IMPROVING THE DESIGN OF RESIDENTIAL BUILDINGS IN KUWAIT

A.W. SADEK¹, S. AL-FADALA¹ and N. AL-MUTAIRI²

¹Kuwait Institute for Scientific Research
²Kuwait Foundations for Advancement of Sciences

(Received May 2006 and accepted October 2006)

Keywords: Structural design; Reinforced concrete; Kuwaiti villas; Reinforcement; Flexure; Shear; Columns.

1. INTRODUCTION

It is well known that construction activities, which can be chronologically classified as pre-construction (analysis and design), during construction and post-construction (usage and maintenance), have significant impact on the direct and indirect costs of construction. The present study addresses the impact of the pre-construction tasks, analysis and design, on the quality of design and cost functions of residential buildings in Kuwait.

In general, the significance of this study comes from the fact that the cost and safety of residential buildings affect almost every citizen in Kuwait. Moreover, with the absence of a uniform national building code, current practices of analysis and design of buildings involve inconsistencies and non-uniformities. First, it is up to the designer to select whatever foreign code to be followed in specifying the loads and designing structural members. Second, based on a preliminary review of existing design documents and personal contacts with several design offices it is found that it is common here in Kuwait to over-design structural members to allegedly provide safer buildings. It should be stated that every design code stipulates, explicitly or implicitly, certain factors of safety adequate for the local conditions. Over-designing beyond these limits is a waste of materials and money as it provides for unjustified levels of safety.

The present paper is the first attempt to examine the impact of design practices on the quality and cost of residential buildings. The presented work is part of a series of research programs conducted at Kuwait Institute for Scientific Research whose findings are expected to serve the immediate objective of providing recommendations on improving the quality of building designs and most important reducing the cost of buildings construction in Kuwait. On the other hand the findings of the present study will form the basis for a large scale endeavor leading to the development of the much needed National Building Code for Kuwait.

Focusing on Kuwaiti villas the objective of the study was set as to assess the current practices of structural design of residential buildings in Kuwait at the different stages of evaluation of loads, materials properties, analysis and design procedures, details and specifications. Further, the assessment of over-designing was limited to the super-structure, i.e. reinforced concrete skeleton members, namely slabs, beams and columns.
2. BUILDINGS SAMPLE

The buildings sample consists of six residential units of villa type, whose details are presented in Table 1. Detailed design documents (Sadek et al., 2001) were compiled and reviewed as the existing design documents. All villas are of two to three stories, with or without basements, and the structural system is invariably of the reinforced concrete skeletal type that is most commonly used in construction. Date of construction of the selected unit ranges from 1983 to 1998 and the locations of the units cover several districts within Kuwait City. Selected units are either government or private projects designed by either governmental or private design agencies. Different systems of framing plans are included in the sample, such as solid slabs, ribbed slabs, flat plates and their combinations. Hence, it is believed that the selected sample is representative of the residential Kuwaiti villas.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Location</th>
<th>Project Type</th>
<th>Design Agency</th>
<th>No. of Floors</th>
<th>Built-up Area/Floor [m²]</th>
<th>Type of Infill</th>
<th>Structural System</th>
<th>Framing Plan</th>
<th>Foundation Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Coastal line-sector B1</td>
<td>Governmental</td>
<td>Governmental (PHA)</td>
<td>G+1</td>
<td>169</td>
<td>Concrete blocks</td>
<td>R.C. Skeleton</td>
<td>Projecting Beams + Solid Slabs</td>
<td>Isolated</td>
</tr>
<tr>
<td>2</td>
<td>Sara</td>
<td>Private</td>
<td>Private</td>
<td>B+G+1+R</td>
<td>382</td>
<td>Hebel blocks</td>
<td>R.C. Skeleton</td>
<td>Projecting Beams + Solid Slabs</td>
<td>Isolated</td>
</tr>
<tr>
<td>3</td>
<td>Different</td>
<td>Governmental</td>
<td>Governmental (PHA)</td>
<td>G+1</td>
<td>168</td>
<td>Concrete blocks</td>
<td>R.C. Skeleton</td>
<td>Mixed Flat Plates/Projecting Beams + Solid Slabs</td>
<td>Isolated</td>
</tr>
<tr>
<td>4</td>
<td>Locations</td>
<td>Governmental</td>
<td>Governmental (PHA)</td>
<td>G+1</td>
<td>121</td>
<td>Concrete blocks</td>
<td>R.C. Skeleton</td>
<td>Projecting Beams + Solid Slabs</td>
<td>Isolated</td>
</tr>
<tr>
<td>5</td>
<td>Ardiyah</td>
<td>Private</td>
<td>Private</td>
<td>B+G+1+R</td>
<td>166</td>
<td>Concrete blocks</td>
<td>R.C. Skeleton</td>
<td>Solid Slabs</td>
<td>Isolated</td>
</tr>
<tr>
<td>6</td>
<td>Qurain</td>
<td>Private</td>
<td>Private</td>
<td>B+G+1+R</td>
<td>229</td>
<td>Concrete blocks</td>
<td>R.C. Skeleton</td>
<td>Solid Slabs + Ribbed Slabs</td>
<td>Isolated</td>
</tr>
</tbody>
</table>

PHA: Public Housing Authority, B: Basement, G: Ground Floor, R: Roof, R.C.: Reinforced Concrete

3. ACCURATE DESIGN

Detailed computer-based structural analysis of the selected residential units was performed to find the straining actions in the resisting elements. For this purpose, the commercial software STAAD III (STAAD III, 1990) was employed. Several structural modeling approaches were attempted, namely 3D, 2D and 1D as shown in figure 1, and compared before determining critical values of the straining actions in the members.

The selected buildings were redesigned according to ACI-318 provisions (ACI, 1989) while considering the same concrete dimensions as stated in the existing design and hence the reinforcement was accurately found by the present redesign process. This approach of maintaining the existing concrete dimensions in the accurate design stage is essential to have a common reference between the existing and accurate design,

Figure 1. 3D, 2D and 1D modeling approaches
and, hence a meaningful comparison is possible. One shortcoming of this approach is that, although in some cases, the existing statical systems and/or proportions might not be appropriate, no comments or alternative suggestions are made. This can be justified by the absence of all design documents and other practical considerations or constraints that possibly interfered in the selection of the existing system. The following design loads criteria were used in the analysis.

Concrete strength for typical construction was specified by cube strength $f_{cu}=25$ MPa. In some buildings, concrete for columns was specified by $f_{cu} = 30$ MPa. Reinforcement minimum yield stress was taken as 420 MPa.

**Dead Loads**

- **Density of structural concrete** = 2500 kg/m$^3$
- **False ceiling and ducts** = 100 kg/m$^2$
- **Flooring** = 150 kg/m$^2$
- **Roofing** = 250 kg/m$^2$

**Ribbed one-way slabs (depth 35 cm)**

- **Uniform own weight** = 400 kg/m$^2$
- **Exterior concrete blocks wall (20 cm)** = 490 kg/m$^2$
- **Interior concrete blocks wall (15 cm)** = 435 kg/m$^2$
- **Aerated autoclaved concrete blocks (20 cm)** = 100 kg/m$^2$

**Live Load**

- **Intensity on all floors** = 200 kg/m$^2$
- **Lateral Loads**
  - **Wind loads** = none
  - **Earthquake loads** = none

**4. COMPARATIVE STUDY**

**4.1 Slabs**

**Solid Slabs**: Main and secondary reinforcements in all solid slabs found in the buildings sample were accurately evaluated and compared with the corresponding values as per the existing design. A sample comparison of the reinforcement in the existing design and the accurate design of the solid slabs in one of the units, unit 6, is presented in Table 2. The reinforcement is shown for the maximum +ve moment (field moment) in both directions. It is clear from the table that the reinforcement used is ranging between 10-50% more than the required value.

**Flat Slabs**: Flat plates encountered in the sample were of irregular configurations (columns are not of equal spacing and not aligned), and hence the simplified code method for determining the straining action by dividing the plate into column strip and field strip was not applicable. Analysis may be performed by using the equivalent frame method, outlined in the ACI-318 (1989), or more accurately, using a three-dimensional finite element model utilizing plate elements to determine the straining actions. The latter approach was employed to model flat plates using the general-purpose finite element program SAP 2000 (1995). Shell elements were used to represent the flat plate and beam elements were used to represent the marginal/drop beams.

The flat slab thickness used was 18 cm and the concrete cover was assumed to be 1.5 cm for the purpose of design calculations. Maximum values for positive (field) bending moments were used to determine the required bottom reinforcement directly, whereas maximum values for negative reinforcement were averaged for a width of 50 cm for design purposes. This procedure is a common practice to avoid designing on a very local sharp peak that typically occurs at supports as a result of the finite element discretization. The high negative moment value that is averaged is typically at limited locations that are already covered within the column cross section.

A sample comparison between the reinforcement for the flat slab system for unit 3 in the existing structure and the accurate design is given in Tables 3 and 4. The reinforcement is shown for the maximum field moment in two perpendicular directions for the first floor slab in Table 3, whereas the maximum -ve moment (on top of columns) in both directions is presented in Table 4. It is clear from the table that the percentage of increase in the reinforcement used ranges from 0 to 156% more than the required value based on the maximum moment at each strip/location.
4.2 Beams

**Flexure Reinforcement:** Detailed comparison of the existing and accurate flexure reinforcements of beams in the six units was performed. Comparison was made in terms of the reinforcement ratio of the existing reinforcement to the accurate reinforcement determined assuming the same cross sections as per the existing drawings. The reinforcement ratio is calculated for the bottom reinforcement corresponding to the maximum +ve bending moment (field moment) and the top reinforcement corresponding to the maximum –ve bending moment. A sample result is shown in Fig. 2 for unit 1. Generally, ratios higher than one indicate overdesigning and vice versa. However, and from a practical point of view, differences of up to 15 to 20 percent can be tolerated and should not be used to condemn the existing design. Examining Fig. 2, it can be seen that the values of the reinforcement ratio are typically substantially larger than one and reaching, in some cases, up to four and hence a clear case of overdesigning the flexure reinforcement of beams is evident.

To make the comparison more visible, the average values of the reinforcement ratio for the six units are shown in Fig. 3. Averaging is made over all beams and all floors for top and bottom reinforcement so that a single ratio can be obtained for each building as shown in Fig. 3. Overall average flexure reinforcement ratios range from 1.16 to 2.04, i.e. flexure reinforcement is over-designed on the average by as much as 15 to 100%. This is an alarming finding and current practices need improvement in order to achieve cost-effective designs.

**Shear Reinforcement:** Shear reinforcement in beams is in the form of stirrups provided to resist shear stresses. Shear reinforcement ratios of existing to accurate reinforcements were calculated for all beams in each unit. Average ratios similar to the one presented for flexure reinforcement are shown in Fig. 4 for the six units. The average values are ranging between 1.08 and 2.05, i.e. shear reinforcement in beams are over-designed by as much as 8 to 100%.

### Table 4. Comparison between Existing and Accurate Top Reinforcement for Flat Slab (for –ve moment) in the Second Floor of Unit 3

<table>
<thead>
<tr>
<th>Strip Location</th>
<th>Direction 1</th>
<th>Direction 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$A_{e}$ existing</td>
<td>$A_{a}$ accurate</td>
</tr>
<tr>
<td>Col. Location</td>
<td>$A_{e}$ existing</td>
<td>$A_{a}$ accurate</td>
</tr>
<tr>
<td>2$^\circ$E</td>
<td>16 20 14 20 30</td>
<td>14 20 14 20 0</td>
</tr>
<tr>
<td>4$^\circ$E</td>
<td>16 20 12 20 78</td>
<td>16 15 14 20 74</td>
</tr>
<tr>
<td>6$^\circ$S</td>
<td>16 15 14 20 74</td>
<td>14 20 14 20 0</td>
</tr>
<tr>
<td>2$^\circ$W</td>
<td>12 20 12 20 0</td>
<td>14 20 14 20 0</td>
</tr>
<tr>
<td>4$^\circ$W</td>
<td>12 20 12 20 0</td>
<td>14 20 12 20 36</td>
</tr>
</tbody>
</table>

### Figure 2. Flexure reinforcement ratio of existing to accurate design of beams in unit 1

### Figure 3. Average flexure reinforcement ratios for the six units

### Figure 4. Average shear reinforcement ratios of the six units

#### 4.3 Columns

The design of columns was based on ACI-318, assuming axial load with minimum eccentricity. Two approaches were used for the comparison between the accurate and existing designs. In both approaches, the concrete dimensions were fixed. The first approach assumed the minimum reinforcement necessary to sustain the given loads, given that the concrete dimensions are unchanged.
Improving the Design of Residential Buildings in Kuwait

It was noted that, even with the minimum reinforcement, the column capacities were much larger, indicating that the concrete dimensions chosen for the existing design are excessive. Shown in Figs. 5 and 6, are reinforcement ratios, existing to accurate, of columns in the six units, and it can be seen that ratios are generally low except for some cases, such as units 1, 3 and 4. Average column reinforcement ratios of all units are shown in Fig. 7. Apart from units 1 and 4, ratios are rather low. A second approach was used by finding the capacity of the existing columns. Ratios of the existing column capacity to the actual load acting on columns are shown in Figs. 8 and 9. Ratios are very high and can reach up to ten. This clearly indicates the oversizing of columns. Average ratios range from 2.8 to 4.3 as shown in Fig. 10.

Axial load used for the column design was based on the summation of beam reactions (in case of slab and beam type system) or summation of the SAP 2000 finite element analysis output for the reaction of each floor (in case of the flat plate system). Slenderness effect for the columns is neglected based on the height-to-width ratio of the columns. The large difference in the capacity versus load in small-sized columns is due to the minimum dimension of columns used (20 x 40 cm) in the existing design. This is an acceptable common practice to ensure proper column forming during construction. It is also possible that the increase in column size was due to architectural requirements.

5. OVERALL ASSESSMENT

In an attempt to have an overall view of the findings, the six units are classified according to the type of the project, either governmental (G) or private (P) and the type of the design agency. According to these criteria, three classifications are possible, namely: GG (governmental project and design agency), PP (private project and design agency) and GP (governmental project and private design agency). Shown in Fig. 11 is the mean, maximum and minimum beam flexure reinforcement ratios for each of the GG, PP and GP classes. Figure 12 shows the same parameters for the beam shear reinforcement, and Fig. 13 shows the column main reinforcement. Excluding the beam flexure reinforcement case, it can be seen that a larger dispersion of the ratios is evident for the PP class, as indicated by the larger difference between maximum and minimum values compared with the GG class. This can be attributed to the larger variation in the quality of design practices exercised by private consultants, which confirms the non-uniformity of designs that comes as a natural consequence of the absence of a national building code.
Figure 8. Column capacity-to-actual-load ratio in units 1, 2 and 3

Figure 9. Column capacity-to-actual-load ratio in units 4, 5 and 6

Figure 10. Average capacity-to-actual-load ratio of columns in the six units

Figure 11. Overall flexure reinforcement ratio in beams

Figure 12. Overall shear reinforcement ratio in beams

Figure 13. Overall columns reinforcement ratio
6. CONCLUSIONS

1. Based on the comparison of existing and accurate designs of the buildings sample, all structural members are invariably found to be highly over designed. Maintaining the same concrete dimensions as per the existing design, the reinforcement provided is found to be much higher than required and determined by the accurate design conducted in the present study. The average percentages, as obtained for the entire buildings sample, of increase of reinforcement are 30% for slabs, 60% for beams and 60% for columns.

2. Columns are substantially oversized in terms of concrete dimensions, the carrying capacity of these elements as per the existing design is found to be much higher than the actual applied loads by as much as 240% for columns, on the average.

3. In an attempt to have an overall view of the findings, the six units are classified into three categories, GG (governmental project and design agency), PP (private project and design agency) and GP (governmental project and private design agency). Although these categories are invariably over designed, a larger dispersion of the ratios is evident for the PP class as compared to the GG class. This can be attributed to the larger variation in the quality of design practices exercised by private consultants, which confirms the non-uniformity of designs that comes as a natural consequence of the absence of a unified national building code.

4. Serviceability requirements of allowable deflections and crack widths are satisfied in the existing designs and this finding is rather expected in view of the apparent over designing of different structural elements.

REFERENCES

1. ACI-318. 1989. Building Code Requirements for Structural Concrete. American Concrete Institute, Detroit, Michigan, USA.

