $F_u = 5.2$ t/cm²

Section Properties

- $A_f = 4.97$ cm²
- $A_e = 4.32$ cm²
- $I_x = 206.1$ cm⁴
- $i_x = 6.43$ cm
- $S_x = 24.4$ cm³
- $S_{xe} = 24.4$ cm³
- $I_y = 24.27$ cm⁴
- $i_y = 2.21$ cm
- Weight = 3.9 kg/m

Applied Straining Actions

- $N = 0$ ton
- $M_x = 19.8$ t.cm.
- $Q = 0.91$ ton
- height = 3 m

Check of Stresses

- $f_c = 0$ t/cm² < $F_C = 2.1$ t/cm² Safe
- $f_b = 0.81$ t/cm² < $F_b = 2.1$ t/cm² Safe
- $\tau = 0.33$ t/cm² < $\tau_a = 0.73$ t/cm² Safe

Interaction Check

$$\frac{f_{ca}}{F_c} + \frac{f_{bex}}{F_{bex}} A_1 + \frac{f_{bey}}{F_{bey}} A_2 = 0.386$$
External Stud

Section Name 170SG60-150

Section Dimensions

- $H = 170$ mm
- $B = 60$ mm
- $D = 20$ mm
- $t = 1.5$ mm
- $r = 3$ mm

Steel Properties

Steel Type 52

- $F_{y} = 3.6$ t/cm²
- $F_{u} = 5.2$ t/cm²

Section Properties

- $A_{f} = 4.97$ cm²
- $A_{e} = 4.32$ cm²
- $I_{x} = 206.1$ cm⁴
- $i_{x} = 6.43$ cm
- $S_{x} = 24.4$ cm³
- $S_{xe} = 24.4$ cm³
- $I_{y} = 24.27$ cm⁴
- $i_{y} = 2.21$ cm
- Weight = 3.9 kg/m

Applied Straining Actions

- $N = 5.84$ ton
- $M_{x} = 4.5$ t.cm.
- $Q = 0.2$ ton
- height = 3 m

Check of Stresses

- $f_{c} = 1.35$ t/cm² < $F_{C} = 2.1$ t/cm² Safe
- $f_{b} = 0.18$ t/cm² < $F_{b} = 2.1$ t/cm² Safe
- $\tau = 0.18$ t/cm² < $\tau_{a} = 0.73$ t/cm² Safe

Interaction Check

$$\frac{f_{c}}{F_{c}} + \frac{f_{b,cx}}{F_{b,cx}} A_{1} + \frac{f_{b,cy}}{F_{b,cy}} A_{2} = 0.73$$
Internal Stud 1

Section Name 170SG60-150

Section Dimensions

- \( H = 170 \text{ mm} \)
- \( B = 60 \text{ mm} \)
- \( D = 20 \text{ mm} \)
- \( t = 1.5 \text{ mm} \)
- \( r = 3 \text{ mm} \)

Steel Properties

- Steel Type 52
- \( F_y = 3.6 \text{ t/cm}^2 \)
- \( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

- \( A_f = 4.97 \text{ cm}^2 \)
- \( A_e = 4.32 \text{ cm}^2 \)
- \( I_x = 206.1 \text{ cm}^4 \)
- \( i_x = 6.43 \text{ cm} \)
- \( S_x = 24.4 \text{ cm}^3 \)
- \( S_{xe} = 24.4 \text{ cm}^3 \)
- \( I_y = 24.27 \text{ cm}^4 \)
- \( i_y = 2.21 \text{ cm} \)
- Weight = 3.9 kg/m

Applied Straining Actions

- \( N = 7.5 \text{ ton} \)
- \( M_x = 0 \text{ t.cm.} \)
- \( Q = 0 \text{ ton} \)
- height = 3 m

Check of Stresses

- \( f_c = 1.736 \text{ t/cm}^2 \) < \( F_c = 2.1 \text{ t/cm}^2 \) Safe
- \( f_b = 0 \text{ t/cm}^2 \) < \( F_b = 2.1 \text{ t/cm}^2 \) Safe
- \( \tau = 0 \text{ t/cm}^2 \) < \( \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check

\[ \frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{b,cx}} A_1 + \frac{f_{bcx}}{F_{b,cx}} A_2 = 0.826 \]
Internal Stud 2

Section Name 300SG80-200

Section Dimensions

- $H = 200$ mm
- $B = 80$ mm
- $D = 25$ mm
- $t = 2$ mm
- $r = 4$ mm

Steel Properties

- Steel Type 52
- $F_y = 3.6$ t/cm²
- $F_u = 5.2$ t/cm²

Section Properties

- $A_f = 9.934$ cm²
- $A_e = 8.14$ cm²
- $I_x = 1277.3$ cm⁴
- $i_x = 11.339$ cm
- $S_x = 85.15$ cm³
- $S_{xe} = 82.5$ cm³
- $I_y = 77.318$ cm⁴
- $i_y = 2.79$ cm
- Weight = 7.79 kg/m

Applied Straining Actions

- $N = 14.2$ ton
- $M_x = 0$ t.cm.
- $Q = 0$ ton
- Height = 3 m

Check of Stresses

- $f_c = 1.74$ t/cm² < $F_c = 2.1$ t/cm² Safe
- $f_b = 0$ t/cm² < $F_b = 2.1$ t/cm² Safe
- $\tau = 0$ t/cm² < $\tau_a = 0.73$ t/cm² Safe

Interaction Check

$$\frac{f_{ca}}{F_c} + \frac{f_{b,cx}}{F_{b,cx}} A_1 + \frac{f_{b,cy}}{F_{b,cy}} A_2 = 0.83$$
Vertical and Diagonal Bracing elements

Section Name: 200x200x2

Section Dimensions

H = 200 mm  
B = 200 mm  
t = 2 mm

Steel Properties

Steel Type: 52  
\( F_y = 3.6 \text{ t/cm}^2 \)  
\( F_u = 5.2 \text{ t/cm}^2 \)

Section Properties

\( A_e = 12.9 \text{ cm}^2 \)  
\( I_x = 1035.1 \text{ cm}^4 \)  
\( i_x = 8.08 \text{ cm} \)  
\( S_x = 103.5 \text{ cm}^3 \)  
\( I_y = 1035.1 \text{ cm}^4 \)  
\( i_y = 8.08 \text{ cm} \)

Weight = 12.43 kg/m

Applied Straining Actions

\( N = 23 \text{ ton} \)  
\( M_x = 0 \text{ t.m. height = 3 m} \)  
\( Q = 0 \text{ ton} \)  
\( k_x = 1 \)  
\( k_y = 1 \)  
\( L_x = 300 \text{ cm} \)  
\( L_y = 300 \text{ cm} \)  
\( k_x L_x / i_x = 37.128 \)  
\( k_y L_y / i_y = 37.128 \)

Check of Stresses

\( f_c = 1.78 \text{ t/cm}^2 < F_c = 1.91 \text{ t/cm}^2 \) Safe  
\( f_y = 0 \text{ t/cm}^2 < F_y = 2.1 \text{ t/cm}^2 \) Safe  
\( \tau = 0 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \) Safe

Interaction Check

\[
\frac{f_{ec}}{F_c} + \frac{f_{bcx}}{F_{bcx}} A_1 + \frac{f_{bcy}}{F_{bcy}} A_2 = 0.933
\]
Horizontal bracing member

Section Name 200x220x2

Section Dimensions

\[
\begin{align*}
H &= 200 \text{ mm} \\
B &= 220 \text{ mm} \\
t &= 2 \text{ mm}
\end{align*}
\]

Steel Properties

Steel Type 52

\[
\begin{align*}
F_y &= 3.6 \text{ t/cm}^2 \\
F_u &= 5.2 \text{ t/cm}^2
\end{align*}
\]

Section Properties

\[
\begin{align*}
A_e &= 14.14 \text{ cm}^2 \\
I_x &= 1286.4 \text{ cm}^4 \\
ix &= 8.79 \text{ cm} \\
S_x &= 116.94 \text{ cm}^3 \\
l_y &= 1113.5 \text{ cm}^4 \\
i_y &= 8.18 \text{ cm}
\end{align*}
\]

Weight = 13.062 kg/m

Applied Straining Actions

\[
\begin{align*}
N &= 0 \text{ ton} \\
M_x &= 2.26 \text{ t.m.} \\
Q &= 1.88 \text{ ton}
\end{align*}
\]

Span = 3.6 m

Check of Stresses

\[
\begin{align*}
f_C &= 0 \text{ t/cm}^2 < F_C = 1.29 \text{ t/cm}^2 \quad \text{Safe} \\
f_b &= 1.93 \text{ t/cm}^2 < F_b = 2.1 \text{ t/cm}^2 \quad \text{Safe} \\
\tau &= 0.24 \text{ t/cm}^2 < \tau_a = 0.73 \text{ t/cm}^2 \quad \text{Safe}
\end{align*}
\]

Maximum deflection due to LL = 13 \text{ mm}

\[
L/200 = 18 \text{ mm}
\]
Stair Beam

Section Name 200C75-200

Section Dimensions

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>200 mm</td>
</tr>
<tr>
<td>B</td>
<td>75 mm</td>
</tr>
<tr>
<td>D</td>
<td>25 mm</td>
</tr>
<tr>
<td>t</td>
<td>2 mm</td>
</tr>
<tr>
<td>r</td>
<td>4 mm</td>
</tr>
</tbody>
</table>

Steel Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Type</td>
<td></td>
</tr>
<tr>
<td>Fy (t/cm²)</td>
<td>3.6</td>
</tr>
<tr>
<td>Fu (t/cm²)</td>
<td>5.2</td>
</tr>
</tbody>
</table>

Section Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (cm²)</td>
<td>7.66</td>
</tr>
<tr>
<td>Iₓ (cm⁴)</td>
<td>471.64</td>
</tr>
<tr>
<td>Sₓ (cm³)</td>
<td>47.16</td>
</tr>
<tr>
<td>Sₓₑ (cm³)</td>
<td>47.155</td>
</tr>
<tr>
<td>Weight (kg/m)</td>
<td>6.019</td>
</tr>
</tbody>
</table>

Applied Straining Actions

<table>
<thead>
<tr>
<th>Action</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mₓ (t.m.)</td>
<td>0.42</td>
</tr>
<tr>
<td>Q (ton)</td>
<td>0.64</td>
</tr>
<tr>
<td>Span (m)</td>
<td>2.62</td>
</tr>
</tbody>
</table>

Check of Stresses

<table>
<thead>
<tr>
<th>Stress</th>
<th>Value (t/cm²)</th>
<th>Comparison</th>
<th>Fb (t/cm²)</th>
<th>Safe</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_b</td>
<td>0.89068</td>
<td>&lt;</td>
<td>2.1</td>
<td>Safe</td>
</tr>
<tr>
<td>τ_a</td>
<td>0.16</td>
<td>&lt;</td>
<td>0.73</td>
<td>Safe</td>
</tr>
</tbody>
</table>

Maximum deflection due to LL = 10 mm
L/200 = 26 mm
Appendix B

Research Papers

Appendix B1_ Paper on Cold Formed Members (AISC: NASCC/SSRC Conference 2013)

Appendix B2_ Paper on fire of Cold Formed Steel (ASCE Structure Congress 2013)

Appendix B3_Paper on Novel Systems Design (ICCSEE 2013)

Appendix B4_ Paper on Cost and Sustainability Analysis (ICCSEE 2013)
September 22, 2012

Subject: Abstract Submittals for the 2013 SSRC Annual Stability Conference

Dear Metwally Abu-Hamd:

We are pleased to inform you that the SSRC Program Committee has accepted your abstract submission for presentation at the SSRC 2013 Annual Stability Conference in St. Louis, Missouri.

Title of Abstract: Buckling Strength of Axially Loaded Cold Formed Built-Up I-Sections

Authors: Metwally Abu-Hamd

Please email SSRC Headquarters (ssrc@mst.edu) no later than Friday, October 12th 2012 to confirm your willingness to prepare a paper, attend, and present at the conference. The preliminary program is attached.

The allotted time for each paper (including time for questions and discussion) is 20 minutes. The presenting author for all papers will have their conference registration fees waived, and will be eligible for reimbursement for partial travel costs, not to exceed $400.00 USD. A reimbursement form and guidelines for reimbursement are attached.

Written manuscripts are required for all presentations, instructions are attached. The deadline for submission of full manuscripts is January 21, 2013.

Please note, all papers submitted for the Vinnakota Award must be presented by the student and must be submitted by the deadline to qualify for consideration.

Thank you for your participation in the 2013 Annual Stability Conference. We look forward to seeing you on Tuesday April 16th at the SSRC Annual Meeting, and in the SSRC Track at the AISC-NASCC Conference April 17th through the 19th.

Sincerely,

Todd A. Helwig, SSRC Program Committee Chair

Chair
Prof. Ronald D. Ziemian
Bucknell University
Department of Civil Engineering
Lewisburg, PA 24061
Ph. 570-577-1784  FAX: 570-577-3415
ziemian@bucknell.edu

Vice-Chair
Prof. Benjamin W. Schafer
Johns Hopkins University
Department of Civil Engineering
Baltimore, MD 21218
Ph. 410-516-7801  FAX: 410-516-7473
schafer@jhu.edu

Missouri S&T Faculty Liaison
Dr. Roger A. LaBoube
laboube@mst.edu

Administrative Secretary
Christina Stratman
ssrc@mst.edu

Headquarters: Missouri University of Science and Technology, 1401 N. Pine St., 301 Butler-Carlton Hall, Rolla, MO 65409-0030 Phone: 573-341-6610  FAX: 573-341-4476  Web: http://stabilitycouncil.org
Incorporating the SSRC Annual Stability Conference
and the Technology in Steel Construction Conference

AMERICA’S CENTER
CONVENTION COMPLEX

St. Louis Missouri

NASCC
THE STEEL CONFERENCE

sponsored by

www.aisc.org/nascc
April 17–19 2013
### Stability Under Fire Conditions

**S1** W 3:15 p.m.– 4:15 p.m.  
Room 276  
1.0 PDHs  
**Moderator:** Ronald Ziemian, Bucknell University

- Welcome to the 2013 SSRC Annual Stability Conference  
  R. Ziemian, Bucknell University, Lewisburg, PA
- Performance of Steel Shear Tab Connections at Elevated Temperatures  
  M.S. Seif, J.A. Main and T.P. McAllister, National Institute of Standards and Technology, Gaithersburg, MD
- Stability of Cold-Formed Steel Compression Members Under Thermal Gradients  
  J.C. Batista-Abreu and B. W. Schafer, Johns Hopkins University, Baltimore, MD

### Stability of Frames and Systems

**S3** Th 8:00 a.m.– 9:30 a.m.  
Room 276  
1.5 PDHs  
**Moderator:** Dinar Camotim, Technical University of Lisbon

- On Frame Stability Analysis  
  A.S. Doria, Petrobras, Brazil; M. Malite, University of Sao Paulo, Sao Paulo, Brazil; L.C.M. Vieira, Jr., University of New Haven, West Haven, CT
- Experimental and Analytical Study on Failure Modes of Structural Steel Scaffolds  
  Maheeb M.E. Abdel-Ghaffar, Abdullah N.S. Mahmoud, Cairo University, Cairo, Egypt
- Analysis of Locally/Distortional Buckled Beams  
  Xi Zhang and Kim J.R. Rasmussen, University of Sydney, Sydney, Australia
- System Reliability of Steel Frames Designed by Inelastic Analysis  
  Shabnam Shayan, Kim J.R. Rasmussen and Hao Zhang, University of Sydney, Sydney, Australia

### Cold-Formed Steel Member Stability

**S4** Th 10:00 a.m.– 11:30 a.m.  
Room 276  
1.5 PDHs  
**Moderator:** Roger LaBoube, Missouri University of Science and Technology

- Buckling Strength of Axially Loaded Cold-Formed Built-Up I-Sections  
  Metwally Abu-Hamm, Basel El-Samman, Cairo University, Giza, Egypt
- Elastic Buckling of Thin-Walled Steel Columns with Periodic Perforations  
  F.H. Smith and C.D. Moen, Virginia Tech, Blacksburg, VA
- Distortional Post-Buckling Strength of Cold-Formed Steel Columns: How does the Cross-Section Geometry Affect it?  
  Alexandre Landesmann, Federal University of Rio de Janeiro, Rio de Janeiro, Brazil; Dinar Camotim, Technical University of Lisbon, Lisbon, Portugal; Cilmar Basaglia, University of Sao Paulo, Sao Paulo, Brazil
- Shape Optimization of Cold-Formed Steel Columns with Manufacturing Constraints and Limited Number of Rollers  
  J. Leng, Z. Li, J.K. Guest and B.W. Schafer, Johns Hopkins University, Baltimore, MD
Buckling Strength of Axially Loaded Cold Formed Built-Up I-Sections

Mtwally Abu-Hamd¹ and Basel El-Samman²

ABSTRACT

This paper presents a numerical procedure using finite element analysis for the calculation of axial strength of cold formed steel built-up I-sections composed of two back-to-back channels. The material nonlinearity of the flat and corner portions of the section were incorporated in the model. The effects of initial local and overall geometric imperfections as well as the membrane residual stresses have been taken into consideration in the finite element model. The results of the nonlinear finite element analysis were compared with the available experimental results, and with the calculated theoretical buckling capacities based on the AISI design provisions. A parametric study was carried out using the developed finite element model to study the effects of member and cross-section geometries and imperfection values on the strength of cold-formed steel built-up I-columns. The column strengths predicted from the parametric study were compared with the design strengths calculated using the American Specification. The results of the parametric study showed that the design provisions specified in the American Specifications are generally conservative for long and medium length columns, but may give un-conservative estimates for some of the short columns.

1. Introduction

Cold-formed steel members are widely used in building construction, such as wall studs, floor joists, truss members and other structural applications. Cold-formed steel sections are usually formed in single C, Z, and hat sections. The cross sections of these members can be also formed by connecting two or more sections together, for examples, an I-section formed by connecting two channel sections back-to-back, and a box section formed by connecting two channel sections in the flanges.

Axially loaded cold formed members may fail by global, local and/or distortional buckling due to their high plate width-to-thickness ratio. Flexural buckling tends to occur in slender members due to global geometric imperfections, Fig. 1. As the slenderness ratio becomes smaller, geometric local imperfections cause the failure to become more localized as in a thin plate subjected to an in-plane membrane stress, resulting in a transition from global buckling to local and/or distortional buckling, Fig. 2.

¹Professor, Faculty of Engineering, Cairo University, Egypt, abuhamd@eng.cu.edu.eg
²Ph. D. Candidate, Faculty of Engineering, Cairo University, Egypt.
The design provisions for cold-formed sections under pure compression are stated in Sections C4 of the 2007 North American Specification for the Design of Cold-Formed Steel Structural Members; AISI (2007). The nominal axial strength $P_n$ is taken as follows:

$$P_n = \text{smaller of } (P_{ne}, P_{nd})$$  \hspace{1cm} (1)

Where,

- $P_{ne} =$ nominal strength for yielding, flexural, flexural-torsional, and torsional buckling according to section C4.1,
- $P_{nd} =$ nominal distortional buckling strength according to section C4.2.

Note that flexural-torsional buckling does not occur in most cases of doubly-symmetric built-up members with a sufficient number of intermediate, symmetrical fasteners.
Similarly, distortional buckling may be present only for sections having exceptionally wide flanges as studied by Piyawat (2011).

Alternatively, Appendix 1 of the AISI Specification presents a different design procedure based on the direct strength method (DSM), Schafer (2008). According to this method the nominal axial strength $P_n$ is taken as follows:

$$P_n = \text{minimum of } (P_{ne}, P_{nl}, P_{nd})$$  \hspace{1cm} (2)

Where,

$P_{ne} = \text{nominal strength for yielding, flexural, flexural-torsional, and torsional buckling according to section 1.2.1.1,}$

$P_{nl} = \text{nominal local buckling strength according to section 1.2.1.2.}$

$P_{nd} = \text{nominal distortional buckling strength according to section 1.2.1.3.}$

The direct strength method relies on the calculation of elastic buckling stresses from a rational elastic buckling analysis buckling analysis such as finite strip method, see Schafer (2010), or using finite element methods. The DSM method is highly favorable than the traditional effective method because it does not require the calculation of the effective width for cross-section. In addition, the DSM uses realistic estimates of the local and global buckling loads based on consideration of the entire cross section rather than considering individual elements. However, the method has not been calibrated for built-up I-shaped members.

Built up sections formed by connecting two components are subjected to shear-induced relative deformations between the combined components. Furthermore, built-up members may buckle globally either as one single component or as one combined section. For these reasons, additional specific design provisions for built-up compression members are stated in Section D1.2 of the AISI specifications.

If the buckling mode produces shear forces in the connectors between the members, the slenderness ratio $(KL/r)$ used to calculate the elastic buckling stress should be replaced with a modified value $(KL/r)_m$ calculated from:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2}$$  \hspace{1cm} (3)

where $(KL/r)_m$ is the overall modified slenderness ratio of the entire section about the built-up member axis; $(KL/r)_o$ is the overall (unmodified) slenderness ratio of the entire section about the built-up member axis; $a$ is the longitudinal spacing between intermediate fastener or spot welds connecting the two components; and $r_i$ is the minimum radius of gyration of the full unreduced cross-section of the individual component. Furthermore, Section D1.2 of AISI-2007 provides the requirements for the fastener or stitch weld strength and spacing. First, the ratio $(a/r_i)$ is not to exceed 0.5$(KL/r)_o$. Second, the member-end connectors or weld should have a certain length. Lastly, the intermediate fasteners or welds at any longitudinal member tie location should have a transmitting force of 0.025 of the nominal axial capacity in any direction.
The additional provisions stated in Section D1.2 are similar to those used in the AISC Specifications (2010). They are based on research conducted on hot rolled sections where the member axial strength is governed mainly by global buckling and rarely influenced by local buckling. Cold formed members, because of their high plate width-to-thickness ratios, may fail by local and/or distortional buckling at stress levels much lower than the corresponding global buckling stresses.

At the same time, limited test data are available on cold formed built-up I-sections. Stone and LaBoube’s (2005) tested some cold-formed, built-up I-sections constructed from steel studs. The members tested were cold formed C-channels intermittently connected back to back with screws to model a typical, cold-formed, I-shaped wall stud. Additional work of cold-formed built-up C-channels was conducted by Brueggen and Ramseyer (2003), Whittle (2007) and Biggs (2008) on smaller c-channels in open and closed-sections with intermediate welded stitch attachments. Their research on the built-up stub columns concluded that the AISI design method is conservative for compact members but often un-conservative for members with slender elements. Brueggen and Ramseyer (2003) recommended that additional tests be performed to determine the effects of length and location (double-or single-sided), spacing, and number of weld attachments on the behavior of welded built-up members. Piyawat (2011) studied the axial capacity of cold formed built-up I-columns with exceptionally wide flanges where distortional buckling may govern the design.

The previous review of current design provisions and available test results shows that there is a need to investigate the appropriateness of the current design rules for cold formed built-up I-shaped members.

The main objective of this paper is to develop a numerical procedure that can be used to calculate the axial capacity of cold-formed built-up I-section using finite element method. The finite element program ANSYS (2010) was used in the analysis. The results obtained from the numerical analysis were first compared with some available test results. A parametric study was then performed to investigate the effect of cross-section geometry and geometric imperfections on the strength of these sections. The results obtained from the parametric study were compared with the design strengths calculated using the AISI provisions.

2. Numerical Analysis

The finite element method has proven to be a very successful tool for calculating the post buckling capacity of cold formed steel members. The geometrical and material non-linear behavior present in such a case requires two types of analyses. The first type of analysis is an eigenvalue analysis that estimates the buckling modes and buckling frequencies as the solution to an eigenvalue problem. In this problem the material behavior is assumed to be elastic and the member is assumed to have perfect geometry. The lowest buckling modes predicted from the eigenvalue analysis are subsequently used to model the geometric imperfections. The second type of analysis is a nonlinear load–displacement analysis of the real member under the action of applied loads in the presence of initial geometrical imperfections, residual stresses and material nonlinearity. The ultimate loads and failure modes are determined from this analysis when it reaches a limit point located on its equilibrium bath; the corresponding load parameter value and deformed configuration
provide the member ultimate strength and failure mode, respectively. The finite element program ANSYS was used in the present study to model the cold-formed steel columns as described in the next section.

2.1 Finite Element Model

The member was modeled using the 4-node finite strain shell element, shell 181, built in ANSYS element library. This element accounts for six degrees of freedom per node and allows for stress stiffening, large deformation, as well as material non-linearity. It is well suited for linear, large rotation, and/or large strain nonlinear applications. In order to choose the finite element mesh that provides accurate results with minimum computational time, convergence studies were conducted. It is found that the mesh size of 25 mm×10 mm (length by width) provides adequate accuracy and minimum computational time in modeling the flat portions, while a finer mesh was used at the corners as shown in Fig. 3.

![Figure 3 Finite Element Model](image)

The material behavior provided by ANSYS allows for a multi-linear stress–strain curve to be used. The first part of the multi-linear curve represents the elastic part up to the proportional limit stress with a known Young’s modulus, taken equal to 203000 MPa in the present study, and Poisson’s ratio, taken as 0.3. Von-Mises yield criteria with isotropic hardening was used.

2.2 Modeling Geometric Imperfections

Cold-formed members always contain initial geometric imperfection during their fabrication either by cold rolling or press braking. Geometric imperfections may be classified into two categories; global imperfections along the member length and local
imperfections of the cross section. Typically $L/1000$ is used as the magnitude and a global buckling mode shape, see Fig. 1, is used as the distribution shape to approximate global imperfections, where $L$ = member length. The common approach in considering local cross-sectional imperfections is to use a portion of the thickness of the member as the magnitude and the local and distortional buckling mode shapes, see Fig. 2, as the distribution of these imperfections (Schafer and Pekoz (1998)). Examples of available imperfection measurements categorized into cross-sectional (local and distortional) and global (Bow or weak axis flexure, Camber or strong axis flexure and twist) are given in Zeinoddini and Schafer (2012) as shown in Table 1.

Table 1: Statistical summary of available data on imperfections
(Zeinoddini and Schafer (2012))

<table>
<thead>
<tr>
<th></th>
<th>Local $L$ ($\delta_0/t$)</th>
<th>Distortional $D$ ($\delta_0/t$)</th>
<th>Bow $G_1$ ($L/\delta_0$)</th>
<th>Camber $G_2$ ($L/\delta_0$)</th>
<th>Twist $G_3$ (deg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean</td>
<td>0.47</td>
<td>1.03</td>
<td>2242</td>
<td>3477</td>
<td>0.36</td>
</tr>
<tr>
<td>st.dev.</td>
<td>0.62</td>
<td>0.97</td>
<td>3054</td>
<td>5643</td>
<td>0.23</td>
</tr>
<tr>
<td>25 %ile</td>
<td>0.17</td>
<td>0.43</td>
<td>4755</td>
<td>6295</td>
<td>0.20</td>
</tr>
<tr>
<td>50 %ile</td>
<td>0.31</td>
<td>0.75</td>
<td>2909</td>
<td>4010</td>
<td>0.30</td>
</tr>
<tr>
<td>75 %ile</td>
<td>0.54</td>
<td>1.14</td>
<td>1659</td>
<td>2887</td>
<td>0.49</td>
</tr>
<tr>
<td>95 %ile</td>
<td>1.02</td>
<td>3.06</td>
<td>845</td>
<td>1472</td>
<td>0.85</td>
</tr>
<tr>
<td>99 %ile</td>
<td>3.87</td>
<td>4.46</td>
<td>753</td>
<td>1215</td>
<td>0.95</td>
</tr>
</tbody>
</table>

(*) %ile values are the probabilities that imperfection will be less than the table value

Different mode shape imperfections need to be combined in a proper way. In the traditional modal approach, imperfections are modeled as a linear combination of the first buckling modes using a suitable magnitude for each mode (can be chosen from Table 1, or traditionally chosen as $1/1000$ of length or 10% of thickness). In this paper, $1/1000$ of length and 10% of thickness were used to compare test results with AISI results. Values at 25 %ile and 75 %ile, as recommended by Zeinoddini and Schafer (2012), were used in a sensitivity analysis in the parametric study.

2.3 Modeling of Residual Stresses

Cold formed members always contain residual stresses during their manufacturing process. Coiling, uncoiling, cold bending to shape, and straightening of the formed member lead to a complicated set of initial stresses and strains in the section. Residual stresses may be idealized as a summation of two types, flexural and membrane (Schafer (1998)). Some statistical results for membrane residual stresses are reported in Schafer (1998). The data shows that membrane residual stresses exist primarily in corner regions and their values may reach about 8% $F_y$ at corners and about 4% $F_y$ for flat parts. Opposing this effect, the yield stress $F_y$ is increased at corner regions by about 15% $F_y$ due to cold work of forming as shown by Abdel-Rahman, (1997). On the other hand, measured flexural residual stresses show a large degree of variation. Statistics for flexural residual stress are reported in Schafer (1998) and Meon (2008). Considering these stresses
in the finite element model complicates the analysis considerably as it requires defining the through thickness stresses for each layer. Furthermore, Meon (2008) suggested using kinematic hardening rule instead of isotropic hardening. As the main interest in this paper is to find the ultimate axial load capacity, the present analysis neglects the effect of flexural residual stresses. This assumption would not be correct when considering the deformation behavior and stress distribution across the section. In the present model the effect of membrane residual stresses of the stated representative values on the axial buckling capacity of cold formed built-up members was studied by using different values of the yield strength for corner regions and for flat regions. It was found that the effect on axial buckling strength was less than 1%.

2.4 Boundary conditions and load application

Both member ends of the columns were modeled as hinged ends allowing bending rotation but prevented from translation and twist except for the displacement at the loaded end in the direction of the applied load. The nodes other than the two ends were free to translate and rotate in any directions. The displacements of the two components forming the cross section were coupled at the locations of the connecting screws. The load was applied as an axial concentrated load at the section centeroid at the loaded end.

2.5 Solution Methods

Elastic buckling analysis was carried out using Lanczos solver as recommended by the software. The nonlinear analysis was conducted using Newton-Raphson method with automatic load sub steps determined by the software.

3. Comparison with Test Results

The finite element model developed in the previous section was used to calculate the axial load capacity of 32 cold-formed columns tested by Stone and Laboube (2005). The values of geometrical global and local imperfection were taken as L/1000 and 0.1*t, respectively. The comparison of the ultimate loads (P_{test}/P_y and P_{FE}/P_y) obtained experimentally and numerically is shown in Table 2. The table contains also the results obtained by applying the current AISI provisions of sections C4.1 and D 1.2. Deviations between test results, finite element results and AISI results are shown in Table 3. The results are also plotted in Fig. 4 to to Fig.7 for the four cross sections tested.

Investigation of these results shows that considerable discrepancies exist, but the following conclusions may be stated:

1- Finite element results deviate from the test results by an average of 10.6 % on the conservative side, while the results based on the AISI provisions deviate by an average of 27.1 % on the conservative side.

2- The deviations increase for sections with large plate depth to thickness ratio and decreases for members with small slenderness ratio.
<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Thickness (mm)</th>
<th>Screw spacing, a (mm)</th>
<th>$P_{\text{test}}/P_y$</th>
<th>$P_{\text{FE}}/P_y$</th>
<th>$P_{\text{AISI}}/P_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>304.8</td>
<td>0.317</td>
<td>0.264</td>
<td>0.219</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>609.6</td>
<td>0.326</td>
<td>0.252</td>
<td>0.204</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>609.6</td>
<td>0.303</td>
<td>0.252</td>
<td>0.204</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>609.6</td>
<td>0.319</td>
<td>0.252</td>
<td>0.204</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>762.0</td>
<td>0.291</td>
<td>0.252</td>
<td>0.195</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>762.0</td>
<td>0.308</td>
<td>0.252</td>
<td>0.195</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>762.0</td>
<td>0.341</td>
<td>0.252</td>
<td>0.195</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>914.4</td>
<td>0.289</td>
<td>0.248</td>
<td>0.184</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>914.4</td>
<td>0.253</td>
<td>0.248</td>
<td>0.184</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>1016.0</td>
<td>0.312</td>
<td>0.247</td>
<td>0.177</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>1066.8</td>
<td>0.314</td>
<td>0.247</td>
<td>0.173</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>304.8</td>
<td>0.444</td>
<td>0.447</td>
<td>0.421</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>304.8</td>
<td>0.538</td>
<td>0.447</td>
<td>0.421</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>609.6</td>
<td>0.412</td>
<td>0.432</td>
<td>0.392</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>609.6</td>
<td>0.411</td>
<td>0.432</td>
<td>0.392</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>914.4</td>
<td>0.382</td>
<td>0.42</td>
<td>0.348</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>914.4</td>
<td>0.450</td>
<td>0.42</td>
<td>0.348</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>0.657</td>
<td>0.549</td>
<td>0.497</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>0.579</td>
<td>0.549</td>
<td>0.497</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>0.421</td>
<td>0.549</td>
<td>0.497</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>0.521</td>
<td>0.549</td>
<td>0.497</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>609.6</td>
<td>0.564</td>
<td>0.528</td>
<td>0.474</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>609.6</td>
<td>0.650</td>
<td>0.528</td>
<td>0.474</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>914.4</td>
<td>0.564</td>
<td>0.523</td>
<td>0.437</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>914.4</td>
<td>0.631</td>
<td>0.523</td>
<td>0.437</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>304.8</td>
<td>0.310</td>
<td>0.327</td>
<td>0.256</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>304.8</td>
<td>0.362</td>
<td>0.327</td>
<td>0.256</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>609.6</td>
<td>0.415</td>
<td>0.319</td>
<td>0.239</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>609.6</td>
<td>0.351</td>
<td>0.319</td>
<td>0.239</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>914.4</td>
<td>0.303</td>
<td>0.304</td>
<td>0.215</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>914.4</td>
<td>0.335</td>
<td>0.304</td>
<td>0.215</td>
</tr>
</tbody>
</table>
Table 3: Deviations from Test and AISI Results

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Thickness mm</th>
<th>Screw spacing, a (mm)</th>
<th>% Deviation Test vs FE</th>
<th>% Deviation Test vs AISI</th>
</tr>
</thead>
<tbody>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>304.8</td>
<td>16.7</td>
<td>30.8</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>609.6</td>
<td>22.8</td>
<td>37.3</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>609.6</td>
<td>16.8</td>
<td>32.5</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>609.6</td>
<td>21.1</td>
<td>36.0</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>762.0</td>
<td>13.5</td>
<td>33.2</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>762.0</td>
<td>18.2</td>
<td>36.8</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>762.0</td>
<td>26.1</td>
<td>42.9</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>914.4</td>
<td>14.4</td>
<td>36.3</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>914.4</td>
<td>2.2</td>
<td>27.2</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>1016.0</td>
<td>20.9</td>
<td>43.4</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>1066.8</td>
<td>20.9</td>
<td>44.6</td>
</tr>
<tr>
<td>152.4</td>
<td>1.372</td>
<td>1066.8</td>
<td>21.3</td>
<td>44.9</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>304.8</td>
<td>-0.6</td>
<td>5.4</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>304.8</td>
<td>16.9</td>
<td>21.8</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>609.6</td>
<td>-4.9</td>
<td>4.9</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>609.6</td>
<td>-5.1</td>
<td>4.7</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>914.4</td>
<td>-9.9</td>
<td>9.0</td>
</tr>
<tr>
<td>92.1</td>
<td>1.155</td>
<td>914.4</td>
<td>6.6</td>
<td>22.7</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>16.5</td>
<td>24.4</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>5.1</td>
<td>14.1</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>-30.3</td>
<td>-18.0</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>304.8</td>
<td>-5.5</td>
<td>4.5</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>609.6</td>
<td>6.3</td>
<td>15.9</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>609.6</td>
<td>18.8</td>
<td>27.1</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>914.4</td>
<td>7.2</td>
<td>22.4</td>
</tr>
<tr>
<td>92.1</td>
<td>0.880</td>
<td>914.4</td>
<td>17.2</td>
<td>30.8</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>304.8</td>
<td>-5.6</td>
<td>17.3</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>304.8</td>
<td>9.7</td>
<td>29.3</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>609.6</td>
<td>23.2</td>
<td>42.5</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>609.6</td>
<td>9.1</td>
<td>32.1</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>914.4</td>
<td>-0.2</td>
<td>29.3</td>
</tr>
<tr>
<td>152.4</td>
<td>0.841</td>
<td>914.4</td>
<td>9.2</td>
<td>35.9</td>
</tr>
</tbody>
</table>
Figure 4  Comparison of Axial Strengths Results for Section 152x1.372

Figure 5  Comparison of Axial Strengths Results for Section 92x1.155
Figure 6  Comparison of Axial Strengths Results for Section 92x0.88

Figure 7  Comparison of Axial Strengths Results for Section 92x0.88
4. Parametric study

The comparison presented in the previous section was limited to the available test results which are representative of columns having relatively large overall slenderness ratios; \( \lambda_c \) between 1.1 and 2. In order to study the behavior over a wider range of cross-sections, the developed finite element model was used to conduct a parametric study to investigate the effect of the following design parameters:

1. Variation in the overall slenderness ratio \( \lambda_c \).
2. Variation in the local width-to-thickness ratios of the web and the flange.
3. Variations in the amplitude of geometric imperfection.

For the first design parameter, members having slenderness ratio \( \lambda_c \) between 0.5 and 2.5 were investigated. For the second design parameter, six typical SSMA cross sections with different web depths, flange widths, and thicknesses were used. These sections are 400S137-33, 400S137-68, 600S162-33, 600S162-97, 800S200-33, and 800S200-97. For the third design parameter, an imperfection sensitivity analysis was carried out by applying imperfection values corresponding to 25\%ile and 75\%ile to the finite element model. In all cases, the longitudinal screw spacing was taken equal to one third of the member length to satisfy section D1.2 provisions. A total of 60 cases were investigated. The axial strength predicted by the numerical model were compared with the corresponding design strength as calculated using AISI provisions of Sections C4 and D1.2. Figs. 8-13 show a comparison between the finite element strengths with the nominal design strengths obtained using AISI provisions.

It can be seen that the AISI specifications are generally conservative, except for short columns with \( \lambda_c \) around 0.5, where the AISI specifications overestimates the column strengths. This can be explained by the fact that the member behavior in these regions is governed by local buckling rather than overall buckling. The AISI provisions are based on research conducted on hot rolled sections where local buckling rarely governs the design. Additional numerical and experimental work is needed to study this point.

5- Conclusions

This paper presents a finite element procedure for calculating the axial buckling strength of cold-formed built up I-sections. The initial local and overall geometric imperfections, residual stresses, nonlinear material properties of flat and corner portions have been included in the finite element model. The comparison between the finite element results and the experimental investigation of 32 columns with different geometric dimensions showed that the current AISI design provisions are conservative for members medium and long members. A parametric study of 60 columns was performed using the finite element model to investigate the effect of major design parameters on the behavior. The results of the parametric study showed that the design rules specified in the American Specification are generally conservative for medium and long members but may overestimate the capacity for short members.

Acknowledgment

The research presented in this paper was funded by the Egyptian Science and Technology Development Fund (STDF).
Figure 8  Comparison of FE strengths with design strengths for section 400S137-33

Figure 9  Comparison of FE strengths with design strengths for section 400S137-68
Figure 10  Comparison of FE strengths with design strengths for section 600S162-33

Figure 11  Comparison of FE strengths with design strengths for section 600S162-97
Figure 12  Comparison of FE strengths with design strengths for section 800S200-33

Figure 13  Comparison of FE strengths with design strengths for section 800S200-97
References
AISC Design Specifications (2010), American Institute of Steel Construction.
Schafer, B.W., Li, Z., Moen, C.D. (2010), Computational modeling of cold-formed steel, Thin Walled Structures, Vol. 48, pp 752-762
Extreme Loads on Cold-Formed Steel Framing - Analysis and Design for Earthquake, Blast and Fire

Session ID: EL110
Moderator: Christopher D. Moen, P.E., Ph.D.
Track: Extreme Loading
Date: Thursday, May 2, 2013
Time: 10:00 AM - 11:30 AM

Sponsoring Committee: Committee on Cold-Formed Members

Description:

Blast Resistance of Conventionally Constructed Steel Stud Walls
View Abstract
Casey O'Laughlin, --, Jacobs Technology, Tyndall AFB, FL, United States; Bryan Bewick, Ph.D., P.E., Protection Engineering Consultants, Austin, TX, United States; Eric Williamson, Ph.D., P.E., University of Texas at Austin, Austin, TX, United States

State-of-the-art Review: Fire Performance of Cold-Formed Steel
View Abstract
J Batista-Abreu, -, JHU, Baltimore, MD, United States; M Abu-Hamad, Ph.D., U. of Cairo, Cairo, -, Egypt; L.C.M. Vieira, Jr., Ph.D., Univ. of New Haven, West Haven, CT, United States; Benjamin Schafer, Ph.D., P.E., Johns Hopkins, Baltimore, MD, United States

Advancing seismic performance-based design for light steel framing
View Abstract
N Nakata, Ph.D., JHU, Baltimore, MD, United States; D. Ayhan, -, Istanbul Technical University, Istanbul, -, Turkey; S.G. Buonopane, P.E., Ph.D., Bucknell U., Lewisburg, PA, United States; J Leng, -, JHU, Baltimore, MD, United States; P Liu, -, JHU, Baltimore, MD, United States; R.L. Madsen, P.E., Devco Engineering, -, Oregon, United States; K Peterman, -, JHU, Baltimore, MD, United States; Benjamin Schafer, Ph.D., P.E., Johns Hopkins, Baltimore, MD, United States; C Yu, Ph.D., U. of North Texas, Denton, TX, United States

Axial Hysteretic Modeling of Cold-Formed Steel Members for Computationally Efficient Seismic Simulation
View Abstract
David Padilla-Llano, Graduate Research Assistant, Virginia Tech, Blacksburg, Virginia, United States; Matthew Eatherton, P.E., Ph.D, Virginia Tech, Blacksburg, Virginia, United States; Christopher Moen, P.E., Ph.D, Virginia Tech, Blacksburg, Virginia, United States

Investigation of Cold-Formed Steel Wall Reinforcement Systems to Resist Progressive Collapse
View Abstract
NABIL RAHMAN, Ph.D., P.E., THE STEEL NETWORK, Durham, NC, United States; Ismail Mohamed, etc, North Carolina State University, Raleigh, NC, United States; R Seracino, Ph.D., North Carolina State University, Raleigh, NC, United States
State-of-the-art Review: Fire Performance of Cold-Formed Steel

J Batista-Abreu¹, M Abu-Hamd², L.C.M. Vieira, Jr.³, Benjamin Schafer⁴

¹Civil Eng., JHU, ²Structural Engineering, U. of Cairo, ³Mech., Civil, & Env. Eng., Univ. of New Haven, ⁴Civil Engineering, Johns Hopkins

Session title: Extreme Loads on Cold-Formed Steel Framing– Analysis and Design for Earthquake, Blast, and Fire

Proposed Talk for Structures Congress 2013

State-of-the-art Review: Fire Performance of Cold-Formed Steel

J. Batista-Abreu, M. Abu-Hamd, L. Vieira, Jr., B.W. Schafer

Fire represents one of the most important hazards that building structures must be designed against. Fire resistance of cold-formed steel (CFS) structures is currently insured through adherence to prescriptive building codes and the standardized ASTM (E119) assemblage test. While the level of safety has generally been found acceptable, the lack of an engineering approach to fire resistance of CFS structures impedes progress and stifles innovation:

cost to industry, particularly for ASTM E119 testing, is high and as a result little improvements are sought or found in even basic CFS wall and floor designs;

system-level mechanisms that provide enhanced resistance to fire through re-distribution are neither conceptualized, tested, nor designed in CFS structures due to lack of knowledge to complete such an approach and lack of financial reward for the engineer to do so;

risk consistent multi-hazard based design with fire is largely impossible since fire cannot be reasonably integrated with other hazards without a means to analyze the structure;

as multiple parties work to re-envision buildings to be greener and more sustainable the current prescriptive approach to fire means fire protection is added as a constraint with a small set of known solutions instead of integrated within the larger optimization that needs to be performed.

Recent research has begun to set the stage for performance-based design of the fire resistance of CFS structures. CFS structures provide a compelling and challenging framework for advancing performance-based fire resistance. Compelling, because a significant percentage of the modern building stock uses CFS framed walls with gypsum board for interior fire resistance. When these same walls are load bearing, as in a highly efficient CFS framed building, additional challenges arise. Challenging, because the thin-walled nature of CFS members complicates conventional design significantly and under elevated temperatures the stability response is further modified and must be understood. Current research and recent findings will be summarized in this state-of-the-art review.
From: WASET-Info <info@waset.org>
To: Maged <m_tawfick2003@yahoo.com>
Sent: Sunday, March 31, 2013 10:08 PM
Subject: Re: Toronto June 2013 Oral Presentation ICCSEE 2013 : International Conference on Civil, Structural and Earthquake Engineering

Paper ID Code: CA78000

Letter of Acceptance
Toronto, Canada
June 20-21, 2013

Dear Author,

Herewith, This is kindly to inform you that the peer-reviewed draft paper (see below abstract) has been accepted for presentation as well as inclusion in the conference proceedings at the conference to be held in Toronto, Canada during June 20-21, 2013. The high-impact conference papers will also be considered for publication in the special journal issues at http://www.waset.org/proceedings.php

Conference Registration and Writing Formatted Paper:

1. Registration Form File should be Downloaded at http://www.waset.org/downloads/torontoreg.doc
2. Copyright Transfer Form File should be Downloaded at http://www.waset.org/downloads/copyright.doc
3. Word Template File should be Downloaded at http://www.waset.org/downloads/template.doc
4. Latex Style File should be Downloaded at http://www.waset.org/downloads/latex.zip

Letter of Invitation and Visa Requirements:
If you need an invitation letter to get an entrance Visa, Please fill in the online form to get a letter of invitation at http://www.waset.org/invitation.php

Online Conference Registration Form:
The Conference Program and Certificate of Presentation will be composed using the data entered through the online author registration form. All the conference registration files should be zipped (.zip) or rarred (.rar) and submitted via online form at: http://www.waset.org/author.php

Adam Ariston
Editor-in-Science
International Scientific Council
Tel:++971559099620
http://www.waset.org/

PS: Whilst registered to the conference, for unforeseen circumstances, if you can not attend the conference, the final conference paper will be published in the conference proceedings and posted to your postal address.
### Alternative Structural Systems For Cold Formed Steel Framing Buildings

**Abstract**
Load bearing wall paneled system is the mostly used system in constructing light steel framing buildings made of steel cold formed sections. In this system, the vertical loads are transferred from horizontal joists to series of vertical studs, while lateral loads are resisted either by shear panels or X-bracing system. If opening is present, header beam is added to distribute the load to the adjacent studs. However, there is another systems can be used in which the rigid frame is combined with the vertical studs in resisting the vertical loads. This system can be named as Dual System. The advantage of the this system is that the axial stiffness of the studs can interacts with the bending stiffness of the rigid frame in a manner that minimizing the total weight of steel used, and also it provides flexibility in the size and location of any opening. In this paper, a mid rise building consists of 6 floors was designed using the two mentioned statical system approaches. The building has floor heights equal to 3 meters, while spans between columns ranging from 3 to 5 meters. These values represent the typical dimensions used in residential buildings. For each system, the minimum weight is determined through an optimization study. The lipped channel cross sections are employed for the different structural elements (columns, beams, studs). Finally, a comparison between the different systems is presented showing the advantages and disadvantages of each one.

**Keywords**
Steel, Cold Formed Sections, Strength, Design, Buildings

**Type of Presentation**
Oral Presentation
Abstract:
Load bearing wall paneled system is the mostly used system in constructing light steel framing buildings made of steel cold formed sections. In this system, the vertical loads are transferred from horizontal joists to series of vertical studs, while lateral loads are resisted either by shear panels or X-bracing system. If opening is present, header beam is added to distribute the load to the adjacent studs. However, there is another systems can be used in which the rigid frame is combined with the vertical studs in resisting the vertical loads. This system can be named as Dual System. The advantage of the this system is that the axial stiffness of the studs can interacts with the bending stiffness of the rigid frame in a manner that minimizing the total weight of steel used, and also it provides flexibility in the size and location of any opening. In this paper, a mid rise building consists of 6 floors was designed using the two mentioned statical system approaches. The building has floor heights equal to 3 meters, while spans between columns ranging from 3 to 5 meters. These values represent the typical dimensions used in residential buildings. For each system, the minimum weight is determined through an optimization study. The lipped channel cross sections are employed for the different structural elements (columns, beams, studs). Finally, a comparison between the different systems is presented showing the advantages and disadvantages of each one.

1. INTRODUCTION
Cold-formed shapes can be used for entire buildings and for complete roof, floor and wall systems. They can also be used as individual framing members such as studs, joists and truss members. From structural standpoint, the cold-formed steel can serve as both primary structures and secondary structures. Light gauge steel buildings are built mainly using the load bearing wall panel system. A load bearing stud wall generally consists of the studs and track framed openings, including header and jambs. Average spaces between studs are 60 to 70 cm. The track serves to
provide end support for the studs and wall anchorage. In addition to this system, designers can employ a combined system between the rigid frame and vertical studs. The later system is called dual system. In this paper, a comparative study between the two mentioned primary structural systems is presented. Theoretically, the best design is the one that satisfies the stress and displacement constraints, and results in the least cost of construction. Although there are many factors that may affect the construction cost, the first and most obvious one is the amount of material used to build the structure. Therefore, the main aspect in comparison is the minimum weight of the structure, and this is achieved by an optimization study for each system.

Generally, cold formed sections are usually failed in three basic modes, or classes, Fig.1, (i) local which is normally defined as the mode which involves plate-like deformations alone, without the translation of the intersection lines of adjacent plate elements. (ii) Distortional buckling mode with cross-sectional distortion that involves the translation of some of the fold lines (intersection lines of adjacent plate elements), and (iii) Global buckling mode where the member deforms with no deformation in its cross-sectional shape, in addition to other modes which are formed from the interaction of these basic modes. Local as well as distortional modes are always happened in short members, while long members are failed in overall mode. Moreover local or distortional failure modes are always associated with certain post buckling strength capacity.

![Fig.1: Critical axial load factors, $P_{cr}/P_y$, versus half wave length for two lipped channel sections](image-url)

Fig.1: Critical axial load factors, $P_{cr}/P_y$, versus half wave length for two lipped channel sections
Theoretical and experimental research has been dedicated to characterizing and describing the structural behaviour of these elements, seeking economical and safe design methods. Schafer [1] summarized the column design rules for direct strength method that consider the interaction of local and overall buckling as well as interaction of distortional and overall buckling. Yan and Young [2] provided experimental ultimate loads and failure modes for cold-formed steel channel columns with complex edge stiffeners. They also [3] showed that direct strength method capable of producing reliable design of cold-formed channel columns with complex edge stiffeners when calibrated with resistance factor of 0.85. Young and Rasmussen [4] recommended that when determining the design strength and elastic buckling loads of fixed ended singly symmetric columns loads are assumed to act at the centroid of the effective cross section. W.F. Maia, et al. [5] studied the behavior of simple and lipped angles subjected to centered and eccentric compression. They confirmed the need to consider the coincident local/torsion mode as a global mode for the simple angle. Moreover, for the lipped angle, the torsional-flexural mode should be considered. Lucas et al. [6,7] investigated the influence of the sheeting on the performance of the cold formed sections using finite element methods.

Design codes predicate strength of such members using the effective width approach in which the effect of cross section instabilities (local, distortional) are considered by reducing the gross properties of the section (area, inertia) then the final strength is the product of the critical stresses by the reduced section property. However, recently, AISI-2007 [8] have passed a new design approach called Direct Strength Method, DSM. In DSM, the ultimate strength is defined as the minimum of the nominal local buckling strength, nominal distortional buckling strength, and nominal overall buckling strength. These nominal strengths are determined as function of elastic buckling load that is correspond to each buckling mode.

2. CASE OF STUDY
The case studied includes the optimize design of 6 story residential building. The building covers an area of 315 m$^2$ (including voids), each floor is divided into 4 flats each of which is 63 m$^2$. Fig.2 shows the typical architectural floor plan of the building. The lateral loads are resisted by vertical bracing elements, while primary vertical loads (Dead and Live Loads) are carried by different two systems. These systems are the traditional load bearing wall panels, Fig.3-a, and the proposed one the dual system, Fig. 3-b. The dual system composed of rigid frame and vertical
studs. Floors consist of GRC slabs supported on series of horizontal beams (joists). The beams transmit their loads directly to the systems that carry the vertical loads. These systems are arranged along the vertical axis 1 through 13. Lateral loads are carried by group of vertical bracing systems arranged in the two principal directions of the building. The vertical bracing systems along X-direction are arranged on axis “A”, “D”, “H”, and “I”, while along Y-direction, vertical bracing systems are arranged on axis “1”, “7”, and “13”.

Fig. 2: Typical Floor Plan.

Loads considered in design are dead, live, wind, and seismic loads. Load values and combinations are done according to provisions of uniform building code. Space 3D model has been developed using SAP2000 program. The model has been done considering the following assumptions:

- Beams (joists) are hinged connected to the vertical columns.
- Vertical bracing members are pinned connected to the vertical columns.
- The floor slab moved horizontally in the principle directions as rigid diaphragm.
3. OPTIMIZATION TECHNIQUE

In the formulation of the optimization problem, the objective function is the total weight of the structural members. Therefore, the general formulation of the design optimization problem can be expressed by:

\[
\text{Minimize } F(x) = \sum_{n=1}^{mem} W_n L_n
\]  

(1)

Where:

- \( W \): Mass per unit length of the member under consideration.
- \( L \): Length of the members.
- \( mem \): Total number of members

For discrete optimization problems all possible combinations of the discrete values for the design variables are substituted, and the one resulting in the minimum value for the objective function, while satisfying the constraints, is chosen. This method always finds the global minimum but is slow. However, some newly developed techniques, known as heuristic methods provide means
of finding near optimal solutions with a reasonable number of iterations. Included in this group are Simulated Annealing, Genetic Algorithms, and Tabu Search. Numerous research studies have been done in the field of structural optimization. Goldberg and Samtani [9] performed engineering optimization for a ten member plane truss via Genetic Algorithms. The Simulated Annealing algorithm was applied to discrete optimization of a three-dimensional six-story steel frame by Balling [10]. Jenkins [11] performed a plane frame optimization design based on the Genetic Algorithm. Farkas and Jarmai [12] described the Backtrack discrete mathematical programming method and gave examples of stiffened plates, welded box beams, etc.

In this study the optimization technique used is described as follows. The starting point of the search is a structural configuration that satisfies the stress and displacement constraints. The search begins by evaluating the system weight at the entire neighborhood of the starting point and the corresponding move values, choosing the best move (the one that results in the most weight reduction). The required replacements are then made to the structural properties, and structural analysis is performed. Based on the analysis results, stress and displacement constraints are checked. If all of the constraints are satisfied, the move is feasible and the search algorithm has found a new node. If any of the constraints are not satisfied, the structural configuration is set back to its original form, the second best move is selected, the corresponding changes are made to the structural model, and the analysis and constraint evaluation processes are repeated. This procedure is continued until a move that satisfies all the constraints is found. The search algorithm is now at a new node. It should be noted that a move is not finalized unless all constraints for the structural configuration that is the result of that move are satisfied, therefore, there is no chance of staying in the infeasible region. A grouping method is implemented in the program by simply putting the elements that are desired to have the same section in one group and treating the group as one independent variable. In addition to resulting in more practical designs, the number of independent variables and therefore the time to run the program is reduced. The search method changes sections for the entire group of elements instead of a single structural element.

4. RESULTS
The optimization technique described in the last section has been used to get the minimum weight of the 6 story building. The building has been designed twice, first considering the load bearing wall
panels, and second time when the main structural system is the dual system. The design checks has been done according to provisions that given in the North American Specifications AISI-2007[8]. As the vertical loads transferred mainly to the main systems assigned to axis “1” to “13”, Fig. 2, and the lateral loads resisted by group of vertical bracings, the optimization analysis can be tested first in simple 2D model. Therefore, first we will consider panel along axe “10”, then the analysis will extended to the 3D space model. The section selected from a group of lipped channel sections listed in Table 1.

![Fig. 4: Lipped Channel Section](image)

**Table 1**: Dimensions of Lipped Channel Sections

<table>
<thead>
<tr>
<th>Section</th>
<th>H (mm)</th>
<th>b (mm)</th>
<th>D (mm)</th>
<th>t (mm)</th>
<th>Weight (kg/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100C50-100</td>
<td>100</td>
<td>50</td>
<td>17</td>
<td>1</td>
<td>1.766</td>
</tr>
<tr>
<td>100C50-150</td>
<td>100</td>
<td>50</td>
<td>17</td>
<td>1.5</td>
<td>2.609</td>
</tr>
<tr>
<td>100C50-170</td>
<td>100</td>
<td>50</td>
<td>17</td>
<td>1.7</td>
<td>2.934</td>
</tr>
<tr>
<td>100C50-200</td>
<td>100</td>
<td>50</td>
<td>17</td>
<td>2</td>
<td>3.413</td>
</tr>
<tr>
<td>140C60-150</td>
<td>140</td>
<td>60</td>
<td>20</td>
<td>1.5</td>
<td>3.386</td>
</tr>
<tr>
<td>140C60-170</td>
<td>140</td>
<td>60</td>
<td>20</td>
<td>1.7</td>
<td>3.815</td>
</tr>
<tr>
<td>140C60-200</td>
<td>140</td>
<td>60</td>
<td>20</td>
<td>2</td>
<td>4.449</td>
</tr>
<tr>
<td>160C60-150</td>
<td>160</td>
<td>60</td>
<td>20</td>
<td>1.5</td>
<td>3.621</td>
</tr>
<tr>
<td>160C60-170</td>
<td>160</td>
<td>60</td>
<td>20</td>
<td>1.7</td>
<td>4.082</td>
</tr>
<tr>
<td>160C60-200</td>
<td>160</td>
<td>60</td>
<td>20</td>
<td>2</td>
<td>4.763</td>
</tr>
<tr>
<td>200C75-170</td>
<td>200</td>
<td>75</td>
<td>25</td>
<td>1.7</td>
<td>5.15</td>
</tr>
<tr>
<td>200C75-190</td>
<td>200</td>
<td>75</td>
<td>25</td>
<td>1.9</td>
<td>5.731</td>
</tr>
<tr>
<td>200C75-200</td>
<td>200</td>
<td>75</td>
<td>25</td>
<td>2</td>
<td>6.019</td>
</tr>
</tbody>
</table>

For the load bearing wall panel system, Fig. 3-a, the vertical studs are pin connected to a top and bottom track. Hence, the internal axial forces can be calculated directly, and the values will not be changed by changing the section of the member, so the design has been done directly. The maximum loads arise from the combination of DL+LL, with values ranges from 0.988 at the top
floor to 5.93 at the bottom floor. The best sections assigned for the studs are as follows, 100C50-170 for the first two floors, 100C50-150 for the second two floors, and 100C50-100 for the last two floors. Consequently, the weight of this panel is 0.598 ton. Figure 5 represent the variation of the stress ratios along the different stories of the building. The stress ratios range from 0.25 at the top floors to 0.864 at the bottom floors.

![Graph](image)

**Fig. 5:** Stress ratios at different floor levels for Wall Bearing System.

For the dual system, Fig. 3-b, the relative bending stiffness between the horizontal beams and vertical columns as well as the axial stiffness of the vertical studs will affect the values of the developed internal straining actions (axial force, and bending moments) in the members. Therefore, the optimization technique is needed to get the minimum weight. Initial design has been done for the system depending on a set of assumed sections. This design is considered as the starting point of the optimization analysis. Figure 6 Represent the variation of the weight of the structure with the number of iterations done in the analysis. The figure indicates that the initial weight of the structure is 0.944 ton while the minimum weight achieved is 0.503 ton. Also, it can be observed that the best solutions were obtained within 33 iterations. In addition, Fig. 7 represent the variation of the stress ratios in the members (beam, column, studs) at all story levels for iterations number 24 and 80 (last iteration). In iteration 24 the structure weights 0.599 tons, while in the last iteration the structure weights 0.503 tons (minimum weight). From the figure it can be concluded that, columns are the members that govern the design since stress ratios in both iterations are ranging from 0.86 at first story to 0.96 at
last story. Also, the larger beam stress ratios are always at the higher story, while the stud stress ratios are at the lower story.

Fig. 6: Variation of weight with iteration for Dual System.

The work is then extended to the 3D space model of the 6 story building. In this case the design will consider the floor joists and the vertical bracing members in addition to the main systems that carry the vertical loads. For both cases (wall panels, and dual system) joists are designed as simple beams with variable spans. The spacing between joists ranges from 65 cm to 75 cm. The critical case of

Fig. 7: Stress ratios at different floor levels for Dual System.

a) Stress ratio at iteration no , weight = 0.599 ton  

b) Stress ratio at iteration no , weight = 0.503 ton
loading was DL + LL. Based on this 160C60-170 section was selected for all joists, however, 160C60-300 section was selected for joists that have spans of 5.25m.

Bracing elements are provided to resist lateral loads such as wind load and seismic force, and also ensure the stability of the building. Design of the bracing elements will not depend on the type of the main system that carries the vertical loads, therefore there design will be similar for the two cases. Due to the light weight of steel sections used the wind load become critical than the seismic forces. Thus the bracing elements are designed according to the wind load. Based on the limits that are stated in the design criteria, square hollow section 140x140x4 was selected for the vertical members, and 140x140x2 was selected for the diagonals. Consequently, the minimum weight of the building obtained is 36.17 ton, 32.87 ton when the main systems are the load bearing wall panels and the dual system, respectively.

From the above analysis, it can be concluded that when the analysis is done in 2D the minimum weight of the dual system is 0.503 ton, while for load bearing wall panel system is 0.598 ton. That means there is a reduction in the weight by about 15%. Similarly, when analysis is done in the 3D space model, using dual system leads to reduction in the weight by about 10%, since the minimum weight achieved are 32.87 and 36.17 ton for dual system and load bearing wall panel system, respectively. Moreover, dual system has certain flexibility regarding the size and location of the openings, since the spacing between studs is larger than that in the wall bearing panel system. However, connections between elements (beams, columns) in dual system are rigid or semi rigid to transfer moments from beams to columns. These connections are complex in design and installation especially in the cold formed sections.

5. Summary and Conclusions
In this study, a new structural system has been introduced as an alternative system to the load bearing wall panel system that has been widely used in construction of cold formed steel building. This system is named as dual system since it is combination of rigid frame and vertical studs. An optimization study was done to get the minimum weight of a 6 story building. The building was designed twice using the load bearing wall panel system and the dual system. The sections used are the lipped channel sections. Results revel that, using dual system leads to
reduction in the total weight of the structure by about 10%. In addition, in dual system, columns are the members that govern the design since stress ratios are ranging from 0.86 to 0.96.

Acknowledgment

The research presented in this paper was funded by the Egyptian Science and Technology Development Fund (STDF) under project no. 3751.

6. References

Letter of Acceptance  
Toronto, Canada  
June 20-21, 2013

Dear Author,

Herewith, This is kindly to inform you that the peer-reviewed draft paper (see below abstract) has been accepted for presentation as well as inclusion in the conference proceedings at the conference to be held in Toronto, Canada during June 20-21, 2013. The high-impact conference papers will also be considered for publication in the special journal issues at http://www.waset.org/proceedings.php

Conference Registration and Writing Formatted Paper:

1. Registration Form File should be Downloaded at http://www.waset.org/downloads/torontoreg.doc
2. Copyright Transfer Form File should be Downloaded at http://www.waset.org/downloads/copyright.doc
3. Word Template File should be Downloaded at http://www.waset.org/downloads/template.doc
4. Latex Style File should be Downloaded at http://www.waset.org/downloads/latex.zip

Letter of Invitation and Visa Requirements:
If you need an invitation letter to get an entrance Visa, Please fill in the online form to get a letter of invitation at http://www.waset.org/invitation.php

Online Conference Registration Form:
The Conference Program and Certificate of Presentation will be composed using the data entered through the online author registration form. All the conference registration files should be zipped (.zip) or rarred (.rar) and submitted via online form at http://www.waset.org/author.php

Adam Ariston
Editor-in-Science
International Scientific Council
Tel:++971559099620
http://www.waset.org/

PS: Whilst registered to the conference, for unforeseen circumstances, if you can not attend the conference, the final conference paper will be published in the conference proceedings and posted to your postal address.
The use of cold-formed steel framing as an alternative to reinforced concrete in mid-rise buildings in Egypt is investigated. The analysis focuses on both cost uncertainties and potential sustainability advantages. Cost uncertainties are modeled based on historical material price fluctuations. The sustainability assessment focuses on material recycling, reuse potential and the use of regional materials. The analysis revealed that overall, CFS buildings have a much better potential for material recycling and reuse compared to their RC counterparts based on common building practices in Egypt. In addition, in spite of large fluctuations in steel prices in Egypt, the overall uncertainty in total building construction cost for CFS is still comparable to its RC counterpart.
Cost and Sustainability Analysis of Cold Formed Steel Residential Buildings

Metrwally Abu-Hamd, Hesham Osman, and Ibraheem Gamal

Abstract—The use of cold-formed steel framing as an alternative to reinforced concrete in mid-rise buildings in Egypt is investigated. The analysis focuses on both cost uncertainties and potential sustainability advantages. Cost uncertainties are modeled based on historical material price fluctuations. The sustainability assessment focuses on material recycling, reuse potential and the use of regional materials. The analysis revealed that overall, CFS buildings have a much better potential for material recycling and reuse compared to their RC counterparts based on common building practices in Egypt. In addition, in spite large fluctuations in steel prices in Egypt, the overall uncertainty in total building construction cost for CFS is still comparable to its RC counterpart.

Keywords—Cold formed steel buildings, Monte Carlo simulation, sustainable construction.

I. INTRODUCTION

Innovation in building construction systems is required in order to meet the challenges facing the built environment. Improved resilience, increased sustainability, and more affordable solutions are being demanded by society in the midst of increasing fiscal turmoil and more prevalent natural hazards. A detailed assessment of the latest construction technologies is warranted in order to truly assess their environmental and cost implications. Cold Formed Steel (CFS) buildings are a worthy alternative to existing tradition building systems and offer advantages owing to their modularity, light weight and ease of assembly.

The objective of this study is to undertake an assessment of the sustainability and costs associated with CFS buildings based on construction practices and cost trends in Egypt. A traditional (Reinforced Concrete) and CFS prototype was designed and used as the basis for comparison in this study. The sustainability analysis involved a detailed review of LEED 2009 for New Construction & Major Renovations. The review focused on the identification of LEED credits that are directly impacted by the choice of the building structural system. The cost assessment involved a sensitivity analysis regarding the impact of fluctuations in material prices on the expected construction cost of reinforced concrete (RC) and CFS buildings. This analysis can be used as a measure of inherent risk that building developer can be exposed to when investing in a particular type of design.

II. BACKGROUND

A. Cold-formed Steel Buildings

In recent years society has begun to re-evaluate our built environment with the objective of achieving higher performance, specifically, to minimize loss attributed to natural hazards and to seek sustainable solutions, for long-term needs in the built environment. Low-rise structures in general, and residential housing in particular, represent by far the greatest percentage of the world’s building stock. In the past, cultural norms largely drove the materials and systems employed for residential housing, today the situation is in flux and the potential to move towards a more scientific performance-basis exists.

Among the available construction systems that recently has seen significant increase in use, the light (cold-formed) steel framing systems (Figure 1) have proven to be a worthy alternative to traditional systems. Potential advantages of such light steel framing systems include the high degree of dimensional exactness of the members, high strength-to-weight ratio of the members, high recycled content, and ease of construction. These qualities have lead cold-formed steel studs and tracks to be the framing method of choice for non-load bearing walls in mid- and high-rise construction worldwide. This proposal explores the use of cold-formed steel framing in residential housing with a focus on (a) developing new non-proprietary systems for light cold-formed steel framing with the potential to greatly improve building performance and flexibility, and (b) providing a series of building archetype studies that explore framing solutions in the U.S. and Egypt from economic, environmental, and sustainability metrics.

FIGURE I
COLD FORMED STEEL FRAMED HOUSE
B. Sustainable Construction

Sustainable construction is a field of engineering that aims to incorporate sustainable development concepts into traditional construction practices. In spite of the foundation of knowledge in this field being continuously evolving, sustainable construction is not yet standardized practice within the construction industry. Many studies have documented damages caused by the construction industry to the surrounding environment. These studies have also acknowledged the great opportunity to improve the sustainability of existing construction practices. [1], [2].

Accordingly, sustainable construction has been introduced as: “a holistic process in which the principles of sustainable development are applied to the comprehensive construction cycle, from the extraction and beneficiation of raw materials, through the planning, design, and construction of buildings and infrastructure, until their possible final deconstruction, and management of the resultant waste [3].”

An evaluation tool is a mechanism through which the sustainability of a building (or building component) can be assessed against certain criteria. This evaluation is of utmost importance since there are numerous possible actions that can be taken in order to improve the sustainability of a building construction process. For example, actions that can be considered include: 1) Specify equipment, materials, and products based on performance, 2) Use recycled materials to reduce use of raw materials and divert material from landfills, 3) Use local and regional materials as much as possible, 4) Minimize site impact by specifying location of trailers, equipment, storage, traffic, 5) Monitor construction site energy and water use, and 6) Develop a construction waste management and recycling plan.

In this regard a distinction is made between two common tools that are used for the sustainability evaluation of buildings: 1) Sustainable building rating systems, and 2) Life Cycle Assessment. Green building rating systems serve two functions of promoting high performance buildings and creating the demand for sustainable construction [4]. Presently, the most widely accepted green building rating system in the United States is the LEED system, developed by the U.S. Green Building Council. There are other green building assessment systems used worldwide, such as the Building Research Establishment’s Environmental Assessment Method (BREEAM), and the Comprehensive Assessment System for Building Environmental Efficiency (CASBEE).

III. DEVELOPMENT OF BUILDING PROTOTYPES

The building prototype used in the present study is commonly used in affordable housing projects in Egypt. This model provides housing units with an area of 63 square meters per unit in a 6 story buildings having either 4 or 6 flats per floor. These building usually have the following characteristics:

- a) Reinforced Concrete Skeleton:
  The typical residential house has a 10 to 12 cm reinforced concrete slab cast in situ with a reinforced concrete footings, beams and columns.

- b) Ceramic Tile Flooring
  The flooring of the conventional residential house consists of ceramic tiles fixed on top of 3 cm sand layer and 2 cm cement mortar.

- c) Brick Walls
  The walls of conventional residential house consist of 25 cm brick walls for the exterior walls and 12 cm brick walls for interior walls. The wall finishing is plastered paint.

In the present study, an alternative design was performed using cold formed steel construction for the building skeleton instead of reinforced concrete. Also, in order to provide an efficient construction system that can be executed easily in a very short time, precast glass-fiber reinforced concrete panels were used for the floor and walls instead of the traditional time consuming construction systems.

IV. SUSTAINABILITY ANALYSIS

The sustainability analysis involved a detailed review of LEED 2009 for New Construction & Major Renovations. The review focused on the identification of LEED credits that are directly impacted by the choice of the building structural system. Subsequently, a quantitative and qualitative comparison of relevant credits based on common building practices in Egypt was conducted. The comparison focused on Cold Formed Steel (CFS) and Reinforced Concrete (RC) buildings as RC buildings are the most commonly used building system in Egypt. The analysis approach was based on the ‘intent’ of the credit rather than the suggested LEED calculation procedure. Based on the review up to 3 credits were found to be most relevant to the analysis:

1) Materials & Resources: Material Reuse
2) Materials & Resources: Recycled Content
3) Materials & Resources: Regional Materials

The following sections discuss the results of this analysis.

A. Material Reuse

The main intent of this credit is to reuse building materials and products to reduce demand for virgin materials and reduce waste, thereby lessening impacts associated with the extraction and processing of virgin resources (USGBC, 2009). Currently LEED awards up to 2 credits for this category based on the percentage of reused material utilized on the building (5% reuse, 1 credit and 10% refuse, 2 credits). In this study, main building materials for a traditional RC and the proposed CFS design were assessed based on their potential for reuse based on existing common building practices in Egypt. A weighted
average based on % cost was calculated for each building. The percentage of materials reuse for the CFS building prototype was found to be 76.25% surpassing LEED’s threshold of 10% required for 2 credits.

### B. Material Recycling

The main intent of this credit is to increase demand for building products that incorporate recycled content materials, thereby reducing impacts resulting from extraction and processing of virgin materials. Currently LEED awards up to 2 credits for this category based on the percentage of reused material utilized on the building (10% recycled content, 1 credit and 20% recycled content, 2 credits). Main building materials for a traditional RC and the proposed CFS design were assessed based on the values of their recycled content according to common manufacturing practices in Egypt. A weighted average based on % cost was calculated for each building.

Tables 3 and 4 shighlight that both RC and CFS have a substantial percentage of recycled content based on current building material manufacturing practices in Egypt. Both prototypes will be eligible for obtaining at least 2 LEED credits. This is primarily due to the utilization of a significant amount of steel in both buildings and the large cost of steel compared to other building materials in Egypt. The CFS has more than 70% more recycled content as shwon by the analysis.

### C. Regional Material

The main intent of this category is to increase demand for building materials and products that are extracted and manufactured within the region, thereby supporting the use of indigenous resources and reducing the environmental impacts resulting from transportation. Distance thresholds used in LEED were quite high (500 miles) and have been deemed unacceptable for a country with the size of Egypt. As such, instead of specifying a fixed distance threshold that must be met, this research developed a Material Transportation Distance Indicator (MTDI) that can be used to assess the overall distance moved by material from point of manufacturing to point of consumption. The indicator is calculated as per the following equation:

\[ MTDI = \sum_{i=1}^{N} C_i \times d_i \]  

Where \( C_i \) is the percentage cost for building material \( i \) and \( d_i \) is the shortest distance from the manufacturer of building material \( i \) and the project location. \( N \) is the total number of building components being analyzed. The analysis involved six main cities in Egypt that span the various geographical areas of the countries. MTDI was calculated for each city based on actual land-based transportation distances from point of manufacturing to point of consumption. Truck-based transportation was used as this is the predominant form of freight transportation in the country.

The comparison revealed that in most Egyptian cities RC buildings outperform CFS buildings in terms of average distances for material transportation. This is mainly due to the
fact that cement manufacturing and concrete production is undertaken in a large number of locations throughout the country as compared to steel manufacturing that is largely centralized in the north. Also, cities in Upper (southern) Egypt have relatively high average material transportation distances for both designs due to their large proximity from major steel manufacturing facilities in the north.

V. COST ANALYSIS

One of the main challenges of introducing a new form of building construction to any market is managing uncertainty associated with the various aspects of the building process. Uncertainty of construction cost due to fluctuations in material prices has been reported to be one of the most significant risks in construction projects and has been studies by several researchers [5], [6].

In this research, historical trends of material prices in Egypt were collected based on national records available in the Central Authority for Public Mobilization and Statistics (CAPMAS). Data was obtained from 2003-2012. Data from 2003-2008 was available in quarters while from 2008-2012 data was only available on a yearly basis. The data was subsequently used to forecast material process from 2012-2017. Two prediction models were utilized: 1) Autoregressive Moving Average (ARMA) and 2) Exponential Smoothing. The ARMA model was found to be more suitable for the data being analyzed.

ARMA models are widely used models in time-series analysis and forecasting. The simplest AR model is the first order autoregressive, or AR(1) model:

\[
y_t = -a_1 y_{t-1} + e_t
\]

(2)

Where \( y_t \) is the mean adjusted series in year \( t \), \( y_{t-1} \) is the series in the previous year, \( a_1 \) is the lag -1 autoregressive coefficient, and \( e_t \) is the noise. The residuals \( e_t \) are assumed to be random in time (not autocorrelated), and normally distributed. The AR(1) model has the form of a regression model in which \( y_t \) is regressed on its previous value. In this form, \( a_1 \) is analogous to the regression coefficient, and \( e_t \) to the regression residuals. Figures 2 and 3 show the ARMA forecast for steel and cement prices. Fluctuations in steel reinforcement process are noticeable while cement exhibited a constant upward trend.

The forecasted data was subsequently fitted to the most appropriate probability distribution using Anderson-Darling goodness of fit tests. The test measure the compatibility of a random sample (in this case forecasted data from the ARMA model) with a theoretical probability distribution function.

The Anderson-Darling statistic \( A^2 \) is defined as

\[
A^2 = -n - \frac{1}{n} \sum_{i=1}^{n} (2i-1) \left[ \ln F(X_i) + \ln(1 - F(X_{n-i+1})) \right]
\]

(3)

\( H_0 \): The data follow the specified distribution.
\( H_1 \): The data do not follow the specified distribution.

The hypothesis regarding the distributional form is rejected at the chosen significance level (alpha) if the test statistic, \( A^2 \), is greater than the critical value obtained from a table. Chosen probability distributions are shown in Table 6.

Finally, a Monte Carlo simulation (5000 trials) was performed using these distributions to obtain expected
probability distribution for entire building based on percentage cost of each building material. Monte Carlo simulation is a well-established quantitative technique that allows the simulation of risk in any number of variables due to inherent uncertainty in their values. Cost variations in both buildings were found to follow the normal distribution. Results revealed that RC building prototype had a mean expected cost of approximately 1.1 million EGP and a standard deviation 86,000 EGP while the CFS building had a mean of 0.83 million EGP and a standard deviation of 74,000. Results revealed that in spite large fluctuations in steel prices in Egypt, the overall uncertainty in total building construction cost for CFS is still comparable to its RC counterpart. Not only are CFS an economically attractive alternative in Egypt, they exhibit a cost risk similar to that of traditional RC buildings.

<table>
<thead>
<tr>
<th>Material</th>
<th>Distributions Type</th>
<th>Distribution Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Reinforcement</td>
<td>Normal</td>
<td>Mean=4,315.07, Std. Dev.=746.91</td>
</tr>
<tr>
<td>Cement</td>
<td>Beta</td>
<td>Minimum=23.74, Maximum=36.17, Alpha=1.04511, Beta=0.92449</td>
</tr>
<tr>
<td>Sand</td>
<td>Beta</td>
<td>Minimum=22.62, Maximum=49.69, Alpha=0.98668, Beta=0.98668</td>
</tr>
<tr>
<td>Aggregate</td>
<td>Beta</td>
<td>Minimum=46.32, Maximum=133.81, Alpha=0.96632, Beta=0.97008</td>
</tr>
<tr>
<td>Brick</td>
<td>Beta</td>
<td>Minimum=304.42, Maximum=461.34, Alpha=0.98668, Beta=0.98668</td>
</tr>
</tbody>
</table>

This paper presented a cost and sustainability assessment for cold formed steel buildings. A prototype six storey low-cost housing module was used as the basis for comparison. The comparison was based on existing building practices in Egypt and a comparison was undertaken with a reinforced concrete building which is the common construction system in Egypt. The sustainability assessment focused on material reuse, recycling and sourcing regional materials. The analysis revealed that overall, CFS buildings have a much better potential for material recycling and reuse compared to their RC counterparts. In most Egyptian cities, sourcing materials for a RC building was found to be more sustainable due to the relatively shorter distances from material manufacturing to consumption. This is primarily due to the centralization of steel manufacturing in the north of the country.

The cost assessment focused on conducting a sensitivity analysis based on fluctuations in material prices and its effect on overall building construction costs. Actual historical material prices were used and 5-year forecasts were developed using an Autoregressive Moving Average model. Forecasts were then fitted to probability distributions using the Anderson-Darling goodness of fit test. Results revealed that in spite large fluctuations in steel prices in Egypt, the overall uncertainty in total building construction cost for CFS is still comparable to its RC counterpart.

Acknowledgment
The research presented in this paper was funded by the Egyptian Science and Technology Development Fund (STDF) under project no. 3751.

REFERENCES
Proceedings of the

Egypt-United States Workshop on

Use of Light Steel Framing in Residential Buildings

9,10 December 2012

Faculty of Engineering
Cairo University

Part 1/2
PREFACE

In Egypt, it is estimated that at least 400,000 housing units are needed every year. If conventional reinforced concrete construction continues to be employed alone, 50% of annual demand will not be met, leaving the situation to worsen.

In addition to demands for more residential housing units, the demands for better residential housing must be recognized. These demands exist along two vectors: (1) sustainability: fully consider the important balancing act between economic, environmental, and social constraints when considering materials, means and methods, and long-term function of the structure, and (2) resiliency: increase the performance of residential structures against natural hazards, particularly earthquakes and wind events. This confluence of greater demand in number and performance, for residential housing makes today an excellent time to question if traditional residential construction methods should be augmented by new systems.

To meet such a challenge, it is necessary to explore the latest construction technologies, and to create innovative building systems that have the potential to bring high-performance affordable housing within reach of new markets, particularly in developing regions. Beyond being affordable, these systems have to be flexible enough to suit local climate and site conditions, cultural and living habits, and spatial standards. Construction solutions also should reduce or eliminate the need for skilled personnel on the site, and ideally should be assembled with simple tools and erectable without machinery. Among the available construction systems that satisfy the previous conditions, light (cold-formed) steel framing systems have proven to be a worthy alternative to traditional systems.

Potential advantages of such light steel framing systems include the high degree of dimensional exactness of the members, high strength-to-weight ratio of the members, high recycled content, and ease of construction. These qualities have lead cold-formed steel studs and tracks to be the framing method of choice for non-load bearing walls in mid- and high-rise construction worldwide.

This workshop explores the use of cold-formed steel framing in residential housing with a focus on creating the necessary awareness in the society to include such systems in the solutions adopted to solve the existing housing problem.
Objectives of the Workshop:

• To create awareness in the society about the worldwide applications of light steel framing in residential buildings.
• To give an overview about the design, production and erection considerations related to such systems.

The scientific material included in this proceedings aims at providing the attendance with a general overview of the covered topics. Materials included contain some parts by the individual speakers in each topic and the rest have been collected from available resources such as books, journals, and public domain web sites. All these resources are hereby acknowledged.

Organizing Committee:

Egypt
Prof Dr Metwally Abu-Hamd
Prof Dr Mohammed Badr
Dr Maged Hanna

United States
Prof Dr Ben Schafer
Dr. Zhanjie Li
Final Program

Day 1: Sunday December 9, 2012
08:00 – 09:00 Registration
09:00 – 09:30 Opening Session
Session 1: Application of Light Steel Framing (LSF) in Residential Buildings
Moderator: Ben Schafer
09:30 – 09:45 Current Housing Status in Egypt (M. Abu-Hamd)
10:10-10:15 Questions and Discussion from the Audience*
10:15 – 11:15 Worldwide Applications of LSF in Residential Building Construction (G. Richards)
11:15 – 11:30 LSF Applications in the Arabian Gulf Area (N. El-Hajj)
11:30 – 12:15 Coffee Break

Session 2: Resources for the Engineering and Design of Light Steel Framing
Moderator: George Richards
12:15 – 12:30 Egyptian Design Codes and LSF (M. Abu-Hamd)
12:30 – 13:00 International Design Codes and Manual Design Aids (B. Schafer)
13:00 – 13:30 Overview of CFS Design Software** (N. Rahman)
13:30– 14:15 Coffee Break

Session 3: Design Examples of LSF Residential Buildings
Moderator: Don Allen
14:15 – 15:00 Alternative Solutions for floors and walls. (M. Badr and N. El-Hajj)
15:00 – 15:30 Design Case Studies (M. Abu-Hamd /M. Hanna)

Day 2: Monday December 10, 2012
Session 4: Connections and Nonstructural Details for LSF
Moderator: Nader El Hajj
09:00 – 09:45 Connections in LSF (N. Rahman)
09:45 – 10:00 Architectural and Services Details (D. Allen)
10:00 – 10:30 Fire & Acoustic Performance (B. Schafer \N. El-Hajj)
10:30 – 11:00 Sustainability Assessment (D. Allen and Zhanjie Li)
11:00 – 11:45 Coffee Break

Session 5: Production and Erection Technology - American Case Studies
Moderator: Ben Schafer
11:45 – 12:15 DSi (D. Allen)
12:15 – 12:45 TSN (N. Rahman)
12:45 – 13:15 FRAMECAD (N. El-Hajj)
13:15 – 13:45 BORM (G. Richards)
13:45 – 14:30 Coffee Break

Session 6: Implementation of LSF in Egypt
Moderator: Metwally Abu-Hamd
14:30 – 15:00 Present Capabilities of LSF in Egypt (Alex form)
15:00 – 15:15 Implementation of LSF in Egypt (M. Abu-Hamd)
15:15 – 15:30 Closing Remarks (All)

*Questions and discussion from the audience is highly encouraged. Five minutes is set aside for shorter talks (less than 30 minutes) and 10 minutes of questions and discussion is set aside for longer talks.

**Join us in the exhibition space for more information about design software associated with the U.S. participants and their companies.
Speakers Biography

**Metwally Abu-Hamd, Cairo University, Egypt**

Dr Abu-Hamd is a Professor of Steel Structures and Bridges in the Structural Engineering Dept of the Faculty of Engineering at Cairo University, Egypt. He received his B.Sc. from Cairo University, Egypt in 1971 and his M.Sc. and Ph.D. in 1975 and 1977 from Duke University, USA. On the academic level, Dr Abu-Hamd served as Department Head from 2007 to 2009 and served as Director of the Civil Engineering Research and Studies Center from 1994 to 2005. He authored two text books on the design of Plate Girder Bridges and Steel Bridges. His research interests include optimization of steel structures, composite columns, local stability of plate girders and cold formed steel construction. Dr Abu-Hamd has published numerous research papers and supervised several M.SC. and Ph.D. theses in the field of Structural Steel Design. He received the Cairo University Award for Distinguished Research in Structural Engineering in 1985 and the Egyptian Government State Prize for Engineering Sciences in 1989. Dr Abu-Hamd is chairman of the Egyptian Universities Committee for Promotion of University Professors since 2011. On the professional level, Dr Abu-Hamd is a member of the Permanent Committee of the Egyptian Code of Practice for Steel Structures and Bridges since 1984 and a registered Professional Engineer since 1991. He has been working as a Consulting Engineer since 1980 and was involved in the structural steel design of many major projects in Egypt.

**Don Allen, DSi Engineering, USA**

After working for a product manufacturer, a specialty engineer, and full-service structural engineering firm, Don Allen, P.E. spent 8 years as Technical Director for the Steel Stud Manufacturers Association, the Steel Framing Alliance, and the Cold-Formed Steel Engineers Institute. Having returned to private practice in 2012, Mr. Allen still has a special interest in the structural role of materials in sustainable construction. He is a LEED® Accredited Professional, a member of ASCE Structural Engineering Institute (SEI) Committee on Sustainability, the ASCE SEI Committee on Cold-Formed Steel, and chairs the General Provisions subcommittee of the American Iron and Steel Institute (AISI) Committee on Framing Standards. Allen has given presentations on steel throughout North America, as well as China, Colombia, South Africa, and Hawaii. He was instrumental in the recent development of the SSMA Code Compliance Certification program, and chairs the education Task Group of the Structural Engineers Association of Georgia Structural Engineers’ Emergency Response committee. Allen currently oversees engineering marketing for DSi Engineering in Norcross, Georgia.
Mohamed Ragaee Badr, NHBRC, Egypt

Mohamed Ragaee Badr has B.Sc. in 1982, M.Sc. in 1990 and Ph.D. in 1996 in repair of steel under load. He is the Professor and Head of Structure and Steel Construction Institute in National Housing and Building Research Center in Egypt. He has over 30 years of experience in steel research and rehabilitation of structures and in design, supervision of construction of steel and concrete projects.

Nader El Hajj, FRAMECAD Middle East, USA

Nader El Hajj is a licensed structural engineer with a Master's of Science degree in Structural engineering and an MBA. He has over 30 years of experience in the design, construction and analysis of building materials. He is the director for the Dubai based FRAMECAD Middle East. Prior to FRAMECAD, Mr. El Hajj was the Director of the Research Centre for the National Association of Home Builders in the USA. He managed research and development projects for alternative building materials and emerging technologies. He also managed tasks related to design, construction, analysis, and field evaluation of construction materials in housing. Mr. El Hajj developed standardized design and other prescriptive specifications for light gauge steel framing. Mr. El Hajj is active in writing engineering standards, codes, and specifications for the building industry. Mr. El Hajj authored several publications on the subject of housing design and cold-formed steel framing. Mr. El Hajj serves on several industry-related steel committees.

Zhanjie Li, Johns Hopkins University, USA

Dr. Li received his B.S. in Civil Engineering from Shanghai Jiao Tong University in China and his M.S. in Structural Engineering from both Harbin Institute of Technology in China and Johns Hopkins University and Ph.D. in Structural Engineering from Johns Hopkins University. His Ph.D. research was on Finite Strip Modeling of Thin-walled Members by developing a new finite strip method that accounts for general boundary conditions and a modal identification method for shell finite element analysis quantify the buckling and failure modes. Currently he is working as a post-doctoral researcher on US Egypt Cooperative Research: Use of Cold-Formed Steel in (Mid-rise) Residential Housing focusing on a novel cold-formed steel framing system and study of its sustainability features. Dr. Li is a member of Structural Stability Research Council, Cold-Formed Steel Engineers Institute and American Institute of Steel Construction.
Nabil Rahman, The Steel Network, USA

Nabil A. Rahman, Ph.D., P.E. is the Director of Engineering and R&D for The Steel Network, Inc. in Durham, NC. He also currently the chairman of the Cold-Formed Steel Engineers Institute (CFSEI), an international coalition of CFS design engineers with 900+ professional and student members. Dr. Rahman has vast experience in CFS product development, design software development, as well as the analysis and protection of CFS structures against extreme loads (progressive collapse, blast, Impact and hurricanes). He has given numerous continuing education seminars on Design of Cold Formed Steel Framing Systems to engineering associations around the US and internationally in Europe and South America. He has participated in several vulnerability, blast and progressive collapse assessments of commercial and military buildings. He serves as a member of the Committee on Specification and Committee on Framing of the American Iron and Steel Institute (AISI) and a member of ASCE Committee on Cold-Formed Steel.

George Richards, P.E., BORM Group of Companies, USA

George is a technical strategist who has been analyzing and formulating design solutions for over 30 years. His expertise lies in research, design, and the development of construction methodologies, techniques, tooling, manufacturing, intellectual property and specialized product development. George has brought his innovations to bear on over 250,000 homes and buildings in America, Asia, the Middle East & Africa. George brings a unique approach and vision to the industry. Always focused on the most practical and efficient solutions, he brings valuable insight into the optimization of material usage and reduction of waste. He has played a pivotal role in the research and development of cold-formed steel framing through collaborations with both industry alliances and university research centers. George is also well known for his work in improving the efficiency of concrete construction. George holds a Bachelor's degree in Architectural Engineering from California Polytechnic State University, San Luis Obispo. He is a member of the North American Steel Alliance, American Steel Institute, SEAOC (Structural Engineers Association of California) and is former president of the CFSEI (Cold-Formed Steel Engineers Institute). He is a registered engineer in the state of California, and has served on many committees that have written and published building codes.
Ben Schafer, Johns Hopkins University, USA

Professor Schafer received his B.S.E. in Civil Engineering from the University of Iowa with Honors and Distinction and his M.S. and Ph.D. in Structural Engineering from Cornell University. Before joining academia he worked as a practicing structural engineer at Simpson, Gumpertz & Heger, Inc. in Boston, MA and is licensed as a Professional Engineer. He currently holds the titles of Professor, Swirnow Family Faculty Scholar, and Chair of the Department of Civil Engineering at Johns Hopkins University. Professor Schafer serves on numerous technical committees related to cold-formed steel structures, is a past-president of the Cold-Formed Steel Engineers Institute, and is the current Vice-Chair of the Structural Stability Research Council. He has received a National Science Foundation CAREER Award, the ASCE Collingwood and Huber Prizes, and Teaching Awards. For further information on Professor Schafer's activities please see www.ce.jhu.edu/bschafer.

Maged Tawfick Hanna, NHBRC, Egypt

M.T. Hanna, Ph.D. is an associate professor in Structure and Metallic Construction Department at Housing and Building National Research Center, Cairo, Egypt. He obtained his B.Sc. degree from Cairo university in 1994, and the M.Sc. as well as the Ph.D. degrees from Ain-Shams University in 1999 and 2004, respectively, in the field of elastic stability of planer steel frames, and behavior of slender I-section beam-columns. He spent one academic year (2008-2009) as a visitor at Johns Hopkins University, USA. Dr. Hanna has several papers published in international journals and conferences. In-addition to the academic experience, Dr. Hanna shared in the design of several steel and concrete buildings. He is also a member of the Egyptian Engineering Syndicate and the Egyptian Steel Bridges Code Committee.
# Table of Contents

Preface
Workshop Program
Speakers Biography

## Session 1: Application of Light Steel Framing in Residential Buildings

1.1 Current Housing Status in Egypt (Metwally Abu-Hamd)
1.2 Overview of LSF in North American Building Construction (Don Allen)
1.3 Worldwide Applications of LSF in Residential Buildings (G. Richards)
1.4 LSF Applications in the Arabian Gulf Area (Nader El-Hajj)

## Session 2: Resources for the Engineering and Design of LSF

2.1 Egyptian Design Codes and LSF (Metwally Abu-Hamd)
2.2 International Design Codes and Manual Design Aids (Ben Schafer)
2.3 Overview of CFS Design Software (Nabil Rahman)

## Session 3: Design Examples of LSF Residential Buildings

3.1 Alternative Solutions for floors and walls. (Mohammed Badr and Nader El-Hajj)
3.2 Design Case Studies (Metwally Abu-Hamd /Maged Hanna)

## Session 4: Connections and Nonstructural Details for LSF

4.1 Connections in LSF (Nabil Rahman)
4.2 Architectural and Services Details (Don Allen)
4.3 Fire and Acoustic Performance (Ben Schafer and Nader El-Hajj)
4.5 Sustainability Assessment (Zhanjie Li and Don Allen)

## Session 5: Production and Erection Technology: American Case Studies

5.1 DSi (Don Allen)
5.2 TSN (Nabil Rahman)
5.3 FRAMECAD (Nader El Hajj)
5.4 BORM (George Richards)

## Session 6: Implementation of LSF in Egypt

6.1 Present Capabilities of LSF in Egypt (AlexForm).
6.2 Implementation of LSF in Egypt (Metwally Abu-Hamd)
Report from US Team to NSF on the Workshop

As Egypt Finds its Future, Engineers Gather to Envision Future Housing

Ben Schafer, Chairman of Civil Engineering Dept at Johns Hopkins University, USA, writes about his experience in Egypt - US Workshop on use of Light Steel Framing in Residential Housing:

Egypt is in political turmoil. Making a difference in these times is probably asking too much from a small joint USAID/NSF research project, but for two days this week (9-10 December) approximately 200 Egyptian engineers, architects, builders, and manufacturers and even a few ministers from the Egyptian government met at the University of Cairo to imagine a future for Egypt that takes advantage of the latest technology: light (cold-formed) steel framing. The workshop was organized by Professor Metwally Abu-Hamd of the Cairo University and Professor Benjamin Schafer of Johns Hopkins University as part of an ongoing grant that the two professors are leading: "US Egypt Cooperative Research: Use of Cold-Formed Steel in Residential Housing".

The workshop was led by the United States and Egyptian project teams and included key participation from the United States Industrial Advisory Board. Four members of that board: George Richards (BORM), Don Allen (DSi), Nader ElHajj (Framecad), and Nabil Rahman (TSN), came to Cairo to share their experiences with making light steel framing a reality not only in the United States, but around the world. Their talks were augmented by research talks from the project team (Egypt: Metwally Abu-Hamd, Maged Hanna, Mohammed Badr; United States: Benjamin Schafer, Zhanjie Li). An industrial exhibition also complemented the workshop and demonstrated to the Egyptian engineers that the manufacturing base was already in place to make light steel framing happen in Egypt.

Being in Egypt, at the same time as protestors marched in Tahrir square, only made the Egyptian participants even more gracious with their United States participants. Benjamin Schafer summarizes: "It was a once in a lifetime chance to demonstrate that we care about an Egyptian future, without being political, instead we simply set down to the business of becoming better engineers and learning new skills so we can provide society with more from less."

More about the project can be found at the project websites: www.ce.jhu.edu/bschafer/us-egypt-cfs and www.esss-eg.org.