# *Case study: Evaluation of a two-level residence structural plan (stilt house with open space on the first floor) and compliance recommendations for earthquake resistance in Puerto Rico using ACI318-14 code*

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***Abstract*** *— The construction of two-level residences on columns in Puerto Rico is quite common. This type of structure is built for the purpose of having an open space on the first floor (ground level) while the main residence is built on the second level. In this case study, an original design was evaluated against existing codes to bring it to compliance. Recommendations are presented with the objective of ensuring that the structure is adequately designed as an earthquake-resistant moment frame according to code ACI318-14. The first analysis considers the properties of materials, live loads, dead loads, and seismic loads based on the structure's own weight (applied at a distance equivalent to 5% of the centroid). It also considers the P Delta effects. All these loads are factored in and combined in order to decide which one is the worst-case scenario. The capacity of each element should be verified with the maximum load obtained from the analysis; if not compliant, the structure must be redesigned. When the cross section of an element changes in size, components, or material property, it is necessary to redo the analysis. This iterative process should be carried over until the sections obtained have a capacity (multiplied by a reduction factor) greater than the maximum demand. Furthermore, it is necessary to comply with important aspects such as the minimum spacing of transverse steel (shear), minimum dimensions of elements, maximum and minimum and maximum amounts of longitudinal steel (flexure), location of development lengths and splices, and verification of strong column-weak beam criteria ($Mn_{Columns} > 1.2Mn_{Beams}$), among others.*

***Key terms*** *— earthquake resistant, stilt houses, strong column-weak beam, two-level houses*

## INTRODUCTION

On December 28, 2019, and progressing into 2020, the southwestern part of Puerto Rico was struck by seismic activity [1]. The largest and most damaging of this sequence occurred on January 7, 2020, (4:24AM AST), with a magnitude of 6.4. Days before January 7 (and days after), several tremors greater than 5.0 were registered in Puerto Rico. This earthquake left 8,000 people homeless [2]; about 40,000 others (just in the municipality of Ponce) camping in front yards, public areas, open parks, and government roads, because they did not feel safe in their houses, even though some had not collapsed; 28 refugee government centers; and $3.1 billion in financial losses [1].

Damage to structures was noticeable, especially those on columns distributed without walls in the ground level. Many of these structures in the southern part of the island where the seismic fault has been more active (e.g. Ponce, Guánica, Yauco, Guayanilla, Lajas) collapsed entirely. Many of the structures that did not collapse presented structural cracks with displacements that make their repairs very expensive or practically impossible.

As of the day of publication of this article, there is still seismic activity. Seismic resistant requirements are mandatory and recent events make it more relevant. This article details approach to relation to various codes, especially chapter 18 (earthquake resistant structures) of the ACI318-14.

## OBJECTIVE

The main objective of this article is to evaluate a construction plan for a typical two-story residential design (elevated house supported by columns), analyze its elements and redesign them if necessary, based on earthquake resistance. This article seeks to provide the most important recommendations of ACI 318-14 for similar structures and thus, ensure that they remain safe and stable after seismic events.

## Earthquakes in Puerto Rico

Puerto Rico is located in an active seismic area [1], due to the large number of geological faults around it. This activity usually occurs due to the interaction between the Caribbean tectonic plate and the North American plate. The island has two different seismic regions, but the west-southwest is more prone to seismic activity. An average 3 to 5 earthquakes are recorded daily, and an average of 1 to 3 events of magnitude 5.0 per year.

The average lapse between destructive earthquakes in Puerto Rico is every 83 years (1787, 1867, 1918, 1943 and 1946).

## Law Enforcement

Law 135 for Certification of Plans in Puerto Rico ("Ley de Certificación de Planos de Puerto Rico") was created in June 15, 1967 [3]. Its purpose was to authorize the Administration of Regulations and Permits (ARPE) the implementation of a system to certify construction project plans. Law 161 of 2009 created the integrated permit system of the government of Puerto Rico and was amended 8 years later by Law 19-2017, with the purpose of creating a uniform digital system to evaluate requests that are submitted in relation to construction projects, permits, consults, inspections, licenses, and certifications in PR [4]. All these requests are already handled through a unified and digital system, which today is known as the "Oficina de Gerencia y Permisos" (OGPe). This government agency took the place of ARPE.

The main regulation of OGPe, called "Reglamento Conjunto de Obras," establishes in section 2.7.1 that every plan or design will be certified by a responsible licensed engineer [5]. As a territory of the United States, Puerto Rico uses its codes (including ACI318) to establish the minimum standards by which buildings can be legally constructed. They are adopted and enforced to help safeguard public health and safety.

## Main Information of the Case Study

The case in analysis is based on a two-level residential building. The first level is open (without walls) and has a distribution of 14 columns with 3 different geometric sections aligned in different directions. All columns continue to the second level, which is partially closed with masonry walls and includes the residence. All the beams have the same section. The plan and general data of the original design are shown on figure 1 and table 1.

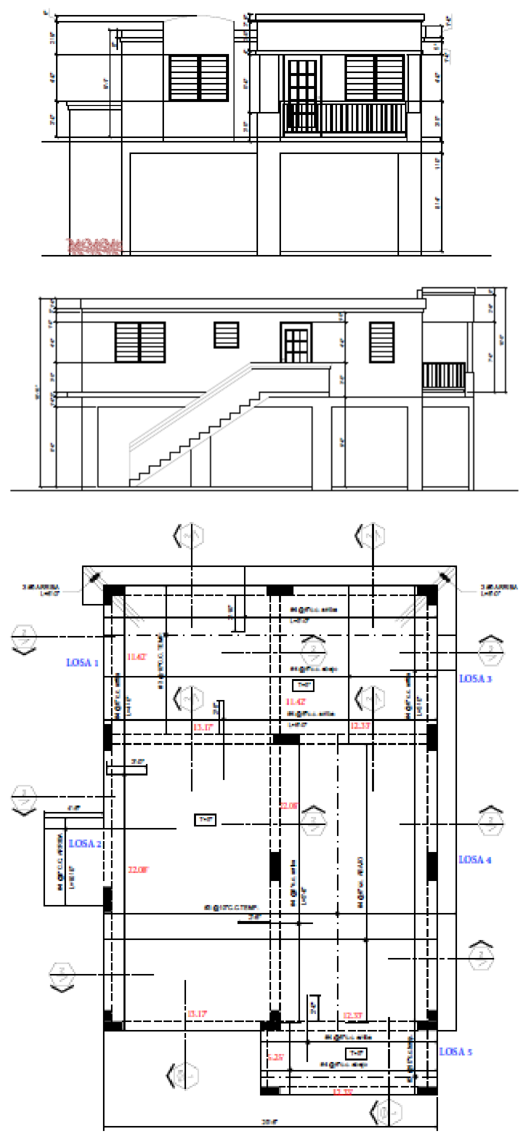


**Figure 1**
**Original plans**

**Table 1**
**General data of the original design**

| | |
|---|---|
| Materials | Properties |
| Concrete | -F'c = 3,000 psi |
| Steel | -Fy = 60,000 psi, Fyt = 60,000 psi |
| Elements | Measurements |
| Column #1 | -8"x24" / 8#5 long. / #3@6" crossties |
| Column #2 | -8"x18" L / 12#5 long. / #3@6" crossties |
| Column #3 | -8"x18" / 6#5 long. / #3@6" crossties |
| Beams | -8"x12" / 4#5 long. Top / 4#5 Long. Bottom / #3@6" crossties |
| Slabs | -5" thickness |
| Loads | Description |
| Live | -50 psf |
| Dead | -calculated per element |
| Dead Added | -20 psf (typical) |

## Important Modeling Assumptions

- The plans show the construction of masonry walls on the second floor. It is unknown whether the original design considered the detrimental effects that these walls often have on columns when they are subjected to lateral loads. Masonry walls can negatively influence the structure's seismic response and could cause damages if it is not properly designed. To avoid this, the computer model was made without considering the block walls (SMRF).
- [8] recommends a "fuse" anchorage by means of a separation equivalent to the drift obtained in the displacement analysis. Being a residential house (19 ft high, small lateral displacement), it is suggested to leave a 3-inch gap between the block walls and the columns, which can be covered with plycem.
- The moments of inertia and cross-sectional area of members have been calculated in accordance to table ACI318-14/6.6.3.1.1(a) [9]. These factors reduce inertia by $0.70I_G$ and $0.35I_G$.
- Structural slabs are considered as "diaphragm" in SAP2000 software. The main reason is to apply a restriction in the "z" axis.

## Response Spectrum Analysis

The residence used in this case study is in Ponce. However, there are other residential areas affected in this seismic region; hence the value and relevance of this article. The data used to obtain the RSD is described on figure 2 and table 2.

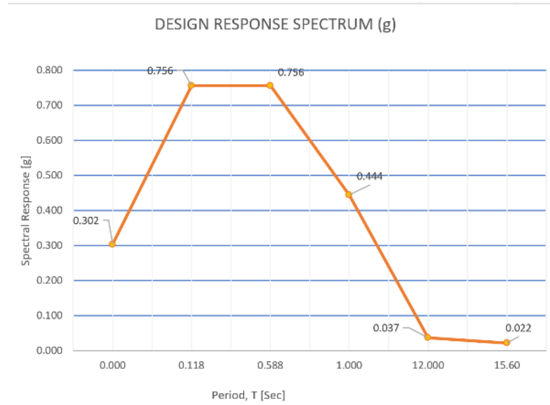


**Figure 2**
**Response Spectrum Design (RSD)**

**Table 2**
**Summary of SRA procedure for the first analysis**

| | |
|---|---|
| Location | -Ponce, PR |
| Site | -D (assumed in absence of additional data) |
| $S_S$ = 1.05 | -SRA at 0.2 sec **(PRBC, 1613.5, P.48)** |
| $S_1$ = 0.34 | -SRA at 1.0 sec **(PRBC, 1613.5, P.49)** |
| $F_A$ = 1.08 | -short period site coef. **(ASCE7/16, 11.4-1)** |
| $F_V$ = 1.96 | -long period site coef. **(ASCE7/16, 11.4-2)** |
| $S_{MS}$ = 1.134 | -site coefficient **(ASCE7/16, 11.4-1)** |
| $S_{M1}$ = 0.666 | -site coefficient **(ASCE7/16, 11.4-2)** |
| $S_{DS}$ = 0.756 | -design parameter **(ASCE7/16, 11.4-3)** |
| $S_{D1}$ = 0.444 | -design parameter **(ASCE7/16, 11.4-4)** |
| T = 0.2 | -fundamental period **(ASCE7/16, 12.8-8)** |
| R = 8.0 | -SRMF response m.f. **(ASCE7/16, 12.2-1)** |
| $I_e$ = 1.0 | -importance factor **(ASCE7/16, 11.5-1)** |
| $C_S$ = 0.09 | -seismic response **(ASCE7/16, 12.8-2)** |
| $C_S$ = 0.50* | -seismic response **(PRBC, A.2.1, P.63)** |
| $W_T$ = 335 kip | -total weight of the structure (calculated) |
| $W_{T1}$ = 180 kip | -total weight of the 1st floor (calculated) |
| $W_{T2}$ = 155 kip | -total weight of the 2nd floor (calculated) |
| $C_{V1}$ = 0.367 | -dist. factor 1st floor **(ASCE7/16, 12.8-2)** |
| $C_{V2}$ = 0.633 | -dist. factor 2nd floor **(ASCE7/16, 12.8-2)** |
| $F_1$ = 61.5 kip | -dist. force 1st floor **(ASCE7/16, 12.8-11)** |
| $F_2$ = 105.9 kip | -dist. force 2nd floor **(ASCE7/16, 12.8-11)** |
| V = 167.5 kip | -seismic base shear **(ASCE7/16, 12.8-1)** |

Although the procedure established by ASCE7/16 [6] was used to calculate the response spectrum design, we found that the value $C_S$ (0.09 in this case) is less than the minimum established in the 2016 PR Building Code, $C_S$=0.5 [10]. Finally, the PRBC was applied, which is also more conservative.

Both codes (PRBC2016 and ASCE7/16) require considering the effect of accidental torsion [6, 10]. This effect was taken in consideration by two lateral forces per level: (1) 100%*$F_i$ and (2) 30%*$F_i$ (the latter applied in the orthogonal direction). It is sometimes difficult to calculate the exact center of mass and rigidity. To account for this inaccuracy, both forces were applied at each level and were located at a point of eccentricity, not less than 5% of the maximum dimension of the plane normal to the direction of the load. The centroid calculation is described on table 3 and figure 3.

**Table 3**
**Centroid computation**

| CASE STUDY: CENTROID CALCULATION | | | | |
|---|---|---|---|---|
| (FOR 5% ECCENTRICITY PERPENDICULAR TO THE MAIN DIRECTION OF DISPLACEMENT) | | | | |
| AREA | Xci | Yci | Xci*A | Yci*A |
| 945.00 | -10.50 | -37.25 | -9922.50 | -35201.25 |
| 64.7 | -4.67 | -57.38 | -301.98 | -3714.03 |
| 27.0 | -26.25 | -41.84 | -708.75 | -1129.55 |
| 1036.7 | | Σ | -10933.2 | -40044.8 |
| ΣXci*A | -10.546 | | ΣYci*A | -38.626 |
| Total Area | Xc [FT] | | Area Total | Yc[FT] |
| MODEL DIMENSIONS | | | 5% ECCENTRICITY | |
| L = | 40.25 FT | | 2.01 FT | |
| W = | 27.00 FT | | 1.35 FT | |

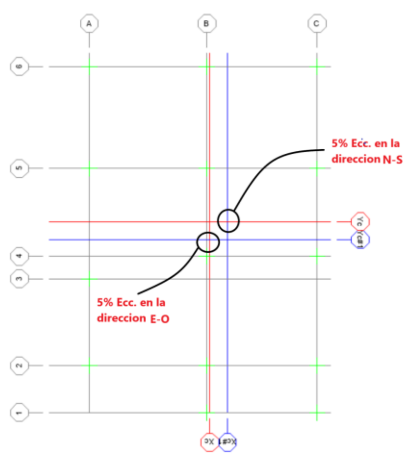


**Figure 3**
**Illustration of 5% eccentricity with static seismic loads**

## FIRST ANALYSIS: ORIGINAL SECTIONS

The main purpose of this analysis is to find the maximum moments, axial loads, and shears under the worst combination of factored loads. After running the model with the calculated loads in the two main directions (N-S and E-O), we see that the most critical forces and moments were developed with load combination # 4, "Comb4 (1.2D + 1.6L)". Important: At first glance, a series of structural details are observed in the design plan. These details need to be corrected for compliance with code ACI318-14 and will be taken into consideration in the second (final) analysis. For verification purposes in the first analysis, results and comments are presented below. See original sections in figure 4.

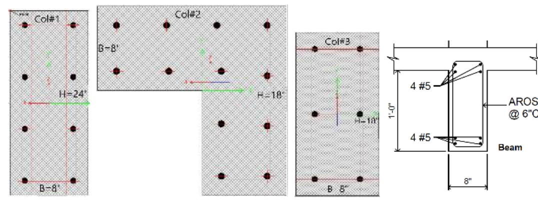


**Figure 4**
**Original plan sections**

### Original Beams

$M_{U(NEG)}$ = -47.24 kip*ft & $M_{U(POS)}$ = 32.11 kip*ft

$V_U$ = 24.34 kip

Shear:

$$Vc_b := 2\sqrt{\frac{f'c}{psi}} \cdot psi \cdot B_b \cdot d_b = 9.202\ kip \quad , \quad V_{CBmax} := 8 \cdot \sqrt[2]{\frac{f'c}{psi}} \cdot psi \cdot B_b \cdot d_b = 36.807\ kip$$

$$S_{bmax} := \max\left(\frac{A_v \cdot fyl}{0.75\sqrt[2]{\frac{f'c}{psi}} \cdot B_b \cdot psi}, \frac{A_v \cdot fyl}{50\ psi \cdot B_b}\right) = 40.166\ in \quad , \quad S_{bfinal} := min\left(S_{bmax}, s_b\right) = 2.625\ in$$

The calculations suggest hoops #3@2", which will make it difficult to pour concrete later (3" minimum). This was verified manually by increasing the rebar's hoops to #4, but the spacing did not change. It is necessary to increase the beam section, (width=10") which increases the beam inertia and shear capacity. Similarly, the minimum spacing d/4 or 4" must be met by ACI318-14,18.6.3.1 [9]. Therefore, a second run with B=10" and H=14" is recommended (increasing "d" also). Equations used: ACI318-14, 21.2.1(b), 9.5.1.1, 22.5.1.2, 22.5.10.5.3, 22.5.10.5.5, 9.7.6.2.2 & 9.6.3.3 [9]

Flexure:

$$As_{min1} := \frac{3 \cdot \sqrt{\frac{f'c}{psi}}}{fy} \cdot psi \cdot B_b \cdot d_b = 0.23\ in^2 \quad , \quad As_{min2} := \frac{200\ psi \cdot B_b \cdot d_b}{fy} = 0.28\ in^2$$

$$\rho_{bal} := \frac{0.85\ \beta_1 \cdot f'c}{fy}\left(\frac{87000}{87000 + \frac{fy}{psi}}\right) = 0.021 \quad , \quad \rho max_{bal} := 0.75 \cdot \rho_{bal} = 0.016$$

$$\rho_{top} := \frac{0.85\ f'c}{fy}\left(1 - \sqrt{1 - \frac{2.353 \cdot \frac{M_{UBneg} \cdot -1}{kip \cdot ft} \cdot 12 \cdot 1000}{\phi_b \cdot \frac{B_b}{in} \cdot \left(\frac{d_b}{in}\right)^2 \cdot \frac{f'c}{psi}}}\right) = 0.0143$$

The amount of steel for **negative** moment meets ductile failure criteria. The analysis (~ 47 K-Ft) suggests using a longitudinal distribution steel (As=1.202in$^2$) of 4#5 to resist the maximum negative moment demand. This is the same amount of steel as in plan, it is validated:

$$\rho_{bottom} := \frac{0.85\ f'c}{fy}\left(1-\sqrt{1-\frac{2.353 \cdot \frac{M_{UBpos}}{kip \cdot ft} \cdot 12 \cdot 1000}{\phi_b \cdot \frac{B_b}{in} \cdot \left(\frac{d_b}{in}\right)^2 \cdot \frac{f'c}{psi}}}\right) = 0.0091$$

The amount of steel for **positive** moment meets ductile failure criteria and the analysis (~32K-Ft), suggests using 3#5. The plan suggests the same amount of steel in top and bottom (4#5), along the entire length of the beam. This assumption could increase the compression capacity, affecting ductility. Length of development, length of hooks and splices will be calculated in the second analysis.

## Original Columns

Table 4 shows the maximum forces in columns.

**Table 4**
**Maximum forces in columns (1st run)**

| FINAL LOADS RESULTS FOR COLUMNS (1ST RUN - SAP2000 MODEL) | | | | | |
|---|---|---|---|---|---|
| COL#1 | | COL#2 | | COL#3 | |
| M3,MAX(-)= | -56.60 | M3,MAX(-)= | -21.13 | M3,MAX(-)= | -8.8181 |
| M3,MAX(+)= | 36.76 | M3,MAX(+)= | 21.49 | M3,MAX(+)= | 8.8303 |
| M2,MAX(-)= | -30.44 | M2,MAX(-)= | -14.96 | M2,MAX(-)= | -2.27 |
| M2,MAX(+)= | 24.82 | M2,MAX(+)= | 19.14 | M2,MAX(+)= | 2.67 |
| V,MAX= | 10.11 | V,MAX= | 4.53 | V,MAX= | 1.667 |
| P,MAX= | 192.96 | P,MAX= | 151.77 | P,MAX= | 49.927 |

## Column #1

The interaction diagram in figure 5 shows how the strong axis resists Pu~193K and Mu~56 k-ft.

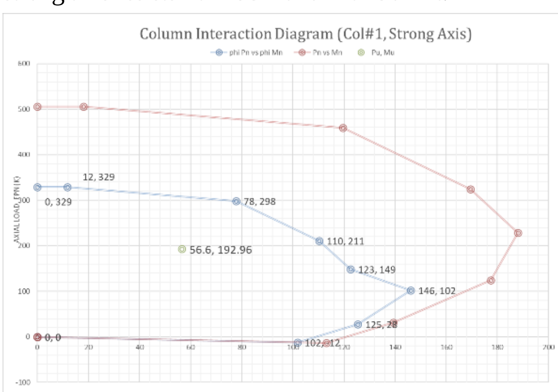


**Figure 5**
**Interaction diagram: original col#1 - b8" x h24" (strong axis orientation)**

The interaction diagram in figure 6 shows how the weak axis does not resists Pu~193K and Mu~56 k-ft. Increasing the column section is recommended. However, design must also comply with ACI318-14, 18.7.2.1 (b=12", b/h>0.4) [9].

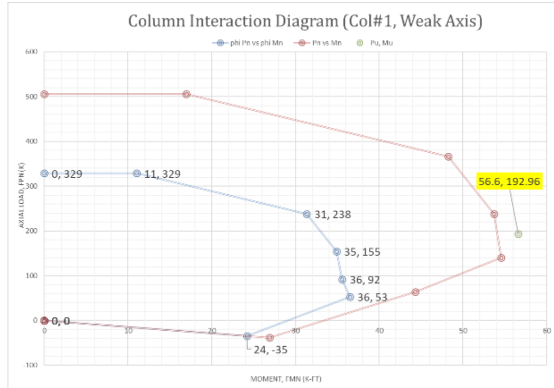


**Figure 6**
**Interaction diagram: original column#1 - b24" x h8" (weak axis orientation)**

## Column #2

The interaction diagram in figure 7 shows how the original "L" column 18"x8" (both axis) resists Pu~151.77K and Mu~21.49 k-ft. However, shear reinforcement will be verified in second analysis. To comply with section ACI318-14, 18.7.2.1 [9] ($B_{MIN}$ = 12", b/h>0.4) increasing all cross-sections is recommended.

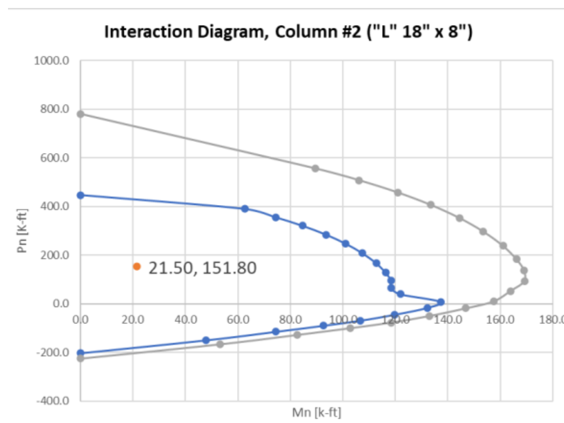


**Figure 7**
**Interaction diagram: original column #2- "L" b18"x h8"**

## Column #3

The interaction diagram in figure 8 shows how the strong axis resists Pu ~ 49.93K & Mu ~ 8.83 k-ft, both from first analysis.

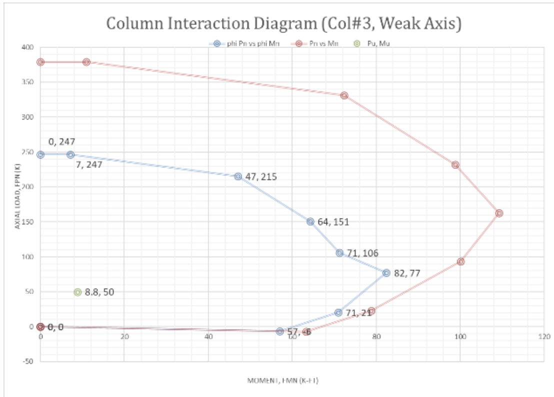


**Figure 8**
**Interaction diagram: original column #3 - b8"xh18" (strong axis orientation)**

The interaction diagram in figure 9 shows how strong axis resists Pu~49.93K and Mu~8.83 K-ft. However, shear reinforcement should be verified. To comply with section ACI318-14,18.7.2.1 [9], increasing the section in the second run ($B_{MIN}$ =12", b / h> 0.4) is recommended.

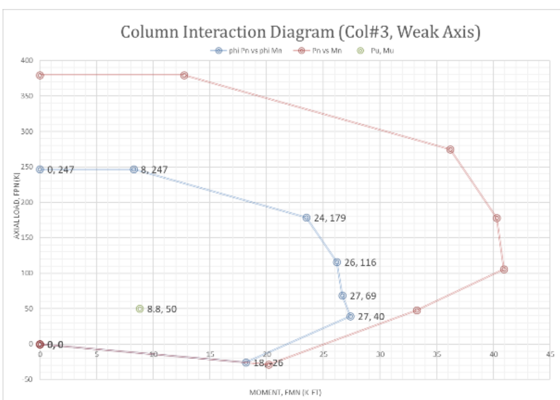


**Figure 9**
**Interaction diagram: original column #3- b18" x h8" (weak axis orientation)**

The plan establishes a general transverse reinforcement design for all the columns, *# 3 @ 6"*. This spacing changes for the zones near the joints, where the spacing is reduced to 3" x L=24". Because it is necessary to increase all the sections and rerun the model to get the new moments and forces, the shear design will be evaluated in the second analysis.

## Conclusions to Be Considered Later

So far, the results of a structural design to build a two-story house in PR have been analyzed. For full compliance with ACI318-14, these are the most important conclusions of the first analysis:

- Special force resisting frames (case studied in this article) shall satisfy sections 18.2.3 through 18.2.8 and 18.6 through 18.8 [9].
- The beam section (8x12") should meet the dimensions required in ACI318-14, 18.6.2.1 [9]. Second run will have a width B = 10" and height H = 14", which will allow a greater effective depth. Increasing "H" also improves minimum spacing distance in ties/hoops, d/4 [9].
- Lap splices of longitudinal reinforcement are prohibited along lengths where flexural yielding is anticipated [9]. Figure 10 shows splices within the beam-column joints. This condition is not allowed, ACI318-14, 18.6.3.3 [9].

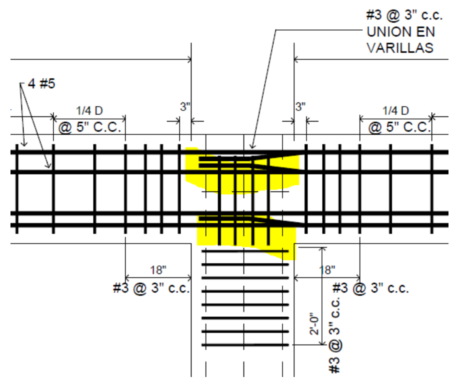


**Figure 10**
**Splices at the wrong location**

- The first hoop in beams shall be located not further than 2" from the face of a supporting column. Spacing of the hoops shall not exceed the least in ACI318-14, 18.6.4.4 [9].
- Positive moment strength at joint face shall be at least 50% the negative moment. Both the (-) and the (+) moment strength at any section across member length shall be at least 1/4 of the maximum moment, ACI318-14, 18.6.3.2 [9].
- All column widths will be increased from 8" to 12". None of the column's sections met this requirement, ACI318-14, 18.7.2.1 [9].
- Design must meet ACI318-14, 18.7.3.2 strong column weak beam requirement, $\Sigma M_{NC}>1.2\Sigma M_{NB}$ [9]. It was found that about 50% of the original columns do not meet this requirement.

## SECOND ANALYSIS: UPDATED SECTIONS

The cross-sections of the four structural elements have been changed in size and rebar configuration (figure 11, table 5). The increase in sectional areas brings an increase in inertia, dead weight, and, finally, structural strength. A second model analysis is necessary to find maximum moments, axial loads, and shears under the worst combination of factored loads. The Spectrum Response design was calculated again (table 6). After running the new model (loads in the directions N-S and E-O), it shows that the most critical forces and moments were developed with the same load combination, "Comb4 (1.2D + 1.6L)". This second analysis incorporated all the details that needed to be corrected for better compliance with code ACI318-14.

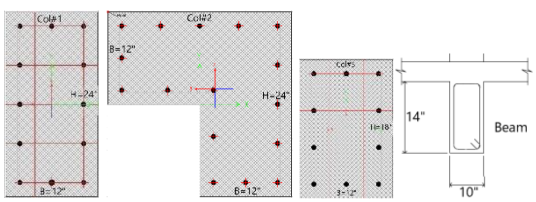


**Figure 11**
**Updated cross-sections with their new measures**

**Table 5**
**Update cross-sections measurements**

| Elements | Measurements |
|---|---|
| Column #1 | -12”x24” / 12#5 long. / ties t.b.d. |
| Column #2 | -12”x24” L / 16#5 long. / ties t.b.d. |
| Column #3 | -12”x18” / 10#5 long. / ties t.b.d. |
| Beams | -12”x14” / long. And transversal (ties) t.b.d |
| Slabs | -5” thickness |

**Table 6**
**Summary of SRD procedure for the second analysis**

| | |
|---|---|
| Location | -Ponce, PR |
| Site | -D (assumed in absence of additional data) |
| Factors | Remain the same from the 1st run |
| Seismic loads | |
| $W_T$ = 380 kip | -total weight of the structure (calculated) |
| $W_{T1}$ = 207 kip | -total weight of the 1st floor (calculated) |
| $W_{T2}$ = 173 kip | -total weight of the 2nd floor (calculated) |
| $C_{V1}$ = 0.374 | -dist. factor 1st floor **(ASCE7/16, 12.8-2)** |
| $C_{V2}$ = 0.626 | -dist. factor 2nd floor **(ASCE7/16, 12.8-2)** |
| $F_1$ = 71.2 kip | -dist. force 1st floor **(ASCE7/16, 12.8-11)** |
| $F_2$ = 118.9 kip | -dist. force 2nd floor **(ASCE7/16, 12.8-11)** |
| V = 190.1 kip | -seismic base shear **(ASCE7/16, 12.8-1)** |

### Updated Beams

$M_{U(NEG)}$ = -50.43 kip*ft & $M_{U(POS)}$ = 27.14 kip*ft
$V_U$=26.05 kip (from second analysis)
$V_E$=23.48 kip (probable V, ACI318-14, 18.6.3.2) [9]

Shear:

- Required shear strengths were determined using a stress of 1.25*Fy in the longitudinal reinforcement, ACI318-14, 18.6.5 [9].
- The increase in beam section (b=10” and H=14”) increased shear capacity.
- Section ACI318-14, 18.6.3.3 [9] states that any splice must have continuous confinement, with the smallest spacing between d / 4 or 4", in our case, $S_{plice}$= (14-1.5)/4 ~ 3”.
- Section ACI318-14, 18.6.4.3 states the minimum confinement spacing shall be maintained for 2*d=25” [9].
- First hoop must be placed 2” from the joint:

$$S_{bfinal} := min\left(\frac{d_b}{4}, 6 \cdot d_{b5}, 6\ in\right) = 3.125\ in$$

- After the first hoop, the design suggests transverse steel #3@3" (hoops) by 2*d = 2 * 12.5" = 25":

$$S_{bnormal} := \text{floor}\left(\frac{\frac{d_b}{2}}{in}\right) in = 6\ in$$

- After the hoops #3@3” x 25”, the design suggests transverse steel #3@6” by the remaining length to the center of the beam.
- Lap splices shall not be placed in joints nor a distance 2*d from the joint [9].

Flexure:

ACI318-14, 9.6.1.2 [9],

$$As_{min1} := \frac{3 \cdot \sqrt{\frac{f'c}{psi}}}{fy} \cdot psi \cdot B_b \cdot d_b = 0.342\ in^2 \quad , \quad As_{min2} := \frac{200\ psi \cdot B_b \cdot d_b}{fy} = 0.4167\ in^2$$

$$\rho_{bal} := \frac{0.85\ \beta_1 \cdot f'c}{fy}\left(\frac{87000}{87000 + \frac{fy}{psi}}\right) = 0.021 \quad , \quad \rho max_{bal} := 0.75 \cdot \rho_{bal} = 0.016$$

$$\rho_{top} := \frac{0.85\ f'c}{fy}\left(1 - \sqrt{1 - \frac{2.353 \cdot \frac{M_{UBneg} \cdot -1}{kip \cdot ft} \cdot 12 \cdot 1000}{\phi_b \cdot \frac{B_b}{in} \cdot \left(\frac{d_b}{in}\right)^2 \cdot \frac{f'c}{psi}}}\right) = 0.0079$$

The amount of steel for **negative** moment met ductile failure criteria. The analysis (~ 50 K-Ft) suggests using a longitudinal distribution steel

(As=0.989 $in^2$) of 4#5 to resist the maximum negative moment demand. This is the same amount of steel as in plan, it is validated.

$$\rho_{bottom} := \frac{0.85\ f'c}{fy} \left(1 - \sqrt{1 - \frac{2.353 \cdot \frac{M_{UBpos}}{kip \cdot ft} \cdot 12 \cdot 1000}{\phi_b \cdot \frac{B_b}{in} \cdot \left(\frac{d_b}{in}\right)^2 \cdot \frac{f'c}{psi}}}\right) = 0.0041$$

The amount of steel for **positive** moment meets ductile failure criteria. The analysis (~+27K-Ft), suggests using a longitudinal distribution (As=0.507 $in^2$) of 2#5 for the maximum positive moment demand. Following the section ACI318-14, 18.6.3.2, the final longitudinal distribution suggested will be:

- From the joints: 4#5 Top (- moment) and 2#5 Bottom (+ moment, 50%As [-])
- Center of span: 2#5 Bottom (+ moment) and 2#5 Top (- moment, minimum of 25%As, $M_{max}$ but not less than 2 rebars)

Length of Development:

- In hooks (ACI318-14, 25.4.3.1):

$$L_{dh} := \max\left(\left(\frac{fy \cdot \psi_e \cdot \psi_c \cdot \psi_r}{50\ \lambda \sqrt[2]{\frac{f'c}{psi}} \cdot psi}\right) \cdot d_b, 8\ d_b, 6\ in\right) = 13.693$$

- In longitudinal tension (ACI318-14, 25.4.2.2):

$$L_{db} := \left(\frac{fy \cdot \psi_t \cdot \psi_e}{25\ \lambda \sqrt[2]{\frac{f'c}{psi}} \cdot psi}\right) \cdot d_b = 27.386\ in$$

- In splices (ACI318-14, 25.5.2.1):

$$L_{ds} := 1.3 \cdot L_{db} = 36.4\ in$$

## Updated Columns

Table 7 shows the maximum forces in the updated columns.

**Table 7**
**Maximum forces in columns (2nd run)**

| FINAL LOADS RESULTS FOR COLUMNS (2ND RUN - SAP2000 MODEL) | | | | | |
|---|---|---|---|---|---|
| COL#1 | | COL#2 | | COL#3 | |
| M3,MAX(-)= | -59.56 | M3,MAX(-)= | -24.02 | M3,MAX(-)= | -10.2188 |
| M3,MAX(+)= | 41.43 | M3,MAX(+)= | 24.93 | M3,MAX(+)= | 9.849 |
| M2,MAX(-)= | -37.94 | M2,MAX(-)= | -16.36 | M2,MAX(-)= | -3.08 |
| M2,MAX(+)= | 29.91 | M2,MAX(+)= | 21.17 | M2,MAX(+)= | 3.76 |
| V,MAX= | 10.92 | V,MAX= | 5.29 | V,MAX= | 1.837 |
| P,MAX= | 215.87 | P,MAX= | 187.24 | P,MAX= | 63.32 |

## Updated Column #1

The interaction diagram in figure 11 shows weak axis resists Pu~216K and Mu~60 k-ft (The strong axis orientation resists these loads by default).

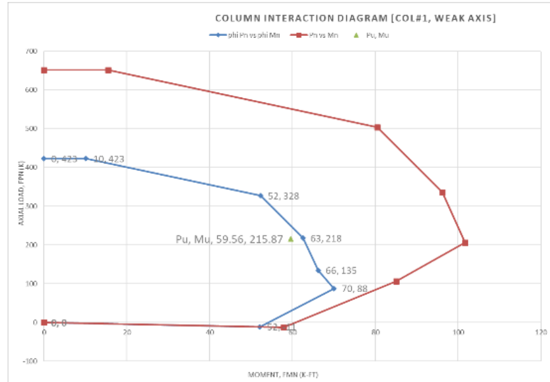


**Figure 11**
**Interaction diagram: updated col#1 – b12"x h24" & 12#5**

Transverse steel (shear design):

- $L_O$ from joints (ACI318-14, 18.7.5.1) [9]

$$L_{o1} := \max\left(d_{c1}, \frac{L_c}{6}, 18\ in\right) = 22.5\ in$$

- $L_{Omax}$ spacing (ACI318-14, 18.7.5.2) [9]

$$S_o := 4\ in + \frac{14\ in - h_{x1}}{3} = 6.97\ in \quad , \quad S_{o1} := min\left(\frac{x_{o1}}{4}, 6 \cdot d_{\theta 5}, S_o\right) = 3.00\ in$$

- Amount of steel (ACI318-14, 18.7.5.4) [9]

$$A_{sh1} := 2 \cdot \theta_4 = 0.4\ in^2$$

$$x_{01} := \max\left(0.3 \cdot \left(\frac{A_{g1}}{A_{ch1}} - 1\right) \cdot \left(\frac{f'c}{fyt}\right), 0.09 \cdot \left(\frac{f'c}{fyt}\right)\right) = 0.008$$

- The hoop's spacing beyond $L_O$ should not exceed the lesser of 6*$d_{bar}$ and 6" (18.7.5.5) [9]
- Use hoops #4@3" over the entire length of col#1

Column length splices:

$$l_{d1} := \max\left(1.3\left(\frac{fy \cdot \psi_t \cdot \psi_B}{25\ \lambda \cdot \sqrt{\frac{f'c}{psi}} \cdot psi}\right) \cdot d_b, 12\ in\right) = 42.722\ in$$

## Updated Column #2

The interaction diagram in figure 12 shows column #2 resists Pu~187K and Mu~25 k-ft.

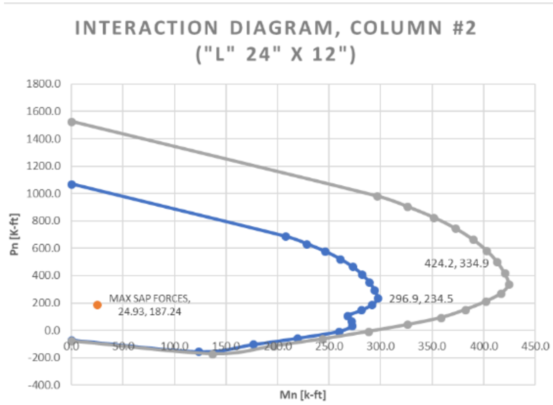


**Figure 12**
**Interaction diagram: updated col#2 - b12"xh24" & 16#5**

Transverse steel (shear design):

- $L_O$ from joints (ACI318-14, 18.7.5.1) [9]

$$L_{o2} := \max\left(d_{c2}, \frac{L_c}{6}, 18\ in\right) = 22.5\ in$$

- $L_{Omax}$ spacing (ACI318-14, 18.7.5.2) [9]

$$S_o := 4\ in + \frac{14\ in - h_{x1}}{3} = 6.97\ in$$

$$S_{o2} := min\left(\frac{x_{o2}}{4}, 6 \cdot d_{\theta 5}, S_o\right) = 3.00\ in$$

- Amount of steel (ACI318-14, 18.7.5.4) [9]

$$A_{sh2} := 3 \cdot \theta_4 = 0.6\ in^2$$

$$x_{02} := \max\left(0.3 \cdot \left(\frac{A_{g2}}{A_{ch2}} - 1\right) \cdot \left(\frac{f'c}{fyt}\right), 0.09 \cdot \left(\frac{f'c}{fyt}\right)\right) = 0.007$$

- The hoop's spacing beyond $L_O$ should not exceed the lesser of 6*$d_{bar}$ and 6" (18.7.5.5) [9]
- Use hoops #4@3" + 1#4 cross-hook over the entire length of column #2

Column length splices:

$$l_{d2} := \max\left(1.3\left(\frac{fy \cdot \psi_t \cdot \psi_B}{25\ \lambda \cdot \sqrt{\frac{f'c}{psi}} \cdot psi}\right) \cdot d_b, 12\ in\right) = 42.722\ in$$

### Updated Column #3

The interaction diagram in figure 13 shows weak axis resists Pu~63K and Mu~10 k-ft (The strong axis orientation resists these loads by default).

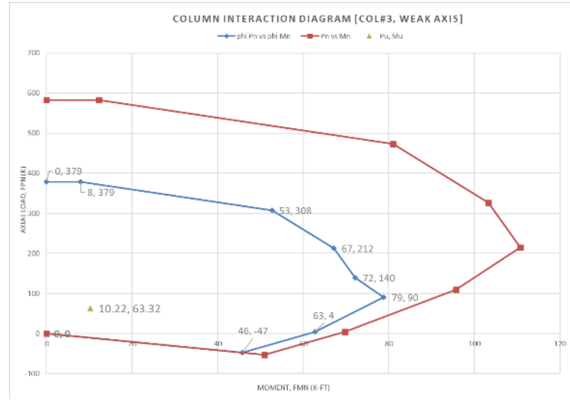


**Figure 13**
**Interaction diagram: updated col#3 – b12"xh18" & 10#5**

Transverse steel (shear design):

- $L_O$ from joints (ACI318-14, 18.7.5.1) [9]

$$L_{o3} := \max\left(d_{c3}, \frac{L_c}{6}, 18\ in\right) = 18.5\ in$$

- $L_{Omax}$ spacing (ACI318-14, 18.7.5.2) [9]

$$S_o := 4\ in + \frac{14\ in - h_{x1}}{3} = 6.97\ in$$

$$S_{o3} := min\left(\frac{x_{o3}}{4}, 6 \cdot d_{\theta 5}, S_o\right) = 3.00\ in$$

- Amount of steel (ACI318-14, 18.7.5.4) [9]

$$A_{sh3} := 3 \cdot \theta_4 = 0.6\ in^2$$

$$x_{03} := \max\left(0.3 \cdot \left(\frac{A_{g3}}{A_{ch3}} - 1\right) \cdot \left(\frac{f'c}{fyt}\right), 0.09 \cdot \left(\frac{f'c}{fyt}\right)\right) = 0.009$$

- The hoop's spacing beyond $L_O$ should not exceed the lesser of 6*$d_{bar}$ and 6" (18.7.5.5) [9].
- Finally use hoops #4@3" + 1#4 cross-hook over the entire length of column

Column length splices:

$$l_{d3} := \max\left(1.3\left(\frac{fy \cdot \psi_t \cdot \psi_B}{25\ \lambda \cdot \sqrt{\frac{f'c}{psi}} \cdot psi}\right) \cdot d_b, 12\ in\right) = 42.722\ in$$

## FINAL ANALYSIS AND CONCLUSIONS

- This case study redesign satisfies ACI318-14, 18.2.3 through 18.2.8 and 18.6 through 18.8 [9].
- The plan's original beam (8x12") was increased to meet the minimum dimensions required in ACI318-14, 18.6.2.1 [9]. It is important to ensure from the beginning that the effective depth is d>14". The final dimension suggested (10x14")

allows a greater effective depth. For a lower beam height, it is necessary to consider the design as a T-beam.

- The final suggested longitudinal reinforcement in beams is 4#5 top steel (negative moment) and 3#5 bottom steel (positive moment). Joints should have 2#5 bottom steel (50% of negative moment) and center of the beam should have 2#5 top steel (at least 25% of the maximum moment), ACI318-14, 18.6.3.2 [9].
- The transversal reinforcement in beams is suggested to be #3@2" from the joint, #3@3" for a distance 2d=25" and #3@6" until the center.
- The bases (widths) of all columns increased from 8" to 12" to comply section ACI318-14, 18.7.2.1 [9]. This requirement needs to be met from the beginning of design.
- The longitudinal reinforcement in columns is suggested to be column #1-12#5, column #2-16#5 and column #3-10#5
- The transversal reinforcement in columns is suggested to be column #1-#4@3, columns #2&#3-#4@3" with 1 hook#4 (both directions). First hoop shall be located 2" form the joint ACI318-14, 18.7.5 [9]
- Lap splices of reinforcement are prohibited in beam-column joints. This condition is not allowed, ACI318-14, 18.6.3.3 [9].
- Final design (2$^{nd}$ run) complies with section ACI318-14, 18.7.3.2, better known as strong column-weak beam, $\Sigma M_{NC}>1.2\Sigma M_{NB.}$ Approximately 50% of the original columns' sections did not meet this capacity requirement in the first analysis, even for the first floor. The capacity in all elements shall ensure that the beams fail first, so the inertia of the columns must be always greater (more rigid). Columns that do not meet these criteria on the top floor (because there are no columns coming from the roof structural slab) must be confined to the minimum tie spacing (in this case study, 3") along the entire length of the column. In this case study, all columns have minimum spacing due to $6*D_{Long,bar}$, ACI318-14, 18.7.5.5 [9].

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