Thornton Tomasetti

Analysis and Design of Reinforced Cast-in-Place Concrete Diaphragms

October 16, 2017

Diaphragm Design

using strut- and-tie analysis

1. Background
2. Case Study Introduction
3. Example ‘traditional’ diaphragm design
4. Example of Alternate design approach using strut and tie
5. Conclusions

Background

Uncertainty in Current Diaphragm Stiffness and Design

Table 4. Stiffness parameters for Upper Bound and Lower Bound Models

Stiffness Parameters	UB	LB
Diaphragms at the podium and below		
$E_c\, I_{eff}$	0.5	0.20 to 0.25
$G_c\, A$	0.5	0.20 to 0.25

2014 LATBSDC Alternative Analysis and Design Procedure with 2015 Supplements 19

Table 5. Stiffness parameters for Upper Bound and Lower Bound Models

Stiffness Parameters	UB	LB
Diaphragms at the podium and below		
$E_c\, I_g$	0.25	0.10
$G_c\, A$	0.5	0.25

100% DRAFT

2017 LATBSDC Alternative Analysis and Design Procedure 39

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Floor diaphragms – Seismic bulwark or Achilles' heel

J.M. Scarry

Structural Engineer, Auckland

2014 NZSEE Conference

ABSTRACT: Floor diaphragms form a critical component of seismic resistant buildings, but unfortunately, in the main their analysis and design in New Zealand leaves much to be desired. No worse example exists than the CTV Building in Christchurch. Despite the critical importance of diaphragms, there is a paucity of code provisions and design guidance relating to them.

Using generic examples, the author describes a number of common diaphragm design deficiencies. These include diaphragms where valid load paths do not exist; diaphragms where the floors are not properly connected to the lateral load resisting elements, diaphragms that lack adequate flexural capacity and where re-entrant corners are not properly accounted for, and transfer diaphragms into which the reactions from the walls above cannot be properly introduced or transmitted.

Three main types of diaphragm action are discussed – 'inertial,' 'transfer' and 'compatibility.' These are, respectively, the direct inertial load on a floor that must be carried back to the lateral load resisting elements, the transfer forces that occur when major changes in floor area and lateral load resisting structure occur between storeys, and the compatibility forces that must exist to force compatible displacements between incompatible elements, such as shear walls or braced frames and moment frames, or as a result of redistribution.

The author presents a simple Truss Method that allows complex diaphragms to be analysed for multiple load cases, providing accurate force distributions without the multiple models that rigorous Strut and Tie methods would require.

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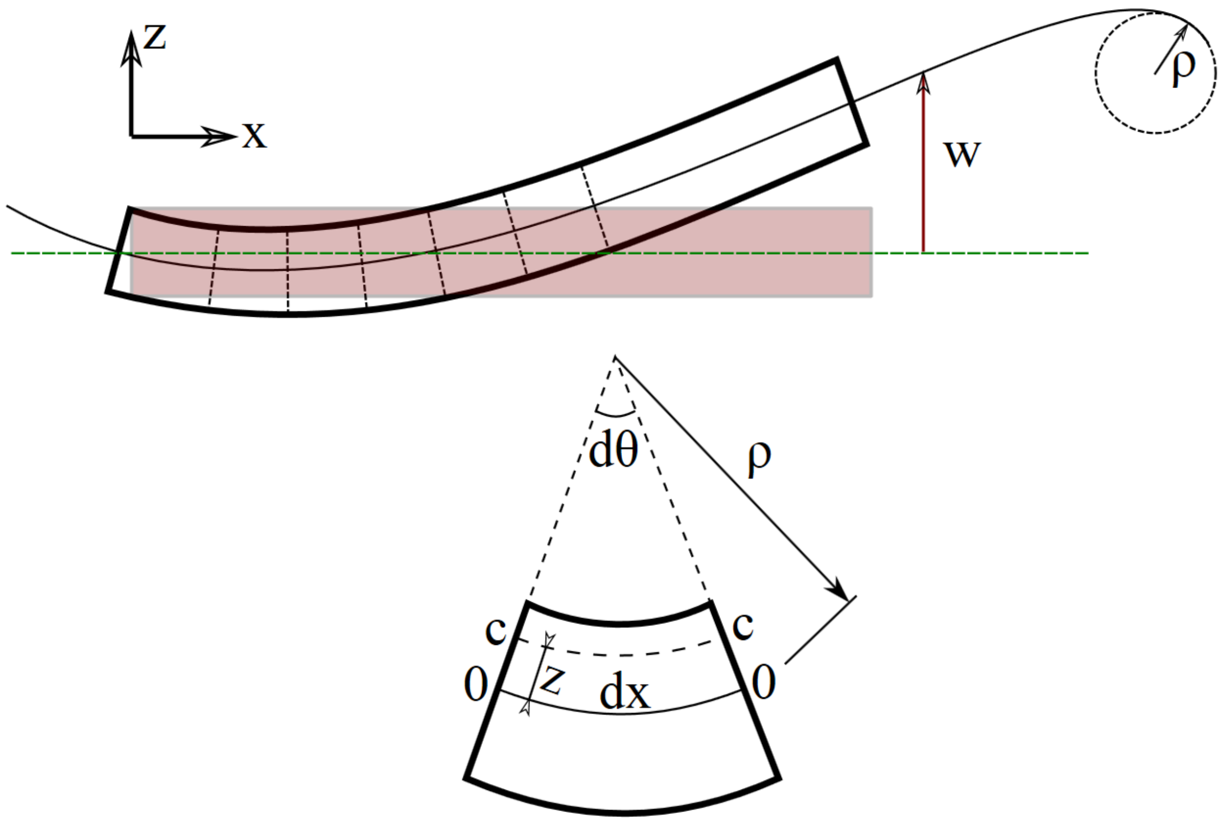
Euler-Bernoulli Beam Theory

Plane Sections Remain Plane



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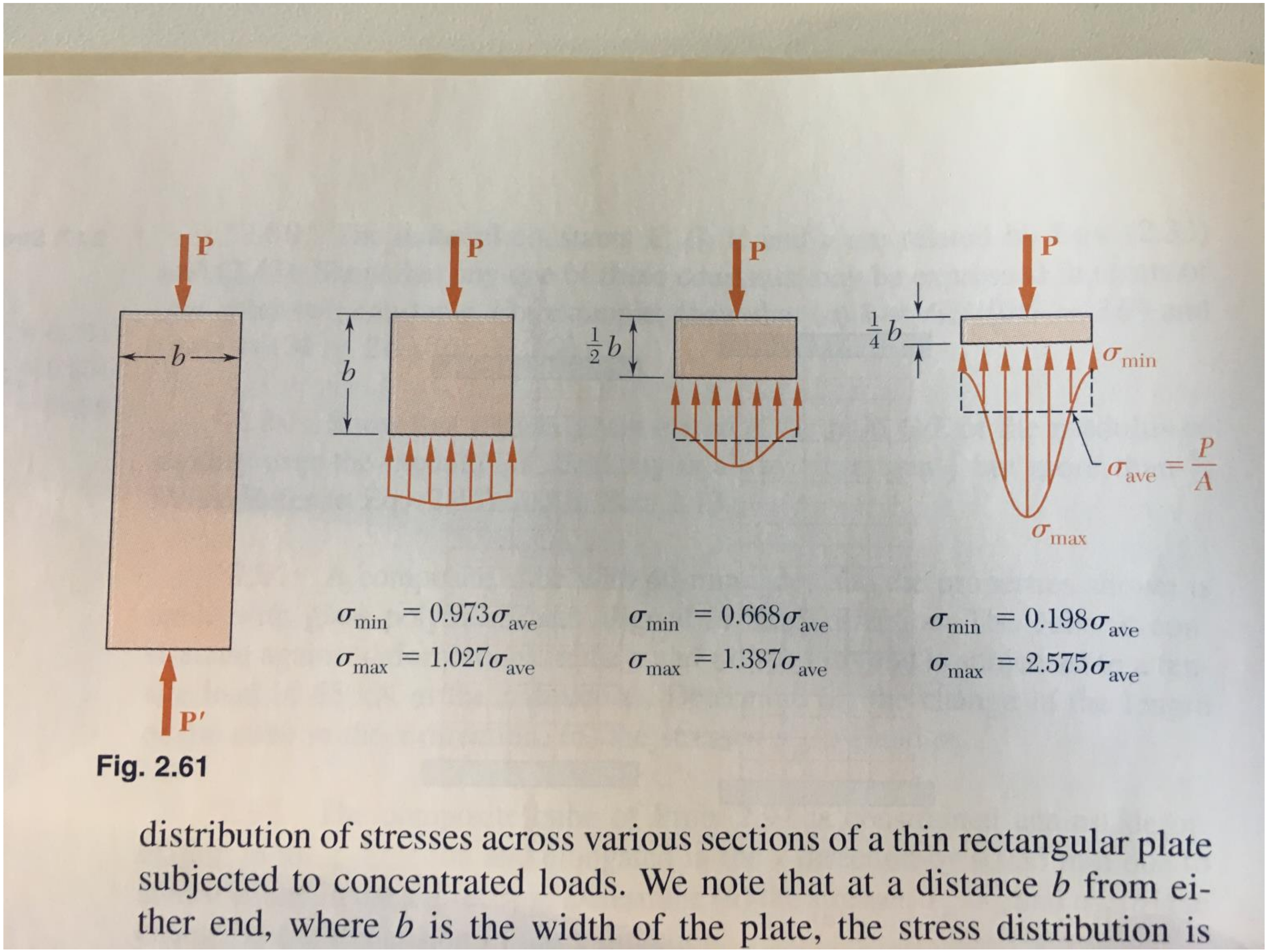
St. Venant's principale

Mechanics of Materials, Fourth Edition



$\sigma_{min} = 0.973\sigma_{ave}$
$\sigma_{max} = 1.027\sigma_{ave}$

$\sigma_{min} = 0.668\sigma_{ave}$
$\sigma_{max} = 1.387\sigma_{ave}$

$\sigma_{min} = 0.198\sigma_{ave}$
$\sigma_{max} = 2.575\sigma_{ave}$

Fig. 2.61

distribution of stresses across various sections of a thin rectangular plate subjected to concentrated loads. We note that at a distance b from either end, where b is the width of the plate, the stress distribution is

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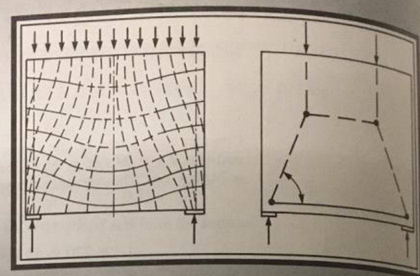
Strut-and-Tie method

- The Strut-and-Tie considers all load effects (Moment, Axial, Shear) simultaneously
- The model provides a rational approach by representing a complex structural behavior with an appropriate simplified truss of 1-D members
- MacGregor's 1992 book on Concrete Design provide one of the early and detailed examples of how to implement Strut-and-Tie.
- The method was incorporated into the 2002 version of ACI-318 in Appendix A

18 Discontinuity Regions and Strut-and-Tie Models



18-1 INTRODUCTION

Definition of Discontinuity Regions

Structural members may be divided into portions called *B-regions*, in which beam theor applies, including linear strains and so on, and other portions called *discontinuity region* or *D-regions*, adjacent to discontinuities or disturbances, where beam theory does n apply. D-regions can be *geometric discontinuities*, adjacent to holes, abrupt changes i cross section, or direction, or *statical discontinuities*, which are regions near concentrate loads and reactions. Corbels, dapped ends, and joints are affected by both statical and ge metric discontinuities. Up to this point, most of this book has dealt with B-regions.

For many years, D-region design has been by "good practice," by rule of thumb empirical. Three landmark papers by Professor Schlaich of the University of Stuttgart a his coworkers [18-1], [18-2], [18-3] have changed this. This chapter will present rules a guidance for the design of D-regions, based largely on these and other recent papers.

Saint Venant's Principle and Extent of D-regions

St. Venant's principle suggests that the localized effect of a disturbance dies out about one member-depth from the point of the disturbance. On this basis, D-regions are sumed to extend one member-depth each way from the discontinuity. This principle is c ceptual and not precise. However, it serves as a quantitative guide in selecting dimensions of D-regions.

Figure 18-1 shows D-regions in a number of structures, some of which have B-regi (bending regions) between two D-regions. Figure 18-2 shows examples of D-regions. D-regions in Fig. 18-2b and c extend one member-width from the discontinuity as sugg ed by St. Venant's principle. Occasionally, D-regions are assumed to fill the overlapp region common to two members meeting at a joint. This definition is used in the traditi al definition of a joint region.

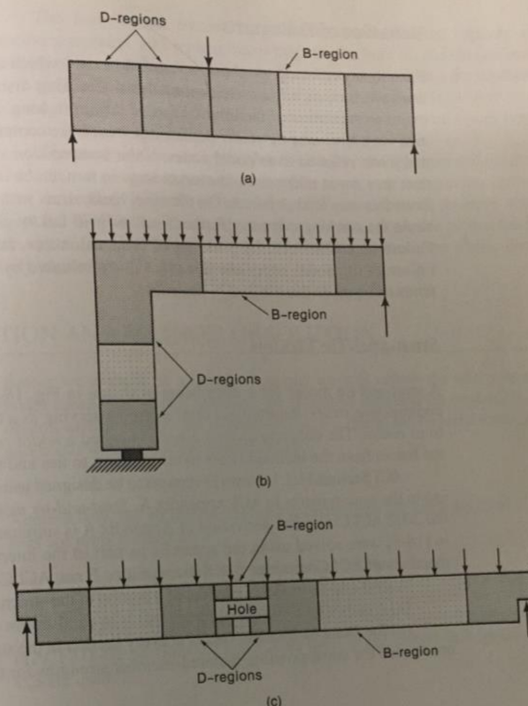
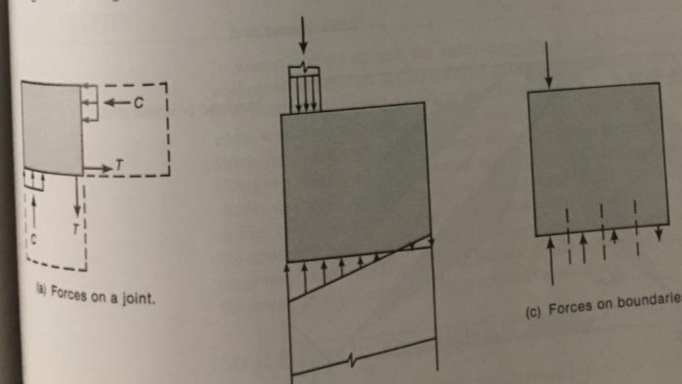


Fig. 18-1
B-regions and D-regions.

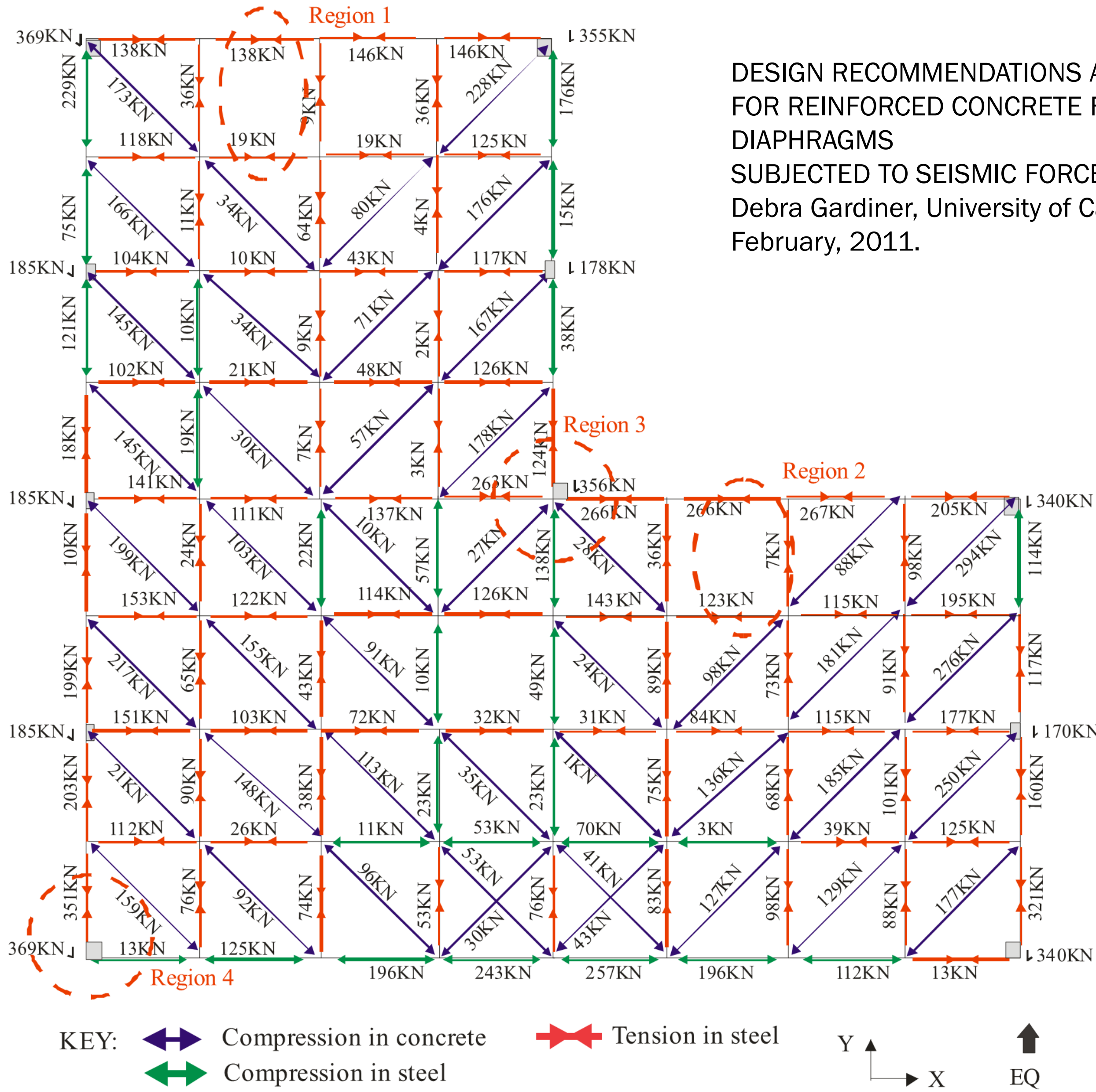


(a) Forces on a joint.

(b) Stresses on boundaries.

(c) Forces on boundaries.

Fig. 18-2
Forces on boundaries of D-regions



DESIGN RECOMMENDATIONS AND METHODS FOR REINFORCED CONCRETE FLOOR DIAPHRAGMS SUBJECTED TO SEISMIC FORCES, PhD Thesis, Debra Gardiner, University of Canterbury, February, 2011.

Figure 8-23 Strut and tie solution for L-shaped floor 3.5x3.5m mesh

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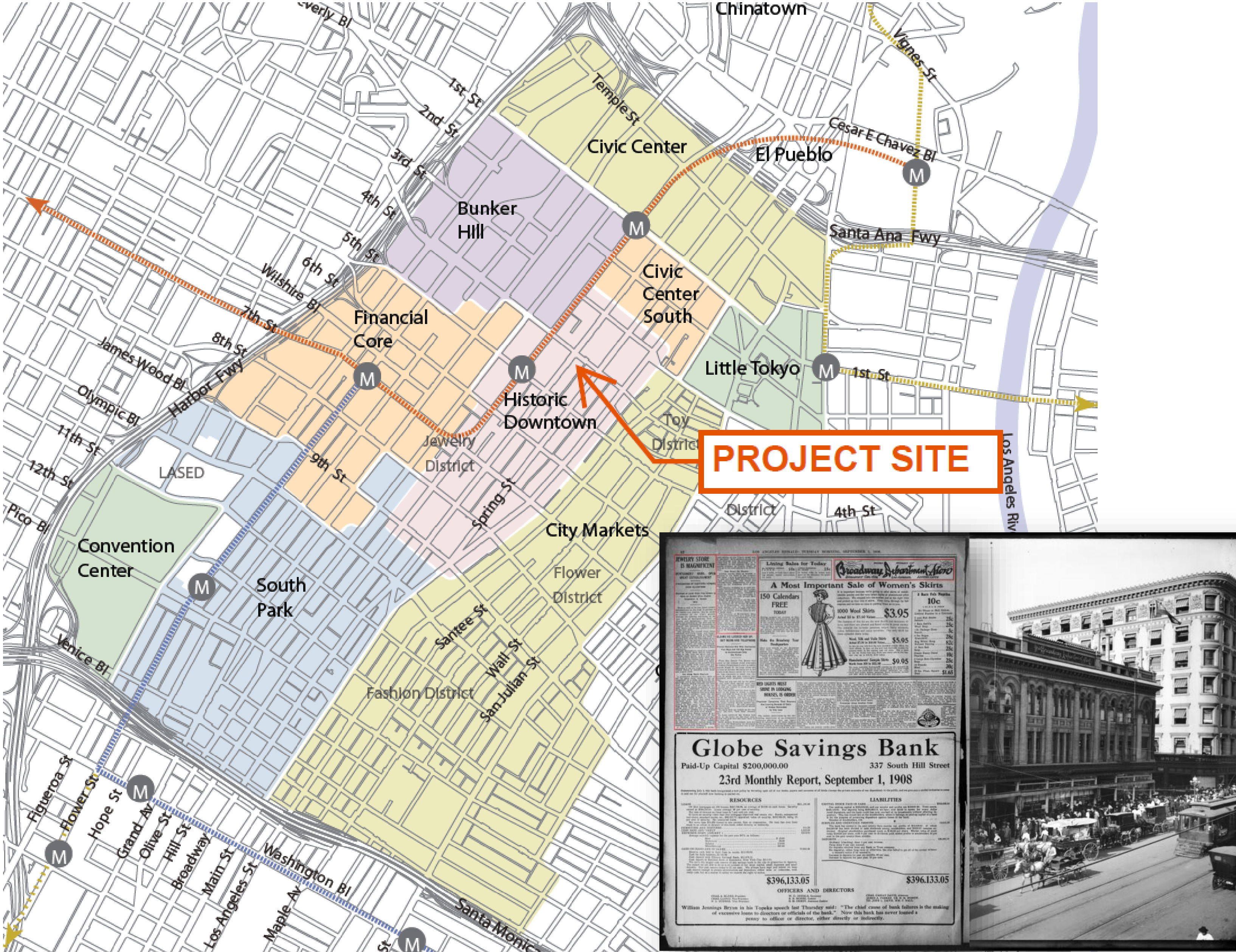
Case Study

Perla on Broadway

4th & Broadway, Downtown Los Angeles

- Guided by strict planning constraints in the historic core of downtown Los Angeles
- 35-story tower with
- Tall 11-story Podium
- Large 4-story Atrium
- Creates unique challenges for diaphragm design.
- Including large openings and transfer forces

Historic 4th and Broadway



JEWELRY STORE IS MAGNIFICENT

Living Sales for Today

Broadway Department Store

A Most Important Sale of Women's Skirts

150 Calendars FREE TODAY

1000 Wool Skirts $3.95

$5.95

$9.95

Globe Savings Bank

Paid-Up Capital $200,000.00 337 South Hill Street

23rd Monthly Report, September 1, 1908

RESOURCES

LIABILITIES

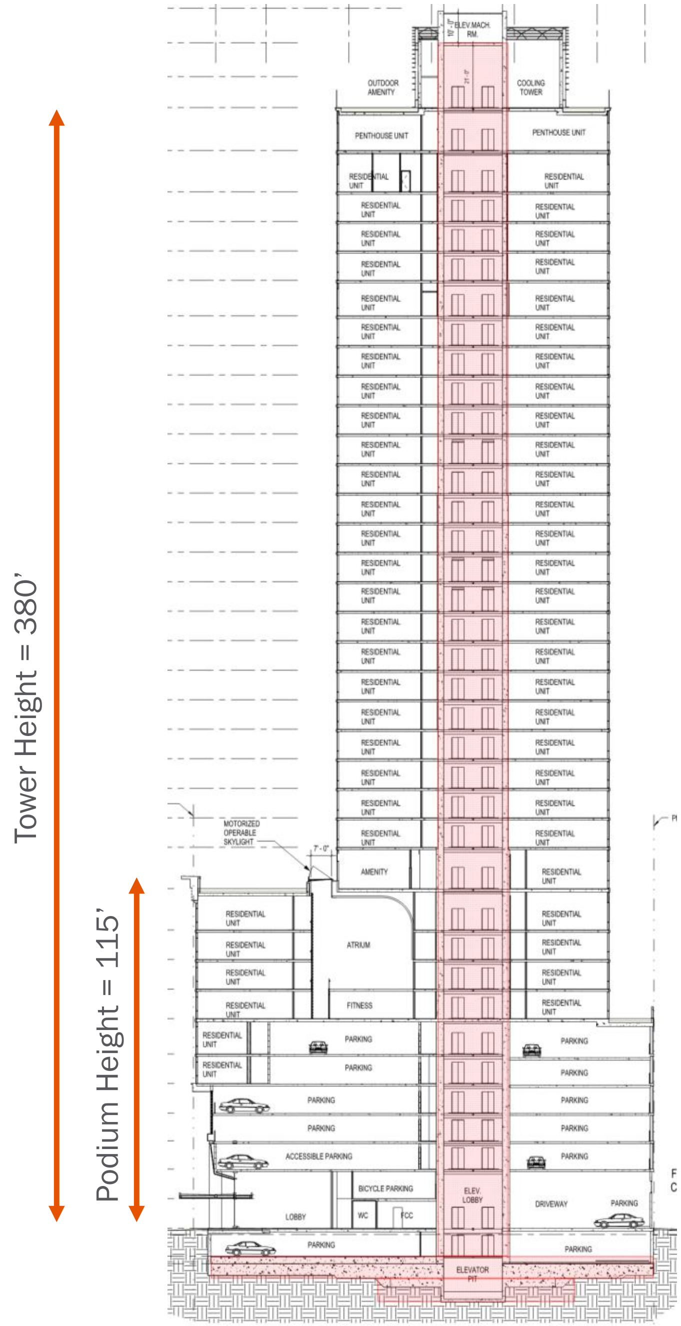
$396,133.05 $396,133.05

OFFICERS AND DIRECTORS

William Jennings Bryan in his Topeka speech last Thursday said: "The chief cause of bank failures is the making of excessive loans to directors or officials of the bank." Now this bank has never loaned a penny to officer or director, either directly or indirectly.

Historic 4th and Broadway

Tower Section



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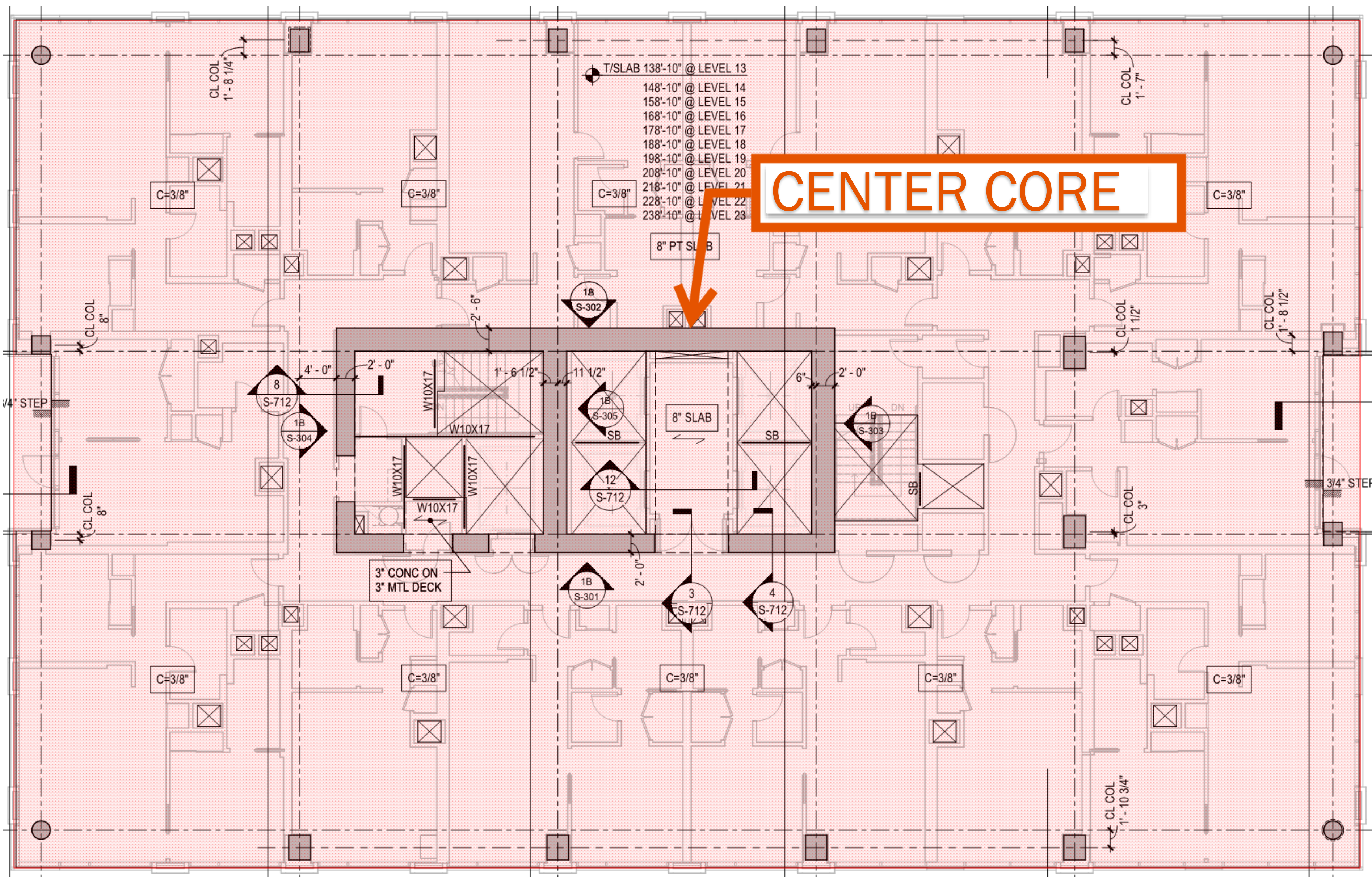
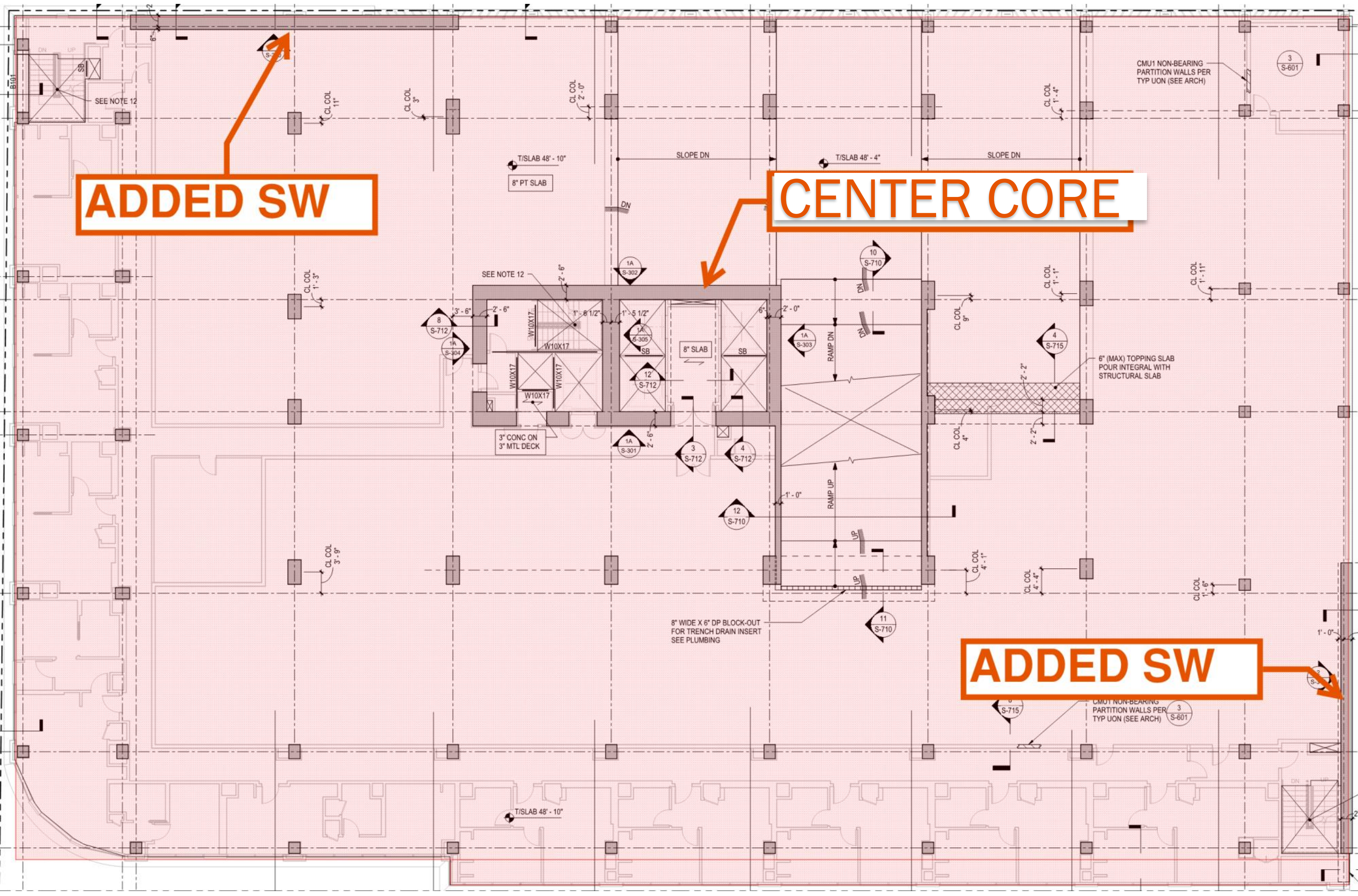
Tower Floor Plate

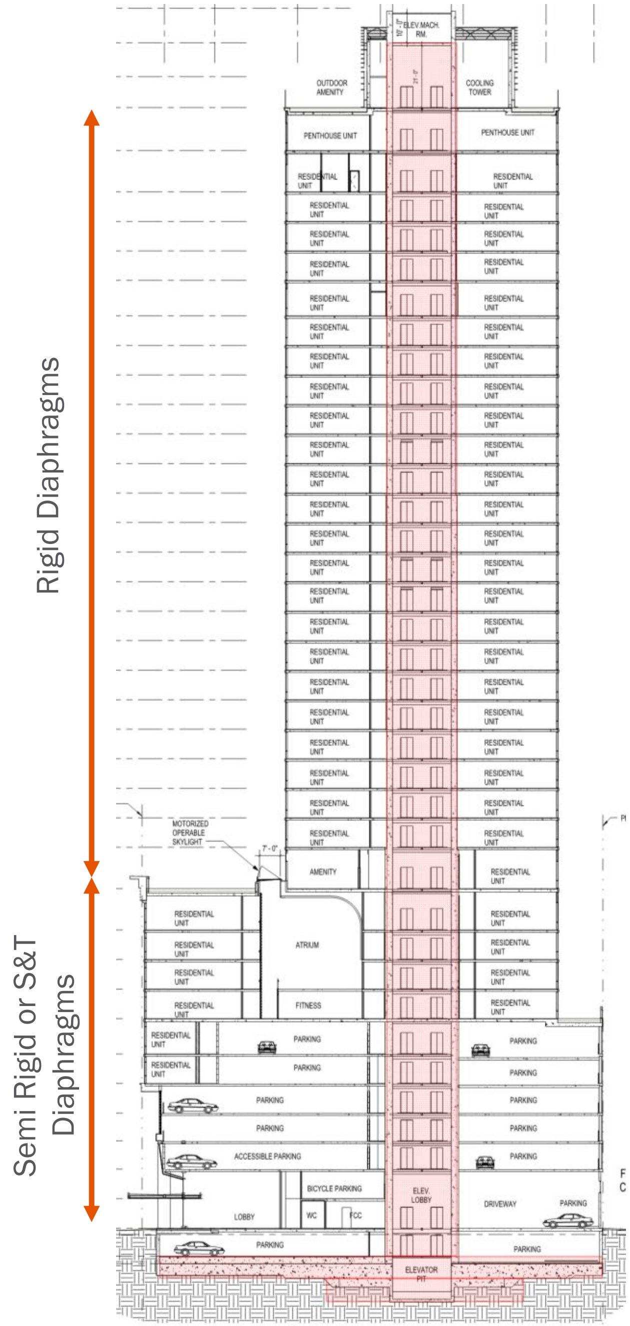


Podium Floor Plate



Diaphragm Design

Tower Section



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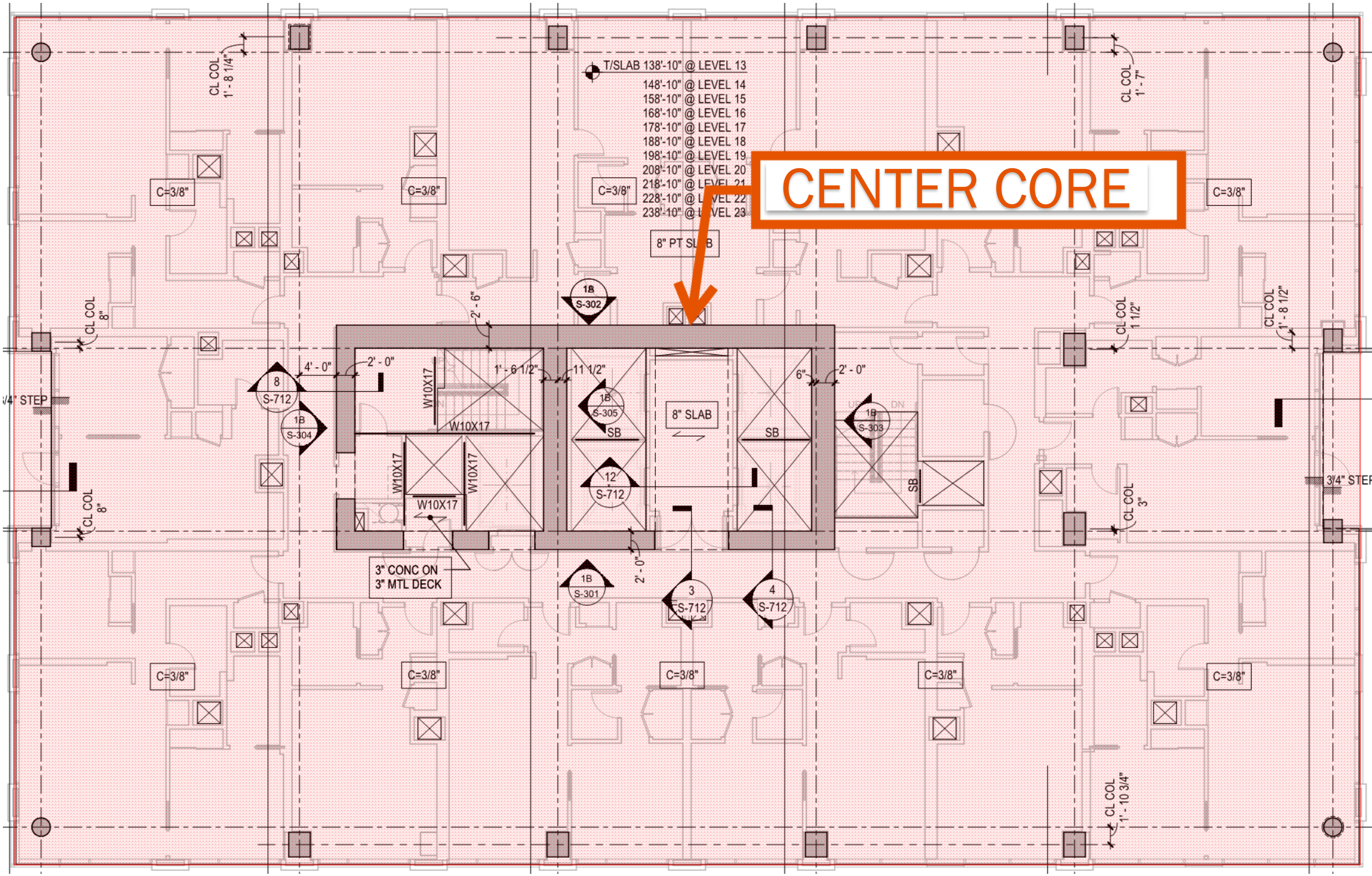
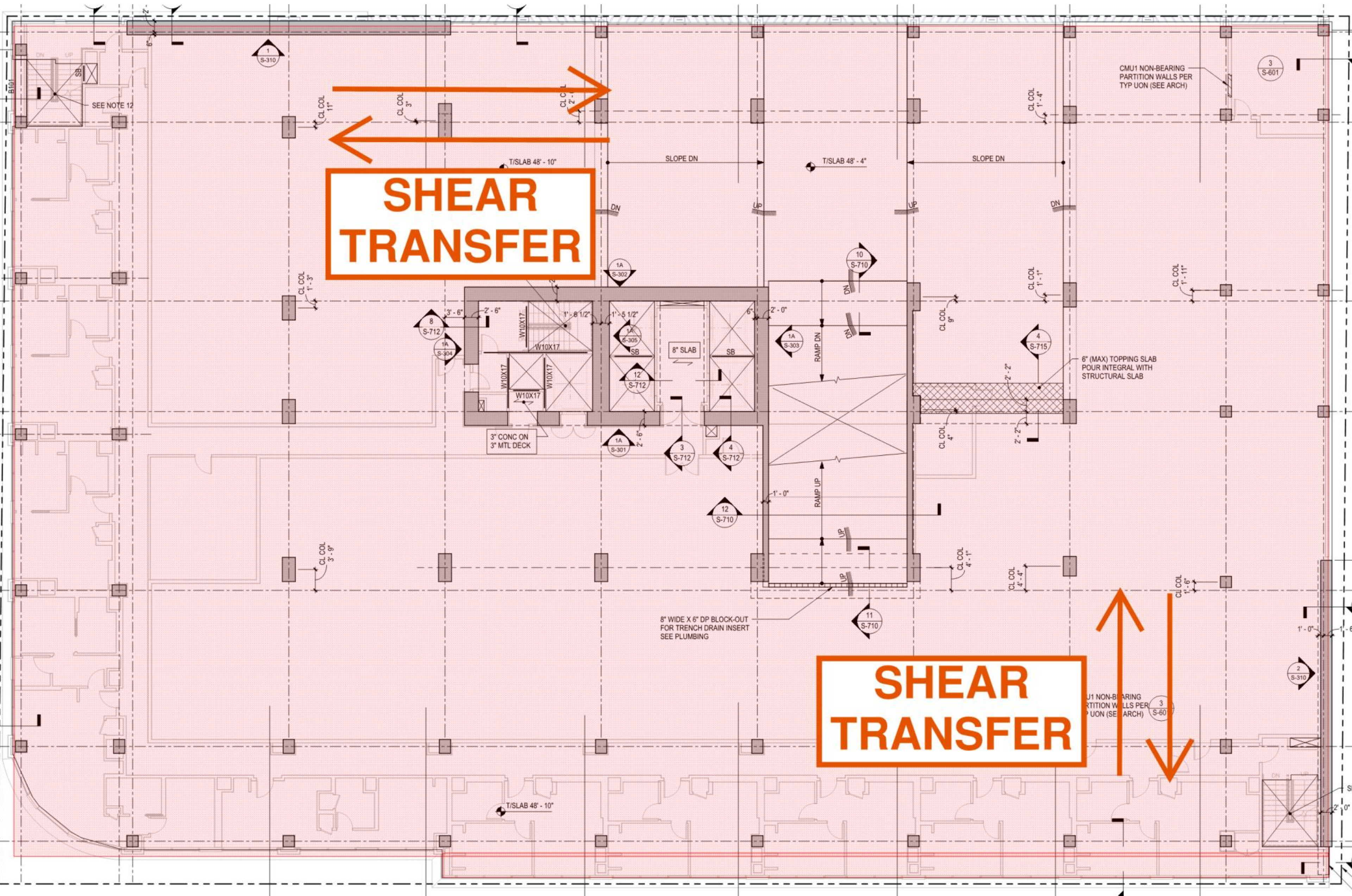
Rigid Diaphragm



Semi Rigid or S&T Diaphragm



Rigid Diaphragm vs
Semi Rigid or Simple Truss Diaphragm

Rigid Diaphragm

Forces extracted at vertical elements and a diaphragm load path is assumed or modeled outside of the primary analysis program

Semi Rigid or S&T

Diaphragm forces are extracted from within the diaphragm using section cuts or axial S&T forces

Traditional Diaphragm Design

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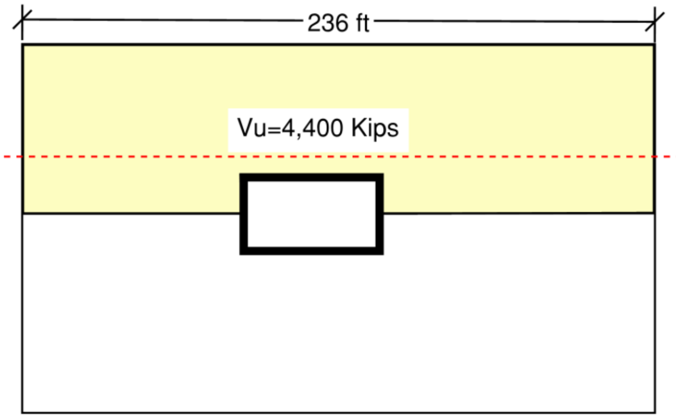
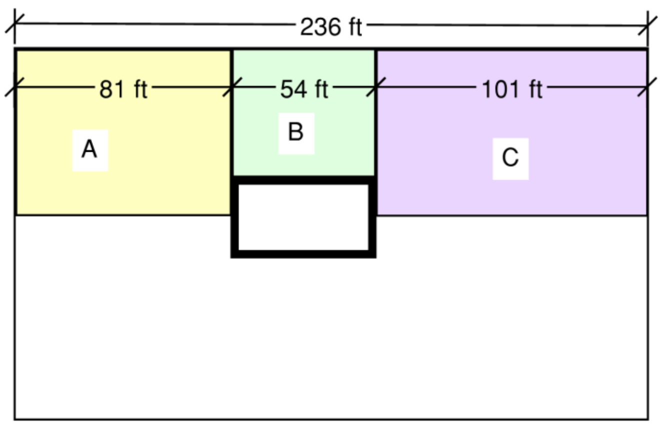
Detailed calculation for slab-Wall connection at GL 3.1

Slab moves to the left



Assume uniform distribution of shear force, the (3) slab panels proportionally receive forces:

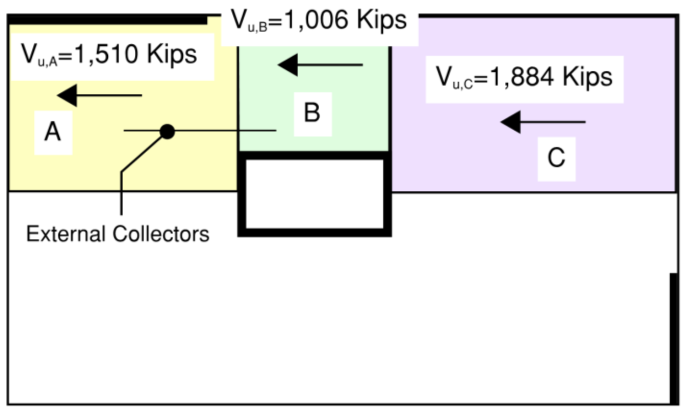


$V_{u,A}$ = 81'/236' x 4,400 k = 1,510 k

$V_{u,B}$ = 54'/236' x 4,400 k = 1,006 k

$V_{u,C}$ = 101'/236' x 4,400 k = 1,884 k

Slab moves to the left ←



PANEL A

Post-tensioning tendons within the slab transfers:

0.4 x 16 k/ft x 270 ksi / 175 ksi x 48'= 474 k

Remaining force is 1,510 - 474 = 1,036 k, resisted by external collectors.

Use (12) # 10. Tension force: 12 x 1.27 in^2 x 1.17 x 60 ksi = 1,069 k > 1,036 k ✓

PANEL C

Force delivered to shear walls through direct bearing :

0.4 x 1.3 x 5.5 ksi x 8" x 60" = 1,372 k < 1,884 k

Remaining force is 1,884 - 1,372 = 512 k, transferred to panel B as panel C pushes on it.

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PANEL B

- Total shear force transferred to slab-wall dowels at panel B is :

$$1{,}006 + \underbrace{(474 + 1{,}036)}_{\text{from A}} + \underbrace{(512)}_{\text{from C}} = 3{,}028 \text{ k}$$

Over 54 ft of slab-wall interface, creates stress 56 k/ft < 65 k/ft corresponding to

$8 \sqrt{f'_{c,exp}}$ within the 8" thick slab. Provided are dowels 2 x #6 @ 6" minus #5 @ 6" for

gravity, is 2 x 62 - 44 = 80 k/ft > 56 k/ft $\checkmark$

- Total shear force received by panel B is :

$$1{,}006 + \underbrace{(474)}_{\text{From A}} + \underbrace{(512)}_{\text{from C}} = 1{,}992 \text{ k}$$

Over 54 ft length of panel B, it is 37 k/ft > 16 k/ft which is the shear strength of unreinforced concrete.

Place shear reinforcement #5 @ 12" corresponding to a shear capacity of 22 k/ft.
Total shear capacity is 16 + 22 = 38 k/ft > 37 k/ft.

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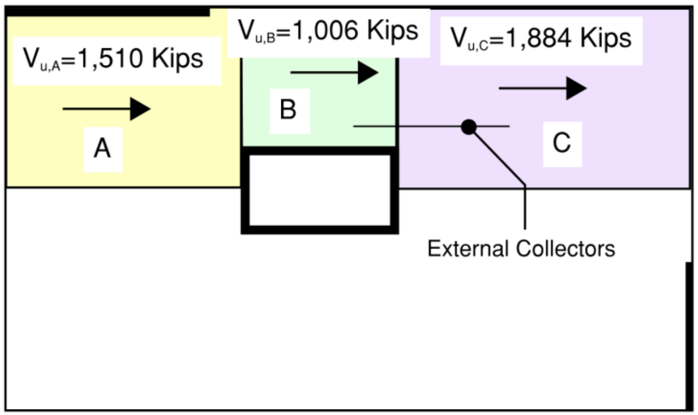
Slab moves to the right →



PANEL A

Force delivered to shear walls through direct bearing is :

0.4 x 1.3 x 5.5 ksi x 8" x 60" = 1,372 k < 1,510 k

Remaining force is 1,510 - 1,372 = 138 k, transferred to panel B as panel C pushes on it.

PANEL C

Post-Tensioning tendons within the slab Transfers:

0.4 x 16 k/ft x 270 ksi / 175 ksi x 48' = 474 k

Remaining force is 1,884 - 474 = 1,410 k, resisted by external collectors.

At L2-Use (16) #10. Tension force: 16 x 1.27 in^2 x 1.17 x 60 ksi = 1,426 k.
L3-Use (12) #10 L5-Use (14) #10
L4-Use (12) #10 L6-Use (14) #10

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PANEL B

- Total shear force transferred to slab-wall dowels at panel B is :

$$\underbrace{1{,}006+ (138)}_{\text{from A}} + \underbrace{(474+1{,}410)}_{\text{from C}}= 3{,}028 \text{ k}$$

See corresponding calculations for opposite slab move.

- Total shear force received by panel B is :

$$\underbrace{1{,}006 + (138)}_{\text{From A}} + \underbrace{(474)}_{\text{from C}}= 1{,}618 \text{ k}$$

See corresponding calculations for opposite slab move.

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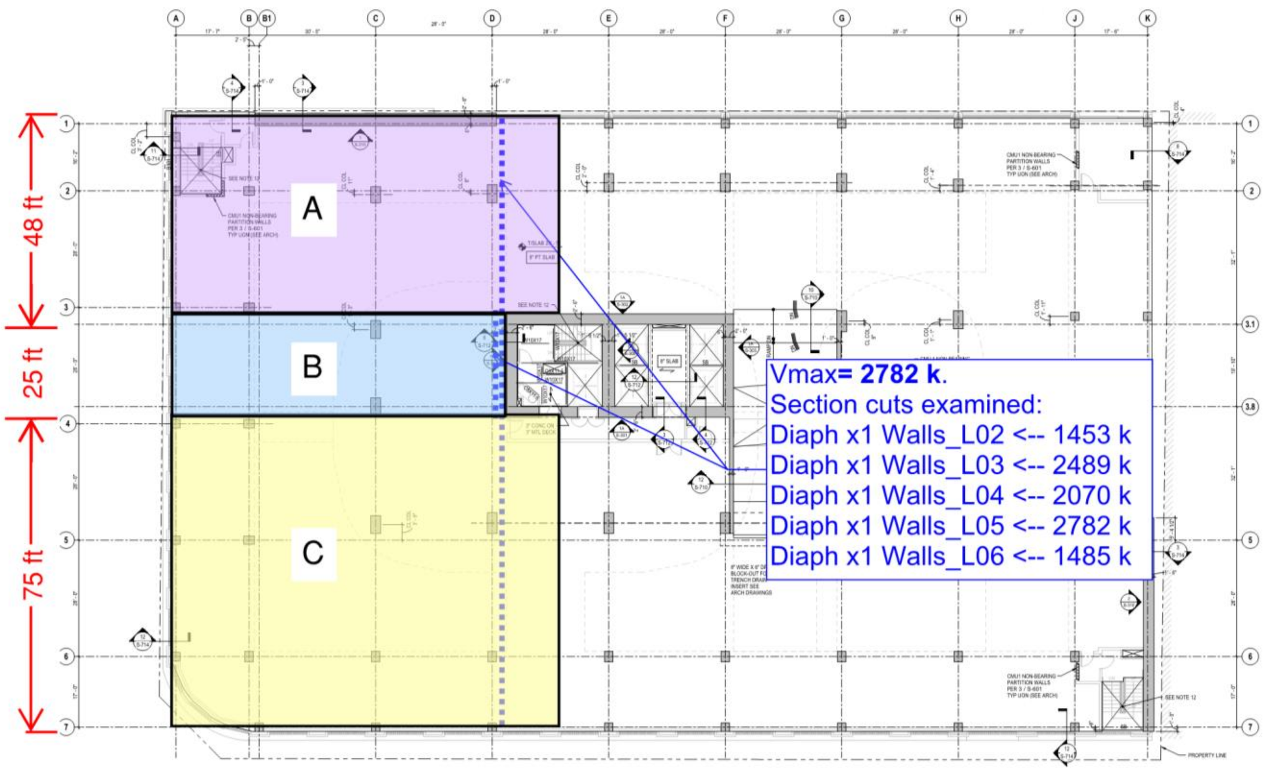
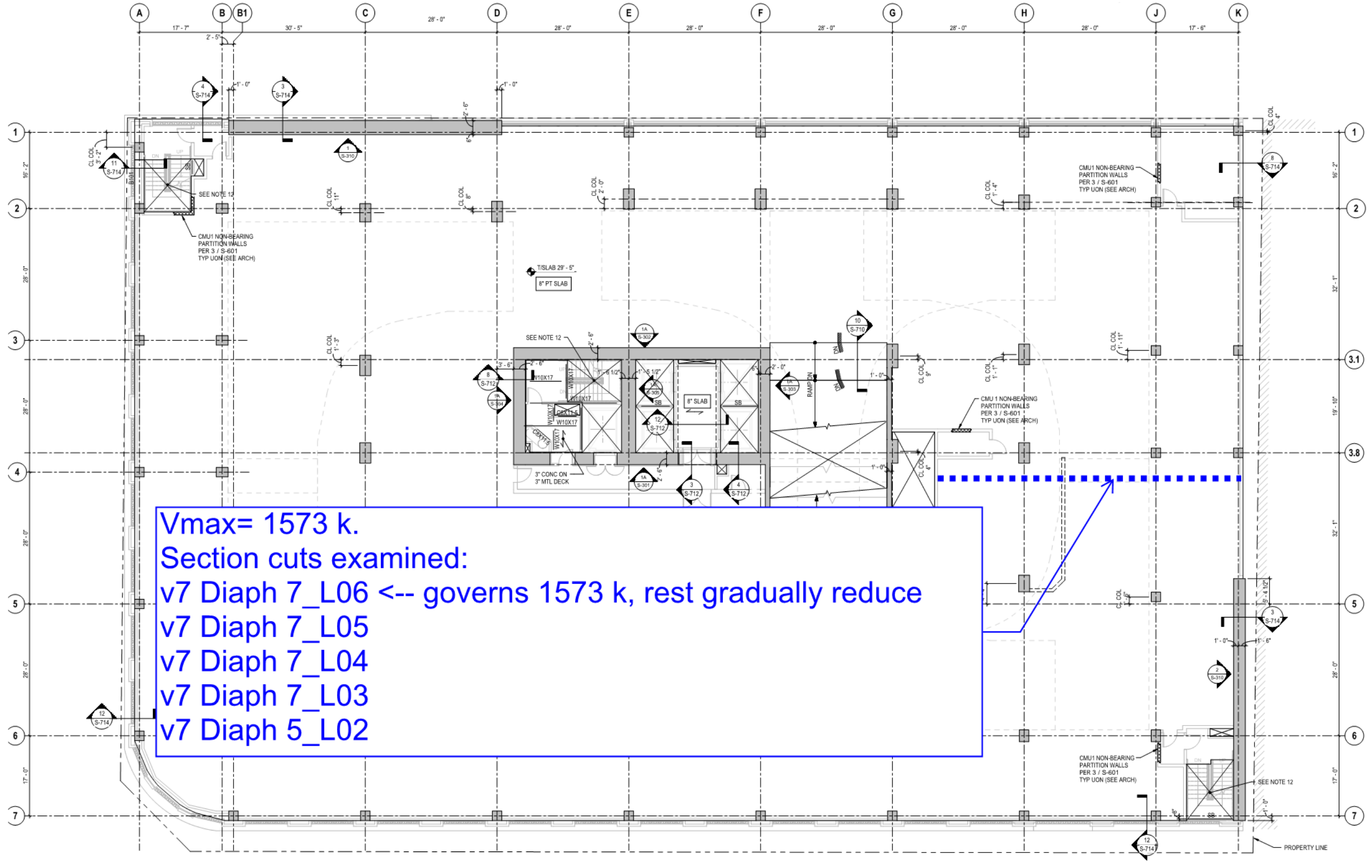


ASSUMED LOAD DISTRIBUTION

- (3) effective areas based on contributing lengths:

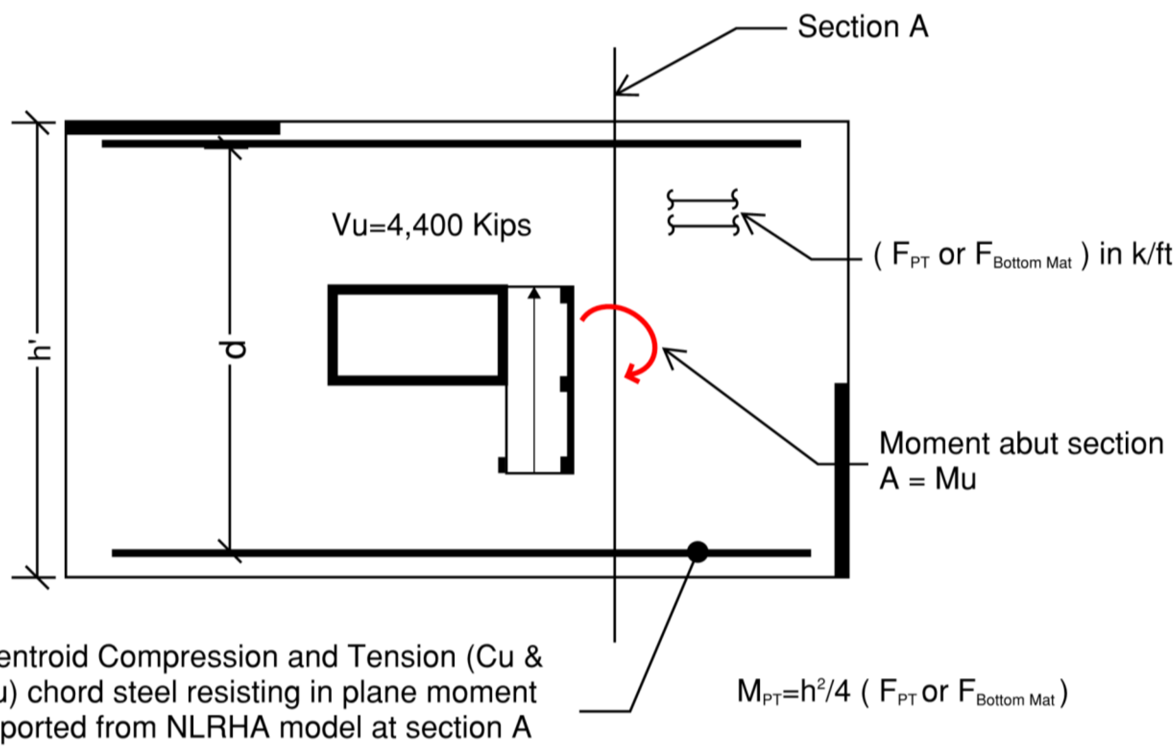
  Panel A: [48ft, 902 k]
  Panel B: [25ft, 470 k]
  Panel C: [75ft, 1410 k]



Over 75 ft is 21 k/ft > 16 k/ft. Place #4@24" distributed reinf which has 7 k/ft, total 23 k/ft, ok.

OK

Chord reinforcement in the diaphragm is designed using section cuts in the NLRHA model at levels II an below.
The moment about section cuts taken at critical location is averaged for (13) ground motions.
At transfer levels, this mean force is amplified by a factor of 1.5x .
at all other levels, the mean force is used for design.



Chord forces and reinforcement are designed using a simple beam analogy in accordance with NIST GCR 16-917-42

$C_u = T_u = M_{u\,-}\ M_{PT} / d$

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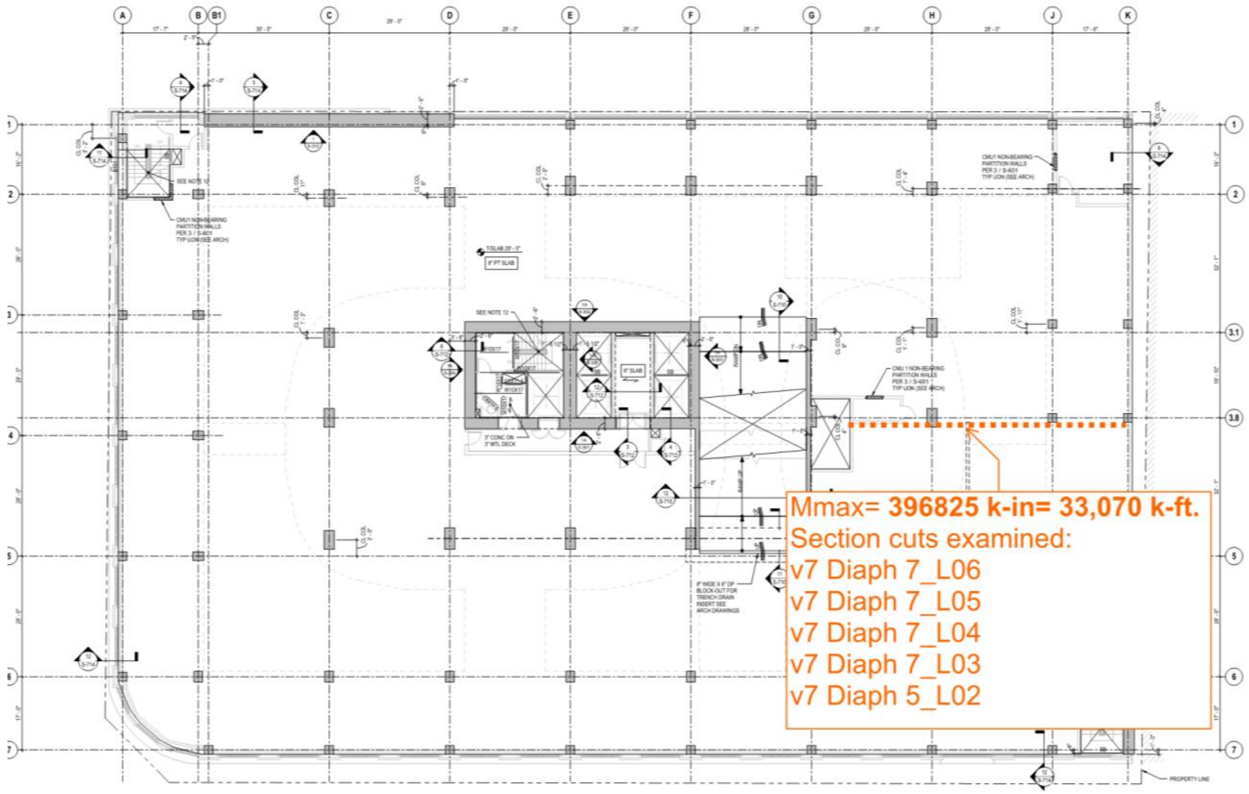

PT takes Mpt = h ^ 2 / 4 x fy = (75^2) / 4 x (9.87)= 13,879 k-ft, ok.
Provided are collectors at least (6)#10 at a distance 65 ft have moment 34671 k-ft.
Total strength 13879 + 34671 = 48,550 k-ft.

OK

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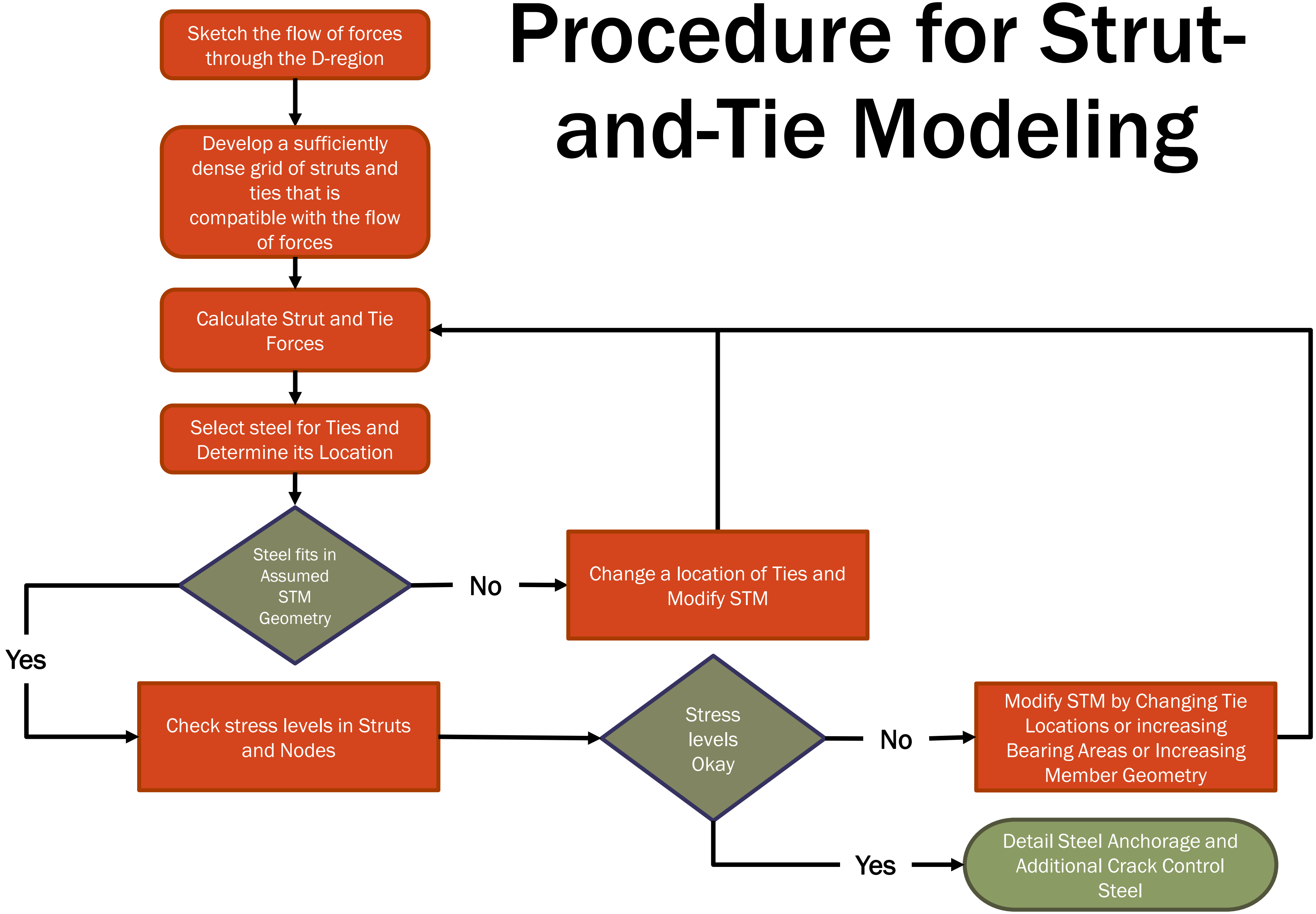
Alternate
Approach

Procedure for Strut-and-Tie Modeling



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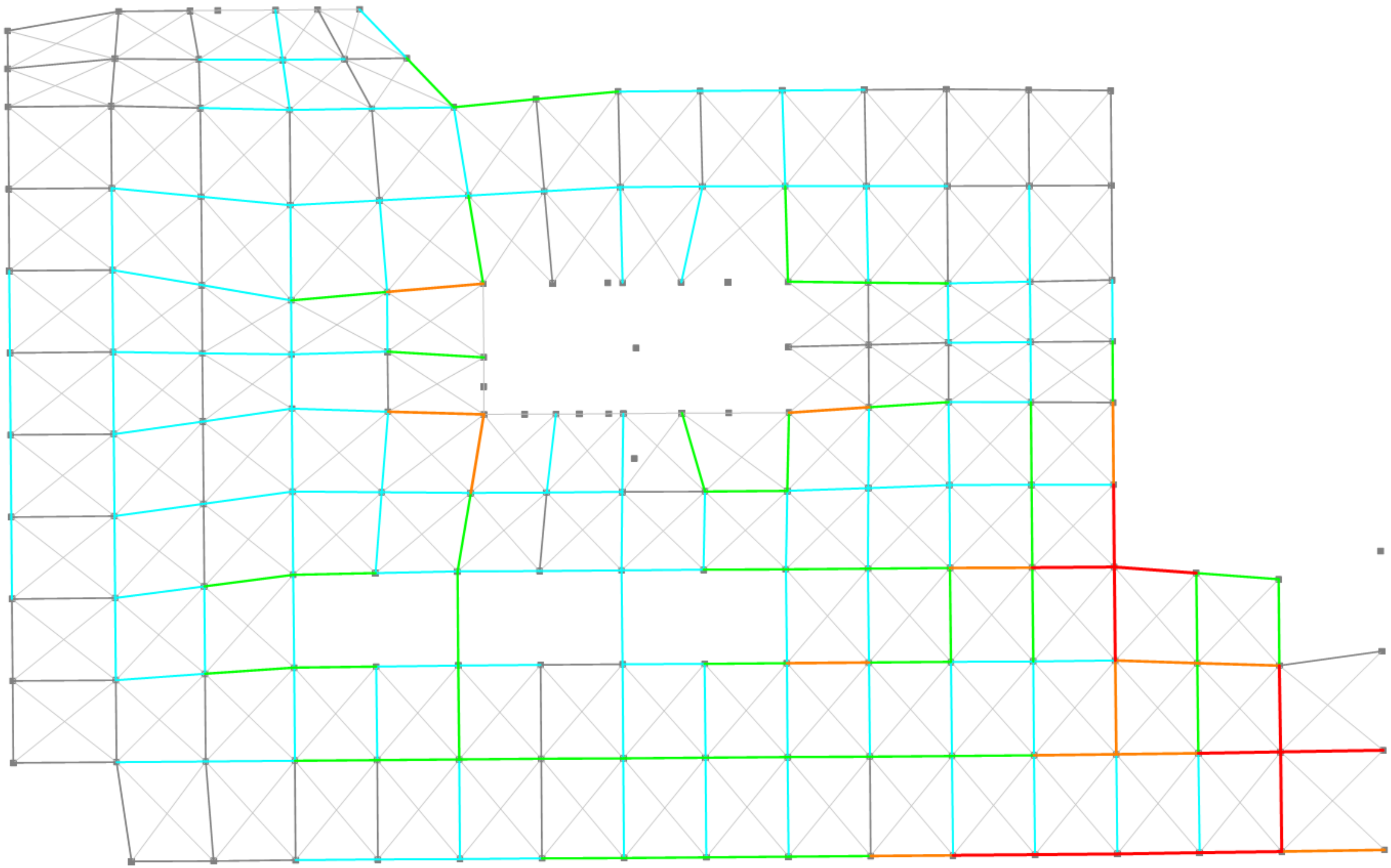
Diaphragm Tie Forces at Level 11



Axial Force in Diaphragm Ties

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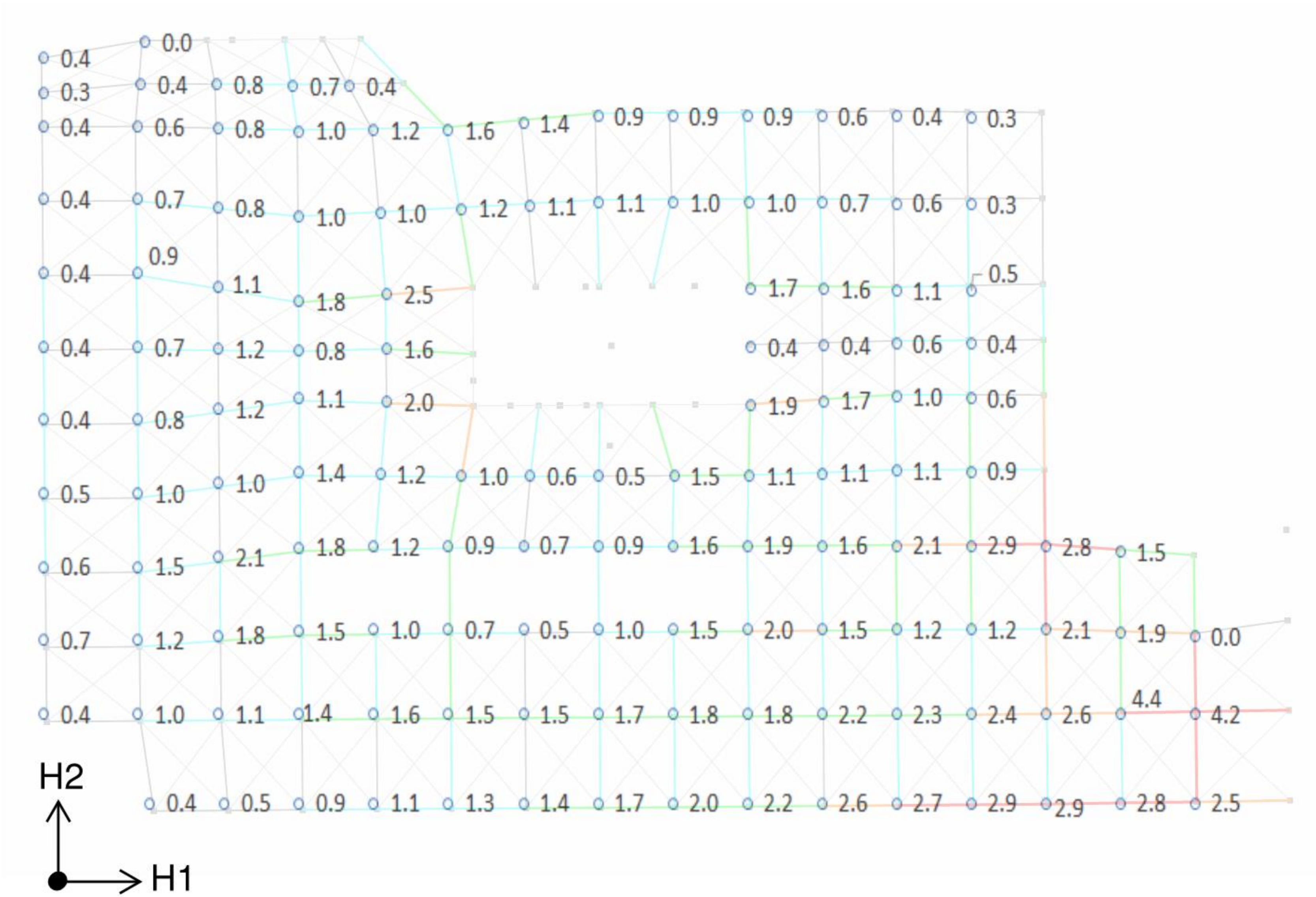
Tension Steel in L11 Diaphragm Ties - H1 Direction



Tension Steel Required in Diaphragm Ties H1 direction

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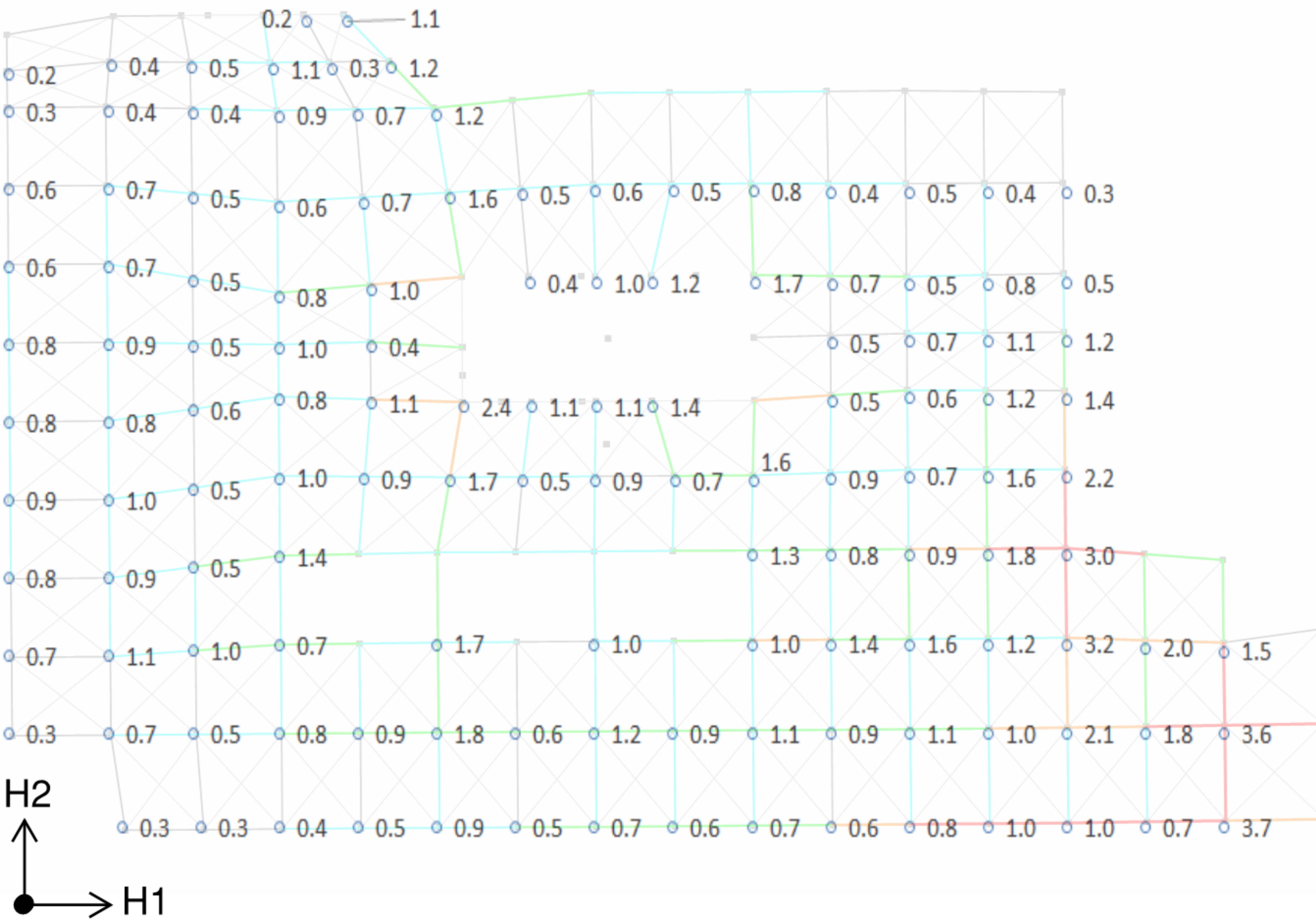
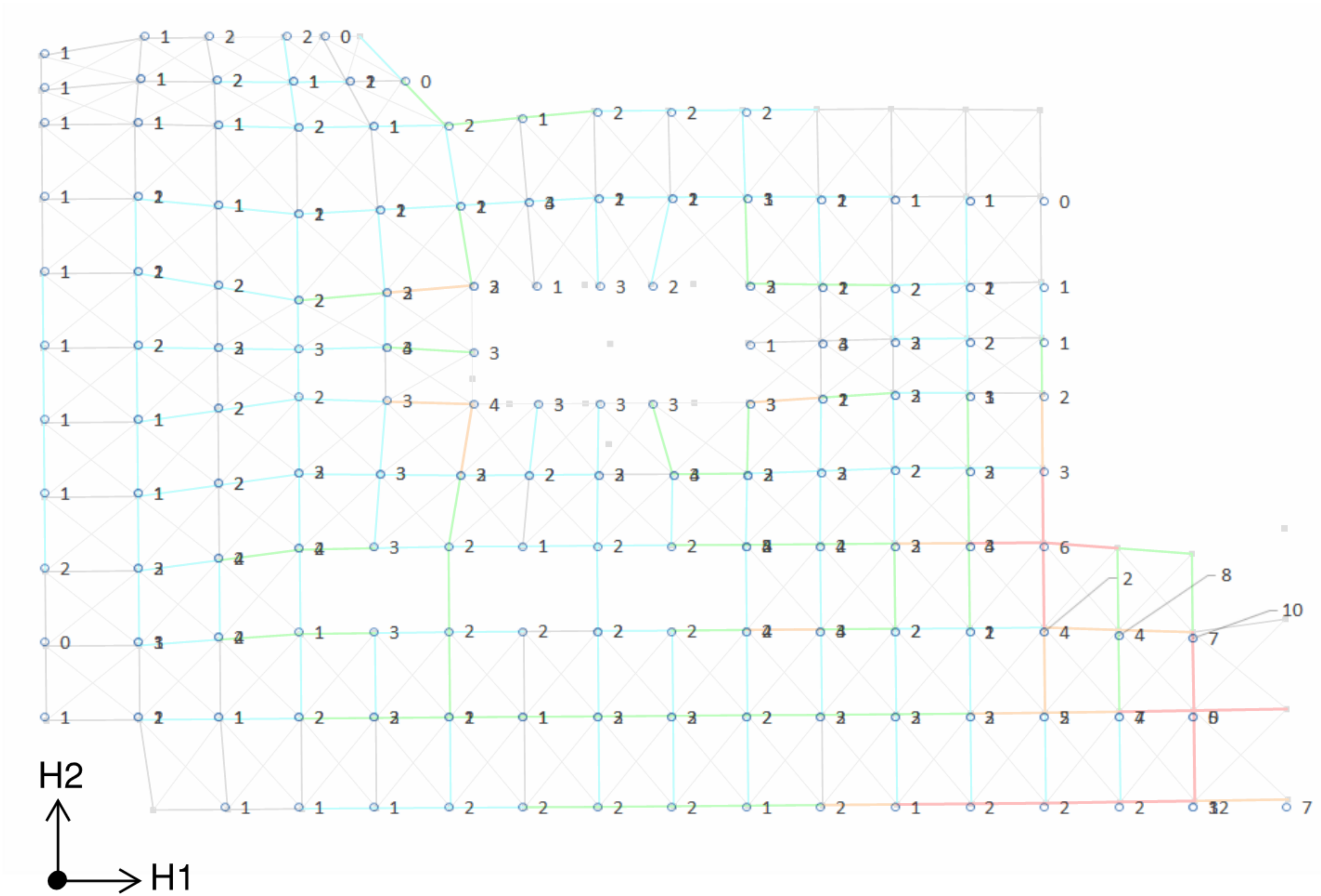
Tension Steel in L11 Diaphragm Ties - H2 Direction



Tension Steel Required in Diaphragm Ties H2 direction

Compression Width Required in L11 Diaphragm Diagonal Struts



Compression Width Required for Diaphragm Struts

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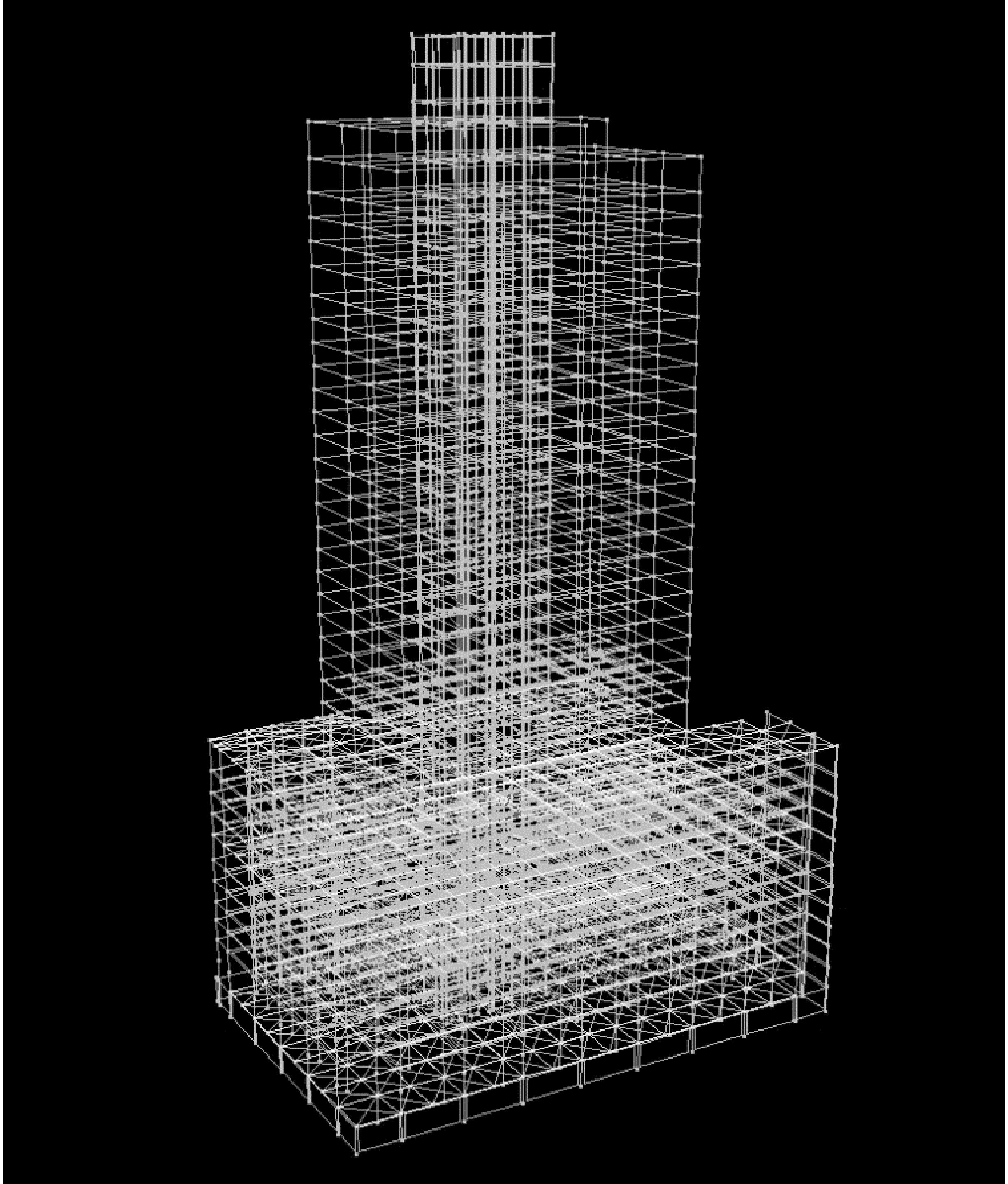

Conclusions

Conclusion

1. This approach provides a simplified analysis model that can account for shear, axial, and moment forces through a resultant one dimensional element in tension or compression.
2. Provides a graphical representation of the diaphragm load path.
3. Allows for the extraction of diaphragm forces without having to define explicit section cuts.
4. Provides explicit modeling of collectors to assist the designer in the determination of collector reinforcing and extents.
5. Provide explicit deformation in long collector elements where the stiffness of the collector can be calibrated to the amount of steel provided.
6. Future studies - Allows for nonlinear analysis of diaphragm elements
7. Future studies – provides a detailed distribution of diaphragm forces that can lead to more economical, safe designs.

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