STRUCTURAL DESIGN OF POOL HOUSE IN THE CITY OF COLUMBIA

By

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This report presents a structural design for a one-story pool house in the City of Columbia. The most important part of the project was determining critical horizontal and vertical loads, and designing each member in accordance to those governing loads. The loads we analyzed were dynamic, dead, live and seismic. By means of the LRFD method, the greatest possible load combination was calculated, combining the previously mentioned loads. These load combinations were different for each analysis, as the loading pressures will change depending on what structural system is being analyzed. This comprises the large part of the report and of the project. The second and final step was designing timber members to satisfy the structural demands produced by those load combinations. The pool house was engineered in three different sections: the roof, the walls, and the foundation. The design process also started with the roof system, then the walls and cladding, and finally finished on the foundation.
1. INTRODUCTION

1.1 OBJECTIVE

This report strives to provide a clear and concise description of the work done over the semester in designing the structure for a pool house. All information is contained within the report concerning the challenges faced, the calculations made, and the lessons learned. Along with this, is all information about the structural design itself, from choice of wood, to spacing, to dimensions, etc.

1.2 PROJECT DESCRIPTION

The proposed project includes a structural design for a one-story pool house located at a residential community in the City of Columbia. The Community needs a safe and serviceable pool house to serve its residents. Detailed drawings were provided to us by the architect. The pool house is to be framed in timber as it is a small building, not possessing a high structural demand satisfied by stronger materials such as steel. For the structural design, the vertical and horizontal loads were calculated for the most vulnerable members using different load combination and the highest governing one was chosen for the final design.

The project required some other aspects of civil engineering, namely geotechnical engineering which includes foundation design. A geotechnical report and a boring log were provided, and all bearing capacity calculations were based upon this. The foundation is very basic and is a reinforced concrete strip footing poured at the site. No other design or construction recommendations were made for the foundation beyond material, shape, thickness, and dimensions. Cost estimate is a very crucial part in any construction project because it gives a
clear financial picture to the client. A detailed cost estimate for the project is included. This estimate includes only the cost of materials for the timber frame and the concrete for the foundation. It does not include architectural materials, construction costs, and cost of services.

All the calculations for the structural, geotechnical and cost estimation are included in the excel spreadsheets filed with this report. Along with these, will be brief comments on the process of formulation and other general details. These comments will be included in the Appendices. Structural designs are provided at the end of the report. Drawing sheets are organized into A-sheets, S-Sheets, and Detail Sheets. A-Sheets, which are the architectural drawings include all drawings provided by the architect. S-Sheets are the structural drawings. These are the sheets detailing the layout of the timber frame, timber roofing system, and the shear walls. Detail sheets include connection design and general comments from our group. Foundation drawings detail the specific design and cross section of the foundation for the pool house.

The structure’s roofing system will be comprised of rafters with a king rafter providing the spinal connection, a nailing board. Roofing materials laid upon the rafters will be listed and are specified by the Architect. The walls are to be framed by timber studs and cladded in a brick veneer. Shear walls will be installed directly to the studs to resist wind pressures and the resulting tension forces.

1.3 PROJECT SCOPE

The client wishes to build a pool house to serve the community and for members to utilize. The following has been tasked of Top Down Timber Design:

- Structural design of the building frame
  - Roofing system
- Wall design
- Connection Design
  - Foundation design for pool house
  - Cost estimation for materials

Due to the nature of the building, timber is the material of choice for economy. All timber design is based upon the guidelines set forth by the American Wood Council in the NDS 18.

1.4 CODES AND ORDINANCES

We, Top Down Timber Design, adhered to the following codes set forth by proper authorities:

- ASCE 7-16
- IBC 2015 with SC Modifications
- Code of Ordinances, City of Columbia
- NDS 18
- AAMA/WDMA/CSA 101/I.S.2/A440-11
- NAFS-11

2. CONSTRUCTION SPECIFICATIONS

All lumber shall be Type 2 Southern Pine. For fastening studs to the top and bottom plates, use 16d galvanized nails. For fastening rafters together, use 8d galvanized nails. Plywood sheathing is to be fastened with 10d nails. Foundation concrete is to be poured on site. For rafter to plate connection and plate to stud connection use Simpson Hurricane ties.
3. ROOF DESIGN

3.1 ROOF DESCRIPTION

As indicated on the architectural drawings, the roof has a (8:12) slope. A system of 12” spaced rafters connected to a nailing board is used. The area of the roof is 2,873 ft². A 2”x14” size member was selected for the nailing board. The structure has a cross-gabled roof without any windows.

3.2 ROOF MATERIALS AND COMPONENTS

The roof system is comprised of 2x12” Southern Pine No. 2 wood rafters. On top of the rafters is a ¾” thick plywood layer. An extruded polystyrene insulation layer will cover the plywood. On top of the polystyrene will sit a layer of weather-proof felt material. Presidential Shake Shingles will be the final layer on the roof.

3.3 ROOF CALCULATION

All the loads acting on pool house were determined following the standards, codes and procedures given by International building codes, Design for wood structures 6th edition and ASCE 7-16.

3.3.1 ROOF DEAD LOAD

The roof dead load for the pool house was determined using Design for Wood Structures 6th edition. The roof was made of different components which includes Shingles, re-roofing, plywood, insulation, felt and contingency. Sheet A-2 in the architectural plan set determines individual material types and these specs were used to calculate the dead load on the roof.
CertainTeed Presidential Shake® shingles were used as it was specified by manufacturers which have dead load of 3.55 psf. Re-roofing has a dead load of 3.55 psf to account conservatively for a new layer of shingles being placed in the future. 3/4 inch thick plywood was used for the roofing which account for 1.5 psf of the dead load [Appendix B; Breyer]. Extruded polystyrene insulation was used in accordance to Appendix B; Breyer which has a small impact of 0.2 psf on the dead load. Felt layer is an essential component of roofing system, it is to be installed under the shingles to provide water proofing in case of a leakage which has a dead load of 0.3 psf. A contingency load of 3 psf is placed on the roof in order to account for unknown mechanical, plumbing, and electrical loads which are not specified at this step in the design process. Total dead load of the roof is sum of all the above which is equal to 12.85 psf.

3.3.2 ROOF LIVE LOAD

Live loads for the pool house was determined using ASCE 7-16. Live loads are loads which act along the floor or the roof elements throughout the life of the structure. Live loads are not the permanent part of the structure, and they vary largely depending on the time and season. Roof live load for pool house accounts for construction workers who are building the roof and weight of the maintenance workers who will be working on the roof in the future. The roof live load of 20 psf for a one-story building was given by IBC Table 1607.1. However, we reduced it to 16 psf using slope reduction factor of 0.8 [IBC Sec. 1607.13.2.1].

3.3.3 ROOF DYNAMIC LOAD

Dynamic loads are loads which are induced by environmental parameters such as winds, snow, rain, etc. In simple words they are loads imposed directly or indirectly by the environment. Environmental loads for the pool house was calculated using ASCE 7. Ground snow load for
Columbia was 10 psf based on IBC Fig. 1608.2. Snow load was reduced to 9.08 psf using slope reduction factor (Cs) of 0.91. The rain load on the building will be zero as roof members have such a severe slope that no ponding will occur.

3.3.4.1 WIND LOAD - ROOF

Wind loads will influence the roofing system. The approach was to calculate the greatest possible force for the most vulnerable portion of the roof. This calculation was used for all rafters. There were two scenarios to observe: the wind creates both a negative and positive force. The positive force was used for bending moment calculations and the negative force was used for roofing connection design. The greatest positive force was 11.3 psf and the greatest negative force was –17.76 psf.

4. WALL FRAME DESIGN

4.1 WIND LOAD – WALL

Wind pressures will provide the greatest structural demand on the walls. Like the roof, the wall was analyzed such that each member was designed to withstand the greatest possible load at the most vulnerable situation. By ASCE 7-16, the wind pressure was calculated. For walls, a separate wind calculation had to be made and this is titled the “Components and Cladding” wind pressure. This pressure was calculated to be 49.92 psf. The walls will have a 3/4” plywood sheathing that acts as a wind resistance system. The wind pressure that acts upon this system was calculated to be 29.3 psf.
4.2 SEISMIC LOAD – WALL

Seismic pressures will also provide the a demand on the walls. According to ASCE 7-16, the seismic loading consists of both a horizontal (Eh) and vertical effect (Ev). Through this design method, values of 8.4 kips and 5.7 kips were obtained for the vertical and horizontal effects, respectively. When converting these loads to psf, the values are 3.14 and 2.92 psf, respectively. However, when calculating the load combinations from IBC Table 1605.2, the seismic load combinations did not govern. Therefore, the seismic forces did not affect the sizing of structural members. The calculations can be found in the Appendices.

4.3 WALL DESIGN

The wall frame is to be made of 2”x4” studs at a 16” spacing. This spacing is within the maximum spacing set forth by the IBC Table R602.3(5). These specifications meet the structural demand offered by the wind pressure analyzed under LRFD. The calculations can be found in the Appendices.

4.4 EXTERIOR COLUMNS

There is an exterior “porch/awning” attached to the front of the pool house. This is detailed in the A-sheets and therefore did not require any design beyond the selection of an inner steel member of the columns that support the awning. Using the previously listed loadings from Section 3.3 under LRFD analysis, the member selected was an HSS4 x 2 x 1/8 to meet the structural demand, using a roof pressure of 48.92 psf.
5. CONNECTION DESIGN

All connections were designed in accordance to NDS 18. Design consisted of two major components for the nails: Withdrawal Force and Shear Force. However, since there is no pullout force applied to our nails, the withdrawal force will not be a factor. Strength characteristics were based off Southern Pine connections, and the diameter of the nails (0.148 in). After finding the nominal shear force for one nail, the number of nails for each side needed to be determined. By multiplying the calculated wind pressure (psf), by the area of each side of the building (ft²), a wind force was calculated. By dividing this force by our shear force, we were able to find the minimum number of nails required for one side of the building. Through this method, our minimum number of nails were 47, 49, 51, and 51 for each side.

For the framing connections, we decided to use Simpson brand hurricane ties. These ties can both connect rafters to plates and studs to plates. The rating for the ties is above the uplift and lateral forces. The rafter-to-plate connections shall be Simpson H1’s. The stud-to-plate connections shall be Simpson H6’s. The detail for these can be found below.

There are to be at a minimum, 4 ½” diameter foundation anchors with 2” square washers on each wall, or the number of anchors it takes to have a minimum spacing of 6’ on each wall. Two
anchors are to be embedded 12” away from either side of a plate end. All anchors are to be embedded at a minimum of 7” into the concrete [IBC 23-1 Sect. 2308.3.1].

6. COST ESTIMATION

The cost estimation was made purely upon the cost of materials alone. This cost estimation does not include any prices for workmanship, engineering, or transportation. All numbers were based off average prices from several online stores and lumber yards. The materials included in this cost estimation are the lumber used for the framing, plywood for sheathing, the concrete used in the foundation, and the nails used in the connections. This estimation does not include any of the prices for the brick veneer, roofing materials, interior installations, permitting fees, plumbing, erosion control devices, etc. The total cost was calculated to be $18,710.43.

7. FOUNDATION DESIGN

7.1 GEOTECHNICAL ANALYSIS

For each project, a geotechnical analysis must be conducted in order to analyze the site’s composition. Unfortunately, we were not given a specific location to run a subsurface investigation ourselves or visit the site. Instead, a geotechnical report was provided to the group including the boring logs which were used for the foundation analysis. There were some assumptions made, necessary to complete the design process. A series of calculations were used to come up with the recommendations based on the available site data.
7.2 SOIL CLASSIFICATION

The site was assumed to located in Richland County in South Carolina. Based on this assumption, the elevation above sea level found to be around 394’. The soil condition was reported to be Silty Clay (CL-ML), Silty Sand (SM), Clayey Sand (SC), Lean Clay (CL), Sand (SP). Groundwater was encountered at 10’. As for seismic consideration, the site has been classified to be class D.

7.3 FOUNDATION

Since no foundation specification was given, the group calculated the allowable bearing capacity for the foundation based on the dead and live load of the structure. After that, the given borehole information was used to calculate the bearing capacity of the soil assuming a continuous foundation would be used. Because our foundation supports a one story building, a shallow foundation was selected instead of a deep foundation. A continuous strip footing foundation was chosen with reinforcing #4 Rebar.

7.3.1 BEARING CAPACITY

Understanding bearing capacity is essential for designing the foundation. Bearing capacity is the ability of the soil to support the vertical load. Applying too much load to the soil will develop a shear plane which may cause the superstructure to tip over. So, based on the boring log that was provided to the group, the ultimate bearing capacity was calculated and found to be 20.17 kip/ft² based on Terzaghi's Bearing Capacity equation.

\[ q_{uu} = c'N_cF_{cs}F_{cd}F_{ci} + qN_cF_{qs}F_{qd}F_{qi} + \frac{1}{2} \gamma B N_{\gamma}F_{\gamma s}F_{\gamma d}F_{\gamma i} \]
7.3.1.1 COHESION(C)

After that, in order to use equation (1), cohesion which is defined as the force that holds together molecules within the soil need to be calculated. Typically, cohesion is determined in the laboratory from the Direct shear test where the Unconfined compressive strength $S_{uc}$ can be determined using the Triaxial test or the unconfined compressive strength test. However, the group was given the unconfined compressive strength for the clay soils, so based on that, cohesion was calculated:

$$C = \frac{S_{uc}}{2}$$

7.3.1.2 BEARING CAPACITY FACTOR ($N_c, N_q, N_y$)

$N_c, N_q, N_y$ known as Terzaghi’s bearing capacity factors. Those factors depend on the soil friction angle. After calculating the friction angle, table below is used for selecting the $N_c, N_q, N_y$ values.
7.3.1.3 FRICTION ANGLE ($\phi$)

Calculating the friction angle is one of the most important steps for calculating the bearing capacity of the soil. Three different empirical equations have been used to calculate the friction angle.

- Peck, Hanson, and Thorburn (1974):

$$
\phi' \text{ (deg)} = 27.1 + 0.3(N_1)_{60} - 0.00054(N_1)_{60}^2
$$

- Schmertmann (1975):

$$
\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_o}{P_a}\right)}\right]^{0.34}
$$

- Hatanaka and Uchida (1996):

$$
\phi' = \sqrt{20(N_1)_{60}} + 20
$$

In order to use the equations that were introduce above, $N_{60}$, $(N_1)_{60}$ need to be calculated based on the following equations

\[ N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60} \]

where $N_{60}$ = standard penetration number corrected for field conditions
$N$ = measured penetration number
$\eta_H$ = hammer efficiency (%) 
$\eta_B$ = correction for borehole diameter
$\eta_S$ = sampler correction
$\eta_R$ = correction for rod length
\[(N_1)_{60} = C_N N_{60}\]

\[C_N = \left[ \frac{1}{\left( \frac{\sigma'}{P_a} \right)} \right]^{0.5}\]

Where \(P_a\) (atmospheric pressure) is equal to 2000 lb/ft², and \(Q\) (the effective vertical stress) was calculated for each layer based on the boring log information.

### 7.3.1.4 SHAPE FACTORS

\[F_{ca} = 1 + \left( \frac{B}{L} \right) \left( \frac{N_q}{N_c} \right)\]

\[F_{qs} = 1 + \left( \frac{B}{L} \right) \tan \phi'\]

\[F_{ys} = 1 - 0.4 \left( \frac{B}{L} \right)\]

DeBeer (1970)
7.3.1.5 DEPTH FACTORS

\[
\frac{D_t}{B} \leq 1
\]

Hansen (1970)

For \( \phi = 0 \):

\[
F_{cd} = 1 + 0.4 \left( \frac{D_t}{B} \right)
\]

\[
F_{qd} = 1
\]

\[
F_{yd} = 1
\]

For \( \phi' > 0 \):

\[
F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}
\]

\[
F_{qd} = 1 + 2 \tan \phi' \left( 1 - \sin \phi' \right)^2 \left( \frac{D_t}{B} \right)
\]

\[
F_{yd} = 1
\]

\[
\frac{D_t}{B} > 1
\]

For \( \phi = 0 \):

\[
F_{cd} = 1 + 0.4 \tan^{-1} \left( \frac{D_t}{B} \right)
\]

\[
F_{qd} = 1
\]

\[
F_{yd} = 1
\]

For \( \phi' > 0 \):

\[
F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}
\]

\[
F_{qd} = 1 + 2 \tan \phi' \left( 1 - \sin \phi' \right)^2 \tan^{-1} \left( \frac{D_t}{B} \right)
\]

\[
F_{yd} = 1
\]

7.3.1.6 INCLINATION FACTORS

\[
F_{ci} = F_{qi} = \left( 1 - \frac{\beta^\circ}{90^\circ} \right)^2
\]

Meyerhof (1963); Hanna and Meyerhof (1981)

\[
F_{yi} = \left( 1 - \frac{\beta}{\phi} \right)
\]

\( \beta \) = inclination of the load on the foundation with respect to the vertical
7.3.1.7 WATER TABLE
Since the water table have been identified to be at 10 ft below the ground, no modification for the unit weight of the soil will be needed because the water table is deeper than 3 times the width of the base of the foundation. The water table appeared in one boring log in a sand layer.

\[ q = \gamma D_f \]

7.3.1.8 ULTIMATE BEARING CAPACITY RESULTS
After running the calculation from the boring log data, the ultimate bearing capacity for each layer was calculated. The lowest strength value was observed to be in layer 1 from B4.

<table>
<thead>
<tr>
<th>Layer</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
<th>B4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>95 kip/ft²</td>
<td>148 kip/ft²</td>
<td>24 kip/ft²</td>
<td>20.17 kip/ft²</td>
</tr>
<tr>
<td>Layer 2</td>
<td>145 kip/ft²</td>
<td>74 kip/ft²</td>
<td>658 kip/ft²</td>
<td>1366 kip/ft²</td>
</tr>
<tr>
<td>Layer 3</td>
<td>29 kip/ft²</td>
<td></td>
<td>92 kip/ft²</td>
<td></td>
</tr>
</tbody>
</table>

Based on the lowest value of the ultimate bearing capacity which is found to be 20.17kip/ ft², the allowable bearing capacity calculated as following assuming the safety factor to be 3:

\[ q_{all} = \frac{q_{ult}}{F.S} \]

\[ q_{all} = \frac{20.17(kip/ft²)}{3} = 6.7 \text{ kip/ft}² \]

Then the allowable load was calculated

\[ P_{all} = q_{all}B*L = 6.7 \text{ kip/ft}² * 3\text{ft} * 256\text{ft} = 5145 \text{ kip} \]

This value was compared with the total dead load of the structure which is found to be 250 kips. This means that our foundation will be able to support the dead load and live load of the structure.
APPENDICES
LUMBER IS SOUTHERN PINE, NO.2. RAFTERS ARE 2X12" MEMBERS. STUDS ARE 2X4" MEMBERS.
RAFTER SPACING IS AS MARKED. OTHERWISE, SPACING IS 1' FOR RAFTERS.
STUD SPACING IS AS MARKED. OTHERWISE, SPACING IS 16" FOR STUDS.
2X4" AND 2X12" BLOCKING AT MIDSPAN OF ALL STUDS AND RAFTERS RESPECTIVELY (SEE SHEET S-3)
Lumber is Southern Pine, No. 1. Rafters are 2x12" members. studs are 2x4" members. rafter spacing is as marked. Otherwise, spacing is 1' for rafters. stud spacing is as marked. Otherwise, spacing is 16" for studs. 2x4" and 2x12" blocking at midspan of all studs and rafters respectively (See Sheet S-3).
LUMBER IS SOUTHERN PINE, NO.1. RAFTERS ARE 2X12" MEMBERS. STUDS ARE 2X4" MEMBERS.
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LUMBER IS SOUTHERN PINE, NO. 2. RAFTERS ARE 2X12" MEMBERS. STUDS ARE 2X4" MEMBERS. 2X12" BLOCKING AT MIDSPAN OF ALL RAFTERS (SEE SHEET S-3).
NOTES:
1. CONTINUOUS STRIP FOOTING FOUNDATION IS 3 FT WIDE AT A DEPTH OF 3FT.
2. EXTERIOR COLUMNS HAVE A CONCRETE FOOTING THAT IS 25"X25" AT A DEPTH OF 3FT.
3. A CONCRETE SLAB WILL BE POURED OVER THE STRIP FOOTING AND WILL ALSO BE THE FINISHED FLOOR FOR THE STRUCTURE.
4. BOTTOM PLATE IS TO BE ANCHORED TO THE CONCRETE WITH 2" Dia. STEEL BOLTS.
5. THERE ARE TO BE AT LEAST 4 BOLTS ON EACH WALL OR ENOUGH BOLTS ON EACH WALL SUCH THAT THE SPACING IS UNDER 6'. THERE ARE TO BE TWO BOLTS AT 12" AWAY FROM EITHER SIDE OF A PLATE END.

SECTION A-1:
NOTES:
1. THE FOUNDATION IS A CONTINUOUS STRIP FOOTING.
2. CRUSHED ROCK SHALL BE PLACE UNDER THE CONCRETE SLAB.
3. THE FOOTING IS A POURED CONCRETE.
4. 2" BOLT MUST BE EMBEDDED AT MINIMUM 2" INTO THE CONCRETE.

SECTION A-2:
NOTES:
1. SPREAD FOOTING IS POURED CONCRETE.
USE 3/4" PLYWOOD, NAILED AT 12" INTERVALS ON EACH STUD AND 6" SPACING AROUND EDGE OF PANELS.

PVC LOUVER INSTALLATION TO TAKE PLACE AFTER SHEATHING IS NAILED. CUT OUT PORTHOLE AND INSTALL PER PRODUCT SPECIFICATIONS.