

PODIUM SLAB DESIGN EXAMPLE

SUPPORT FOR LIGHT-FRAMED SUPERSTRUCTURE

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APPENDIX A : Punching Shear Calculation

S.0 PRELIMINARY CONSIDERATIONS

S.0.1 APPLICATION AND FUNCTION

A common type of construction for medium density residential or commercial occupancy, where land is relatively expensive hybrid construction. The lower one or two levels are parking or retail. The bottom structure supports three to five levels of light frame residential or commercial superstructure. Where there is height restriction, it is not uncommon the move the parking level below grade. Figures S.01-1 through 3 are examples of hybrid construction.



FIGURE S.0.1-1 Hybrid Construction; the Concrete Slab Supports Light Framing Superstructure (P1084)

FIGURE S.0.1-2 Hybrid Construction (P863)
The post-tensioned podium slab supports five levels of light framing



FIGURE S.0.1-3 Hybrid Construction; View of Finished Building (P1415)
 In this hotel, the large ground level restaurant serving a popular area, and the parking levels below grade called for wide open spans. The post-tensioned podium slab over the ground level supports the light framed hotel rooms with short spans and multitude of non-aligned walls with the ground level supports.

The uppermost construction level of the concrete level that supports the light-framed superstructure is referred to as podium slab. The distinguishing feature of this slab compared to the typical residential or office floor slabs is the following:

- ❖ The load on a podium slab is much higher than the common floors. The podium slab carries the load of several floors it supports. The wall and column supports of the superstructure do not generally line up with the supports of the podium slab. The podium slab is in effect a transfer plate.
- ❖ The arrangement of the walls and columns above the podium slab is generally selected to suit the occupancy above the podium – mostly residential or office occupancy. The support arrangement below the podium slab is mostly configured for parking or retail.
- ❖ The allocation of space below the podium slab – such as breakdown into parking stalls – remains generally unchanged over life of the structure. Those of the upper level light framing are likely to change due to re-modeling of space. This necessitates the design of the podium slab to account for re-positioning of the loads from above.
- ❖ In many instances, the podium slabs feature exposed areas for access or landscaping. The exposed regions require special treatment, both in regards to the value of design loads from landscaping and modification is slab geometry. In many instances, a slab of uniform thickness below the superstructure does not apply.

S.0.2 Geometry and Details

In its simplest form, podium slabs are covered entirely by a light frame superstructure. Access to the level below the podium is through stairwell opening and elevators. In this case the slab will be of uniform thickness. Drops at entrance to balconies and landscaping often requires change in slab thickness at discontinuities.

S.0.3 Preliminary Sizing

The initial assumption for thickness of the podium slab depends on the number of levels that the slab supports and the concrete strength used.

For common residential and commercial occupancy; light framed superstructure either wood or metal studs; concrete strength 5000 psi (34.47 MPa) or higher; the following initial podium slab thickness (h) is recommended.

2 level over podium; $h = L/40 \sim 8.5\text{-in. (216 mm)}$; Column round 14-in. (356 mm)
 3 levels over podium; $h = L/38 \sim 10\text{-in. (254 mm)}$; Column round 16-in. (406 mm)
 4 levels over podium; $h = L/35 \sim 11\text{-in. (279 mm)}$; Column round 16-in. (406 mm)

Column drops or punching shear reinforcement is generally required for the thicknesses suggested.

S.1 GEOMETRY AND CONSTRUCTION

Ava is a four story apartment building in Northern California. The typical longitudinal section of the building is shown in Fig. S.0.3-1.



FIGURE S.0.3-1 Elevation; Simplified View of Podium Slab Supporting Four Levels of Light Framing (PTS941)

The following covers the gravity and lateral design of the concrete members of the structure.

S.1.1 Occupancy

The upper four levels are residential. The ground level is parking. A post-tensioned podium slab separate the parking below from the light residential framing above.

S.1.2 Geometry

The post-tensioned podium slab, the supporting concrete walls and columns, along with the ground-supported mat foundation form an integrated concrete box that supports the superstructure.

The plans of the parking level and the podium slab are shown in Figs. S.1.2-1 and 2. The foundation of the structure is a conventionally reinforced mat slab supported directly on soil. A 3D view of the concrete box is shown in Fig. S.1.2-3.

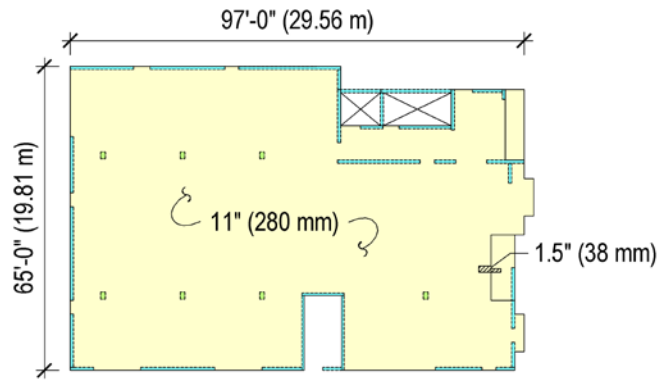


FIGURE S.1.2-1 Plan; Podium Slab (PTS942)

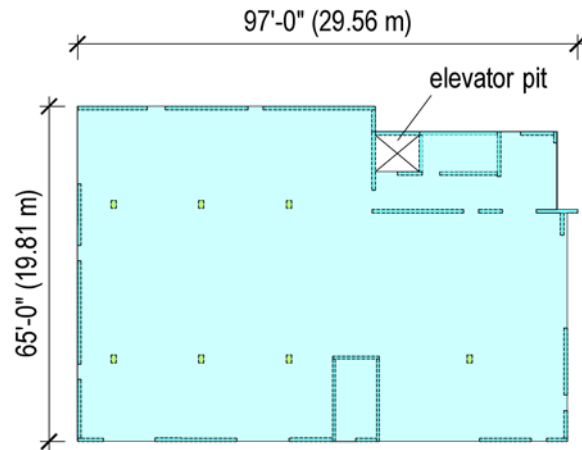


FIGURE S.1.2-2 Plan; Parking Ground Level (PTS940)

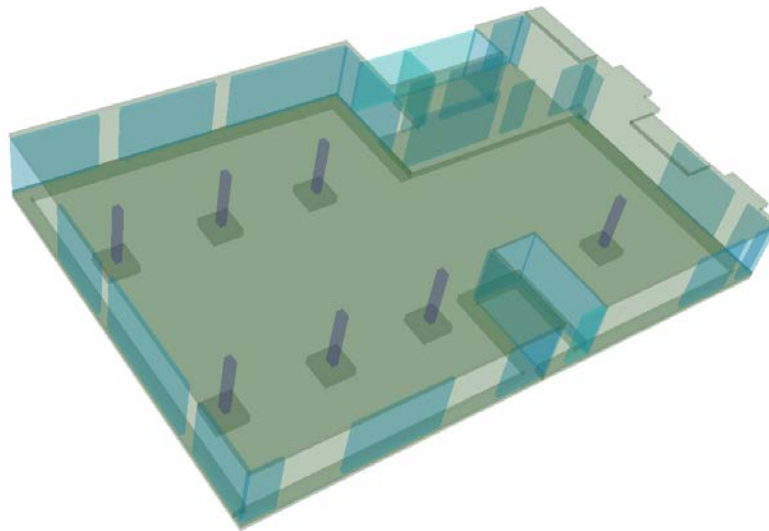


FIGURE S.1.2-3 3D See-Through View of the Concrete Box (P1278)
The concrete box supports four levels of superstructure

The lateral force resisting system of the concrete frame consists of shear walls acting with the podium slab and the foundation mat. The walls designated to resist seismic forces are identified as SWL in Fig. S.1.2-4.

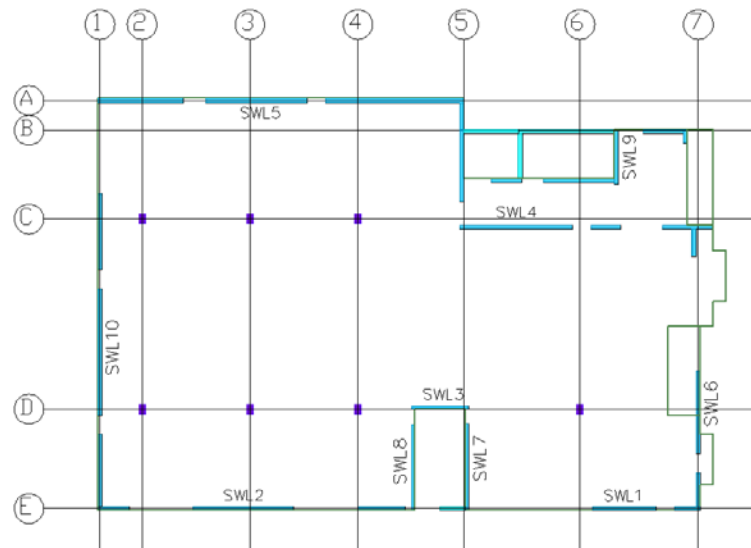


FIGURE S.1.2-4 Plan of the Podium Slab Identifying the Walls Designated to Resist Seismic (Lateral) Forces (P1393)
The walls not identified as shear wall (SWL) will be designed to resist gravity loads only

S.1.3 Sizing

For the four level light framing, and colum supports spaced at 30 ft (9.14 m), using the guidelines S.0.3 the following dimensions are selected.

| | |
|---------------------------|--|
| Podium slab thickness | 11-in. (280 mm) |
| Balconies | 9.5-in. (242 mm) |
| Columns | 12x18-in. (306x459 mm) |
| Walls | 8-in. (204 mm) |
| Foundation slab | 10-in. (255 mm) |
| Foundation slab perimeter | 24-in. wide; 18-in. deep (612 by 459 mm) |
| Column base | 60x60-in. on plan; 18-in. deep (1530 mm) |

S.2 MATERIAL PROPERTIES

S.2.1 Concrete

| | |
|--|---------------------------------|
| Weight | 150 psf (24 kN/m ³) |
| A. Podium Slab | |
| Cylinder Strength at 28 days for slabs | = 6000 psi (41.37 MPa) |
| Modulus of Elasticity | = 4,696 ksi (32,379 MPa) |
| Creep Coefficient | = 2 |
| B. Columns and walls | |
| Cylinder Strength at 28 days for slabs | = 5000 psi (34.47 MPa) |
| Modulus of Elasticity | = 4,287 ksi (29,559 MPa) |
| C. Mat foundation | |
| Cylinder Strength at 28 days for slabs | = 5000 psi (34.47 MPa) |
| Modulus of Elasticity | = 4,287 ksi (29,559 MPa) |

S.2.2 Post-Tensioning

| | |
|---|---|
| MATERIAL | |
| Low Relaxation | |
| Strand Diameter | = 0.50 in nominal (13 mm) |
| Strand Area | = 0.153 in ² (99 mm ²) |
| Modulus of Elasticity | = 29007 ksi (199,955 MPa) |
| Guaranteed Ultimate Strength (f_{pu}) | = 270 ksi (1,862 MPa) |
| Average Effective Stress (f_{se}) | = 175 ksi (1,206 MPa) |
| System | unbonded |
| Angular Coefficient of Friction (μ) | = 0.07 |
| Wobble Coefficient of Friction (K) | = 0.001 rad/ft (0.0033 rad/ft) |
| Anchor Set (Wedge Draw-in) | = 0.25 inch (6mm) |

S.2.3 Nonprestressed Reinforcement

| | |
|-----------------------|----------------------------|
| Yield Strength | = 60 ksi (413.7 MPa) |
| Modulus of Elasticity | = 30,000 ksi (206,850 MPa) |

S.2.4 Soil Support

| | |
|--|--------------------------------------|
| Bulk modulus | = 200 pci (0.054 N/mm ³) |
| Friction coefficient between concrete and soil | = 0.4 |

Horizontal spring stiffness
 Allowable pressure for service condition = 2000 psf (157.60 kN/m²)
 Allowable pressure for gravity and lateral forces = 2.600 psf (204.88 kN/m²)

S.3 LOADS

The loads on the podium slab consist of the weight of the superstructure; weight of the slab; and seismic/wind forces transferred from the superstructure to the slab.

The load from the superstructure typically does not line up with the podium slab supports. It is common that the shear walls from the superstructure deliver the overturning moment from the seismic or wind forces as concentrated loads. These loads typically act at the end of the walls. Hence, in addition to the horizontal and vertical forces there are up and down forces at the tip of shear walls terminating over the podium slab.

In addition, the details of the up-and-down forces from the lateral loads of the superstructure are not always known at the time the podium slab is being sized, and goes through the first design stage. Future remodeling, also can change the point of application of the loads from the superstructure.

For the foregoing reasons, to start with it is expedient to design and detail the podium slab for smeared (distributed) loading that represents the actions from the superstructure.

At preparation of the construction drawings, or changes in occupancy, the podium slab is checked against the final position and value of concentrated forces from the superstructure. Should it be necessary, adjustments are made to the overall reinforcement of the podium slab.

S.3.1 Gravity Loads

TABLE S.3.1-1 Smeared Superimposed Load on Podium Slab (T210)

| Description | DL over podium psf (kN/m ²) | LL over podium psf (kN/m ²) | LL reduced psf (kN/m ²) |
|-----------------------|---|---|--|
| Roof | 40 (1.92) | 20* (0.96) | 16* (0.77) |
| 4 th level | 40 (1.92) | 40 (1.92) | 24 (1.15) |
| 3 rd level | 40 (1.92) | 40 (1.92) | 24 ² (1.15) |
| 2 nd level | 40 (1.92) | 40 (1.92) | 24 (1.15) |
| Podium/first | ----- | 40 (1.92) | 40 (1.92) |
| MEP | 5 (0.24) | | |
| Total | 165 (7.90) | 180 (8.62) | 128 (6.13) |

* Roof not accessible for assembly

Balconies: DL from selfweight and cover 125 psf (5.99 kN/m²) ; LL 60 psf (2.87 kN/m²)

Slab thickness 11-in. (279 mm)

Slab selfweight = 11 x 150/12 = 137.5 psf (6.58 kN/m²)

² Reduced using ASCE 7

Design the podium slab for the following superimposed loads

DL = 165 psf (7.90 kN/m²)

LL = 128; assume 130 psf (6.22 kN/m²)

For the balcony:

Selfweight = $(9.5/12)0.15 = 0.118$, assume 0.12 psf (5.75 kN/m²)

LL = 0.06 ksf (2.87 kN/m²)

S.3.2 Lateral Loads

Seismic forces govern the design for lateral loads. These are handled in the Section on Seismic Forces.

The lateral loads are handled in the section on Lateral Design

S.4 STRUCTURAL SYSTEM

The podium slab, walls/columns and the mat foundation are treated as a contiguous concrete box structure.

Collectively, they resist the gravity and the lateral loads on the structure. The concrete box rests on the foundation soil. The foundation soil resists the gravity and lateral loads.

S.4.1 Boundary Conditions; Releases

The connectivity of the members is selected for better prediction of the internal forces, and improved performance of the structure. The assumed load resisting structural system including the connectivity is shown in Fig. S.4.1-1. It includes the following:

- ❖ Top and bottom slabs that act as diaphragm in resisting the seismic forces;
- ❖ Designated shear walls between the top and bottom slabs;
- ❖ Full shear release between the podium slab and supporting non shear walls;
- ❖ Moment release at the connection of the columns to the slab and the mat foundation;
- ❖ Hinged connections between the superstructure to the podium slab.

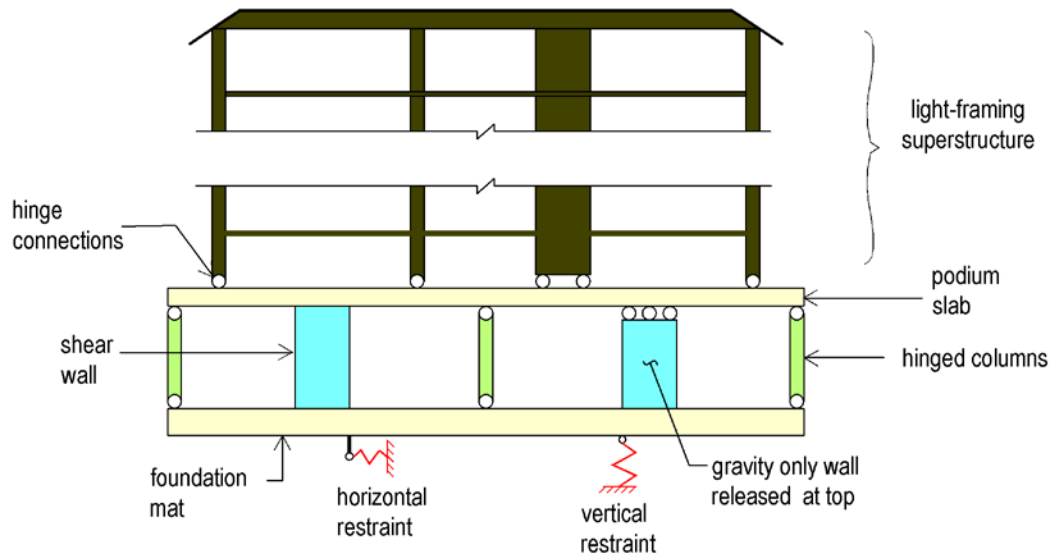


FIGURE S.4.1-1 Elevation; Structural system of the building (PTS938)

S.5. DESIGN PARAMETERS

S.5.1 Applicable Codes

The following codes are referenced in the design of the concrete box.

ACI 318-19
 ASCE7-16
 IBC- 2019
 EC2 -2004

S.5.2 Soil Parameters

The soil report recommends the following parameters

Allowable bearing pressure under dead and live load 2000 psf (95.76 kN/m²)
 Allowable pressure is one third more for transient loads (2660 psf/127.36 kN/m²)
 Friction coefficient between soil and foundation 0.3
 Passive soil pressure 400 pcf (64 kN/m³)

S.5.3 Seismic/Wind Data

For the location and configuration of the building seismic forces governs.

Based on the location and features of the project the seismic base shear coefficient C_s for the concrete box structure was determined to be 0.3

Base shear (V) for the concrete box $V=0.3W$

Where, W is the weight of the structure.

S.5.4 Cover to Reinforcement

A. Corrosion Prevention:

Nonprestressed reinforcement:

Cover to top bars (enclosed areas) = 1.00- in. (25 mm)

Cover to bottom bars (enclosed areas)= 1.00- in. (25 mm)

Prestressed Tendons:

Top cover = 0.75-in. (19 mm)

Bottom cover

Interior spans = 0.75-in. (19 mm)

Exterior spans = 0.75-in. (19 mm)

B. Fire Rating:

2-hr fire rating required. End spans assumed unrestrained for the configuration of the structure. Interior spans assume restrained.

Prestressing tendons

Top cover; all spans = 0.75-in. (19 mm)

Bottom cover; interior spans = 0.75-in. (19 mm)

Exterior spans = 1.75-in. (44 mm)

S.6 PODIUM FLOOR DESIGN OUTLINE

The following describes the steps in design of the podium slab. Section S.7 follow the steps for detailed design.

S.6.1 Design Parts

The design parts are:

- Service
- Safety
- Transfer of Prestressing
- Detailing

S.6.1.1 Service Condition (SLS)

A. Allowable Deflections:

- Allowable immediate live load deflection
 - Immediate deflection from live load not to exceed (span/360)
- Allowable total deflection
 - Allowable total deflection of the finished floor not to exceed span/240.
 - This applies to the visible finished floor.
 - Camber or topping may be used, if necessary.
 - Factor of 2 is used to magnify the immediate deflection to account for the effects of creep and shrinkage, when estimating the long-term deflection³.

B. Allowable Stresses:

Maximum tensile stress

³ ACI 318-14 R24.2.4.1

- Due to prestress plus sustained loads = $6 \sqrt{f_{ci}}$
- Due to prestress plus total loads = $6 \sqrt{f_{ci}}$

Maximum compressive stress

- Due to prestress plus sustained loads = $0.45 f_c$
- Due to prestress plus total loads = $0.60 f_c$

C. Minimum overall reinforcement:

The minimum reinforcement is satisfied through imposition of minimum 125 psi (0.86 MPa) precompression in two orthogonal directions.

S.6.1.2 Strength Condition (ULS)

The following design checks are performed

- A. Bending capacity
- B. Safety against cracking moment
- C. Punching shear

S.6.1.3 Transfer of Post-Tensioning (initial condition)

- A. Load combination
- B. Stress check; rebar addition

S.6.1.4 Construction Detailing

- A. Allocation of rebar for transfer of column moment

S.7 PODIUM SLAB DESIGN DETAILS

S.7.1 Validation of Analysis Model

S.7.1.1 Geometry of Podium Slab and its Supports

The plan of the podium slab showing its thickness, openings and drops at the balconies is shown in Fig. S.1.2-2.

The supports of the podium slab are shown in Fig. S.7.1.1-1

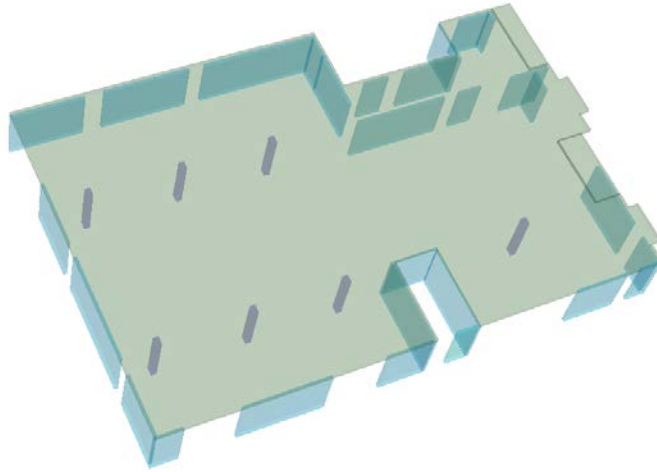


FIGURE S.7.1.1-1 Podium Slab Showing the Gravity Load Bearing Supports
(P1359)

All the supports shown resist vertical loads from gravity. The transfer of shear and moment between the slab and each support depends on the definition of force release at each of the connections.

All the supports shown resist gravity loads. The transfer of shear and moment depends on the connection release between the slab and its support. The type of connection for transfer of load is outlined in S.10.4.2

Next, the model generated for the analysis will be validated by reviewing its deflection under selfweight.

S.7.1.2 Displacement under selfweight

The evaluation of the displacement under selfweight of the podium is used to validate the geometry of the analytical model created for the design of the podium.

A. Discretization: The discretization of the podium slab using a fine quadrilateral finite element mesh is shown in Fig. S.7.1.2-A-1⁴

⁴ ADAPT Builder-Edge www.adaptsoft.com

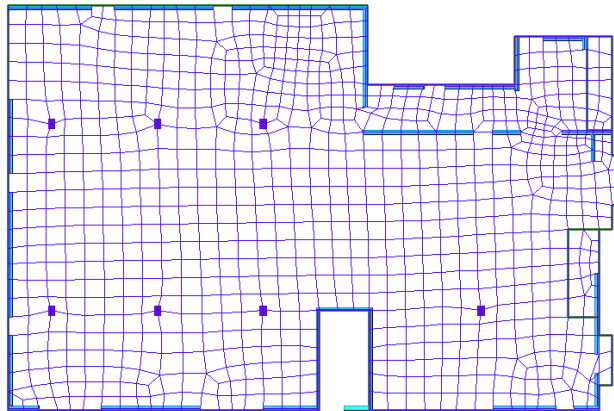


FIGURE 7.1.2.A.1 Discretization of the Podium Slab into Finite Elements for Analysis (P1360)
 Quadrilateral organic cells adjust to the geometrical irregularities of the plan

B. Deflection: The shape of deflection under selfweight is used to visually verify the anticipated qualitative response of the model. The deflected contour verifies the proper modeling of the supports, as well as the overall response of the floor slab under selfweight.

The deflection under selfweight is shown in Fig. 7.1.2.B-1

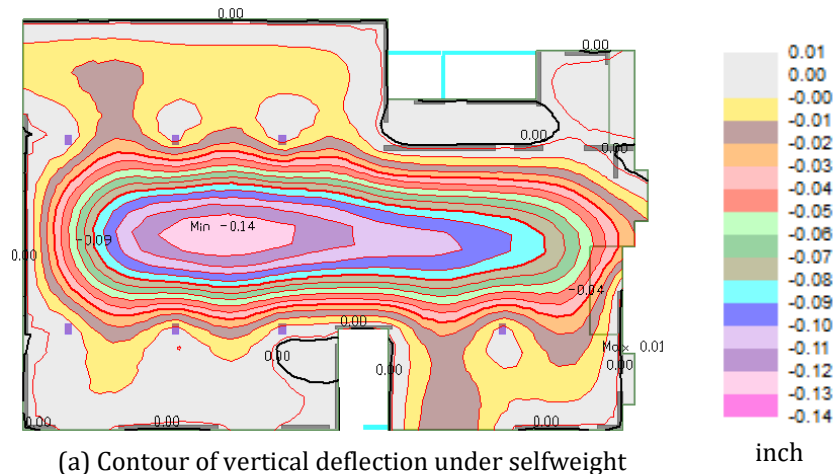


FIGURE 7.1.2.B-1 Downward Deflection of the Podium Slab Under Selfweight (P1410)

S.7.1.3 Validation of Slab Thickness and Concrete Strength

Before continuing with the design of the podium slab, it is important to verify that the selection of slab thickness and concrete strength meet the requirements of the code and can lead to a code-compliant⁵ design.

Live load deflection of the slab shall not exceed span/360⁶

⁵ ACI 318-14

⁶ ACI 318-14 Table 24.2.2

The magnitude of the live load deflection is independent from the amount and distribution of post-tensioning. This is particularly true for the designs based on essentially crack free condition. This is the target condition of ACI 318 for post-tensioned two-way floor systems.

The deflection contour of the podium slab under live load is shown in Fig. S.7.1.3-1

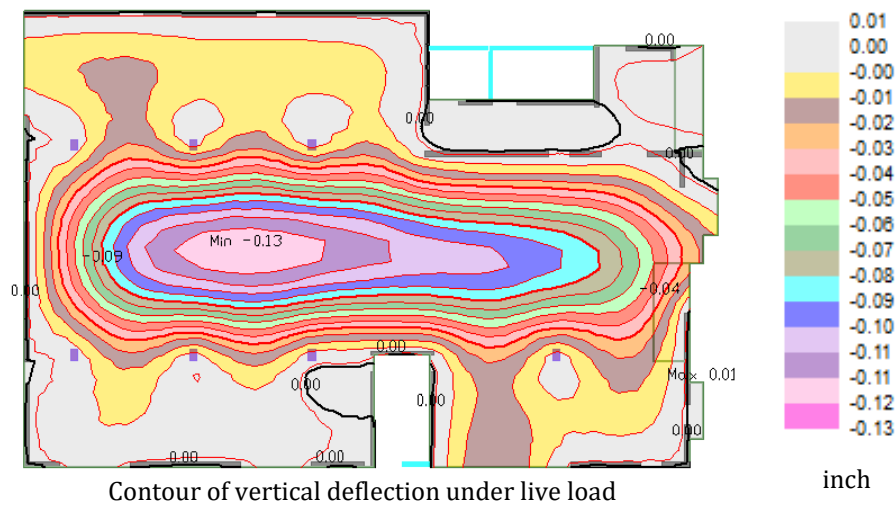


FIGURE S.7.1.3-1 Downward Deflection of the Podium Slab Under Live Load (P1411)

The deflection shown is for the total live load supported by the podium slab.
Maximum deflection 0.13-in. (3 mm)

Max deflection under live load is 0.13-in. (3 mm)
Span in Y-Y direction is 30 ft (9.144 m)
Deflection ratio = $0.13 / (12 \times 30) = 1 / 277 < 1 / 360$ OK

The thickness of slab and the concrete strength are adequate for live load deflection

S.7.2 Design for Gravity Loads

S.7.2.1 Selection of Post-Tensioning: The design is carried out by specifying the layout of the post-tensioning tendons. Depending on the outcome of the design, the layout to initiate the design may be revised.

The assumptions for the initial tendon layout are based on (i) selection of an average precompression, and (ii) mapping the tendons to provide the maximum uplift in critical spans. In the course of design, the tendon profile may have to be modified in non-critical spans to avoid over-balancing.

The maximum uplift is achieved by tendons being at the high point over the supports; and at lowest point in the span. At slab edges, tendons are anchored at slab centroid.

For podium slab of this structure, the optimum average precompression is typically between 200 to 250 psi (1.38 To 1.72 MPa).

Assume 200 psi (1.38 MPa) average precompression

Required force per lineal ft is: $200 \times 11 \times 12 / 1000 = 26.4 \text{ k/ft}$ (305 kN/m)

The effective stress, after all losses, for each post-tensioning tendon is 175 ksi (1,206 MPa)

Force /tendon = $175 \times 0.153 = 26.78 \text{ k}$ (119.10 kN)

Comparing the required force per ft (26.4 k) (305 kN/m) to the force provided by each tendon (26.7k; 119.1 kN), the design is initiated by assuming 1 tendon per ft (306 mm) of the slab width.

S.7.2.2 Arrangement of Post-Tensioning Tendons: Arrange the tendons grouped (banded) in one direction and distributed in the orthogonal direction.

In the distributed direction, place the tendons in bundles of three at 36-in. o.c (918 mm o.c)

In the banded direction select the number of tendons based on the tributary of each tendon band (design strip).

Along gridline 3

Tributary = $0.5(16 + 31) = 23.5 \text{ ft}$; use 24 tendons

Along gridline 2

Tributary = $0.5(20 + 31) = 25.5 \text{ ft}$; use 26 tendons

Along gridlines 1 and 4, the primary function of the tendons is to provide precompression

Use 4 tendons each along the gridlines 1 and 4.

The arrangement of the tendons assumed is shown in Fig. S.7.2.2-1 and S.7.2.2-2.



FIGURE 7.2.2-1 Plan; Arrangement of Tendons (P1357)

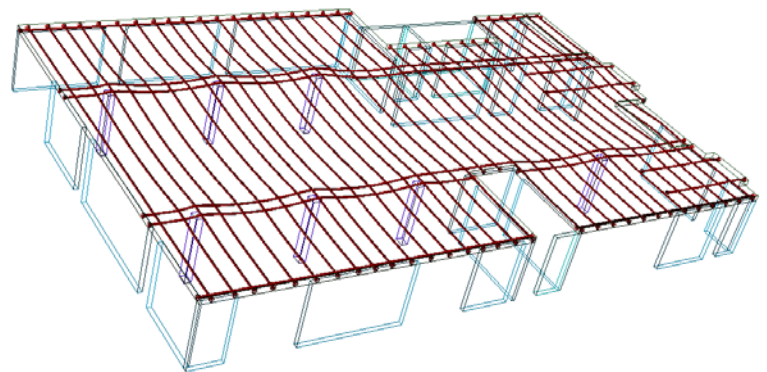


FIGURE 7.2.2-2 3D view of tendon layout (P1364)

Tendons are grouped in one direction (X-direction) and distributed in the orthogonal direction

Typical tendon profile is reversed parabola between adjacent supports.

In the typical situation, the low point is at midspan. The inflection points at one-tenth of the span. At the slab edge, tendons are terminated at mid-depth of the slab.

In non-standard situations, tendon profile is adjusted to accommodate the intent of design for the condition at hand. As an example, Fig. 7.2.2-3A shows the tendon profiles in the distributed direction, where the short end span at one end favors a straight profile reaching to the slab edge. In Figure 7.2.2-3B the tendon profile is straight and at high point over the wall [Aalami, 1999].

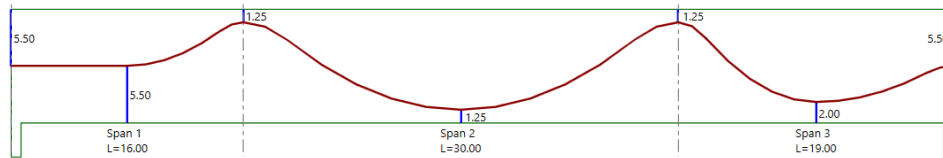


FIGURE 7.2.2-3A. Elevation of a Typical Distributed Tendon Between Gridlines 2 and 4 (P1367)

Numbers show the distance between the slab surface and center of tension. Tendon profile is reversed parabola; inflection point at one-tenth of the span; low point at center; tendon anchored at slab mid-depth

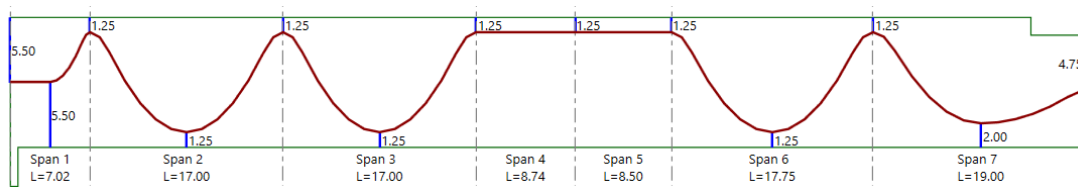


FIGURE 7.2.2-3B Elevation of a Banded Tendon at Gridline D (P1367)

The drop in slab at the far right accommodates the balcony recess. The numbers refer to the distance of tendon's center (CGS) to the top or bottom slab surface. Over the wall the tendon is kept high and straight. Other information, such as friction loss, force along tendon and tendon curvature are not shown for clarity.

S.7.3 Design Strips and Design Sections for Code Check

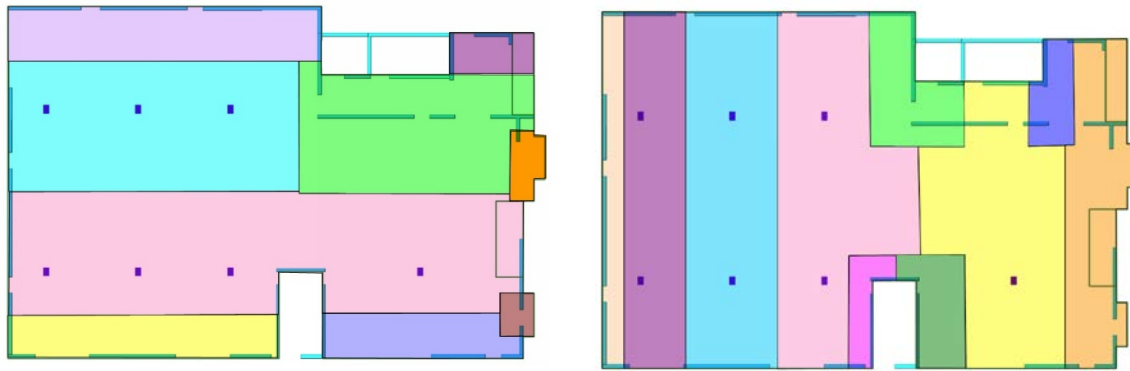
The podium slab is broken down into design strips in each of the two orthogonal directions [Aalami, 2014].

In breakdown of the slab for design it is important that each part of the slab be covered by a design strip.

The design strips can be generated automatically, if specialized software is used⁷; otherwise these are drawn manually.

For the current podium slab the design strips selected are shown in Figs 7.3-1, and 7.3-2.

⁷ ADAPT-Builder www.adaptsoft.com

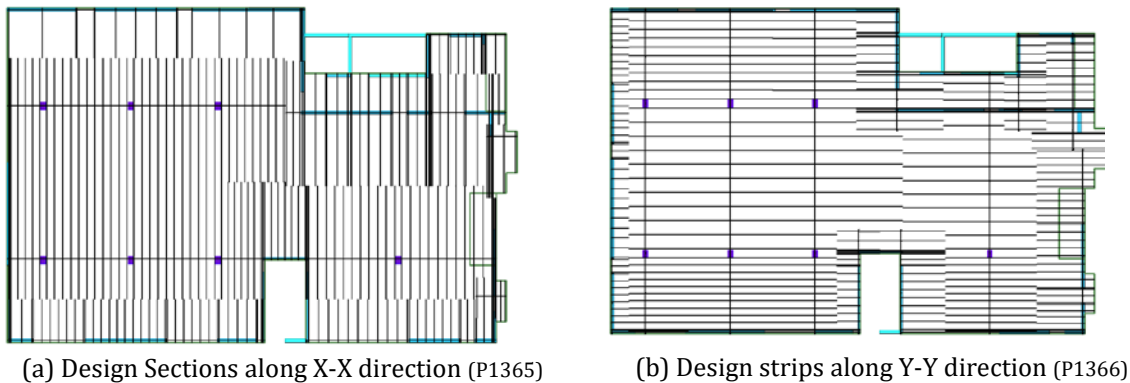


(a) Design strips along X-X direction (P1368) (b) Design strips along Y-Y direction (P1369)

FIGURE 7.3-1 Design Strips Along the two Orthogonal Directions
 Each design strip is associated with a support line along its length (not shown for clarity) Each part of the slab is associated with a design strip

Code compliance check and design for each span is carried out at the face of the support at each span end, and at mid-span, when hand calculation is used. Specialized computer software⁸, in addition to the face of support and midspan, checks the design at multiple locations between the supports – typically 12 to 20 locations. Each design/check location is identified by a cut across the tributary of the design strip.

Figure S.7.3-2 illustrate the selection of design sections in the two orthogonal directions.



(a) Design Sections along X-X direction (P1365) (b) Design strips along Y-Y direction (P1366)

FIGURE S.7.3-2 Design Sections Along the Two Orthogonal Directions
 Design sections for each span are typically at the face of support, at midspan and at several intermediate locations. The forces at each design section are calculated and used for code compliance

S.7.4 Stress Check

After validation of the slab thickness and concrete strength by way of live load deflection, stress check is the next deciding step in the feasibility of a successful design, based on the selected parameters.

⁸ ADAPT-Builder www.adaptsoft.com

For two-way floor slabs, such as the current podium, it is mandatory that the “hypothetical” extreme fiber tensile stresses do not exceed the allowable code values. If they do, the parameters of the design have to be modified to satisfy the mandatory limit on stress values.

S.7.4.1 Load Combinations; hypothetical/Representative Stress: Stress check is carried out for the service condition. For service condition, both ACI 318, and EC2 specify two load combinations, namely “total, or frequent” and “sustained or quasi permanent.” The combinations commonly used for ACI are:

| | | |
|-----------|-----------------------------|-----------------|
| Total | $U = 1.0DL + 1.0LL + 1.0PT$ | (Exp S.7.4.1-1) |
| Sustained | $U = 1.0DL + 0.3LL + 1.0PT$ | (Exp S.7.4.1-2) |

Where, PT stands for forces in the member resulting from prestressing.

The load factors in the preceding combinations are based on common practice. They are not explicitly given in ACI 318.

The extreme fiber hypothetical⁹ stress (f) at a design section is given by:

$$f = (M/Z) + P/A \quad (\text{Exp S.7.4.1-3})$$

Where M is the combined moments from each of the two load combinations; Z is the section modulus; P is the compression acting on the design section; and A is the area of the design section.

S.7.4.2 Bending Stress

A. Bending Stresses: Figure S.7.4.2A-1 illustrates the outcome of the extreme fiber bending stress checks. The diagrams highlight the locations, if any, where the computed stresses exceed the code threshold.

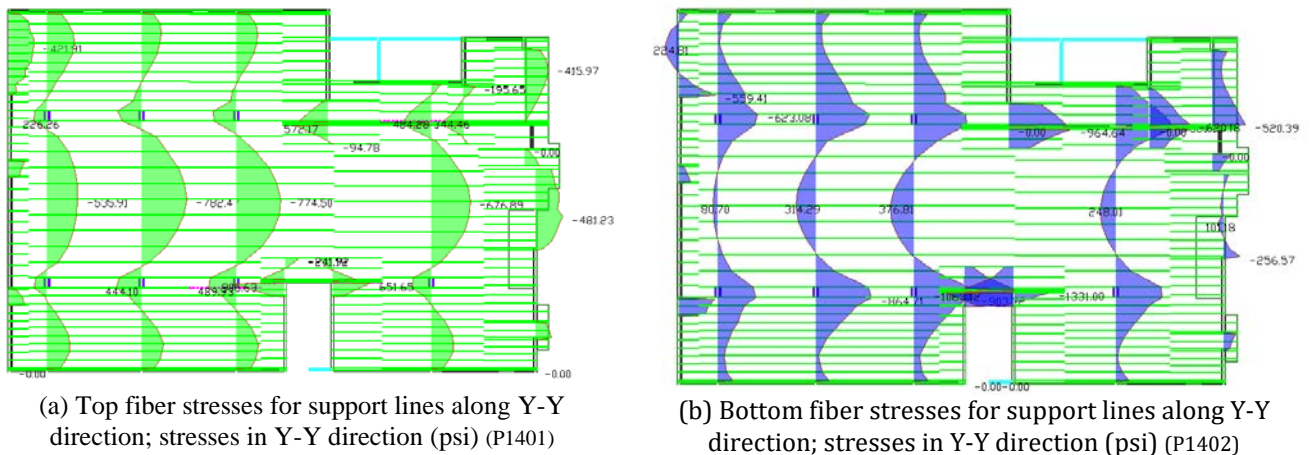


FIGURE 7.4.2A-1 Examples of Top and Bottom Stresses for Design Sections along Y-Y Directions (1 psi = 0.0069 MPa)

Tensile stresses positive; compression negative; allowable tension 464.75 psi (3.20 MPa). The stresses shown are the hypothetical extreme fiber values for each of the

⁹ The values entered in this relationship are those of the entire design section. The extreme fiber calculated does not relate to any specific point of the design section – hence hypothetical

associated design sections. Green lines indicate that the stresses are within the code required values. Red lines, if any, indicate that the computed stress exceeds the allowable value.

B. Precompression: ACI 318 requires a minimum average precompression on a design section for column-supported two-way floor systems. The average precompression (f_p) of each of the design sections is calculated by the total value of the axial force (P) that acts over the cross-sectional area (A) of a design section, divided by the same cross-sectional area.

$$f_p = P/A \quad (\text{Exp S.7.4.2-B1})$$

The calculated along with the minimum required precompression for the design sections in the two directions are shown in Fig. S.7.4.2B-1. The computed precompression meets the minimum required by the code.

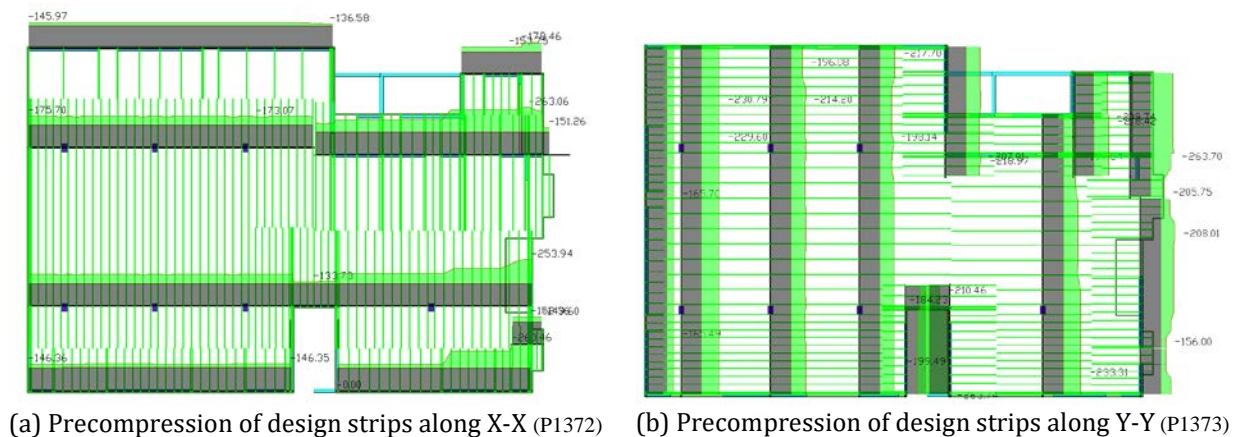


FIGURE S.7.4.2B-1 Distribution of Precompression for Design Sections Along X-X and Y-Y Directions

The green lines (light color) show the computed precompression; the dark regions mark the minimum requirement (125 psi; 0.86 MPa)

From the distributions shown in the preceding, the provided precompression exceeds the minimum required.

S.7.4.3 Minimum Reinforcement

A. Top bars over the support: ACI 318 requires a minimum area of bonded reinforcement over each support in each of the two orthogonal directions. The amount of the bonded reinforcement over each support depends on the local cross-sectional geometry of the floor slab¹⁰. The minimum rebar is calculated and added.

B. Bottom bars in span: Bottom bonded reinforcement in each span is added where the hypothetical extreme fiber tensile stress of the respective span exceeds the permissible value¹¹.

¹⁰ ACI 318-14 Table 8.6.2.3

¹¹ Refer to Section S.11.4.1 for span bottom minimum bars

S.7.5 Deflection Check

Deflections are checked for three conditions.

S.7.5.1. Live load deflection: Live load deflection is independent from the amount and distribution of post-tensioning, since the design is based on gross cross-sectional geometry of the slab.

The value of live load deflection was checked in Section S.7.1.3. It was determined that its value is within the code permissible limit.

S.7.5.2 Long-Term Deflection: The control of long-term deflection is primarily for visual effects and proper functioning of installations on the podium slab. Its limit is set to (span/240).

The load combination for the long-term deflection in the common case is:

$$U = (1 + 2) (D + PT + 0.3L) + 0.7 L < \text{span}/240 \quad (\text{Exp S.7.5.2-1})$$

In this expression “2” is the multiplier for the long-term effects of creep and shrinkage¹². The sustained portion of the live load on the structure is assumed 0.30 of the total design live load.

The significance of the long-term deflection depends on (i) whether it can be observed, and (ii) whether it impacts the function of the installations affected by it. Its evaluation is subject to the finish of the structural slab and the consequence of deflection.

S.7.5.3 Deflection likely to damage non-structural brittle installations: This requires information on the timetable of construction, and details of the construction finish. Typically, no check for this condition is carried out, unless the required information is available.

S.7.6 Podium Slab Vibration

Post-tensioned slabs, generally being thinner than conventionally reinforced slabs of similar configuration, can undergo in-service vibrations that may be perceived by occupants and considered objectionable. This does not apply to podium slabs of the type being designed, however. Podium slabs are (i) generally thicker than single level slabs, and (ii) are subject to large dead load and damping from superstructure. Both result in slab frequencies and damping that cause vibrations beyond the perception range¹³.

Based on the preceding, vibration analysis is not performed.

S.7.7 Podium Slab Strength Check (ULS)

S.7.7.1 Load Combinations: The load combinations for strength check of a post-tensioned member subject to gravity load is:

$$U1 = 1.2D + 1.6L + 1.0HYP \quad (\text{Exp S.7.7.1-1})$$

$$U2 = 1.4D \quad (\text{Exp S.7.7.1-2})$$

Where HYP is the hyperstatic forces from post-tensioning.

¹² ACI 318-14 R24.2.4.1

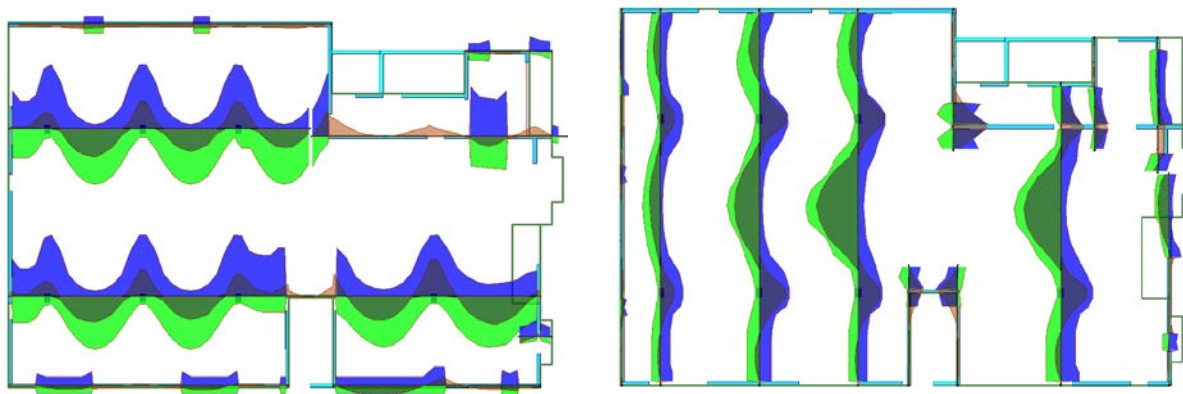
¹³ For vibration evaluation of floor systems refer to TN290

S.7.7.2 Bending Capacity/Demand and Reinforcement: The safety check for strength is carried out at each design section. The demand moment (M_u) is computed and matched against the existing capacity of the same design section (ϕM_n). Where demand exceeds capacity, rebar is added.

The computation of the capacity is based on the available post-tensioning, and additional nonprestressed reinforcement that may have been defined as base reinforcement, or added from the preceding serviceability check.

The predefined “base reinforcement,” can be in form of bottom and/or top mesh or individual bars. None was defined in this project.

Figures S.7.7.2-1 (a) and (b) show the moment demand/capacity of the design sections along in the two principal directions. The demand/capacity display is subsequent to the addition of rebar, where needed.



(a) Moment capacity/demand display for design sections of strips along X-X direction (P1374b)

(b) Moment capacity/demand for design sections of strips along Y-Y direction (P1375b)

FIGURE S.7.7.2-1 Moment demand and Capacity Display along Support Lines in the Principal Directions;

The blue region (upper curve above or to the right of the support line) indicates the negative moment capacity of the support line along the support line length. The green region below the support line (or to the left of the support line) shows the positive (sagging) moment capacity along the support line. The moment demand (ϕM_n) along the support line is marked with the dark fill (between the upper and lower). Over the walls the capacity/demand consideration is not applicable (shown with single fill). For all the design sections displayed, the moment capacity is equal or exceeds moment demand.

S.7.8 Rebar Summary from Design for Gravity Forces

The envelope of the required nonprestressed reinforcement from the serviceability and strength checks of the floor slab is shown in Figs. S.7.8-1 and S.7.8-2.

The rebar plans include the base reinforcement defined by the designer, if any.

The rebar plans at this stage reflect the requirements of the slab to resist the specified gravity loads. The added reinforcement that may be required to resist the lateral loads and structural detailing is handled in the following sections.

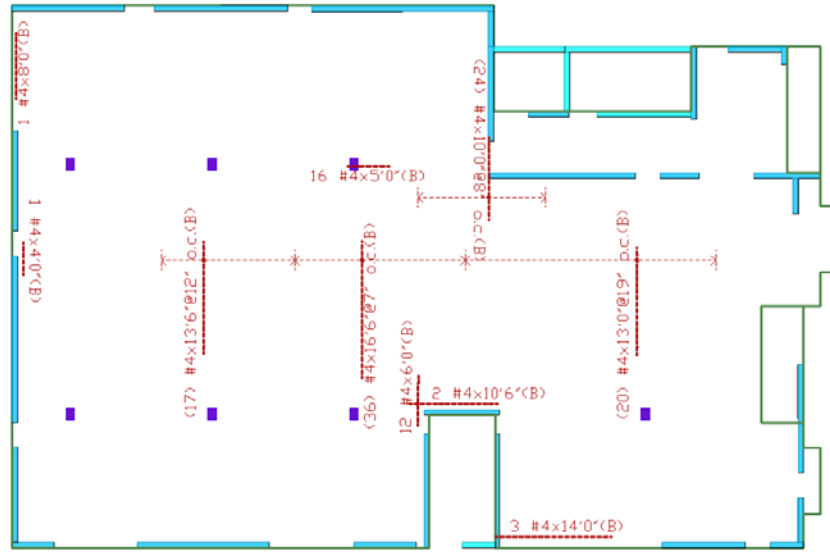
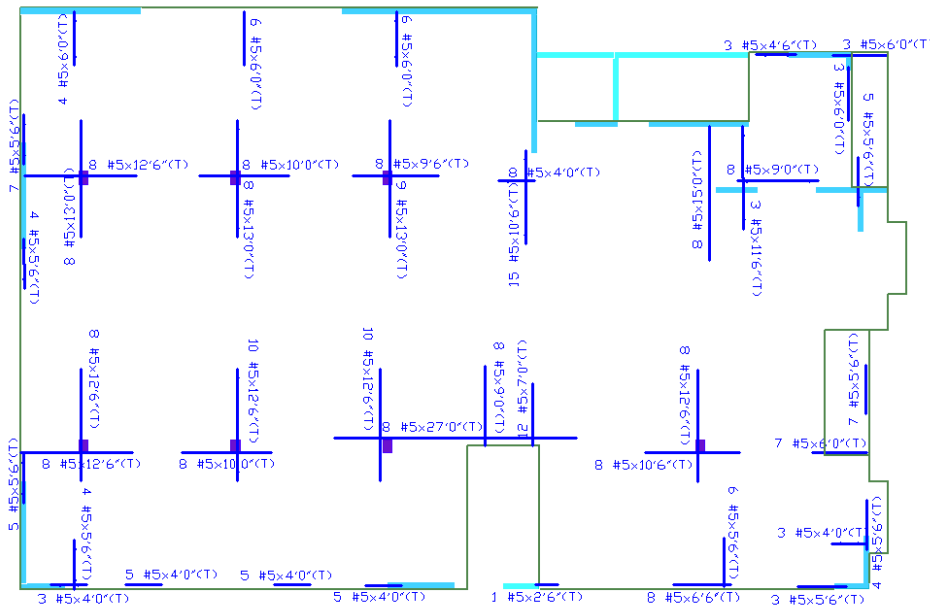


FIGURE S.7.8-1 Bottom Reinforcement Required for Gravity Design (P1376)

The bottom reinforcement shown is in addition to the post-tensioning specified.

(Legend: (17) #4x16' 6" @ 7" o.c is 17 times 12 mm bars each 5.03 m, spaced at 180 mm.)



FLIGURE S.7.8-2 Top Reinforcement Required for Gravity Design (P1377)

The top reinforcement shown is in addition to the post-tensioning specified.

(Legend: (17) #4x16' 6" @ 7" o.c is 17 times 12 mm bars each 5.03 m, spaced at 180 mm.)

S.7.9 Punching Shear Design

The punching shear load combination is the same as the load combination for strength design of the slab (Exp. (Exp S.7.7.1-1 and 2)

The connection of column to the podium slab is modeled as hinged (rotationally free) in the analysis. There is no computed moment transfer between the podium slab and at the top of its supporting columns.

S.7.9.1 Column reactions

The reactions on the columns, and punching shear demand is listed in Tables S.7.9-1.1

TABLE S.7.9.1-1 Column Reactions and Punching Shear Force Demand k (kN) (T200)

| Grid Line | P_D k (kN) | PI k (kN) | Phyp k (kN) | Punching demand V_u k (kN) |
|-----------|-------------------|------------------|-----------------|---------------------------------|
| C/2 | -97.81 (-435.06) | -41.80 (-185.93) | -16.44 (-73.12) | -200.64 (-892.49) |
| C/3 | -150.20 (-668.10) | -64.28 (-285.92) | 8.12 (36.11) | -274.99 (-1223.20) |
| C/4 | -139.99 (-622.72) | -59.94 (-266.61) | -20.95 (-93.20) | -260.15 (-1157.20) |
| D/2 | -90.45 (-402.32) | -38.63 (-171.83) | -20.95 (-93.20) | -191.24 (-850.66) |
| D/3 | -144.40 (-642.33) | -61.83 (-275.02) | 5.66 (25.16) | -266.56 (-1185.70) |
| D/4 | -111.98 (-498.10) | -47.88 (-213.00) | -3.15 (-13.99) | -214.13 (-952.49) |
| D/6 | -149.09 (-663.20) | -63.92 (-284.35) | -8.21 (-36.51) | -289.38 (-1287.20) |

Note: Column compression shown negative

S.7.9-2. Punching shear check: As an example, consider column at gridline D/6.

Computed force from Table S.7.9-1.1 is 289.375 k (1,287 kN)

Computed moment $M_u = 0$

Column dimension 12 x 18-in. (254 x 459 mm)

Slab thickness $h = 11$ -in. (280 mm)

Effective depth $d = 9.38$ -in. (238 mm)

Dimensions of the first critical perimeter at $0.5d$

$c_1 = 12 + 9.38 = 21.38$ -in. (545 mm)

$c_2 = 18 + 9.38 = 27.38$ -in. (698 mm)

Surface of the critical section = $2(21.38 + 27.38) \times 9.38 = 914.73$ in² (5.90×10^5 mm²)

Shear stress at the first critical section

$v_u = V_u/A_c = 289.375 \times 1000 / 914.73 = 316.35$ psi (2.18 MPa)

Allowable concrete stress in shear v_c is 232.38 psi¹⁴ (1.60 MPa)

$V_u = 316.35 > v_c = 232.38$, hence shear reinforcement is required.

The calculated shear reinforcement is shown in Fig. S.7.9-2-1¹⁵

Reinforcement

8 stud rails, each having 6 – 0.75-in. (19 mm) dia studs, spaced at 4.5-in. (115 mm) apart.

¹⁴ See appendix A for the background

¹⁵ Details of the punching shear reinforcement are given in Appendix A

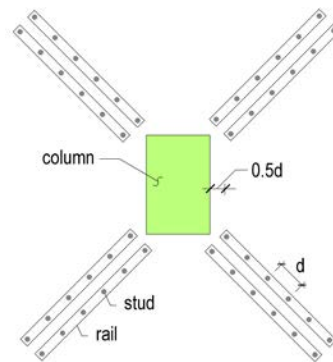


FIGURE S.9.2-1 Punching shear reinforcement (PTS830)
 Shear studs are 0.75-in. (20mm) in diameter; $d=4.5$ -in. (114 mm)

S.7.10 Cracking Moment Safety Check

ACI 318¹⁶ specifically does not require safety check at initiation of cracking for post-tensioned slabs reinforced with unbonded tendons. ACI 318 limits the safety check of this requirement to slabs that are reinforced with bonded tendons. No safety check is carried out¹⁷.

S.7.11 Rebar for Transfer of Unbalanced Moment

The connection of the columns to the slab is assumed hinged. The hinge assumption results in zero moment transfer between the column and the slab. Hence, special reinforcement detailing to account for transfer of column moment does not apply¹⁸.

S.8 COLUMN DESIGN

S.8.1 Column Load Combination

The governing load combination for the columns used is

$$U = 1.2D + 1.6L + 1.0HYP \quad (\text{Exp S.8.1-1})$$

Where, HYP is the hyperstatic reaction from the post-tensioning of the podium slab.

S.8.2 Column Load and Reinforcement

The factored force demand for the columns is the same as that for the punching shear. These are extracted from Table S.7.9.1-1 and listed in Table S.8.2-1 for ease of reference.

Maximum factored column load is from the table -289.375 k (1.287)

Column concrete strength 6000 psi (41.37 MPa)

Using design tables for column reinforcement¹⁹

Use 6#6 bars; $f_y=60$ ksi (413,7 MPa); 2 on the short side and three on the long side.

¹⁶ ACI 318-14 7.6.2.

¹⁷ Refer to TN510 for example of safety check calculation at initiation of cracking

¹⁸ For an example where design for transfer of column moment applies refer to TN511

¹⁹ CRSI Design Handbook; Concrete Reinforcing Steel Institute

$\phi P_n = 495 \text{ k} > 289.375 \text{ k}$; $(2, 202 \text{ kN} > 1,287 \text{ kN})$

use #4 (12 mm) ties at 8" (204 mm) o.c.; change to 4" (102 mm) o.c. (over the top and bottom lengths of the column)

The arrangement of the reinforcement in the column is shown in Fig. S.8.2-1.

TABLE S.8.2-1 Factored Column Loads (T202)

| Column ID | Gridline | Factored Load k (kN) |
|-----------|----------|----------------------|
| 1 | C/2 | -200.64 (-892.49) |
| 2 | C/3 | -274.99 (-1223.20) |
| 3 | C/4 | -260.15 (-1157.20) |
| 4 | D/2 | -191.24 (-850.66) |
| 5 | D/3 | -266.56 (-1185.70) |
| 6 | D/4 | -214.13 (-952.49) |
| 7 | D/6 | -289.38 (-1287.20) |

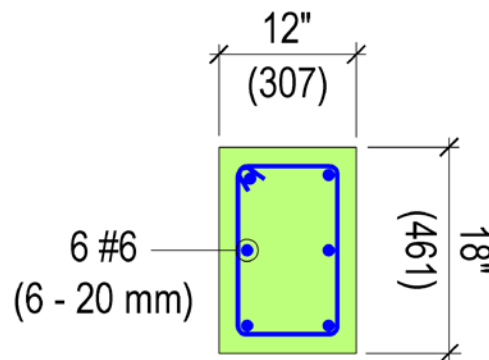


FIGURE S.8.2-1 Column Reinforcement (PTS1399)

Ties #4@8" o.c., (12 mm@ 204 mm o.c.) reduce to 4" o.c. (102 mm) over the up and bottom 24-in. (612 mm) end of the column (12 mm bars @ 200 mm o.c.; reduce to 100 mm spacing over the 610 mm end of the column)

S.9 DESIGN OF GRAVITY WALLS

The gravity walls are all 8-in. (203 mm) thick. The walls are subject to gravity load from the superstructure and podium slab. The lateral loads on the structure are designated to shear walls.

In the computation model of the concrete box, the gravity walls are released at their connection to the podium slab for shear transfer along their longitudinal axis.

S.9.1 Gravity Walls Load Combination

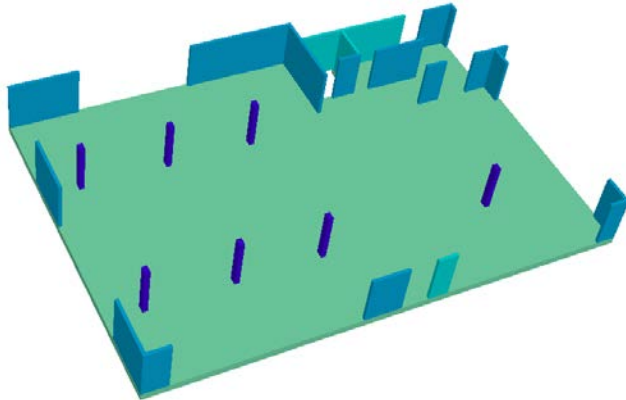
The load combination for the gravity walls is:

$$U = 1.2D + 1.6L + 1.0HYP$$

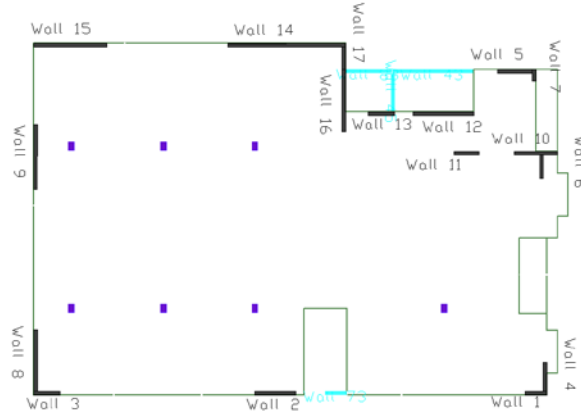
(Exp S.9.1-1)

S.9.2 Gravity Walls Identification

The gravity walls are identified in Fig. S.9.2-1.



(a) Non-shear walls and columns (P1378)



(b) Designation of non-shear walls (P1379)

FIGURE S.9.2-1 Identification of Walls and Columns to be Designed for Gravity Loads Only

S.9.3 Gravity Walls Reinforcement

The factored axial load on the gravity walls and load per unit length of each is listed in Table S.9.3-1

TABLE S.9.3-1 Gravity Walls Factored Loads (T203)

| Wall ID | Axial load P_u k (kN) | Wall length ft (m) |
|---------|-------------------------|--------------------|
| 1 | 1.05 (4.69) | 5.00 (1.52) |
| 2 | 22.90 (101.88) | 7.50 (2.296) |
| 3 | 21.09 (93.830) | 5.00 (1.52) |
| 4 | 11.38 (50.62) | 5.50 (1.68) |
| 5 | 15.30 (68.053) | 6.50 (1.98) |
| 6 | 83.92 (373.29) | 4.30 (1.31) |
| 7 | 18.79 (83.58) | 2.00 (.61) |
| 8 | 19.92 (88.61) | 11.50 (3.50) |
| 9 | 5.16 (22.95) | 12.00 (3.66) |
| 10 | 59.72 (265.63) | 7.50 (2.29) |
| 11 | 128.81 (572.98) | 4.50 (1.37) |
| 12 | -21.29 (-94.72) | -13.50 (-4.12) |
| 13 | -19.27 (-85.74) | -4.50 (-1.37) |
| 14 | 80.69 (358.93) | 24.50 (7.47) |
| 15 | 57.93 (257.69) | 13.50 (4.11) |

| | | |
|----|--------------|--------------|
| 16 | 5.87 (26.12) | 12.50 (3.81) |
|----|--------------|--------------|

Note: Compression is shown positive.

Provide two layers of reinforcement, one on each face, vertical and horizontal
 Use #4@8" o.c. (12 mm @ 200 mm o.c.) 1 each face, each way.
 Maximum force per ft (306 mm) of wall = 13.50 k (58.05 kN)
 Wall concrete $f'_c = 5000$ psi (34.47 MPa)
 Wall capacity by far exceeds demand by far; OK
 By inspection, wall capacity by far exceeds the computed demand.

S.10 DESIGN FOR SEISMIC FORCES

The lateral stiffness of the light frame superstructure is less than 10% of its supporting concrete box. The seismic design of each is handled separately.

The concrete box is assumed to act as foundation of the superstructure for the latter's seismic design.

The base shear from the superstructure is added to the base shear of the concrete box at the level of the podium slab.

Subsequent to the design of the concrete box, the adequacy of the reinforcement in the podium slab is checked for two additional load conditions.

First, the podium slab acts as a diaphragm to distribute the seismic forces it resists or generates among the shear walls that support the podium slab.

Second, the concentrated up and down forces from the seismic frames of the superstructure on the podium slab are resisted by local reinforcement and flexural action of the podium. The localized seismic forces from the superstructure are not intended to call for post-elastic response of the podium slab at the location the loads act.

The podium slab acts elastically in bending to resist the overturning moments of the superstructure. It transfers the base shear of the superstructure to the shear walls through its membrane action.

The lateral design of the concrete box consists of the following steps:

- Estimate the base shear of the superstructure on the podium slab;
- Determine the base shear of the concrete box;
- Add the two base shear parts to arrive at the total base shear of the structure;
- Distribute the total base shear among the shear walls; design the shear walls;
- Check the reinforcement of the podium slab acting as diaphragm for transfer of lateral forces to shear walls;
- Check the adequacy of the podium slab for localized seismic forces it received from the superstructure; and
- Check/design the transfer of seismic forces from the shear walls to the foundation slab.

The applicable load combination for design of the podium slab, shear walls and the transfer of seismic forces to the foundation slab is:

$$U = 1.2\text{Selfweight} + 1.2\text{Dead} + 1.0\text{HYP} + 1.0\text{Live} + 1.0\text{EQ} + 1.0 (5\%\text{EQ_eccentricity}) \quad (\text{Exp S10.1})$$

The inclusion of the live and earthquake components, and the change in sign is selected to maximize the force demand on each of the structural components that resist the seismic forces. The selfweight, dead load and hyperstatic components remain unchanged.

S.10.1 Base Shear of the Superstructure

Average dead weight of the superstructure 165 psf (7.90 kN/m²)

Footprint of superstructure is approximately 6,000 sf (558.15 m²)

Total weight of superstructure $W = 165 \times 6,000/1000 = 990 \text{ k (4,404 kN)}$

Building is located in high risk seismic region (SDC F)

Base shear coefficient of the superstructure is calculated to be $C_s = 0.30$

$$V_{\text{superstructure}} = 0.3W$$

Base shear $V = 0.3 \times 990 = 297 \text{ k}$; assume 300 k (1,334 kN)

S.10.2 Base Shear of the Concrete Box

Base shear coefficient of the concrete box has been determined by separate calculation as $C_s = 0.35$

$$V = 0.35W$$

(i) Podium slab

- Unit weight of 11-in. (280 mm) podium slab= 137.5 psf (6.58 kN/m²)
- Superimposed dead weight = 5.0 psf (0.24 kN/m²)
- Total area of podium slab = 6,280 sf (584.20 m²)
- Weight of the podium slab = $6,280(137.5 + 5)/1000 = 894.90 \text{ k (3,981 kN)}$

Contribution to base shear from podium slab = $0.35 \times 894.90 = 313.21 \text{ k (1,393 kN)}$

(ii) Walls

View of all the walls of the concrete box is shown in Fig. S.7.1.1.

Total length of walls in X-direction 204 ft (62.18 m)

In Y-direction 119 ft (36.27 m)

Total wall length = $204 + 119 = 323 \text{ ft (98.45 m)}$

Wall thickness = 8-in. (204 mm)

Wall height = 10 ft (3.05 m)

Total weight of walls = $323[0.15(8/12)] \times 10 = 323 \text{ k (1,437 kN)}$

50% of the wall weight is considered to contribute to the base shear of the podium slab.

Contribution to base shear from the walls = $0.5 \times 323 \times 0.35 = 56.52 \text{ k (251.4 kN)}$

Total base shear for design of the podium slab and shear walls

$$\begin{aligned} V &= V_{\text{superstructure}} + V_{\text{podium}} + V_{\text{walls}} = \\ &= 300 + 313.21 + 56.52 = 669.73 \text{ k}; \text{ assume } 670 \text{ k (2,980 kN)} \end{aligned}$$

The combined base shear from the superstructure and the podium slab is considered uniformly distributed over the podium slab

Seismic force per unit area of the slab is: $670/6280 = 0.107$ k/sq. ft

S.10.3 Seismic Design forces of shear walls

S.10.3.1 Load Combinations for Seismic Design: The governing load combinations for the force demand on the shear walls is:

$$U1 = 1.2DL + 1.0L + 1.0HYP + 1.0EQ + 1.0EQecc \quad (\text{Exp S.10.3.1-1})$$

Where

HYP = hyperstatic forces from prestressing of the podium floor;

EQ = earthquake force from along X-X or Y-Y direction; and

EQecc = torsion from 5% eccentricity of the earthquake force.

The earthquake force is applied once along X-X direction and once along Y-Y direction. In each instance it is applied once with positive and once with negative sign. Again, in each instance the forces are determined once with positive and once with negative 5% eccentricity of the earthquake force.

S.10.3.2 Application of Seismic Forces: For the purpose of designing the shear walls, the computed base shear is applied to the podium slab. The distribution of the force among the designated shear walls of the box is determined by analysis using commercially available software²⁰.

The base shear is modeled as uniformly distributed force acting over an interior area of the podium slab (Fig. S10.3.2-1). The force is applied in the plane of the podium slab.

The inplane stiffness of the podium slab is many times larger than the stiffness of its vertical supports for horizontal displacement of the concrete box. This rationalizes that for the distribution of seismic forces among the shear walls the point of application of the lateral force on the podium slab is not critical, as long as the resultant of the applied lateral force on the podium is collinear with the base shear. The uniformly loaded patch in Fig. S10.3.2-1 is identified selected to serve the purpose.

²⁰ ADAPT Builder-Edge www.adaptsoft.com

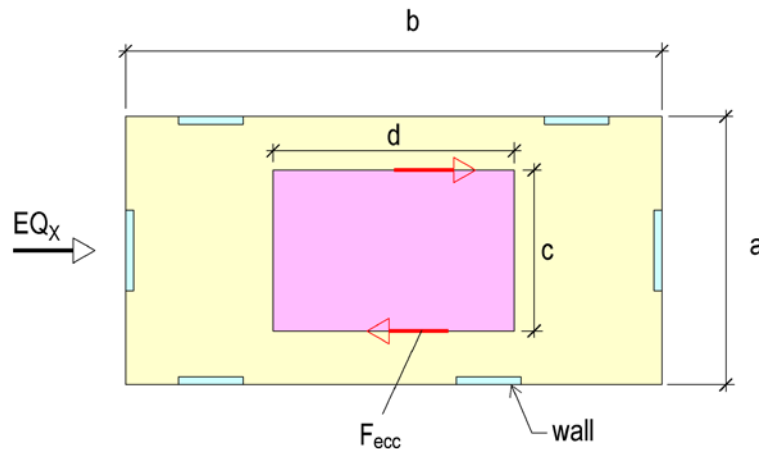


FIGURE S10.3.2-1 Plan; Idealized Representation for Application of Lateral Force and its Eccentricity to the Podium Slab. (PTS912)
 The earthquake force (EQ_x) is applied over the central region (c, d). The eccentricity of EQ_x is simulated by the couple $F_{ecc} c$

The moment due to the accidental eccentricity between the center of mass and center of stiffness is modeled through the couple F for the seismic force along X-X direction. For the eccentricity in the Y-direction a similar couple in the Y-Y direction is selected.

A. Application of EQ_x ; EQ_y : The dimensions of the central load patch selected for the application of seismic forces (Fig. S10.3.2-1) are: 55x25 ft (16.76x7.72 m)

The distributed force over the selected patch is:

$$EQ_x = EQ_y = 670 / (55 \times 25) = 0.487 \text{ ksf (23.28 kN/m}^2\text{)}$$

B. Application of Accidental Eccentricity: The overall dimensions of the structure are (Fig. S10.3.2-1 and Fig. S.1.2-2):

$$a = 65 \text{ ft (19.81 m)}$$

$$b = 97 \text{ ft (29.57)}$$

$$\text{5\% eccentricity for } EQ_x \text{ is: } 0.05 \times 65 = 3.25 \text{ ft (0.99 m)}$$

$$\text{Eccentricity moment} = 670 \times 3.25 = 2,177.5 \text{ k-ft}$$

This moment is represented by the couple F with arm $c = 25$ ft (7.62 m)

$$F = 2,177.5 / 25 = 87.10 \text{ k (387.4 kN)}$$

Similarly, the eccentricity of the seismic force in the Y-direction, and the force of the couple is:

$$\text{5\% eccentricity for } EQ_y = 0.05 \times 97 = 4.85 \text{ ft (1.48 m)}$$

$$\text{Eccentricity moment} = 670 \times 4.85 = 3,249.50 \text{ kft}$$

$$F = 3249.50 / 55 = 59.08 \text{ k (262.80 kN)}$$

S.10.3.3 Shear Wall Design

A: Force on Shear Walls: Using the loads and the load combinations outlined in the preceding sections, the concrete box is analyzed. The force values and the factored design forces for each of the shear walls determined from the analysis is listed in Tables S.10.3.3-1 and 2.

TABLE S.10.3.3.A-1 Shear from seismic forces at the top of shear walls (T204)²¹

| Shear wall | Length ft (m) | EQ-XX Shear in X-XEQ k (kN) | EQ-YY Shear in Y-Y k (kN) | Shear intensity k/ft (kN/m) |
|------------|------------------|-----------------------------------|---------------------------------|--------------------------------|
| SWL1 | 10.00 (3.05) | 71.00 (315.8) | 0 | 7.10 (103.62) |
| SWL2 | 15.87 (4.84) | 144.00 (640.5) | 0 | 9.07 (132.37) |
| SWL3 | 9.00 (2.74) | 64.00 (284.68) | 0 | 7.11 (103.76) |
| SWL4 | 17.70 (5.39) | 253.00 (1125.39) | 0 | 14.29 (208.55) |
| SWL5 | 16.00 (4.88) | 141.00 (627.20) | 0 | 8.81 (128.57) |
| SWL6 | 13.00 (3.96) | 0 | 110.00 (489.30) | 8.46 (123.46) |
| SWL7 | 13.62 (4.15) | 0 | 136.00 (604.95) | 9.98 (145.65) |
| SWL8 | 13.62 (4.15) | 0 | 150.00 (667.23) | 11.01 (160.68) |
| SWL9 | 8.66 (2.64) | 0 | 67.00 (298.03) | 7.73 (112.81) |
| SWL10 | 20.00 (6.10) | 0 | 210.00 (934.12) | 10.50 (153.24) |
| Total | 137.47 (41.90) | 673.00 (2993.64) | 673.00 (2993.64) | |

Note: The values shown in the table are from the load cases EQ in X- or Y-direction. They are intended to verify the equilibrium of the solution. Values used in design account for load cases that include the 5% eccentricities of the force.

Shear intensity: shear force transferred per unit length of wall.

The following illustrates the design of SWL1.

Applicable load combination:

$$U1 = 1.2DL + 1.0L + 1.0HYP + 1.0EQ + 1.0EQecc \quad (\text{Exp S.10.3.3.A-1-1})$$

From the analysis, the following values are obtained at the top of SWL1:

$$DL = 44.00 \text{ k (195.70 kN)}$$

$$LL = 17.83 \text{ k (79.31 kN)}$$

$$HYP = -0.52 \text{ k (2.31 kN)}$$

$$EQX \text{ with 5\% eccentricity} = 107.15 \text{ k}^{22} \text{ (476.62 kN)}$$

$$\text{Shear wall length; height; width} = 10 \text{ ft; } 10 \text{ ft; } 8\text{-in. (3.05 m; 3.05 m; 204 mm)}$$

$$\text{Shear wall weight} = 10 \times 10 \times (8/12) \times 0.15 = 10 \text{ k (44.50 kN)}$$

Actions at the bottom of the shear wall

$$\text{Total dead load} = (44 + 10) = 54 \text{ k (240.20 kN)}$$

²¹ Total shear force applied for the analysis was 673 kips

²² The eccentricity case results in significant shear demand on the wall in this case

$$U1 \text{ (axial)} = 1.2 \times 54.00 + 1.0 \times 17.83 - 0.52 = 82.11 \text{ k (365.24 kN)}$$

$$U1 \text{ (horizontal)} = 107.15 \text{ k (476.62 kN)}$$

Design values at the bottom of the shear wall
 Axial force 82.11 k compression
 Shear 107.15 k (476.62 kN)
 Moment = $10 \times 107.15 = 1,071.50$ kft

B: Reinforcement of Shear Walls: With the dimensions, material and loads known, often commercially available software is used to determine the reinforcement of each shear wall.

Alternatively, the following algorithm conservatively determine the adequacy of the shear walls for specified reinforcement²³.

Concrete strength $f_c = 5000$ psi (34.47 MPa)
 Wall thickness = 8-in. (204 mm)
 Use base reinforcement consisting of a single curtain #5@12" o.c. (16 mm @ 300 mm o.c.) in each direction.
 Estimate the capacity of the wall; compare it with demand; add reinforcement, where required.

For 12-in. (306 mm) length of wall, the capacity in shear is:

$$\Phi V_n = \Phi V_c + \Phi V_s$$

$$\Phi V_c = 0.75 * 2 * f_c^{0.5} * b_w * 12 * 1$$

$$\Phi V_c = 0.75 * 2 * 5000^{0.5} * 8 * 12 * 1 / 1000 = 10.18 \text{ k/ft}$$

$$\Phi V_s = 0.75 * \rho * f_y * b_w * 12$$

$$\rho = 0.31 / (12 * 8) = 0.0032$$

$$\Phi V_s = 0.75 * 0.0032 * 60 * 8 * 12 = 13.82 \text{ k/ft}$$

$$\Phi V_n = 10.2 + 13.82 = 24.0 \text{ k/ft}$$

Assuming effective shear depth $0.8 \times 10.00 = 8.00$ ft (2.44 m)
 Shear demand/ft = $107.17 / 8 = 13.39$ k/ft < 24.0 k/ft OK

For moment capacity, use commercially available software, or the following conservative approximation:
 (i) assume the moment arm (z) to be 0.80h, where h is the depth of the section, namely 10 ft (3.05 m);
 (ii) disregard the contribution of axial compression in reducing the bending reinforcement.

Applied moment $M = 1,071.70$ kft
 $\phi = 0.9$

$$A_s = M_u / (\phi \times f_y \times z)$$

$A_s = 1,071.70 \times 12 / (0.90 \times 0.8 \times 10 \times 12 \times 60) = 2.480$ in²
 Provide 2#7 and 2#8 vertical bars at each end of the shear wall;
 $A_{sprov} = 2 \times 0.6 + 2 \times 0.79 = 2.78 > 2.48$ in² OK

Check for ductility, namely limitation on depth of compression zone "c"
 Compression c

²³ Boundary elements not required; ACI 318-14 18.10.6.2(a)

$$\mu = \phi(0.85af'c)z$$

$$c = a/0.85$$

$$c = \mu/0.85^2 f'c z$$

$$c = 1,071.70 \times 12 / (0.85^2 \times 5 \times 8 \times 0.8 \times 10 \times 12) = 5.562 \text{ in. (142 mm)}$$

$$c/h = 5.562 / (10 \times 12) = 0.05 < 0.35 \text{ OK}^{24}$$

Rebar Requirement

Provide shear reinforcement one layer #5@12" on center in each direction; and jamb reinforcement 2#7 and 2#8 at each end

S.10.4 Diaphragm Design of Podium Slab

The traditional computation of drag and chord forces, followed by design for reinforcement is now superseded/supplemented by the emergence of specialized²⁵ software.

The contemporary software technology can report the capacity of force transfer from the diaphragm to each shear wall based on the available reinforcement. The shortfall can be added. Alternatively, the following conservative method can be used.

The connection of the shear wall to the slab is by way of dowels. The function of the dowels and the required amount is given in Section S.10.4.2

In the current case, the project is assumed to be in the highest seismic zone (SDC F). The podium slab acts as a diaphragm. The slab delivers the seismic forces to the designated seismic force resisting members of its supports through its in-plane (membrane) action.

At this stage the reinforcement from the gravity design of the podium slab is fully defined. Base reinforcement, if any, is also specified. The force demand in the slab from the combination of gravity and seismic event is also known.

S.10.4.1 Diaphragm Action of the Podium Slab

Seismic forces, in addition to other considerations, result in bending of the slab in its plane about an axis normal to the plane. This depending on the support configuration of the slab leads to tensile forces at one edge of the slab and compression at the opposite edge. The forces are commonly referred to as "chord" forces.

The chord and drag forces are additive to those from gravity. If reinforcement provided thus far from other considerations is not adequate to safely resist the added forces from inplane flexure of the slab, more reinforcement for chord forces have to be added. This is explained in greater detail elsewhere²⁶.

S.10.4.2 Transfer of seismic forces to the shear walls

Transfer of seismic forces to the shear wall takes place through the diaphragm action of the wall. The force transfer takes place through (i) reinforcement extending through the end of the wall – referred to as "drag" forces, and (ii) shear through the sides of the wall.

²⁴ ACI 318-11 9.3.2

²⁵ ADAPT Builder; www.adaptsoft.com

²⁶ Refer to TN469

Where the amount of shear transfer through the sides of the wall is not adequate to resist the seismic demand, drag bars are added.

Provide 2#5 bars dowels at 12-in. (2-16 mm bars @ 300 mm) spacing between the shear wall and the slab.

Allowable shear transfer per ft is²⁷:

$$\phi V_n = \phi \mu A_v f_y$$

$$\phi V_n = 0.75 \times 1 \times 2 \times 0.31 \times 60 = 27.9 \text{ k/ft (kN/m)}$$

Shear demand per ft from Table S.10.3.3.A-1 is 11.01 k/ft < 29.9 k/ft OK

Added reinforcement not required.

S.11 FOUNDATION MAT DESIGN

The design of the foundation mat consists of:

- Checking the soil pressure and stability of the mat under the combined and the combined gravity and lateral loads
- Checking the strength of the mat under the gravity and the combined gravity and lateral loads

S.11.1 Geometry and Structural System

The foundation mat consists of 10-in. (260 mm) slab below the entire footprint of the structure (Fig. S.11.1-1). The perimeter of the mat is thickened to 18-in. (457 mm) over 24-in. (614 mm) width. Below the columns, the mat is thickened to 18-in. (457 mm) over a 5x5 ft (1.52x1.52 m) square pad.

Elevator pit at one side of the building interrupts the continuity of the mat

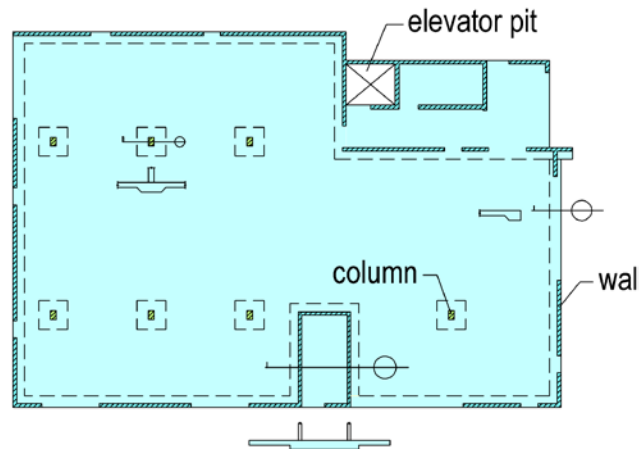


FIGURE 11.1-1 Plan; Geometry of Foundation Mat (PTS943)
The mat slab is thickened around the boundary, below the columns and below the shear walls

²⁷ ACI 318-14 22.9

S.11.2 Loads

The loads on the mat consist of

- Selfweight of the mat
- Distributed live load on the mat; assumed 50 psf (2.39 kN/m²) for parking
- Gravity from the load bearing walls
- Gravity loads from the columns
- Gravity, shear, and moment from shear walls, including horizontal forces from seismic event

Apart from the distributed live load on the mat, the remainder of the loads listed is derived from the preceding analysis/design of the concrete box. View of the structural members that load the mat are shown in Fig. S.11.2-1.

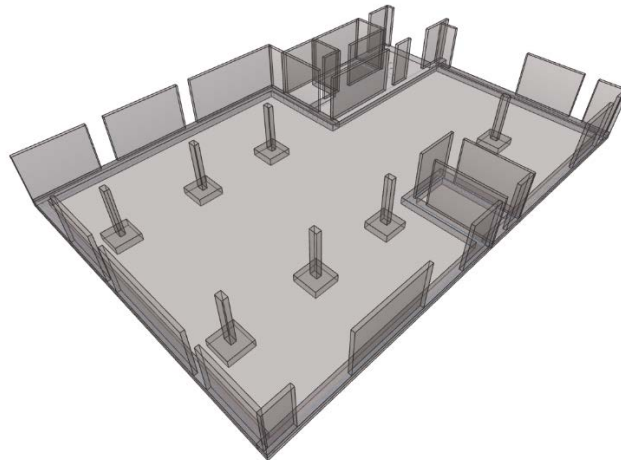


FIGURE S.11.2-1 Foundation mat showing the structural members that transfer load to it (P1385-2)

S.11.3 Soil Pressure

The distribution of soil pressure is determined by analyzing the mat as an elastic thin shell on vertical and horizontal springs. Local increases in mat stiffness from the presence of the walls and columns that are integrally constructed with the mat are included in the mat analysis²⁸.

The compliance of the soil pressure with the allowable value is established by comparing the “computed design” pressure with the permissible value.

Foundation soil does not fail at a “point.” Nor, does its characteristic values, such as bulk modulus, refer to soil response at a “point.” Average pressure over a finite surface area of the soil - referred to as design pressure - provides the entry information to predict the soil response subjected to design loads.

The “design area” is also the function of the stiffness of the member that delivers the pressure to the soil. The stiffer the foundation mat, the larger the design area. Figure S.11.3-1 illustrates the design width for the concrete foundations.

²⁸ ADAPT EDGE www.adaptsoft.com

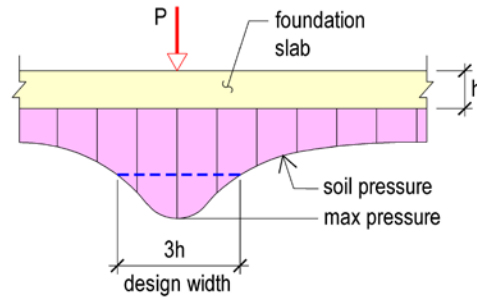


FIGURE S.11.3-1 Definition of Design Width for Compliance with Allowable Soil Pressure (P939)

A. Soil Pressure from Gravity Loads: The distribution of soil pressure under gravity loads is shown in Figs. S.11.3.A-1

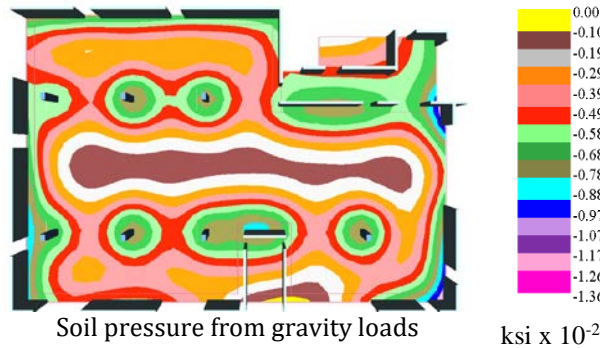


FIGURE S.11.3.A-1 Distribution of Soil Pressure from Gravity Loads (P1386)
 Maximum pressure at a “point” is 1.36×10^{-2} (1.96 k/sf). Allowable pressure is 2 k/sf (95.76 kN/m²) over a “design section.” (1 ksi = 6.895 MPa)

From the analysis results shown above, the design of the foundation mat meets the soil pressure limit set for the gravity load condition

B. Soil Pressure from Combination of Gravity and Seismic Forces: For the configuration of the concrete box, the governing design pressure of the soil occurs for the seismic event in Y- direction with 5% eccentricity.

The applicable load combination is:

$$U = 1.0 (DL + LL + Hyp + EQ + 0.05EQ) \quad (\text{Exp S.11.3.A-1-1})$$

The inclusion of HYP from the post-tensioning of the podium slab is based on the fact that the foundation is an integral part of the entire concrete box. Apart from the DL and LL of the podium slab, the podium slab is loaded with post-tensioning forces. The forces from the post-tensioning act on the mat in the same manner that dead and live loads from the podium slab do.

The distribution of soil pressure under gravity and seismic forces for the preceding load combination is shown in Fig. S.11.3.B-1.

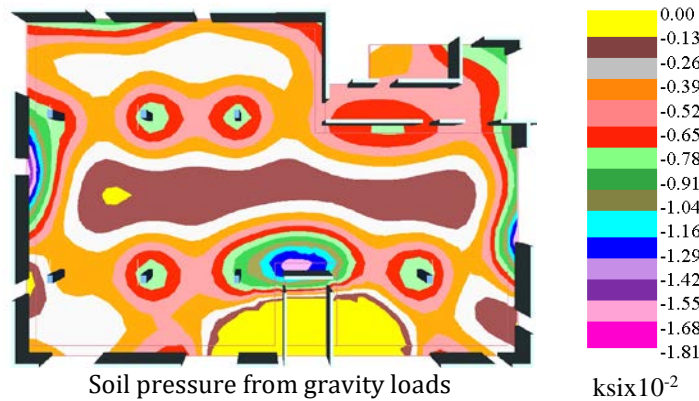


FIGURE S.11.3.B-1 Distribution of soil pressure under the combined action of gravity and lateral loads – Load case EQY (P1394)
 Maximum pressure below the tip of a shear wall is 1.81×10^{-2} ksi (2.60 k/sf)
 (1 ksi = 6.895 MPa)

The allowable pressure for the combined gravity and lateral forces is 2,600 k/sf. The pressure over the “design width” will be less than the value reported at a “point.”

The design of the foundation mat meets the limits on soil pressure.

C. Stability of the Structure Under Seismic Forces: The total horizontal force on the concrete box from the combination of gravity and lateral loads is:

$V = 670 \text{ k (2,980.29 kN)}$

The horizontal force is resisted by the (i) friction between the mat foundation and the soil, (ii) passive pressure of soil on the 18-in deep perimeter beam of the foundation mat, and (iii) the extension of the column footing below the mat proper.

Total area of the mat foundation = 6,280 sf
 Assume conservatively uniform 10-in. (254 mm) mat thickness

| | |
|---|---------------------------|
| Total weight of the super structure = 990 k (4,403.72 kN) | |
| Weight of the podium slab | 313.21 k (1,393.22 kN) |
| Total weight of the walls | 323 k (1,436.77 kN) |
| Weight of mat = $6,280 \times (10/12) \times 0.15 =$ | 785 k (3,491.84 kN) |
| ----- | |
| Total weight ²⁹ | 2,411.21 k (10,725.54 kN) |

Consider conservatively friction coefficient between the mat and underlain soil as 0.35
 Horizontal force required to dislocate the building by overcoming the friction is:

²⁹ Weight of the columns conservatively disregarded

$F = 0.35 \times 2,411.21 = 843.92 \text{ k (3,753.92 kN)} >$ base shear of the structure at the level of podium slab (OK)

In addition, passive pressure from the 18-in. (457 mm) perimeter thickening of the slab and thickening below the columns provide additional resistance.

S.11.4 Strength Check of the Foundation Mat

S.11.4.1 Minimum Requirements and Base Reinforcement

ACI 318 stipulates a minimum amount of reinforcement to be provided in each of the principal directions of the mat. The minimum is 0.0014 times the cross-sectional area of the mat³⁰.

$$A_{smin} = 10 \times 12 \times 0.0014 = 0.168 \text{ in}^2/\text{ft}$$

In addition to the minimum required reinforcement which covers the entire area of the mat, for expediency in construction, it is common in construction to provide the entire area of the mat with a uniform mesh of top and bottom base reinforcement.

The design proceeds by checking the reinforcement requirements of the mat at critical locations with the mesh base reinforcement. The shortfall in reinforcement is added locally.

Provide as base reinforcement #4@14" o.c. top and bottom, each way.

Reinforcement ratio provided is:

$$A_s = (2 \times 0.2 \times 12 / 14) / (10 \times 12) = 0.0029 > 0.0014 \text{ OK}$$

Below the columns, and at the end and below the shear walls reinforcement is often required in addition to the minimum required.

The reinforcement added to the mat's thickening along its perimeter and tip of the shear walls is shown in Figs. (Fig. S.11.4.1-1 and 2).

At each location, the required amount is determined by analysis. For ease of construction, however, the same detail is used for all similar conditions.

The following sections verify the adequacy or determine the shortfall of the specified base reinforcement.

³⁰ ACI 318 8.6.1-1

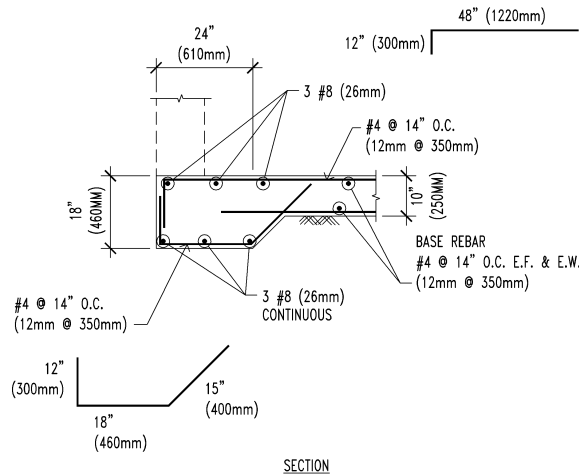


FIGURE S.14.4.1-1 Specified Base Reinforcement at the perimeter thickening of the mat foundation (DET100).

Added reinforcement, where required by analysis is shown on plans

The base reinforcement of the interior shear walls is shown in Fig. S.11.4.1-2.

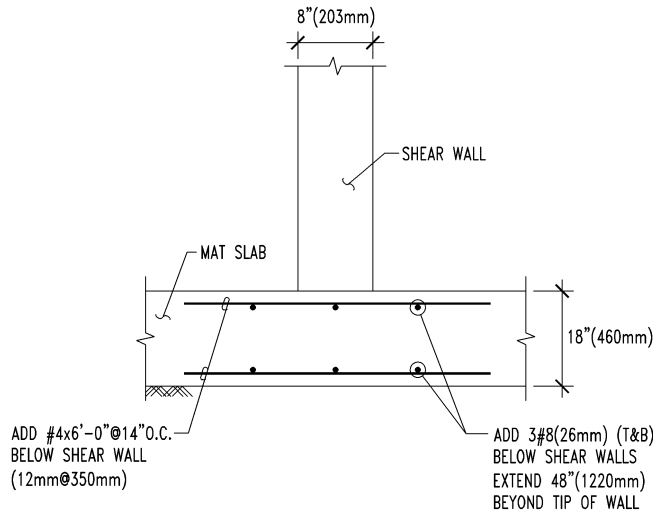


FIGURE S.11.4.1-2 Specified Base Reinforcement below the interior shear walls (DET101)

Added reinforcement, where required by analysis is shown on plans

S.11.4.2 Capacity/Demand Check and Design

The governing load combinations for the strength check of the mat foundation are

$$U1 = 1.2DL + 1.6LL + 1.0HYP \quad \text{(Exp S.11.4.2-1)}$$

$$U2 = 1.2DL + 1.0L + 1.0HYP + 1.0EQ + 1.0EQecc \quad \text{(Exp S.11.4.2-2)}$$

The above is used in several combinations accounting for the change in sign of the lateral loads, and inclusion/exclusion of seismic force eccentricities³¹.

In the general case, for reinforcement verification, the mat foundation can be broken down into design strips and design sections similar to the podium slab (Fig. S.7.3-3). However, having already defined the base reinforcement from other considerations, it is expeditious to verify the adequacy of the reinforcement at typical locations. Add rebar, where required. This is achieved by making design sections at critical locations to check the adequacy of reinforcement.

The design sections selected are shown in Fig. S.11.4.2-1. The design sections include the tip of the shear walls; face of the column, and interior of the mat.

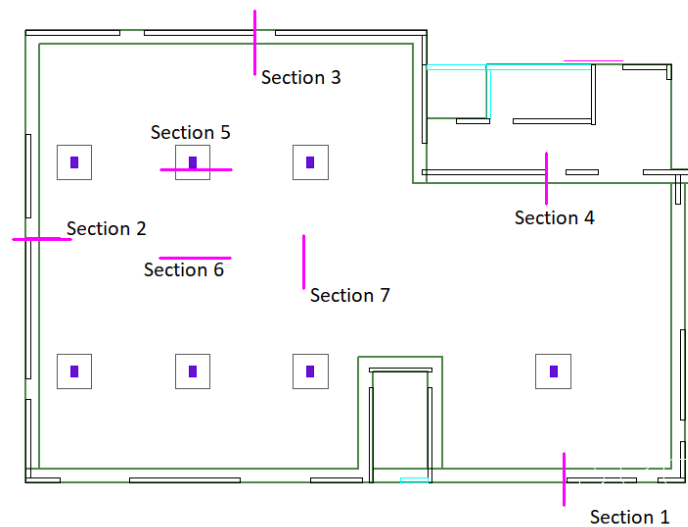


FIGURE S.11.4.2-1 Plan; Mat Foundation Showing the Location of Design Sections Selected to Verify Mat's Strength Adequacy (P1398)
At each design section the capacity of the section is calculated and compared with the demand values

The computed force demand, along with the available moment capacity of each of the design sections is listed in Table S.11.4.2-1.

The values from the Table S.11.4.2-1 confirm that the base reinforcement provided exceeds the force demand.

³¹ In this instance, since the mat is not post-tensioned, in the load combination given the parameter HYP can be replaced by PT, where PT refers to the computed forces in the mat from the application of post-tensioning in the podium slab.

TABLE S.11.4.2-1: Moment capacity/demand values for the design sections of the mat foundation (T209)

| Design section | Moment capacity k-ft (kN-m) | | Moment demand k-ft (kN-m) | |
|----------------|-----------------------------|-------------------|---------------------------|-------------------|
| | Positive moment | Negative moment | Positive moment | Negative moment |
| 1 | 179.37 (243.19) | -211.31 (-286.49) | 86.52 (117.30) | -31.81 (-43.13) |
| 2 | 190.88 (258.79) | -256.38 (-347.60) | 0 | -108.94 (-147.70) |
| 3 | 192.58 (261.10) | -255.49 (-346.39) | 0 | -90.20 (-122.29) |
| 4 | 242.36 (328.59) | -285.21 (-386.67) | 119.36 (161.83) | -66.94 (-90.76) |
| 5 | 272.38 (369.29) | -221.05 (-299.67) | 139.00 (188.46) | 0 |
| 6 | 85.19 (115.50) | -85.19 (-115.50) | 0 | -15.31 (-20.76) |
| 7 | 59.93 (81.25) | -69.54 (-94.28) | 0 | -1.21 (-1.64) |

S.11.4.3 Shear Wall Foundations

The strength check of the shear walls foundations is carried out by evaluating the available strength of the design sections at the tip of each of the shear walls against the force demand from the analysis. Figure S.11.4.2-1 identifies the position of the design sections at the tip of selected shear walls. Specifically, design sections marked 1, 2 and 3 are at the tip of the shear walls viewed more critical. The analysis of these sections, using the base reinforcement provided, concludes that the provided capacity exceeds the force demand (Table S.11.4.2-1).

S.11.4.4 Column Foundations

Below each column, the foundation mat is thickened to a total of 18-in. (459 mm) over a 6 by 6 ft (1.83 by 1.83 m) pad. In addition, 10#5 bars are added in each direction at the bottom of the pad.

The design section marked 5 in Fig. S.11.4.2-1 is used to verify the moment capacity of a typical section through the mat at the face of the column. The outcome reported in Table S.11.4.2-1 confirms that the provided capacity exceeds the force demand. Specifically, for the design section at the face of the column the values are:

Moment demand = 139.00 k-ft

Moment capacity = 272.30 k-ft, OK

The punching shear capacity of the foundation mat is checked in the following.

The columns are assumed hinged at the top and bottom. Hence, the columns are subject to axial gravity loads only.

From the analysis, the maximum factored load on the column at gridline C-3, associated with design section 5 in Fig. S.11.4.2-1 is:

$P_u = 274.17 \text{ k (1,219.56 kN)}$

Column dimension 12x18 in. (306x459 mm)

Depth of section 18-in. (459 mm)

Effective depth $d = 0.8 \times 18 = 14.4$ -in. (367 mm)

Dimensions of the first critical section = (12+14.4); (18+14.4) add up to 26.4 by 32.4-in. (673 by 826 mm)
 Surface of the first critical section = (2x26.4 + 2x32.4)x14.4 = 1,693.44 in²
 Shear stress demand $v_u = 274.17 \times 1000 / 1,693.44 = 161.90$ psi (1.12 MPa)

Allowable value = $\phi v_c = 0.75 \times 4 \times \sqrt{5000} = 212.13$ psi (1.46 MPa) OK.

No shear reinforcement required.

S.12 STRUCTURAL DETAILING

In general, there are three detailing considerations for the podium slab. These are:

- ❖ Detailing of tendons and adjustments in slab thickness associate with steps, changes in elevation, and other features that may be required to account for landscaping, surface drainage or other features of the exposed surface, if any.
- ❖ Generic details commonly used for post-tensioned floor slab to improve the service performance.
- ❖ Details to account for the potential restraint of the supports to free shortening of the podium slab.

S.12.1 Changes in Slab Thickness and Elevation

At balconies, footprint of the superstructure on the podium slab, landscaping, or slopes for drainages, it is often necessary to adjust the slab thickness or slope. Post-tensioning tendons are also adjusted to accommodate the change in slab geometry, while maintaining the design requirements. Figures S.12.1-1 and 2 are two examples.

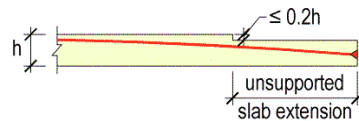


FIGURE S.12.1-1 Drop in Slab Surface at Entrance to Balcony (PTS231)

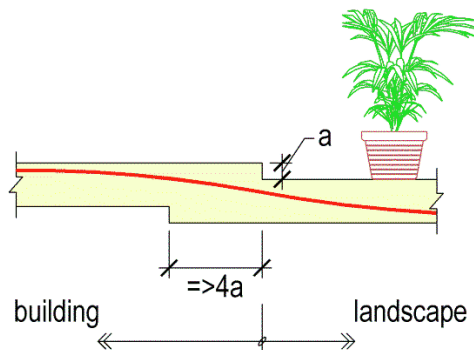


FIGURE S.12.1-2 Detail to Accommodate Change in Slab Thickness at Drop for Landscaping (PTS231)

S.12.2 Generic Details for Post-Tensioned Floors

In addition to the reinforcement determined by computation, rebar is added at the discontinuities in the slab, slab corners, slab edges and several other typical locations for improved service performance.

The reinforcement commonly used for structural detailing is based on the configuration and loading of the slab. The common details are given in a separate Technical Note³².

S.12.3 Details to Allow for Slab Shortening

The contribution of post-tensioning to the bending capacity of a member is directly linked to the precompression in the member from post-tensioning [Aalami, 2019]. Precompression in member can be achieved, if the member is free to shorten under the jacking force. Otherwise, some, or all the force from post-tensioning can be diverted to the restraining members (Fig. S.12.3-1).

There are details specifically developed to allow for shortening of post-tensioned podium slabs. Technical Note³³ gives a selection of commonly used details.

Special details to allow for shortening of a post-tensioned member from prestressing is generally not required at upper levels of a building.

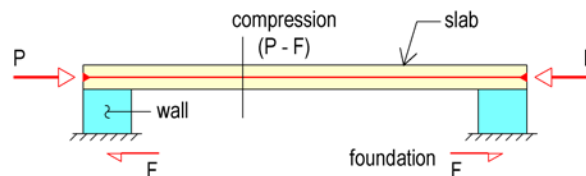


FIGURE S.12.3-1 Elevation; Partial Loss of Prestressing to the Supports at First Elevated Slab (PTS849)
Special details at the first elevated slab between the supports and the slab reduce the loss of force to the foundation.

S.13 FABRICATION (SHOP) DRAWINGS

In USA, the fabrication drawings for the installation of tendons and nonprestressed reinforcement is generally prepared by the post-tensioning and rebar supplier. This is not the case in most other countries, where the structural engineer is expected to provide most of the post-tensioning details required at construction.

The fabrication drawings specify:

- (i) the post-tensioning hardware for stressing and dead ends;
- (ii) the required reinforcement for dispersion of force at the stressing and dead end;
- (iii) means of securing the design tendon profile in place;
- (iv) elongation at stressing; and
- (v) plugging the stressing recess to protect the anchorage hardware against possibility of fire and corrosion.

S.14 BIM APPLICATION TO PODIUM SLAB DESIGN

³² Refer to TN451

³³ Refer to TN451

Gradual move to BIM³⁴ (Building Information Modeling) in design and preparation of construction drawings is revolutionizing the work of the structural engineers in providing seamless transition between the design and construction. Figure S.14-1 is an example of a BIM specifically developed for structural engineers. It shows a view of a BIM-generated view of the current project.

Florian will provide the following figure.

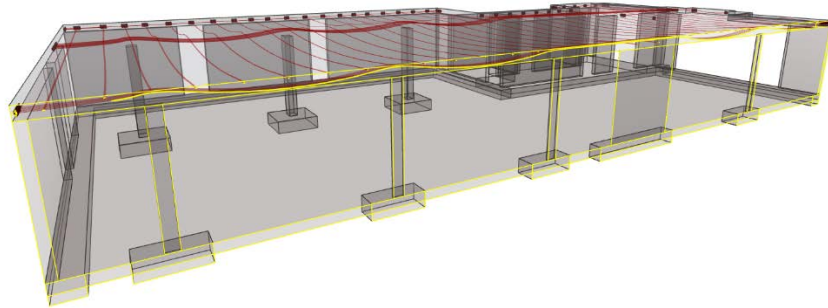


FIGURE S.14-1 One or two BIM views from Visicon (P1409)

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³⁴ www.Visicon.com

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