Technical Report 2
The Residences
Anne Arundel County, Maryland

10/27/2010
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Ryan English - Structural Option
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Executive Summary

This report contains an analysis of four different floor systems for the Residence. The four alternative systems that were included in this studied were:

- Hambro Floor system (Existing Floor System)
- Composite Steel Beams with Composite Deck
- Two-Way concrete Floor
- One-Way Slab

These systems were primarily compared by their building weight, architectural impact, and serviceability. Several other factors were considered in comparison of the systems such as fire protection, constructability, and cost. This study revealed that all three alternative floor systems are to be considered for further research. The Two-Way concrete Floor does pose some problem with the lack of square bay; however the column layout may be altered. All floor systems are to be included for further research.
Introduction

Located in Anne Arundel County, Maryland the Residence is a new construction apartment and retail building part of the Arundel Preserve Town Center Phase I project (Figure 1). The Residence is a five to six story, 300,000 s.f., residential apartment building with 6,000 s.f. retail space surrounding a 5 story precast parking garage. This apartment building houses 242 upscale residential units consisting of studio, one, and two bedroom layouts and two level units. Along with the residential units the building also included a terrace level that contains a clubhouse, health center, and an outside pool. Construction of The Residence began in the fall of 2009 and should be completed in the beginning of 2011. It is owned and managed by the Somerset Construction Company and was designed by KTGY.

The structure of The Residence is comprises of the Hanbro floor system, this system uses a steel bar joist that supports a concrete slab (Figure 2). The floor systems are supported by 6” light gage metal studs bearing and shear walls located throughout the building. A more in-depth structural analysis and detail shall follow in this report.
Figure 1: Site plan, Light Brown-build, Gray-parking garage. Source: Cates Engineering.

Figure 2: Hambro floor joist system. Source: Hambro.
**Structural system**

**Foundation System**

According to the geotechnical report the building rests on Silt-Clay Facies\(^1\) which is identified as clay, silt, and subordinate fine to medium grained muddy sand. The groundwater table was located to be at a minimum 24 feet below existing grade, which is well below the foundation of the building. From the report it was determined that the structures can be supported on shallow spread footings with an allowable bearing pressure of 5,000 pounds per square foot.

The building foundation system uses a 3’-0” wide strip footing with 3’-0”x3’-0” to 15’-0”x15’-0” column footing pads located mainly around the retail space and clubhouse area (Figure 3). The concrete slab on grade was 4” thick reinforced with 6 x 6 W1.4 xW1.4 welded wire fabric. All foundation concrete was to be a 3,000 psi at 28 day strength.

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\(^1\) In geology, facies are a body of rock with specified characteristics.

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**Figure 3: Foundation Plan, Part of the East wing. Source: Construction Documents.**
Floor System

The floor system that was used for the Residence was the Hambro floor joist system (Figure 2). The Hambro floor system uses a specially design steel bar joist with a “S” shape top compression chord that serves three functions, a compression member in the non-composite joist during the construction stage, a chair for the welded wire fabric, and a continuous shear connection for the composite (cured concrete) stage. Detail information of the “s” shape top cord can be seen in Figure 4. The floor slab is a 3” thick 3,000 psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric, this particular floor thickness was chosen to give the system a 2 hour fire rated system. The slab is then supported by a 20” deep Hambro bar joist.

Framing System

The design framing system used in the Residence was light gage steel load bearing walls that are used to support the Hambro floor system and gravity loads in the building. The particular system used was the SigmaStud® load bearing light gage steel stud, a product of The Steel Network Company. The stud design is engineered to have a significant increase in load capacity when compared to the conventional “C” shaped
studs. The Residence uses a 6” wide 18 gage stud with a flange length of 2.5”, as detailed in Figure 5. The exterior wall and interior corridor walls of the Residence are the primary bearing walls in the building; Figure 6 shows the location of the bearing walls in the building.

A=0.772 in²
Iₓ=4.183 in⁴
Iᵧ=0.513 in⁴
Fᵧ=50 ksi
rₓ=2.328 in
rᵧ=0.815 in
E=29,000 ksi

Figure 5: Section of light gage steel stud, with section properties.

Figure 6: Location of bearing walls, See Appendix A for more plans. Source: Construction Documents.
Figure 7: Exterior wall framing details. Source: Construction Documents.

Lateral System

The lateral system used in the Residence was a light gage shear wall system designed and engineered by The Steel Network Company. The system utilizes light gage 50 ksi steel hot dipped galvanized coated straps on both sides of the wall for shear resistance. A 6” wide flat strap was used in lateral system of the Residence. (See figure 8 for a simple framing detail). The shear walls are located all throughout the building (figure 9), with most of the shear wall located in the corridor walls and the walls separating adjacent apartments.

Figure 8: Lateral resistance system. Source: Construction Documents.
Figure 9: Location of the shear walls, Appendix A for more details. Source: Construction Documents.

**Roof System**

The roof system was the same system, Hambro flooring system, which was used for the floor throughout the building. The roof slab is 3” thick 3,000 psi concrete with 6 x 6 W2.9 x W2.9 welded wire fabric, which is supported by a 20” deep Hambro joist.
## Materials Used

### Concrete

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>f'c (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor Slab</td>
<td>Normal Weight</td>
<td>3,000</td>
</tr>
<tr>
<td>Roof Slab</td>
<td>Normal Weight</td>
<td>3,000</td>
</tr>
<tr>
<td>Slab on grade</td>
<td>Normal Weight</td>
<td>3,000</td>
</tr>
<tr>
<td>Footings</td>
<td>Normal Weight</td>
<td>3,000</td>
</tr>
</tbody>
</table>

### Steel

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>W shapes</td>
<td>ASTM A992</td>
<td>50</td>
</tr>
<tr>
<td>Square and Rectangular HSS</td>
<td>ASTM 500</td>
<td>B</td>
</tr>
<tr>
<td>Channels</td>
<td>ASTM A36</td>
<td></td>
</tr>
<tr>
<td>Angles shapes</td>
<td>ASTM A36</td>
<td></td>
</tr>
<tr>
<td>Steel Plates</td>
<td>ASTM A36</td>
<td></td>
</tr>
</tbody>
</table>

### Reinforcement

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformed bars</td>
<td>ASTM A-615</td>
<td>60</td>
</tr>
<tr>
<td>Welded wire Fabric</td>
<td>ASTM A-185</td>
<td></td>
</tr>
</tbody>
</table>
Codes and References

Design Codes

National Model Code:
2006 International Building Code

Design Codes:
Steel construction Manual 13th edition, AISC
American Iron and Steel Institute (AISI) 2008 Design of Cold Formed Steel Structural members
American Concrete Institute (ACI) ACI 530-05, Building Code Requirements for Masonry Structures
American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:
American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design loads for Buildings and other Structures

Thesis Codes

National Model Code:
2006 International Building Code

Design Codes:
Steel construction Manual 13th edition, AISC
American Concrete Institute (ACI) ACI 318-08, Building Code Requirements for Structural Concrete

Structural Standards:
American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design loads for Buildings and other Structures
Load Analysis

Gravity Load

For this report and all further reports the use of the ASCE7-05 design loads will be used. When comparing the design live loads to the minimal ASCE7-05 loads it was found that all loads except the roof live load were identical to the ASCE7-05. Table 1.1 shows the design and ASCE7-05 live loads on the building. The roof live load was design to be 30 psf which is slightly higher than what is stated in ASCE7-05, 20 psf. It is likely that this value was higher to support some of the MEP system on the roof as well as experience of the designers.

Table 1.1: Live Loads

<table>
<thead>
<tr>
<th>Location</th>
<th>Design (psf)</th>
<th>ASCE7-06 (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>Living</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Private Decks/Balconies</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>Corridors Exit stairs</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Light Storage</td>
<td>125</td>
<td>125</td>
</tr>
</tbody>
</table>

Dead loads values we found from a series of sources including, but not limited to ASCE7-05 and manufacturer specification. Design dead load on the building can be found in Table 1.2. A listing of assumed dead loads can also be found in Table 1.3.

Table 1.2: Design Dead Loads

<table>
<thead>
<tr>
<th>Location</th>
<th>Design (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>40</td>
</tr>
<tr>
<td>Living</td>
<td>55</td>
</tr>
<tr>
<td>Private Decks/Balconies</td>
<td>45</td>
</tr>
<tr>
<td>Corridors Exit stairs</td>
<td>45</td>
</tr>
<tr>
<td>Light Storage</td>
<td>45</td>
</tr>
</tbody>
</table>
Table 1.3: Assumed Dead Load

<table>
<thead>
<tr>
<th></th>
<th>Assumed load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>36*</td>
</tr>
<tr>
<td>Joist</td>
<td>5</td>
</tr>
<tr>
<td>Supper impose Dead load</td>
<td>15</td>
</tr>
<tr>
<td>wall</td>
<td>15</td>
</tr>
</tbody>
</table>

* Slab dead load was calculated using a 3" think slab and 145 pcf for concrete

Snow Load

Due to the location of this building being a snow region, snow loads were calculated in accordance to ASCE7-05 section 7. The results of the load calculation can be seen in table 2, with detail calculation and notes can be found in Appendix B.

Table 2: Snow loads

<table>
<thead>
<tr>
<th>Snow load</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground snow load</td>
<td>Pg= 30 psf</td>
</tr>
<tr>
<td>Flat roof snow load</td>
<td>Pf= 21 psf</td>
</tr>
<tr>
<td>Slope roof snow load</td>
<td>Ps= 21 psf</td>
</tr>
</tbody>
</table>
Floor Systems

For this report, a typical interior bay lay out of The Residence will be analyzed for the existing floor system and three alternative floor systems; existing framing plans are provided in Appendix A. Figure 10 shows the layout of the typical interior floor plan that was uses in this report. This particular floor plan was chosen to minimize the need to place columns in the apartments. The design of each floor system along with their advantages and disadvantages shall follow with detail calculation in Appendix C. The effects of lateral loads and sizing of column were not investigated in this report but would need to be done to complete a throw design.

Figure 10: Typical Floor plan lay out.
Hambro Floor system (Existing Floor System)

Description

The Hambro floor system uses a 3” thick 3,000 psi concrete floor slab with 6 x 6 W2.9 x W2.9 welded wire fabric, this particular floor thickness was chosen to give the system a 2 hour fire rated system. The slab is then supported by a 20” deep Hambro bar joist. This is a specially design steel bar joist with a “S” shape top compression chord that serves three functions, a compression member in the non-composite joist during the construction stage, a chair for the welded wire fabric, and a continuous shear connection for the composite (cured concrete) stage.

A typical bay width used in The Residence is approximately 32’-0” with the length of the bay varying with the sizes of apartment units. For this report a 32’-0” x 36’-0” bay size was used to check member sizes, Figure 11 shows the bay layout.
Figure 11: Hambro Floor play lay out.

Material Properties

<table>
<thead>
<tr>
<th>Concrete:</th>
<th>3” Normal Weight concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{c} = 3000$ psi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>$F_y = 60,000$ psi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Welded Wire Fabric 6 x 6 x W2.9 x W2.9</td>
</tr>
</tbody>
</table>

Loading

<table>
<thead>
<tr>
<th>Dead Load (self weight):</th>
<th>41 psf</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDL:</td>
<td>15 psf</td>
</tr>
<tr>
<td>Live Load:</td>
<td>40 psf (Living units)</td>
</tr>
<tr>
<td></td>
<td>100 psf (Corridors)</td>
</tr>
</tbody>
</table>
Advantages

There are many advantages to using the Hambro floor system. The first advantage is the easiness of the construction which the Hambro uses a simple eight step approach to install the system allowing for a shorter construction time. The system also use standard 4’ x 8’ plywood sheets for the bottom formwork for the concrete. The use of bar joists allows significant space for the mechanical duct work, piping, and electrical wires. The overall weight of the system is much less than other system allowing the foundation to be much smaller.

Disadvantages

Only a few disadvantages could be found with the Hambro system. The first being that the contractor must have some understanding of the installation presses of the floor system, even with the simple eight step approach. The system must be installed properly to allow for adequate strength and safety of the system. To aquaria the specified fire rating a ceiling of at minimal ½” gypsum board must be used.
Composite Steel Beams with Composite Deck

Description

The composite metal deck on composite steel beam is a system that combines the strengths of steel in tension and compression of the concrete, to provide an effective system. A typical bay system was used to design the composite steel systems, (see figure 12 for the layout). W-shape girders span from column to column with an infill beam framing into the girder. The metal deck that sits on the beam spans perpendicular to the beam. When using metal decking, composite action is easily obtained. However, extra design steps are needed to obtain composite beam action. For a beam to obtain composite action with the slab, shear studs are required along the length of the beam. The shear studs transfer the load from the concrete slab into the beam. The supporting calculations for the design of the composite steel system may be found in Appendix C: Composite Steel Beams with Composite Deck.
Figure 12: Composite Beam and Deck floor lay out.

Material Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>3” Normal Weight concrete slab on Metal Deck</td>
</tr>
<tr>
<td></td>
<td>$f'c = 3000$ psi</td>
</tr>
<tr>
<td>Decking</td>
<td>17 Gage metal Deck, Valcraft 2VLI17</td>
</tr>
<tr>
<td>Steel</td>
<td>A922 W-Shapes</td>
</tr>
<tr>
<td></td>
<td>Beams: W14</td>
</tr>
<tr>
<td></td>
<td>Girders: W16, W21</td>
</tr>
</tbody>
</table>
Loading

<table>
<thead>
<tr>
<th>Loading</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load (self weight):</td>
<td>51 psf (slab)</td>
</tr>
<tr>
<td></td>
<td>10 psf (Beam)</td>
</tr>
<tr>
<td>SDL:</td>
<td>15 psf</td>
</tr>
<tr>
<td>Live Load:</td>
<td>40 psf (Living units)</td>
</tr>
<tr>
<td></td>
<td>100 psf (Corridors)</td>
</tr>
</tbody>
</table>

Advantages

A composite metal deck on composite steel system has many advantages. The metal deck provides the necessary formwork to place the concrete, with proper beam spacing; no shoring is required during construction. The composite system allows the use of smaller steel members and a thinner concrete slab making it a light weight system. A shorter construction time is achieved with composite beam and deck system compared to other systems.

Disadvantages

A composite beam system does have smaller beams, but the beams are still around 16 inches deep. The space between the ceiling and the bottom of the slab may need to be increase to allow for the mechanical and electrical systems. There is a few more cost associative with the connections of a composite beam system. A faster construction time is achieved with the composite steel; however there is an increase in labor for the placement of the shear studs. To obtain the proper fire rating a spray on fireproofing is required for the structural steel.
Description

The design of the Two-Way reinforced flat slab system is comprised of 11” thick normal weight concrete slab with 2.75” drop panel, Figure 13 shows the layout of the floor system. The typical reinforcement used across the entire system is #8 bars at minimal 12 inches on center.

The slab was designed to resisted flexural, shear, and deflection. The Equivalent Frame Method prescribed by ACI 318-08 was used to design the floor system. The slab thickness of 11” was minimum required in accordance with ACI 318-08 Table 9.5(c). Punching shear and wide beam shear was checked at the columns and drop panels, but was found not to exceed the limits. The preliminary sizes for the columns are 12” square; this however may have been an underestimation, further investigation would need to be conducted to confirm. The system was not design for progresses collapse but would need to be considered. The supporting calculations for the design of the Two-Way Flat Slab system may be found in Appendix C: Two-Way Flat Slab concrete Floor.
Figure 13: Two-Way Flat Slab floor lay out.

Material Properties

| Concrete:                  | 11” Normal Weight concrete with Drop Panels |
|                           | 12” x 12” columns                           |
|                           | f’c = 4000 psi                               |
| Reinforcement             | Fy = 60,000 psi                              |

Loading

| Dead Load (self weight): | 150 psf |
| SDL:                    | 15 psf  |
| Live Load:              | 40 psf (Living units)                       |
|                         | 100 psf (Corridors)
Advantages

A Two-Way Flat Slab system provides a large floor to ceiling height; this allows more space between the ceiling and the bottom of the slab for mechanical and electrical system. No interior beams were used to support the slab; therefore more space could be coordinated with the mechanical and electrical disciplines. Additional fireproofing is not required for the concrete system because it is built into the clear cover of the steel.

Disadvantages

A Two-Way Flat Slab design requires an aspect ratio of less than 2; the corridor bays of the build do not meet this requirement. To achieve this ratio, the bay sizes would have to be change to be squarer; this would have an impact on the architectural design of the apartment units. Construction time for placing the concrete is long because of the increase of time for forming and shoring of the concrete. The weight of the system is much greater than the other systems there for the foundation may have to be redesign for the additional weight.
One-Way Slab

Description

The one-way slab system was designed for an 8” concrete slab that spans a maximum distance of 17’. A girder spans between the columns with a beam framing into the girder. Figure 14 shows the layout of the floor system. The 8” slab was designed to have the following reinforcement; #6 at 12” o.c for flexure steel and #4 at 12” o.c. were provided for temperature steel. The preliminary sizes for the columns are 12” square this however may have been an underestimation; further investigation would need to be conducted to confirm.

Figure 14: One-Way floor lay out.
Material Properties

| Concrete             | 8” Normal Weight concrete 8” Normal Weight concrete  
|                     | 12” x 12” Columns          |
| Reinforcement       | Fy = 60,000 psi             |

Loading

| Dead Lead (self weight): | 110 psf         |
| SDL:                    | 15 psf          |
| Live Load:             | 40 psf (Living units) |
|                        | 100 psf (Corridors) |

Advantages

At this time the only advantage to a one-way floor system is that additional fireproofing is not required for the concrete system because it is built into the clear cover of the steel.

Disadvantages

A one-way slab has many disadvantages when compared to other floor systems. Construction time for placing the concrete is long because of the increase of time for forming and shoring of the concrete. The weight of the system is much greater than the other systems; the foundation may have to be redesigned for the additional weigh.
Conclusion

The analysis of the three alternative floor systems and the existing floor system of the Residence revealed that there were many available systems that can be used for the design of the building. Each floor system presented their own set of advantages and disadvantages. The existing system, the Hambro floor system, provided a low weight, ease of construction, and a low cost system. The composite beam and deck has many of the same advantages and disadvantages that the Hambro floor system has. This system did come at an additional cost for the need to install fire protection and the installation of shear studs. The One-Way and Two-Way concrete system also shared similar advantages and disadvantages. One advantage that both of these systems has is the lack fore addition fire protection; the fire protection is built into the clear cover of the rebar. One drawback of these systems is the increase of weight; this would have an effect on the foundation and seismic load. The Two-Way system does have a problem with the layout of the column, this layout does not have continues square bays. Rearranging the column layout to achieve square bays maybe have an affect the architectural lay out of the apartment units. A comparison of the four systems can be found in the following table. All floor systems are to be included for further research.
## System Comparison

<table>
<thead>
<tr>
<th></th>
<th>Hambro Floor System</th>
<th>Composite Beam &amp; Deck</th>
<th>Two-Way Flat Slab</th>
<th>One-Way Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Weight (Dead)</strong></td>
<td>41 psf</td>
<td>61 psf</td>
<td>150 psf</td>
<td>110 psf</td>
</tr>
<tr>
<td><strong>Architectural Impact</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column Layout</td>
<td>N/A</td>
<td>Good</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td>Floor Depth</td>
<td>20&quot; *</td>
<td>14&quot;-21&quot;</td>
<td>13.75&quot;</td>
<td>19&quot;</td>
</tr>
<tr>
<td>Deflection</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td>Vibration</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td>Constructability</td>
<td>Easy</td>
<td>Easy</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
<tr>
<td>Fire Protection</td>
<td>2 hr.</td>
<td>2 hr.</td>
<td>2 hr.</td>
<td>2 hr.</td>
</tr>
<tr>
<td>Foundation Impact</td>
<td>Little</td>
<td>Little</td>
<td>Major</td>
<td>Major</td>
</tr>
<tr>
<td>Approximate Cost</td>
<td>$17.87 ++</td>
<td>$25.80 **</td>
<td>$19.20</td>
<td>$23.85</td>
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<tr>
<td>Additional Study</td>
<td>N/A</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

* The Hambro Floor system allow mechanical and electrical equipment to pass through the bar joist

** Cost includes Fire Protection cost

+ Cost data attained form RSMeans 2011

++ Cost date is for a typical steel bar joist system, cost data for the Hambro System was unavailable
Appendix A: Building Plans.

Fifth Floor layout. Source: Construction Documents
Exterior and interior wall connection Source: Construction Documents
Appendix B: Snow Load Analysis

ASCE7-05 Section 7

(7.2) Ground snow load

\[ P_g = 30 \text{ psf} \]

(7.3) Flat Roof

\[ P_f = 0.7 \cdot C_e \cdot C_t \cdot I \cdot P_g \]

(7.3.1) Exposure Factor

\[ C_e = 0.9 \]

(7.3.2) Thermal Factor

\[ C_t = 1.1 \]

(7.3.3) Importance Factor

\[ I = 1.0 \]

\[ P_f = 0.7(0.9)(1.1)(1.0)(30) = 20.79 \rightarrow 21 \text{ psf} \]

(7.4) Slope Roof

\[ P_s = C_s \cdot P_f \]

\[ C_s = 1.0 \]

\[ P_s = 21 \text{ psf} \]

Snow Drifting

\[ L_n = 11' - 6'' \]

\[ h_d = \frac{0.343}{\sqrt[4]{P_g}} \sqrt[4]{P_g} + 10 - 1.5 = 2.64' \]

\[ w = 4 \cdot h_d = 10.58' \]

\[ \gamma = 0.13 \cdot P_g + 14 = 17.9 \]

\[ P_d = h_d \cdot \gamma = 47.25 \text{ psf} \]
Appendix C: Floor System analysis

Hambro Floor system (Existing Floor System)
Moment

\[ M = \frac{W}{12} \left( L - \frac{b}{2} \right) \]

\[ W = 1200 \text{ lb} \]

\[ L = 12 \text{ ft} \]

\[ b = 6 \text{ in} \]

\[ t = 3 \text{ in} \]

\[ f'c = 3000 \text{ psi} \]

\[ d = 2 \text{ in} \]

\[ d' = 2\frac{1}{2} \text{ in} \]

\[ \phi = 0.16 \]

\[ P_c = 200 \text{ lb} \]

\[ P_t = 100 \text{ lb} \]

\[ P_t' = 0 \]

\[ P_r = 0 \]

\[ P_r' = 0 \]

\[ P_s = 0 \]

\[ P_s' = 0 \]

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\[ M_k = A_d \times \frac{d}{2} \times h_0 \]
\[ A_d = 0.56 \text{ in}^2 \]
\[ h_0 = 60 \text{ Kip} \]
\[ d = D^2 - 0.8\pi \times 6976 \]
\[ 19.38 \]
\[ 651.17 \text{ K-in} \]
\[ 54427 \text{ K-ft} \]
\[ \Phi M_k = 0.9 \times (54427) = 498.8 \geq 404 \text{ K-ft} \]
\[ M = \frac{1}{6} \times 2.83 \times 6.23 \times 2 \times \frac{1}{2} \times 81' \times 81' = 151,803 \text{ in}^3 \]

\[ A = \frac{50,000 \text{ psi}}{2,000 \text{ psi}} = 25 \text{ psi} \]

\[ \sigma = \frac{151,803 \times 1}{25} = 6,072 \text{ psi} \]

\[ N = 4,360 \text{ psi} \times 2,000 \text{ psi} = 8,720 \text{ psi} \]

\[ E = 4,000 \text{ psi} \]

\[ \gamma = \frac{4,145}{4} \times 10^{-6} \text{ psi} \]

\[ v = 29.9 \text{ psi} \times 10^{-6} \]

\[ I = \frac{29,000}{30,000} \times 3.37 = 9.29 \]

\[ I = \frac{(539)(3)}{3} \times (1728) = 3,764 \]
The Residences
Technical Report 2
Anne Arundel County, Maryland
10/27/2010

Dr. Richard A. Behr

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Equation:
\[ w = \frac{1}{2} (36 + 20) \]

Floor:
\[ 1.2(41) + 1.4(21) + 20 = 101.6 \text{ psf} \]
\[ 16'(101.6) = 1625.6 \text{ plf} \]

Ceiling:
\[ 1.2(50) + 1.6(40) = 131.2 \text{ psf} \]
\[ 16'(131.6) = 2105.6 \text{ plf} \]

Wall:
\[ 1.2(15) = 18.0 \text{ psf} \]
\[ 11'(18) = 198 \text{ plf} \]

Floor + wall:
\[ 2303.6 \text{ plf} \]

- Roof:
  - 1625.6 plf
  - 2303.6 plf
  - 2303.6
  - 2303.6
  - 2303.6
  - 198

13391.6 plf → 13,391,600

Ass. stud are placed 16" OC:
\[ 13,391 \div 12 = 17,791.67/16" \]

Pu = 17.79 k
\[ \phi P_n = F_{cr} A_y (0.9) \]

\[ K_y = \frac{10,132}{132} = 76.7 \text{ ksi} \]

\[ K_x = 1.0 \]

\[ F_{cr} = \frac{9.77 (29000)}{56.7^2} = 89.8 > 0.72 F_{yd} = 22 \]

\[ F_{cr} = \begin{bmatrix} 0.658 \\ 0.658 \end{bmatrix} F_{yd} \]

\[ F_{yd} = \begin{bmatrix} 0.658 \\ 0.658 \end{bmatrix} 50^2 = 39.5 \]

\[ \phi P_n = 39.5 (0.772) (0.9) = 27.7 \text{ ksi} > 17.7 \text{ ksi} \]
Composite Steel Beams with Composite Deck
Design B1 88
\[ LL : \text{SD 10} \]
DL SDL 15 psf
Shb 51 psf
Assg 55 psf
Ex wall 15 psf

\[ w_a = 1.2(15 + 51 + 10) + 1.6(55) = 116 \text{ psf} \]
\[ 1.2(15)(11) = 116 \text{ psf} \]
\[ M_a = \frac{w_a L^2}{8} = \frac{1.17(34)^2}{8} = 169 \text{ k-ft} \]
\[ \Delta_{max} = \frac{M_{max}}{I_{360}} = \frac{3}{4}(34)\frac{360}{12} = 10.13 \text{ in} \]

**Composite Beam**

**Table 3-19**

- W12 x 22
  - 179 > 169
  - 1.16
  - 2.7
  - 1.31
  - 95.1
  - 55.6 (15)
  - 12(10) = 100

- W10 x 25
  - 170 > 169
  - 1.31
  - 95.1

- W19 x 22
  - 179 > 169
  - 3.31
  - 91.2

- W12 x 19
  - 178 > 169
  - 2.27
  - 169
  - 9.8
  - 10.1

- W16 x 19
  - 177 > 169
  - 9.26

- L10 x 19
  - 1.86 > 169
  - 5.01

**Deck is perpendicular**

**Weak Stems**

- C4 = 8.15
- C6 = 13.2" x 10.4"
Design B9, B3, B6, B7

\[ \omega_0 = \left[ 1.2(15+5)+1.6(50) \right] 11'-4'' = 1940 \text{ pcf} \rightarrow 1.94 \text{kN/m} \]

\[ M_u = \frac{\omega_0^2}{8} = \frac{1.94 \times (34)^2}{8} = 2600 \text{kN-m} \]

\[ \Delta_{\text{max}} = \frac{L}{360} = \frac{34(12)}{360} = 1.13'' \]

Composite Beam

\[ I = \text{min} \left( \frac{3n^2(n^2)}{12} \right) \]

Deck is perpendicular, weak stud.

1 stud: 3/4'' stud, \( f'_c = 3 \text{kN/m} \)

\( C_n = 17.2 \text{kN} 

\( y_2 = 5-1 = 4 \)

\( \Delta = W12 \times 30 \)

\( f_{y1} = 0.22 \)

\( Q_n = 29.6 \)

\( 2.96(17.2) = 18(36) \)
Design B4, B5

\[ W_e = 1.6 \times (15 \times 51 \times 10) (8 \times 8') + 1.6 \times (100) (3') \]
\[ + 1.6 \times 100 (3') \]
\[ = 1724 \text{ kips} \rightarrow 1.72 \text{ kips} \]

\[ M_e = \frac{W_e^2}{8} = \frac{1.72 \times (3\text{ kips})^2}{8} = 249 \text{ kips} \cdot \text{ft} \]

\[ D_{max} = 1.13'' \]

Composite Beam

Table 2-19

<table>
<thead>
<tr>
<th>Beam</th>
<th>( I )</th>
<th>( W )</th>
<th>( A )</th>
<th>( d )</th>
<th>( f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>B12</td>
<td>12.23</td>
<td>12.23</td>
<td>12.23</td>
<td>12.23</td>
<td>12.23</td>
</tr>
</tbody>
</table>

For perpendicular

\[ f = \frac{17.2}{117} = 4.2'' \]
\[ P_c = \frac{w_1}{l} \]
\[ W_1 = 1.94 \times 10^3 \text{ kips} \]
\[ L = 39' \]
\[ 2P_c = 66 \text{ kips} \]
\[ M_u = Pa = 66(11-4') = 714 \text{ kips-foot} \]
\[ \Delta_{max} = \frac{440}{3360} = 1.33 \]

**Composite Beam**

\[ b_{eff} = \frac{24002}{8} = 1.48 \]
\[ \frac{340}{2} = 41.0 \]

\[ A_{ss} \approx 1'' \quad Y_z = 4'' \]

**Table 2-19**

<table>
<thead>
<tr>
<th>W18 x 6.5</th>
<th>W18 x 5.5</th>
<th>W21 x 8.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>78.5</td>
<td>77.5</td>
<td>76.7</td>
</tr>
<tr>
<td>0.316</td>
<td>0.118</td>
<td>0.226</td>
</tr>
<tr>
<td>41.84</td>
<td>69.7</td>
<td>48.8</td>
</tr>
<tr>
<td>29.5 (55)</td>
<td>39.2 (54)</td>
<td>37.6 (53)</td>
</tr>
</tbody>
</table>

**Equations**

\[ \frac{45(3+1) + 58(0)}{2} = 2620 \]
\[ 2640 \]

**Conclusion**

NE-01
\[ p = \frac{wL^2}{2} \\
L = 24 \\
= 33 \text{kN} \]

\[ w = 1.2(15 + 11) = 198 \text{ kN} \]

\[ M_u = 403 \]

\[ a_{\max} = 1.13 \]

Composite Beam:

\[ P_{\text{max}} = \frac{241}{2} \approx 51 \text{ in} \]

\[ a = 0.9 \quad a = 1 \quad Y = 41 \]

Table 3-19

| W16 x 36 | W19 x 38 | W14 x 38
|----------|----------|----------|
| 9.27 x 42.7 | 9.27 x 40.8 | 9.27 x 40.8
| Y = 0.212 | Y = 0.280 | Y = 0.280
| G_n = 378 | G_n = 412 | G_n = 412
| n = 21.4 + 22.44 | n = 24.0 + 29.54 | n = 24.0 + 29.54
| 166.4 | 185.2 | 173.6
Check $\alpha$

\[
\alpha = \frac{\sigma_{ct} \cdot y_z}{f_{c,t}} = \frac{61.2 \cdot 1.69}{5.0} = 20.6 > 4'' \text{ OK.}
\]

$M_{b, M_p}$ Beam (Unslabbed)

\[
\phi_b M_p = 1.25 \times 45
\]

$\omega_{p} = 1.2 \left( \frac{y_t}{y_z} + 1.6 \left( \frac{y_t}{y_z} \right)^2 \right) = 4.35 \times 10^6$

\[
\omega_n = 1.2 \left( \frac{y_t}{y_z} + 2.2 \left( \frac{y_t}{y_z} \right)^2 + 1.6 \left( \frac{y_t}{y_z} \right)^3 \right) = 5.54 \times 10^6
\]

\[
\omega_{n, u} = \frac{\omega_n}{8} = 680.2 < 125'' \text{ OK}
\]

\[
\Delta_1 = \frac{s_0}{5.67} \times 2.84 \times 10^6
\]

\[
\Delta_2 = \frac{s}{384} \times \frac{w \theta^2}{E I} = 0.289 \times (2.4)^4 (1728) = 0.8 < 1.13'' \text{ OK.}
\]

\[
\Delta_3 = \frac{s}{384} \times \frac{c(3.141)(3.4)^4 (1728)}{29.000} = 1.6 < 1.13'' \text{ OK, add } \tfrac{3q}{8} \text{ camber}
\]

$81, 82, 83, 86, 87$ Wi4 x 22

Check $\alpha$

\[
\alpha = \frac{279}{800(300)} = 1.07 < \frac{1.07}{2} = 0.536 > 4'' \text{ OK.}
\]

$M_{b, M_p}$ Beam (Unslabbed)

\[
\phi_b M_p = 151
\]

$\omega_{p} = 1.2 \left( \frac{y_t}{y_z} + 2.2 \left( \frac{y_t}{y_z} \right)^2 + 1.6 \left( \frac{y_t}{y_z} \right)^3 \right) = 1.09 \times 10^6$

\[
\omega_n = \frac{\omega_n}{8} = 1.09 \times (3.4)^4 (1728) = 15.7 < 151 \text{ OK.}
\]

\[
\tfrac{3q}{8} = 2.35 > 2.80
\]

$\alpha = \frac{111}{800(300)} = 0.426 < \frac{1.07}{2} = 0.536 < 4' 79'' > 4'' \text{ OK.}
\]

\[
\phi_b M_p = 177 > 156 \text{ OK}
\]
\[ \Delta_{LL} \quad w_{ll} = 50 \left( 11.83 \right) = 593 \text{ KLI} \]
\[ \Delta_{ll} = \frac{6}{384} \frac{0.586 \left( 3.4 \right)^4 \left( 1728 \right)}{29000} = 1.12 < 1.13 \text{ OK} \]
\[ \Delta_{c} = \frac{5}{384} \frac{0.664 \left( 3.4 \right)^4 \left( 1728 \right)}{29000} = 2.15 \geq 1.13 \quad \text{add } 1\frac{1}{4}'' \text{ camber} \]

84, 85: \text{ wiki data...}

Check \( a \)

\[ a = \frac{2B3}{1.85(3\times120)} = 1.09 \quad \gamma_2 = 5 - \frac{1.09}{5} = 4.45 > 4 \text{ OK} \]

Min Beam Unshored Strength.

\[ f_{lmp} = 12.5 \]

\[ W_n = 1.2 \left( 51(8.67)+22 \right) + 1.6 \left( 20(8.67) \right) = 834 \text{ KLI} \]

\[ M_n = \frac{W_n}{8} = 0.824 \left( 3.4 \right)^2 = 120 \quad \leq 125 \text{ kI} \text{ OK} \]

\[ \Delta_{LL} \quad W_{LL} = 50 \left( 5.67 \right) + 100 \left( 3 \right) = 583 \]

\[ \Delta_{l} = \frac{6}{384} \frac{0.583 \left( 3.4 \right)^4 \left( 1728 \right)}{29000} = 1.09 < 1.13 \text{ OK} \]

\[ \Delta_{o} = \frac{5}{384} \frac{0.664 \left( 3.4 \right)^4 \left( 1728 \right)}{29000} = 2.12 \geq 1.13 \quad \text{add } 1\frac{1}{4}'' \text{ camber} \]
\[ \text{Check, } \phi_{21} \leq 55^\circ \]

\[ a_2 = \frac{480}{350(300)} = 1.77 \]

\[ y_2 = 5 - \frac{1.72}{2} = 4.11 > 4 \text{ ok} \]

- Beam unchord strength
  - \[ M_{uc} = 4.73 \]
  - \[ P = 4.01 \]
  - \[ \omega = 1.09 \]
  - \[ L = 34 \]
  - \[ = 18.5^\circ \]
  - \[ 2P = 2.7^\circ \]
  - \[ M_{uc} = 3.7(11.33) = 419 < 473 \text{ ok} \]

\[ \Delta_L = \frac{P_0}{E I} \left( 3L^2 - 4a^2 \right) = \frac{19.2(11.33)}{24(29000)(1540)} \left( 3(300^2) - 4(11.33)^2 \right) (144) = 0.622 < 1.13 \text{ ok} \]

\[ \Delta_C = \frac{0.12(11.33)}{24(29000)(1540)} \left( 3(300^2) - 4(11.33)^2 \right) (144) = 0.064 < 1.13 \text{ ok} \]

\[ G_{16} = 16 \times 36 \]

\[ a = \frac{378}{0.85(300)} = 2.49 \]

\[ y_2 = 5 - \frac{2.49}{2} = 4.55 < 4 \]

- Beam unchord strength
  - \[ M_{uc} = 4.12 > 4.03 \text{ ok} \]

\[ a = \frac{378}{0.85(300)} = 2.49 \]

\[ y_2 = 5 - \frac{2.49}{2} = 4.55 < 4 \]

- Beam unchord strength
  - \[ M_{uc} = 4.12 > 4.03 \text{ ok} \]
\[ \Delta_{UL} = \frac{\Delta}{2} = \frac{0.586 \times 320}{2} = 94.6 \text{ kN} \]

\[ \Delta_{LL} = \frac{9.6 \times (11.33)}{2 \times (27000 \times 10^{6})} \left( \frac{3(24^{2})}{4(11.33)^{2}} \right) \left( \frac{144}{144} \right) = \Delta = 0.26 > 1.13 \text{ OK} \]

\[ \Delta_c = \frac{0.6}{2} = 0.3 \text{ kN/m} \]

\[ \Delta_c = \frac{10.2 \times (11.33)}{2 \times (24000 \times 1448)} \left( \frac{3(24^{2})}{4(11.33)^{2}} \right) \left( \frac{144}{144} \right) = \Delta_c = 1.52 \text{ kN/m} \]
Two-Way Flat Slab concrete Floor

Flat slab with drop panels with edge beams.

Slab Thickness

ACI 318-05 Table 9.5c

\[ h = \frac{1}{36} = \frac{33'}{36} = 1'11'' \]

Drop panel

\[ t = \frac{1}{4}1'' = 2.75'' \]

\[ \frac{3'}{3} = 5.5' \]
Equivalent frame action must be used.

Section A - A slab:

\[ I = \frac{34(12)}{12} \times 45259 \text{ in}^4 \]

Section B - B slab with drop panel:

\[ I = 62.12 \text{ in}^4 \hspace{1cm} A = 4884 \]

Column section:

\[ I = \frac{62544}{1 - \frac{1}{33}} = 66392 \text{ in}^4 \]

From computer analysis: \( M_{u} \) (Kips)

<table>
<thead>
<tr>
<th>Span</th>
<th>Sup</th>
<th>Mid</th>
<th>Sup</th>
<th>Mid</th>
<th>Sup</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>6536</td>
<td>-998.1</td>
<td>-954</td>
<td>-948.1</td>
<td>6585.5</td>
</tr>
</tbody>
</table>

\[ \alpha = \frac{h}{l} = \frac{34}{33} = 1.03 \]

\[ C = 6589 \]

\[ \beta_{p} = \frac{6589}{2 \times 45259} = 0.072 \]
\[ \frac{4488}{396} \cdot 8.25 \approx 7.69'' \uparrow \]

\[ I = I_1 + A d^2 \]

\[ 48254 + 4488(8.25 - 7.69)^2 = 46661 \]

\[ 90.75 + 396(7.69 - 1.375)^2 = 15883 \]

\[ \frac{62544}{62544} \]

5.5 22 5.5

\[ 1.2(15)(34') = 612 \text{ kips} \]

\[ 1.2(13.75)(34') = 50610 \text{ kips} \]

\[ 1.2(5.09) = 6108 \text{ kips} \]

\[ 1.6(40)(58') = 2176 \text{ kips} \]

\[ 100 \cdot 5049 \text{ kips} \]
<table>
<thead>
<tr>
<th>Ext. Span</th>
<th>Int. Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ext. Supp</td>
<td>Middle</td>
</tr>
<tr>
<td>Int. Supp</td>
<td>Int.</td>
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<table>
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<tbody>
<tr>
<td>Ext. 120 k</td>
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<td>392.1 k</td>
</tr>
<tr>
<td>-715.0 k</td>
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<table>
<thead>
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<th>Middle</th>
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</thead>
<tbody>
<tr>
<td>Ext. -10.2 k</td>
</tr>
<tr>
<td>261.9 k</td>
</tr>
<tr>
<td>-238.5 k</td>
</tr>
</tbody>
</table>

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**Required Reinforcement**

<table>
<thead>
<tr>
<th>Mu</th>
<th>Kg</th>
<th>b</th>
<th>h</th>
<th>d</th>
<th>As (in²)</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ext.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>120</td>
<td>204</td>
<td>9</td>
<td>3.33</td>
<td>5 #8</td>
<td>A₀ = 3.95 in²</td>
</tr>
<tr>
<td>Int.</td>
<td>711</td>
<td>204</td>
<td>9</td>
<td>10.89</td>
<td>19 #8</td>
<td>A₀ = 11.06 in²</td>
</tr>
<tr>
<td>Middle</td>
<td>112</td>
<td>204</td>
<td>9</td>
<td>19.75</td>
<td>25 #8</td>
<td>A₀ = 19.75 in²</td>
</tr>
<tr>
<td>Int.</td>
<td>237</td>
<td>204</td>
<td>9</td>
<td>6.58</td>
<td>9 #10</td>
<td>A₀ = 7.11</td>
</tr>
</tbody>
</table>

**Int. Span**

<table>
<thead>
<tr>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ext. 715.5 k</td>
</tr>
<tr>
<td>204</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Middle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ext. 238.5 k</td>
</tr>
<tr>
<td>204</td>
</tr>
</tbody>
</table>
Structural Option
Anne Arundel County, Maryland

10/27/2010

Dr. Richard A. Behr

---

Column
$I = \frac{39425 \times 12}{1} = 30978 \text{ in}^4$

From computer analysis Mean(K-22):

Ext. Span  Int. Span
Ext. Mid  Int. Mid  Int. Mid
-102.7  366.7  -625  -595  200

$\frac{f_y}{f_0} = \frac{20}{33} = 0.61$

$\lambda = 0$

$C = 6589$

$\beta_2 = \frac{6589}{2 \times 39925} = 0.084$

---

Exterior Span, Column step: interior negative 75%, positive 60%, exterior 99%

Interior Span, Negative 75%, Positive 60%
\[
\begin{align*}
A &= 26.40 \\
\frac{y}{y'} &= 8.25 \\
A &= 29.7 \quad (7.55 - 1.375)^2 \\
\end{align*}
\]

\[
\begin{align*}
26.20 + 26.40 (8.25 - 7.55)^2 \\
18.7 + 29.7 (7.55 - 1.375)^2 \\
\end{align*}
\]

\[
\begin{align*}
1.2 (15.20) &= 36 \text{ kip} \\
1.2 (127.5 - 20) &= 3.3 \text{ kip} \\
1.2 (3.06) &= 3.67 \text{ kip} \\
1.6 (40)(17) + 1.6 (100)(3) &= 157 \text{ kip}
\end{align*}
\]
Shear:

Beam shear

\[ V_u = 166 K \]
\[ V_c = 0.75 \times \sqrt{400} \times (12) \times (9) = 174.2 \]
\[ 166 < 174.2 \text{ OK} \]

Punching shear

\[ d = 11.75 \text{ in} \]
\[ V_u = 802.7 K \]
\[ V_c = 0.75 \times \sqrt{400} \times (11.75) \times (9.5) = 211.8 K > 802.7 K \text{ OK} \]

Torsion was not taken into account in this report but could need to be looked into.

\[ d = 11.75 \]

\[ \begin{array}{c}
15 \text{ psf} \\
150 \text{ psf} \\
40 \text{ psf} \\
100 \text{ psf} \\
36.87 \text{ K} \\
31.96 \text{ K} \\
84.86 \text{ K} \\
133.86 \text{ K} \\
189 \text{ K} \\
146 \text{ K} \\
165 \text{ K} \\
582 \text{ K} \\
202.7 K
\end{array} \]
One-Way Slab

\[ f'c = 3,000 \]

Glob. in living units.

4 in. 8

Table 9.5.20

\[ h = \frac{L}{24} \text{ one end} \]

\[ h = \frac{(17-1)(12)}{24} = 8'' \]

Deflection does not need check if using Table 9.5.20

Ass: 12" beams
Try 1#6 @ 12" oc  A₀ = 0.44

M₀ = A₀ fy (d - 0.1d)

A = A₀ fy

M₀ = 0.85 (3) \( \times \) (60)

A = 0.85 (3) \( \times \) (12)

A₀ = 0.85 \( \times \) 863

\( \varepsilon = \frac{E_0 (d - c)}{E_0 (c + 0.01)} = 0.003 \cdot (4 - 1.01) = 0.002 \cdot \frac{1}{1.01} \)

0.85 \( \times \) 863 > 0.002 \( \cdot \) 1.01

\( \gamma = 0.9 \)

M₀ = 0.9 (7.85) = 7.06 kN/m² > 6.96 kN/m²

Shear and temperature reinforcement

A₀ = 0.0012 \( \times \) 0.0018 (12) \( \times \) 0.173 m³ → # 4 @ 12" oc A₀ = 0.2 m²

Slab in corridor

\( \omega = 1.2 (100 + 15) + 1.6 (100) \)

\( = 2.98 \text{ kN/m}^2 \)

M₀ = 2.98 (16) = 9.53

Use 2 # 6 @ 6" oc

Beam B1, B2

19" + 8"

Table 9.5m

h = \( \frac{f}{2t} \)

Bends

Continued

33 (12) \( \frac{\text{m}}{2} \), 19.85 → 19"
\[ M_{max} + \frac{wL^2}{16} = \frac{1.78(250)^3}{12} = 121.1 \text{kN} \]

\[ M(x) = \frac{wL^2}{12} = 176.2 \text{kN} \]

\[ V = \frac{wL^2}{2} = \frac{1.78(250)^2}{2} = 29.87 \text{kN} \]

At midspan, \( M_u = 121.1 \text{kN} \)

\[ A_o = 121.1 \frac{y}{b} = 1.78 \text{ Try } \frac{y}{b} = 1.8 \text{ m}^2 \]

\[ M_o = A_o f_y (d - \frac{y}{b}) = \frac{1.8(60)(17 - 3.5y}{b} = 137.1 \text{kN} \]

\[ a_o = \frac{A_o f_y}{0.85 f_y b} b = \frac{1.8(60)}{0.85(12)} = 3.53 \]

\[ b = 0.85 \]

\[ c = \frac{y}{b} = \frac{5.5}{0.85} = 6.56 \]

\[ \varepsilon_o = \frac{0.003}{6.56} (17 - 4.15) = 0.0095 \geq 0.002 \rightarrow \Phi = 0.9 \]

\[ \Phi M_u = 0.9 (121.1) = 123.41 \geq 121.1 \text{ OK} \]

\[ A_{area} = \frac{3.15^2}{2.5} \text{ kN} \]

\[ 200 \frac{b d}{f_y b} = 0.68 \text{ m} \]

At support, \( M_u = 176.2 \text{kN} \)

\[ A_o = \frac{(176.2)}{9(3.5)} = 4.59 \text{ kN} \]

\[ \frac{y}{b} = \frac{4.59}{4.15} \text{ Try } \frac{y}{b} = 1.8 \text{ m}^2 \]

\[ A_o = 1.8 \frac{y}{b} = 1.8 \text{ m}^2 \]

\[ a_o = \frac{A_o f_y}{0.85 f_y b} b = \frac{1.8(60)}{0.85(12)} = 3.53 \]

\[ b = 0.85 \]

\[ c = \frac{y}{b} = \frac{5.5}{0.85} = 6.56 \]

\[ \varepsilon_o = \frac{0.003}{6.56} (17 - 4.15) = 0.0095 \geq 0.002 \rightarrow \Phi = 0.9 \]

\[ \Phi M_u = 0.9 (176.2) = 158.6 \text{ m} \]
$$A_s = m_{ld} \frac{37 \pi}{4} \frac{6}{200} = 0.56$$

$$f_{yd} = 0.68$$

Beam B2, B8

$$M_{u} (+) = \frac{w l^2}{16} = 245 \text{ ky}$$

$$M_{u} (-) = \frac{w l^2}{11} = 356 \text{ ky}$$

$$V = \frac{w l}{2} = 59.4 \text{ ky}$$

$$A_t = M_{u} + A_{st} = \frac{245}{4 (1.7)} = 3.6$$

Top steel will be required.

Try 4#8 with 2#8

$$A_6 = 4; A_8 = 1.58$$

$$d' = 2.5$$

$$d = 17$$

$$M_n = A_s E_s (0.03 \frac{C-d}{c}) (d-d') + 0.85 f_y' \delta + \beta (d-9)$$

$$A_s f_y' = 0.85 f_y' a_h'$$

$$C = 6.11; a = R; \delta = 0.85$$

$$\varepsilon_t = 0.003 \left( \frac{C-d}{c} \right) = 0.00177 \leq 0.002$$

$$M_n = 259.8 \geq 245 \text{ ky} \text{ OK; } \phi = 0.6$$
At Support $M_u = 3.56 \times 10^4$ kN.m

$$A = \frac{M_u}{\sigma_f} = \frac{3.56 \times 10^4}{550 (16)} = 5.24 \text{ in}^2$$

Try $C = 39 (2 \log A_0)$ with $4 \# 10$

$$A_0 = 6 \times 10^4 \text{ in}^2$$

$$C = 5.13 \quad \beta = 0.85 \quad \alpha = 4.36$$

$$\varepsilon = 100 \times \left( \frac{C - d}{C} \right) = 0.10 \times 1.5 = 0.002$$

$$\phi M_u = 3.56 \times 10^4 \geq 3 \times 10^4 \times 1.4 \text{ kN.m} \quad \phi = 0.51$$

Beam 8x8

$M_u (+) = \frac{8000 \times 15}{16} = 188 \text{ kN.m}$

$M_u (-) = \frac{0 \times 15}{16} = 7.94 \text{ kN.m}$

$V = \frac{15 \times 15}{2} = 45.7 \text{ kN}$

At Mid Span $M_u = 188 \text{ kN.m}$

$$A_n = \frac{M_u}{\sigma_f} = \frac{188}{550 (16)} = 2.76 \text{ in}^2$$

Try $C = 39 (2 \log A_0)$ with $1 \# 7$

$$A_0 = 8 \times 10^4 \text{ in}^2$$

$$C = 5.78 \quad \beta = 0.85 \quad \alpha = 4.91$$

$$\varepsilon = 100 \times \left( \frac{C - d}{C} \right) = 0.0017 \cdot 0.002$$

$\phi M_u = 190.3 \geq 188 \text{ kN.m} \quad \phi = 0.91$
A+ Supports. \( M_u = 2.741.2k \)

\[
\begin{align*}
A_2 &= \frac{M_u}{4d} = \frac{2.741.2}{4(12)} = 41.03 \\
A_2 &= 5.08 \\
A_2' &= 2m^2 \\
C &= 7.32 \\
\beta &= 0.85 \\
\alpha &= 6.22 \\
\varepsilon &= 0.06197 \\
\varepsilon &= 0.0379 < 0.05 \rightarrow \phi = 0.8 \\
\phi M_u &= 2.77k > 2.741.2k \text{ OK}
\end{align*}
\]

Quarter G1, G2, G3, G4.

From computer analysis:
\( M_{u+} = 509 k \)
\( M_{u-} = 517k \)
\( V_u = 61.9k \)

Check T-Beam:
\( M_u, T = 0.85 f_c b h_d (d - h/6) \)
\( 0.85 \cdot 90 \cdot (15)(99) \cdot (8)(17 - 99/6) = 19691.8 > 509 \text{ OK} \text{ T-Beam} \)

Reck Beam with \( b = 99.0 \)
\[ M_u = \frac{A_o f_y (d - c/2)}{2} \]

At support, \( M_u = 517 \)

\[ T_{1/4} = 4\#9, \quad w_{1/4} = 8\#9 \]

Counter G5, G6.

\[ \rho = \frac{\omega - 1}{2} = 59.4 \]

\[ W = 1.2 \left( 137.5 + 15.11 \right) = 0.368 \text{ kN} \]

From computer analysis:

\[ M_u = 270 \text{ kN} \]

\[ M_{u-} = 287 \text{ kN} \]

\[ V_u = 35.9 \text{ kN} \]

At midspan, \( M_u = 270 \)

\[ T_{1/4} = 4\#10, \quad w_{1/4} = 4\#9 \]

\[ T_{1/4} = 277 > 270 \text{ OK} \]

*See chart for 83.84 at supports for detailed calculation.*

At support, \( M_u = 287 \)

\[ T_{1/4} = 6\#8, \quad w_{1/4} = 2\#9 \]

\[ A_e = 9.74 \text{ in}^2, \quad A_s = 2 \text{ in}^2 \]
Unless noticed, all images and figures were created by Ryan English.
Appendix D: Revision of Technical Report 1.

Wind/seismic Load Calculation

Under careful review of the calculation of the wind and seismic load calculation it was found that an analytical error was found in the wind load analysis. It was found that the wrong leeward wind pressure was used in the calculation of the base shear and over turning moment. After recalculation of the wind load, for the E-W direction the base shear was found to be 245 kips with an over turning moment of 8,188 kip-ft and for the N-S direction the base shear was found to be 249 kips with an over turning moment of 7,989 kip-ft. The seismic load was check for errors and none was found. At the same time the seismic load was re-compared to the values for the structural document and was less than 5% off form their values. The seismic base shear was 1355 kips with an over turning moment of 63,704 kip-ft.

Snow Drift

Snow drift was analysis and the calculation can be found in Appendix B.