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Paseo Caribe Condominium Tower and Parking Garage
Proposal

1. Background

Paseo Caribe Condominium Tower and Parking Garage is a multi-use complex currently under construction in Condado, Puerto Rico's prime tourist area. The project consists of a 284,480 square feet residential tower that houses 46 luxury apartments. The tower raises 14 stories above a previously constructed 10 story Parking Garage and overlooks the San Juan bay inlet. Each apartment is 4,800 square feet and they are to be privately owned. There are four apartments per floor; each apartment is approximately 80 feet by 60 feet in plan. The plan is designed to maximize the ocean view.

The Parking Garage is 514,480 square feet and provides 1,283 parking spaces. The parking includes private apartment parking along with valet parking for the neighboring Caribe Hilton Hotel, Paseo Caribe Entertainment Center and the new Convention Center.

Typical floor to floor height for the parking garage is 10 feet and 9 feet 10 inches for the apartment units. The parking and tower reach a total elevation of 244 feet 3 inches. The total square footage of the building is a little less than 800,000 square feet.

2. Original Structure Design

The structure is reinforced cast in place concrete. This is very typical construction practice in the area because of the limited resources and skilled labor required for steel construction. The floor system is a one way 8 inch post tensioned slab. This system was selected for both its strength and the minimum slab thickness.

Lateral forces are resisted by a shear wall system. There is a 10 foot wide core spanning the length of the building at its line of symmetry in the north to south direction. It is comprised of interconnecting 12 inch walls forming 2 elevator shafts and 3 stair enclosures. The walls act together to form a very stiff 10 feet by 150 feet inner tube. At parallel offsets increments from the core, 10 inch walls are used for bearing capacity of the slab-gravity system. This system lends itself well to residential building where repetitive partitions are needed at defined locations. The shear walls eliminate the need for wide concrete columns needed for 21 stories of a typical 27 feet bay span. It also eliminates the need to cast beam for the one-way slab system.

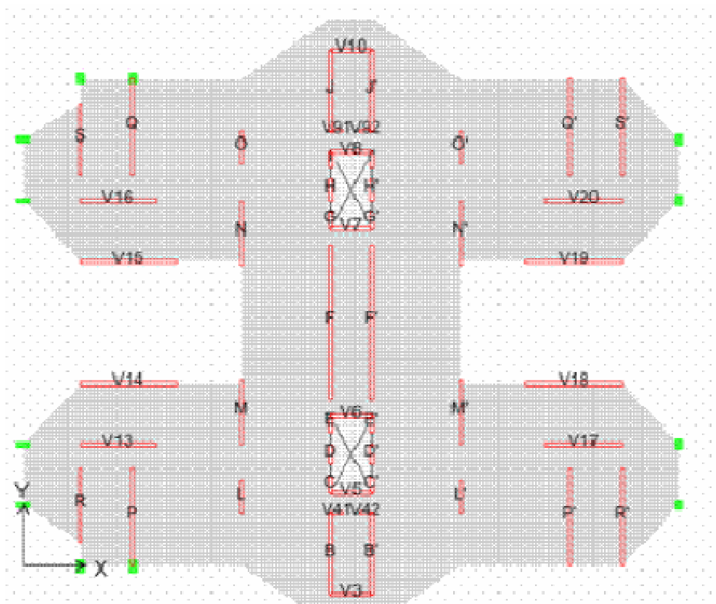

Figure 1 - Typical Apartment Floor Shear Wall Layout

The parking garage has the same structural post – tensioned floor system as the condominium. However, it has a very different lateral system. The lateral loads are resisted by a column frame system. The typical column size is 24 inches by 36 inches with 16 - #7 bar diameter. The grid spacing is a uniform 15 feet (north-south) by 27 feet (east to west).

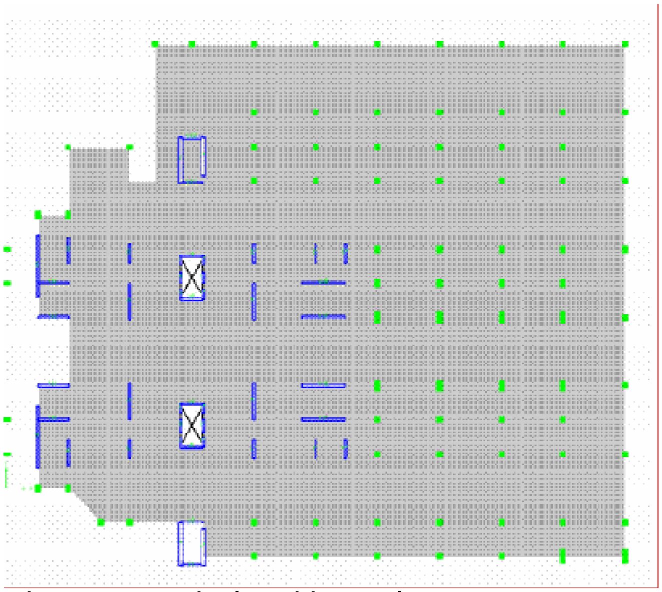

Figure 2 - Typical Parking Column Frame Layout

The floor layout for both structures in this complex is symmetrical and uniform through each level. However, a major point of concern is at the 8th level above ground where the parking garage becomes the condominium tower the lateral system is altered. This floor of transition is very crucial in the lateral system design. As a result of this increase stiffness at this floor level, the floors below this story might be considered as "extreme soft story". The major changes is this story are: (Please refer to Figure 2 for wall labeling references)

- The 2 stair enclosures that extended through the 8 levels of parking lots and form part of the core are shifted at the lobby level 30' each inward toward the center of the building. A 3rd set of stairs was added along the core line and covers the space in-between the two elevator shafts. These changes increase the stiffness at the core.
- Shear walls L, O are extended 8' south over the original wall.
- Shear walls M, N are extended 13' south over the original wall.
- Shear wall V14-V18 extended 8' inward over original wall.

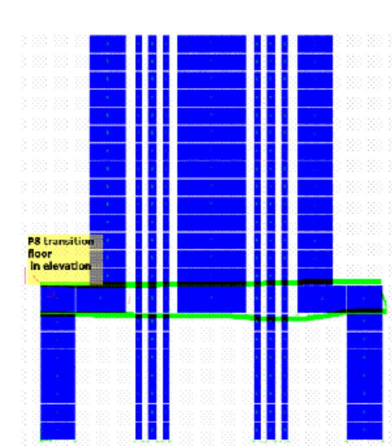

Figure 3 - Building section through core showing transition level

The structure was design following the current code of practice in Puerto Rico: Uniform Building Code 1997. Design loads are based on ASCE 7 – 95 and concrete design follows the provisions specified in ACI 318 – 95. Analysis of the current structural system demonstrates that the system is adequate for both gravity and lateral loading. (Reference 1 and 2).

3. Problem Statement

Results of the analysis show that seismic requirements control the design of the structure. The maximum story shear due to seismic is about 5 times larger than that due to wind.

Wind: 1678 kips
Seismic: 8456 kips

According to UBC 1997 Figure 16.2 Puerto Rico is classified in Seismic Zone 3. The soil profile for the site is Sd. This combination yields one of the highest seismic coefficient, Ca = 0.36. The base shear V is directly proportional to the product of Ca and the building weight, W. It does not help that the building has a very large dead weight due to the large amount of concrete in the 800,000 square feet of slab area and 20,000 linear feet of 10 feet high shear walls. This explains why the post-tensioned system was selected in the attempt to minimize the amount of concrete in the building. ACI provisions do not specify minimum slab thickness requirements for pre-stressed or post-tensioned slabs as long as deflection criteria are met. Prestressing can increase the span range to slab thickness ratio by about 30 to 40 percent. Following ACI Table 9.5(a), a typical one-way reinforced concrete slab is required to have a minimum slab thickness of l/28. For a 27

foot span, the minimum thickness would be 12 inches. This is an additional 50 psf, or 16,886 kips for all 22 floors above ground.

Even with the smaller slab thickness allowed by post-tensioning, the slab contribution to the building's weight is significant:

$$W_{total} = 95{,}132 \text{ kips}$$
$$W_{slab} = 71{,}551 \text{ kips } (75\% \text{ of } W_{total})$$
$$W_{wall} = 23{,}581 \text{ kips } (25\% \text{ of } W_{total})$$

Three quarters of the total weight of the building is in the slab. If the weight of the slab can be reduced, the large seismic lateral forces will also be reduced. This will reduce the amount or shear wall resistance required for lateral loads. A reduced amount of shear walls will further reduce the total weight of the building. It will also allow for a more open – wall free - floor plan as the bearing capacity of the walls is reduced due to a lighter slab dead weight. Furthermore, this reduction in weight will have a positive impact in the foundation of the building. Currently, there are 7 different piles caps along the length of the core, each with 30 piles. Surrounding the core there are other 30 piles caps, each ranging from 6 to 10 piles each. Around the perimeter of the building there are 38 more pile caps, each with 4 mini piles. That is approximately 75 piles caps with over 600 mini piles.

Therefore, a lighter weight floor structure must be designed for both parking garage and apartment units to resist dead loads and minimum design live loads of 60 psf (40 psf live + 20 psf partitions) for the apartments and 50 psf for the parking garage as specified in ASCE 7 – 95 Table 4.1. The new floor structure must be able to span 27 feet while meeting deflection criteria of l/360. The total depth of the slab should be kept to a minimum while maintaining a floor ceiling envelope of less than 18". These will accommodate for a ceiling height of 8 feet 6 inches with a floor to floor height of less than 10 feet. This maximum floor height will allow the building to meet the height limit requirements for Seismic Zone 3 specified by UBC 1997 Table 16-N of 160 feet for a bearing wall system with concrete shear walls (Item 1.2.a).

4. Proposed Solution

Two alternate systems are going to be investigated to minimize the weight of the building, the magnitude of the lateral loads; further reducing the amount of shear and bearing walls in the floor plan. The first alternate system is a pre-cast hollow concrete system with high strength concrete for columns and bearing walls. A concrete system is desirable because of the availability of materials and labor in the island over steel. Two major advantages of this system are the reduced structural depth and the high strength to weight

ratio. Also, prestressed concrete members are crack-free, provide good noise and vibration quality control, and are fire-proof. All of these are important characteristics for a luxurious residential apartment complex.

The preliminary sizing of this alternate system is a 8" x 4' Span Deck – U.L – J917 with a 2" C.I.P topping. For a 27 foot requirement, this slab with of 4 – ½" diameter strands can sustain 84 psf (Refer to Appendix A). This load is larger than the 70 psf required for design. This slab is slightly deeper than the current slab however; the weight of the slab is 82.5 psf compared to 100 psf of the current system. This is a 20% reduction in dead weight. As seen from Figure 4, this system might allow for the elimination of extra bearing walls and the use of a uniform 27 foot bay.

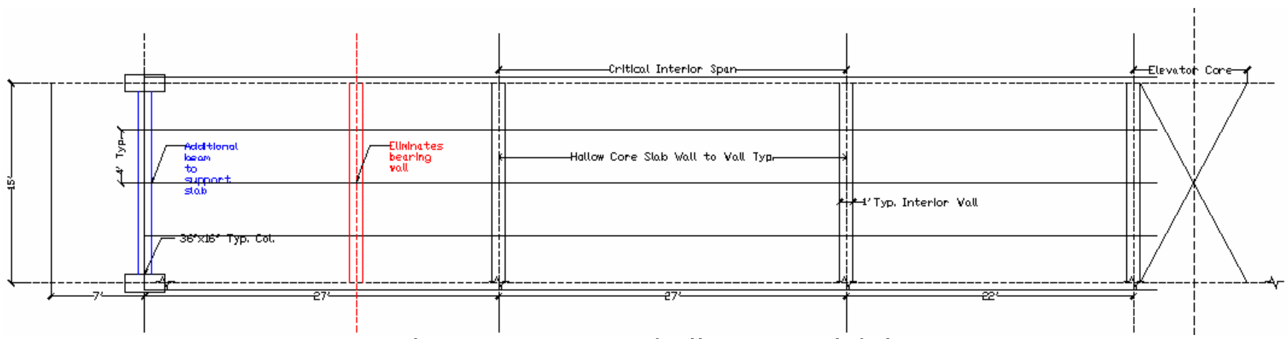

Figure 4: One way hollow core slab layout

Another advantage of pre cast construction is that it lends itself well to parking structures. A similar system can be used for the design of 10 story parking garage. Usually this system consists of double tees. Based on Nitterhouse Product information a preliminary size of 32" by 12' can be used. There are two main structural advantages to this system. First, the weight of the double tee is 83 psf compared to 100 psf of the current system. Second, according the span tables (Refer to Appendix B) the double tees can span up to 56 with and carry 66 psf superimposed load. This load is larger than the 50 psf required for analysis. Furthermore, this would allow for the removal of every other column and more open space for parking spaces. However, this would also require higher strength concrete for the columns.

The second alternate system will explore the use of steel. Steel is not widely used in Puerto Rico. Due to the lack of available materials and skilled labor it is an expensive and lengthy (due to procurement) construction method. However, for purposes of this educational research it will be assumed that the materials and labor are readily available. The system will look at a composite steel floor system. This alternative will explore a composite steel deck and Smartbeam system. The smartbeam is a cellular beam manufactured by SMI Steel Products. It starts with a standard WF beam, which is cut in half diagonally. The two halves are separated, staggered and welded together to form a Smartbeam what weights the same but is 50% stronger and deeper than the original beam.

Picture 1: Smart Beam

The two advantages of this system are its high strength to weight ratio and the lower floor-to-floor height by passing ductwork and utilities through the openings. From previous analysis, the preliminary design of this system for a 27 by 15 foot bay is CB15x12 spaced at 5 feet o.c. The composite design includes a 2.5 inch concrete cover on a 1.5" Lok metal deck and 10 shear studs per beam. The CB15x12 will be supported by W10x12's spanning every 27 feet. The total depth of the system is 18" allowing for a ceiling height of 8 feet 6 inches with a floor to floor height of less than 10 feet.

5. Solution Method

5.1 One-Way Hollow Core Slab

The design of the one way pre cast hollow core slab system is governed by the ACI 18 – 95 Building Code Requirements for Structural Concrete. Hollow core slabs are checked for prestress transfer stresses, handling stresses, service load stresses, deflections, and design ultimate strength in shear and bending. For uniform load cases, the manufacturer's load tables will take into account these various design considerations. Load span tables used as reference are from Nitterhouse Product Catalog (www.nitterhouse.com). From the layout of the system presented in Figure 4, there are two strength considerations that must be check according to these criteria:

- Maximum span: 27 feet; Minimum superimposed load: 60 psf for living spaces
- Minimum span: 22 feet; Maximum superimposed load: 100 psf for corridors and terrace

General design guidelines can be reference to ACI Chapter 18 for flexure design provisions and applicable permissible stresses of extreme tension and compression fibers. Shear provisions can be found in ACI Chapter 11. The code does not allow any shear strength from stirrups, Vs. The nominal concrete strength, Vc, is given by Equation 11-7 in Section 11.3.2.2 for members with an effective prestress force not less than 40% of the tensile strength of the flexural reinforcement. Care must be given to the design of the structural topping. ACI Section 17.5.2.1 limits the horizontal shear strength to 80 psi to be considered for composite action. Finally, deflection limits are based on acceptable values compared to Table 9.5(b) in the ACI. Deflection limits are generally not considered by the manufactures' span load tables and will have to be checked.

Loading conditions other than uniform will be checked based on ACI Section 16.3.1. This section presents load transfer capabilities for concentrated point loads or lines loads. These loads can result from solid partition wall on the slab or heavy equipment on the roof and parking garage.

Subsequently, the slab must be adequate for elastic diaphragm action according to UBC 1997 lateral load results for seismic for Seismic Zone 3. The slab will be assumed to be rigid for strength calculations. However, a simple flexibility check will be performed based on the cracked moment of inertia of the slab and a Virendeel truss model. Minimum tie forces nominal strengths between the slab and shear walls are references in ACI Section 16.5.

Finally, connections adequacy for seismic requirements will be detailed and minimum fire rating requirements will be verified.

5.2 Composite steel and Smart Beam system

The second alternate system of composite steel and Smart Beam system is to be designed in accordance with the AISC Manual of Steel Construction, Load and Resistance Factor Design Second Ed. and per the design procedures outlined in "The Design of Welded Structures" by Omar W. Blodgett. There are 3 basic checks in designing castellated beams: Vierendeel Bending, Shear, and Web Post Buckling. The Smart Beam design is very similar to that of a Vierendeel truss design in that the shear component creates bending strength through the web openings that have to be accounted for in the bending stress calculations. An acceptable web post buckling design procedure has been developed by Drs. Dick Reswood and A. A. Aglan. Their published tables give the ultimate capacities for a variability of opening heights, widths, post widths and thickness based on experimental data. Preliminary sizes will be obtained using SMI Steel Products Composite Castellated Design Program, downloadable from their website (www.smisteelproducts.com). A

detailed gravity analysis will performed using the finite element software program STAAD Pro. The program has in-built capabilities for castellated beams design.

Finally, careful consideration needs to be given to the connection design. Connections for Smart Beams® shall be based on the simple span uniform load design tables as provided by SMI Steel Products. Design values for high strength bearing type bolts with thread allowed across shear plane are specified for use. The final connection shall be designed for 55% of the reaction obtained by using the maximum allowable load as shown in the SMI Steel Products Smart Beam® Design Manual.

6. Task and Tools

6.1 One-way Hollow Core Concrete Slab Alternative # 1

A. Determine critical span to loading conditions
   a. Define required spans based on building plan layout.
   b. Define occupancy of different spaces based on building plans.
   c. Determine fire rating requirements based on the building specifications.
   d. Find live loads on basis of ASCE 7 – 95; Table 4.
   e. Compare maximum load (Case 1) to maximum span (Case 2) conditions.
B. Establish trial member sizes
   a. Obtain load span tables from manufacturer.
   b. Select trial hollow core size based on limiting span/load condition.
   c. Select minimum pre-stressing required for flexure and shear strength.
   d. Select type of structural topping.
   e. Verify fire-rating requirements.
C. Check deflections criteria for trial member size
   a. Calculate initial camber due to pre-stressing with 5% initial losses.
   b. Calculate erection camber.
   c. Calculate change in camber from fabrication to erection (b-a).
   d. Calculate superimposed dead load instant and long term deflections due to creep.
   e. Calculate superimposed live load deflection.
   f. Verify overall deflection is less than L/480.
D. Verify composite strength of structural topping
   a. Determine required compressive strength required for the topping.
   b. Determine if minimum reinforcement is required for strength or temperature and shrinkage control.
   c. Obtain horizontal shear diagrams for slab

   d. Verify maximum shear is less than the 80 psi limit by ACI 318-95 Section 17.5.2.1 or design composite ties.

E. Verify load distribution capacity of slab for concentrated line loads, point loads, and slab openings
   a. Calculate concentrated point loads of heavy machinery in mechanical spaces and line loads on slab due to CMU partition wall to be distributed between slabs.
   b. Calculate the effective resisting section and verify shear and flexural strengths.
   c. Determine opening dimensions parallel to the span and the length of the strand embedment available from the end of an opening to the point of maximum moment for flexural strength at openings and torsional shear on the slab.

F. Design hollow core slab for diaphragm action of lateral seismic loads (Assuming rigid)
   a. Calculate dead weight of the building and obtain lateral seismic forces according to UBC 1997 provisions for Seismic Zone 3 and Soil Type Sd.
   b. Calculate design load for roof diaphragm and shear distributions to walls based on relative rigidities.
   c. Determine chord reinforcement around perimeter boundary elements.
   d. Determine reinforcement for diaphragm web to chord.
   e. Determine longitudinal shear and drag strut reinforcement at each wall connection.
   f. Calculate deflections between end shear walls to verify diaphragm rigidity.

G. Complete lateral analysis on E-tabs by modeling shear walls and slab diaphragm
   a. Calculate shear forces and size reinforcement for lateral shear walls.
   b. Determine story drift

H. Specify and detail connections
   a. Specify connections to concrete walls
   b. Detail connections to columns
   c. Detail connections to exterior enclosure

I. Study vibration and noise control qualities of slab panels
   a. Determine sound transmission loss and STC.
   b. Compare floor values to acceptable noise criteria at 11 frequencies.
   c. Obtain driving frequencies of mechanical equipment.
   d. Calculate minimum required static deflection of the isolators for each equipment

  e. Calculate maximum allowed static deflection of the floor system cause by the weight of the system.

J. Compare cost and schedule impact on project
  a. Determine site layout and sequence
  b. Use Manufacturers Cost Data and R.S. Means for material and labor cost
  c. Use Primavera to estimate overall project cost impact
  d. Use Mc2 to develop a schedule for the erection of the structure

7. Time Table

Week	Task
8-Jan	Material and Consultation
15-Jan	Task A & B
22-Jan	Task C & D
29-Jan	Task D & E
5-Feb	Task E
12-Feb	Task F
19-Feb	Task F & G
26-Feb	Task G
5-Mar	Spring Break
12-Mar	Task H & I
19-Mar	Task I & J
26-Mar	Task J
2-Apr	Conclusions and Report
9-Apr	Presentations