Thornton Tomasetti

Riverview Condominiums

Punching Shear Review

Thornton Tomasetti Q23192.00

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Appendices

1.0 Introduction and Project Overview

At the request of Thayer & Associates, Inc., Thornton Tomasetti ("TT") performed structural analysis of representative reinforced concrete flat plate floor slabs at the Riverview Condominium building located at 221 Mt. Auburn St., Cambridge, MA. Field investigations performed by Simpson Gumpertz & Heger ("SGH") at the Riverview Condominium building during summer 2023 revealed that the as-built steel reinforcement ("rebar") placement at the floor slabs is often set at lower depths than specified in the existing structural drawings. The lower rebar depths substantially reduce the load-carrying capacity of the floor slabs. The Riverview Condominium Board subsequently engaged SGH to perform a comprehensive structural condition assessment and structural analysis of the floor slabs to develop remediation recommendations. TT's role was to peer review SGH's condition assessment and analysis.

TT conducted limited ground penetrating radar ("GPR") scanning at a representative area of the Level 7 floor slab in December 2023 to independently verify the as-built rebar depths. We then performed structural analysis for that floor slab area considering the as-built rebar depths. TT concluded that the representative floor slab area has adequate flexural strength but inadequate punching shear strength. We recommended implementing concrete compressive strength testing and expanding the punching shear analysis using further as-built structural information. TT compiled our GPR scanning and analysis results in a letter issued to Thayer & Associates, Inc. dated December 27, 2023 (**Appendix A**).

In parallel, SGH implemented several structural condition assessment methods including comprehensive GPR scanning of the floor slabs, Schmidt rebound hammer testing, compression testing of concrete core samples, and tension testing of steel rebar samples during Fall 2023 and throughout 2024. SGH also performed a limited exploratory program to expose certain floor slab areas for visual observation, and conducted drone inspections to visually observe the building exterior.

TT maintained regular communication with SGH throughout our peer review to ensure that we agreed with their condition assessment and analysis. SGH consistently shared their field data with us, which we used to refine our structural analysis. TT accompanied SGH on one site visit to view exposed floor slab areas in June 2024.

This report summarizes TT's structural analysis of representative flat plate floor slabs at the Riverview Condominium building as part of our peer review of SGH's evaluation. Our analysis covers most of the building levels and accommodates as-built structural information obtained from our site observations and SGH's field data. We provide comments regarding SGH's condition assessment and analysis of the floor slabs. Finally, we provide conclusions and strengthening recommendations for the floor slabs, in agreement with SGH's evaluation.

1.1 Building Description

The Riverview Condominium building is an 8-story, reinforced concrete structure with flat plate floor slabs, constructed circa 1961 (Figure 1). The existing structural drawings indicate that the building is 52'-6" long by 231'-6" wide and 101'-9" tall. Gravity loads are carried by the flat plate floor slabs to rectangular reinforced concrete columns, which transfer the load down to the foundations. The foundations consist of reinforced concrete pile caps on piles. The lateral force resisting system ("LFRS") in the north-south (short) direction consists of reinforced concrete shear walls. The existing structural drawings suggest that the LFRS in the east-west (long) direction relies on frame action between the floor slabs and columns.

Figure 1: Aerial view of the Riverview Condominium building (Source: Google Maps).

The existing structural drawings indicate that the elevated floor slabs have thicknesses ranging between 5 ½" - 7 ½" thick. Each elevated floor, excluding the roof slab, has a 6'-3" long cantilevered balcony slab extending out from the south face of the building along Grid Line B. The top and bottom slab rebar is specified as either #4 or #5 rebar with horizontal spacing ranging between 5"-12". Figure 2 below shows a typical floor slab plan at Level 7.

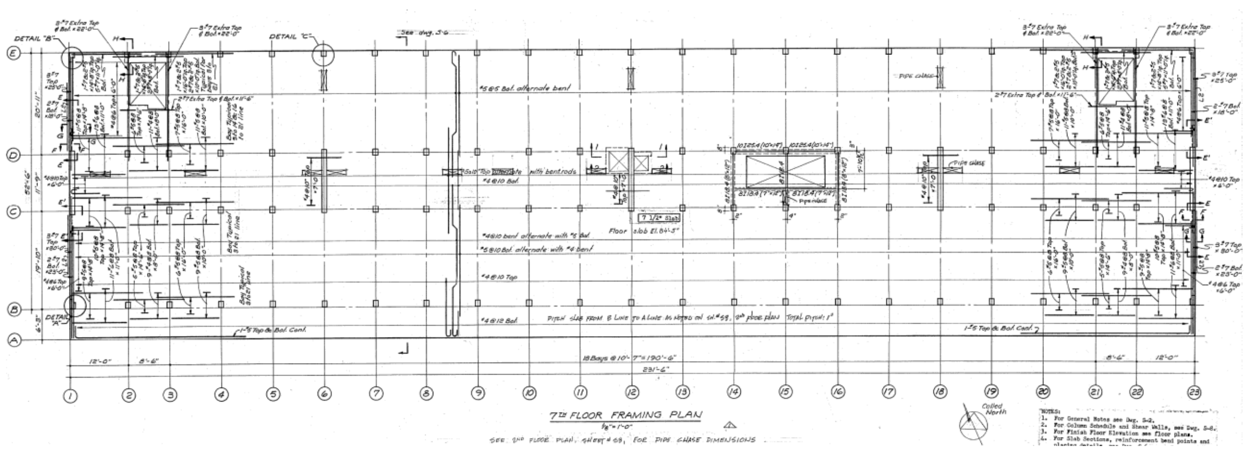


Figure 2: Representative floor slab plan at Level 7.

The existing structural drawings indicate that the typical rectangular concrete columns are 16" deep by 12" wide. The columns along the north face of the building along Grid Line E are 14" deep by 12" wide. The drawings do not indicate any drop panels where the floor slabs connect to the columns.

1.2 Relevant Parties

The relevant parties involved in the structural condition assessment and analysis of the Riverview Condominium building as described in this report are listed in Table 1 below.

Table 1: Relevant Parties.

Role	Name
Building owner	Riverview Condominium Board
Building authority	Inspectional Services Department, City of Cambridge
Property manager	Thayer & Associates, Inc.
Engineer of Record ("EOR") for structural condition assessment and analysis	Simpson Gumpertz & Heger ("SGH")
Structural peer reviewer	Thornton Tomasetti ("TT")

1.3 Scope of Work

The objectives of this peer review were as follows:

- Review existing design drawings for the Riverview Condominium building, focusing on the reinforced concrete flat plate floor slabs.
- Review pertinent codes and guidelines.
- Conduct site visits to perform independent GPR scanning at representative floor slab areas and visually observe exposed floor slab areas.
- Review field data obtained by SGH, including but not limited to photos, GPR scanning data, and material testing data.
- Perform structural analysis of representative floor slabs.
- Review SGH's structural condition assessment and analysis of the floor slabs.
- Provide strengthening recommendations for the floor slabs, in coordination with SGH's evaluation.

2.0 Observations and Document Review

TT reviewed existing design drawings, codes, and guidelines relevant to the structural condition assessment, analysis, and strengthening of the floor slabs. Additionally, we conducted two site visits to gather representative structural information for the as-built structure, and reviewed field data provided by SGH.

2.1 Existing Design Drawings

TT reviewed the following existing design drawings, received from Thayer & Associates, Inc.:

- Architectural drawings entitled "Riverview Redevelopment," dated December 14, 1961, and prepared by Harris and Freeman, Inc. and Milton Schwartz and Associates.
- Structural drawings entitled "Riverview Apartments," dated December 14, 1961, and prepared by Nichols, Norton and Zaldastani Consulting Engineers.

The existing structural drawings indicate the following design live loads ("LL"):

- Roof……………………………………………………………..30 pounds per-square-foot ("psf")
- Residence………………………………………………………60 psf (including partitions)
- Machine rooms……………………………………..………..150 psf

The structural drawings do not explicitly indicate superimposed dead loads (“SDL”) and snow loads (“SL”).

The structural drawings specify the following concrete properties:

- 28-day concrete compressive strength for beams, structural slabs, and columns………………………………...3,000 pounds per-square-inch (“psi”)

The structural drawings specify that the top and bottom rebar in formed slabs shall have 1” protective cover, which establishes the required rebar depths within the slab thickness. The structural drawings specify that the rebar should be deformed, meeting ASTM A305, and have steel grade meeting ASTM-A15 for medium-grade billet steel, or meeting ASTM-A16 for rail steel. The rebar yield strength is 40 kips per-square-inch (“ksi”) for ASTM-A15 and ASTM-A16.

2.2 Building Code Review

The design and construction of the Riverview Condominium building was governed by the City of Cambridge Building Code, adopted on December 27, 1943. Part XXVII of the City of Cambridge Building Code encompasses reinforced concrete design. The existing structural drawings specify that the rebar detailing should have conformed to the American Concrete Institute - Manual of Standard Practice for Detailing Reinforced Concrete Structures (“ACI 315-57”).

Any retrofits to the Riverview Condominium building should conform to the Massachusetts State Building Code 780 CMR, 10th edition (“MSBC-10”), which adopts and amends the 2021 International Building Code (“IBC 2021”) and the 2021 International Existing Building Code (“IEBC 2021”). Other relevant codes and guidelines for the structural condition assessment, analysis, and strengthening of the floor slabs include:

- ASCE/SEI 7-16 – Minimum Design Loads and Associated Criteria for Buildings and Other Structures
- ACI 318-19 – Building Code Requirements for Structural Concrete
- ACI 437R-19 – Strength Evaluation of Existing Concrete Buildings
- ACI 562-21 – Assessment, Repair, and Rehabilitation of Existing Concrete Structures – Code and Commentary
- ASTM C39 – Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
- ASTM A370 – Standard Test Methods and Definitions for Mechanical Testing of Steel Products
- ANSI/AISC 360-16 – Specification for Structural Steel Buildings

2.3 Site Visits

TT conducted an initial site visit on December 6, 2023 to perform GPR scanning of a representative area of the Level 7 floor slab within Unit 705. See **Appendix A** for our letter to Thayer & Associates, Inc. dated December 27, 2023 summarizing our GPR scanning results, site photos, and structural analysis results. Overall, our GPR scanning confirmed that the top rebar layer at representative Level 7 slab-to-column locations is set at substantially lower depths than specified in the existing structural drawings.

TT performed a second site visit on June 26, 2024 with SGH to view exposed floor slab areas on multiple levels as part of their limited exploratory program. We observed slab cracks in the vicinity of several columns along Grid Lines B and E (Figure 3). Various core holes are located around the columns along Grid

Lines B and E. TT did not observe exposed slab-to-column connections along Grid Lines C and D. We observed evidence of concrete spalls and rebar corrosion at the floor slabs along Grid Line E (Figure 4).

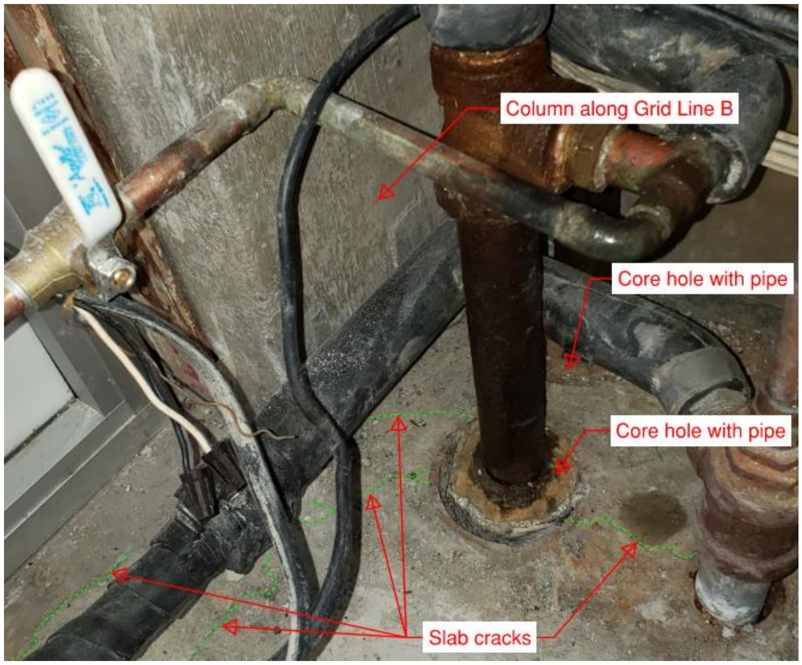


Figure 3: Visually identifiable slab cracks around a column along Grid Line B. At least two core holes are potentially positioned along the critical slab section for punching shear.

Figure 4: Concrete spalls and rebar corrosion near slab edge along Grid Line E.

TT did not observe any apparent evidence of cracking in the drywall finishes or plaster ceilings that would suggest excessive floor deflections. We discerned floor slab deflections at Level 7 within the main corridor of Unit 705 between Grid Lines D and E. We did not obtain explicit deflection measurements of the slab deflections, however the corridor floor exhibited noticeable sloping.

See **Appendix B** for photographs taken by TT.

2.4 Information Provided by Simpson Gumpertz & Heger

SGH shared their comprehensive GPR scanning results which included the top rebar depths at most of the slab-to-column connections. The scanning results confirm that the top rebar is often set at lower depths than specified in the existing structural drawings. The scanning results are in general agreement with our GPR scanning results performed within Unit 705.

SGH also shared structural material properties obtained from their compression testing of concrete core samples and tension testing of steel rebar samples. Their concrete compressive strength testing followed ASTM C39 and ACI 562. Their steel rebar tension testing followed ASTM A370. SGH determined that the as-built concrete design compression strength is 1,960 psi, far below the compression strength of 3,000 psi specified in the existing structural drawings. The tested rebar yield strength was found to meet the as-designed yield strength of 40 ksi.

Overall, TT agrees with SGH's structural condition assessment methods. Their concrete evaluation and testing appear to have followed appropriate published standards by ASTM and the American Concrete Institute.

2.5 Punching Shear

Punching shear is the tendency of a reinforced concrete slab to fail by shearing in a radial pattern around the column supports under the gravity loads. Slab loads are transferred to a supporting column by a combination of internal shear forces and unbalanced bending moments. The shear forces directly induce shear stresses on a critical slab section around the column. Also, a fraction of the unbalanced bending moments contributes shear stresses on the critical section. Following ACI 318-19, the critical slab section around the column is located *d/2* from the face of the column, where *d* is the effective depth of the top rebar in tension with respect to the bottom surface of the slab in compression (Figure 5). The shear stresses at the critical section must not exceed the slab shear strength, otherwise punching shear failure will occur. That is, the punching shear demand-to-capacity ratio ("DCR") must not exceed 1.0. IEBC 2021 allows for a five percent higher critical DCR threshold for strengthening (punching shear DCR must not exceed 1.05) for existing structures.

Top rebar placed lower in the slab at slab-to-column connections reduces the rebar depth *d* and therefore decreases the critical slab section. This in turn increases the shear stresses at the critical section. If the top rebar is placed too low, then the shear stresses at the critical slab section will exceed the slab shear strength and cause punching shear failure (DCR exceeds 1.0, or, for existing structures, DCR exceeds 1.05). TT considered the as-built top rebar depths at the slab-to-column connections from SGH's GPR scanning. We conservatively considered the minimum top rebar depth in calculating the punching shear strength at locations where multiple top rebar depths were measured.

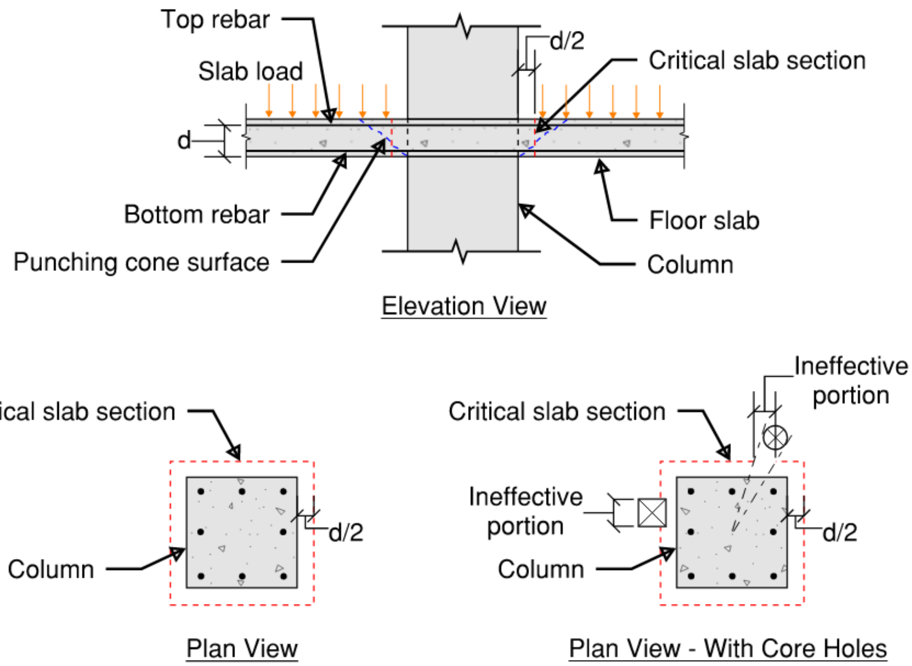


Figure 5: Critical slab section for punching shear at slab-to-column connection.

The presence of core holes positioned along the critical section is another factor that decreases the critical slab section, as depicted in Figure 5. TT did not consider the influence of reduced critical sections from core holes on punching shear capacity due to the unique conditions at each slab-to-column connection.

3.0 Punching Shear Analysis

3.1 Methodology

TT performed structural analysis for most of the floor slabs at the Riverview Condominium building. We used Computer and Structures Inc. SAFE structural analysis software to model Levels 2, 4, 6, 7, and 8. TT built the floor models incorporating structural information from the existing design drawings, and as-built conditions from our site observations and SGH's field data. As-built conditions included the floor slab top rebar depths at the slab-to-column connections and the as-built concrete design compression strength. Thin shell elements were used to model the floor slabs. Design dead load ("DL"), including structure self-weight and superimposed dead load, floor live load, and snow load, were applied to the models and the punching shear DCRs were determined at the slab-to-column connections using linear elastic structural analysis. The load and resistance factor design ("LRFD") methodology was used for the strength checks. We supplemented the computer analysis with hand calculations using Excel spreadsheets.

In coordination with SGH, we classified the slab-to-column connections into two Categories based on the punching shear DCRs (Table 2).

Table 2: Slab-to-Column Connection Categories.

Category	Punching Shear DCR	Strengthening Requirement
1	DCR > 1.05	Strengthening required
2	DCR ≤ 1.05	Strengthening not required

TT evaluated the building for gravity loads only; we did not analyze the building for lateral loads including wind and seismic.

3.2 Material Properties

TT considered the structural material properties listed in Table 3 based on material testing data shared by SGH.

Table 3: Structural Material Properties.

Structural component	Material type	Strength	Weight
Flat plate floor slabs	Concrete	Compressive strength, f_{ceq} = 1,960 psi	145 pounds per-cubic-foot ("pcf")[1]
Rebar	Steel	Yield strength, F_y = 40 ksi	5 pcf[2]

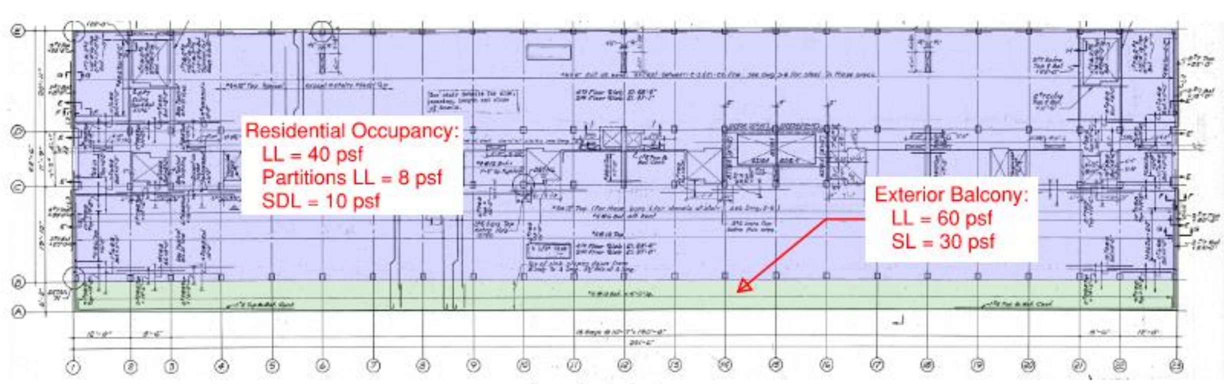
1. Assumed for normal-weight concrete.
2. Assumed rebar weight per cubic foot of concrete.

TT considered the following strength reduction factor for punching shear following ACI 318-19: $\Phi_v = 0.8$.

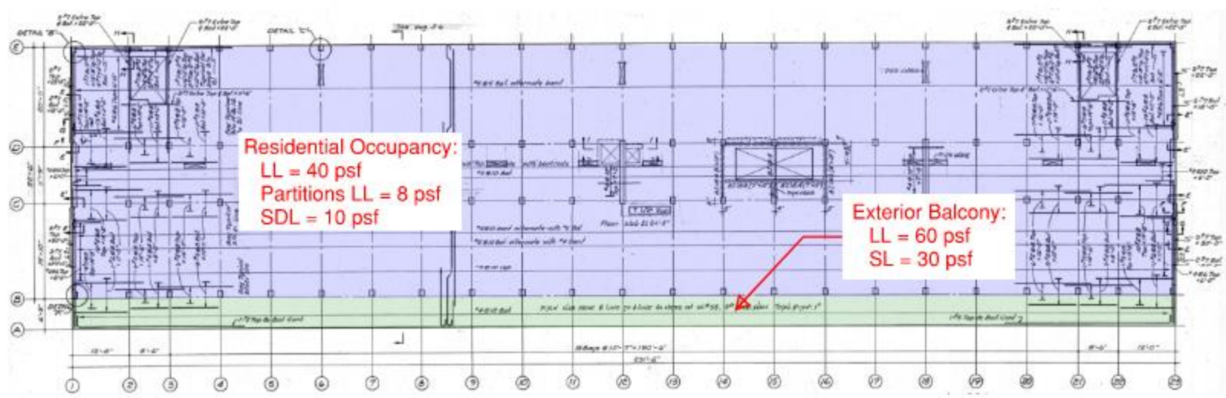
3.3 Loading Assumptions

TT deduced that the actual live load on the floor slabs is markedly lower than the design live load (60 psf), based on our observations of representative apartment units and corridors in the Riverview Condominium building. We considered design live load of 40 psf for residential occupancy following IBC 2021 and ASCE/SEI 7-16 as referenced by MSBC-10. We considered partition live load of 8 psf. Also, we considered superimposed dead load of 10 psf to account for floor finishes, plaster ceilings, and mechanical, electrical, and plumbing ("MEP") weight allowance. The floor slab self-weight was intrinsically accounted for in the models. At the exterior balconies we considered live load of 60 psf following IBC 2021, which is identical to the design live load specified in the existing structural drawings. Also, we considered uniform snow load of 30 psf and no superimposed dead load at the balconies. Finally, at the roof areas we considered roof live load ("LLr") of 20 psf, uniform snow load of 30 psf, and SDL of 10 psf (except at roof areas over exterior balconies).

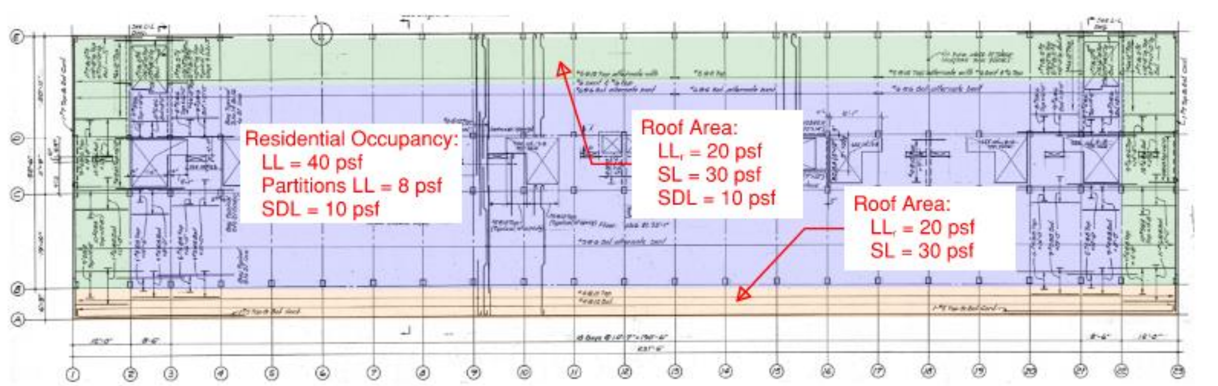
Figure 6 below shows the floor surface loads considered by TT.



Levels 2, 4, 6



Level 7



Level 8

Figure 6: Floor surface loads.

The following LRFD load combinations were considered in the analysis following ASCE/SEI 7-16:

1. 1.4DL
2. 1.2DL + 1.6LL
3. 1.2DL + 1.6LL + 0.5LL_r
4. 1.2DL + 1.6LL + 0.5SL
5. 1.2DL + 1.6LL_r + LL
6. 1.2DL + 1.6SL + LL

3.4 Punching Shear Classification

See **Appendix C** for a summary of our structural analysis results at representative floor slabs classifying the slab-to-column connections into Categories 1 and 2 based on the punching shear DCRs.

TT's structural analysis indicates that a substantial quantity of the as-built slab-to-column connections are classified as Category 1 requiring strengthening. The remaining slab-to-column connections are either classified as Category 2 not requiring strengthening, or do not have as-built structural information and were not evaluated.

We further evaluated the Category 1 slab-to-column connections having DCRs ranging between 1.05 - 1.5 by considering redistribution of the unbalanced bending moments per ACI 318. This redistribution occurs as slab-to-column connections achieve their maximum negative bending moment capacities and any additional imposed loads are resisted by reserve positive bending moment strength at the slab mid-spans. As a result, unbalanced bending moments at the slab-to-column connections are reduced, thereby decreasing the shear stresses on the critical slab sections and providing greater punching shear strength.

TT determined that an exceptionally low quantity of Category 1 slab-to-column connections with DCRs between 1.05 - 1.5 could be re-classified to Category 2 considering bending moment redistribution.

4.0 Discussion

TT agrees with SGH's structural condition assessment and analysis. Our identified slab-to-column locations requiring strengthening are in general alignment with SGH's analysis results.

Inadequate punching shear strength is the primary failure mode requiring remediation. Based on our evaluation of a representative floor slab area at Level 7, we judge that the floor slabs have adequate flexural strength when considering yield line analysis, as described in our letter issued to Thayer & Associates, Inc. dated December 27, 2023 (**Appendix A**).

The floor slabs likely have not failed due to the actual live load being much less than the design live load. In this sense, the floor slabs have been load tested since their construction in 1961. However, the as-built slab-to-column strengths do not satisfy current codes and standards, and should be strengthened as per SGH's reinforcement recommendations.

Visual evidence of slab cracks emanating around several slab-to-column connections, combined with the presence of core holes reducing the critical slab sections at numerous slab-to-column connections, underlines the importance of strengthening the connections. Also, visual evidence of rebar corrosion at the floor slabs along Grid Line E amplifies the urgency of implementing immediate repairs. As such, TT concurred with SGH's October 31, 2024 recommendation to the City of Cambridge Inspectional Services Department to either:

- Implement a live load restriction of 20 psf or less and install emergency shoring at critical slab-to-column connections, followed by permanent strengthening of SGH's identified locations with DCRs equaling or exceeding 1.05.
- Immediately evacuate the Riverview Condominium building to allow for permanent strengthening of SGH's identified slab-to-column connections with DCRs equaling or exceeding 1.05.

5.0 Conclusions and Recommendations

The reinforced concrete flat plate floor slabs at the Riverview Condominium building have reduced as-built capacity due to the following factors:

- Floor slab top rebar is set lower than specified in the existing structural drawings, contributing to reduced punching shear capacity and flexural capacity.
- Floor slab as-built concrete design compression strength is substantially lower than specified in the existing structural drawings, contributing to reduced punching shear capacity.
- Presence of core holes in the slab around the columns, reducing the punching shear critical slab sections.
- Evidence of rebar corrosion, specifically near the slab edge along Grid Line E.

These factors are in alignment with SGH's conclusions.

The floor slab punching shear capacity could be permanently increased by post-installing steel collars to the required columns to increase the critical slab sections around the columns. Alternatively, new reinforced concrete drop panels could be formed and poured at the required columns to increase the critical slab sections around the columns.

Shoring and strengthening should follow engineering plans and details prepared by SGH as they are the EOR for the project. TT recommends further evaluation of the Riverview Condominium building to resist lateral loads including wind and seismic.

6.0 Thornton Tomasetti Statement Regarding SGH's Report

TT has reviewed SGH's final report dated February 7, 2025. We agree with SGH's structural condition assessment and analysis. Our identified slab-to-column locations requiring strengthening are in general agreement with SGH's locations shown in their Appendix J – Conceptual Repair Documents. Any discrepancies between our identified strengthening locations and SGH's locations may be primarily attributed to our considered effective slab thicknesses for computing the punching shear strengths. SGH's concept-level repairs call for reinforced concrete drop panels, which will simultaneously increase the punching shear strength and negative bending moment strength at the slab-to-column connections.

Appendix A: Thornton Tomasetti's Initial Floor Slab GPR and Strength Review Letter

Thornton Tomasetti

Via email: *djohnson@thayerassociates.com*

December 27, 2023

Dwight Johnson
Senior Vice President
THAYER & ASSOCIATES
1812 Massachusetts Avenue
Cambridge, MA 02140

RE: FLOOR SLAB GPR AND STRENGTH REVIEW – RIVERVIEW CONDOMINIUMS – 221 MT. AUBURN ST., CAMBRIDGE, MA

Dear Mr. Johnson,

Per your request, Thornton Tomasetti, Inc. ("TT") has performed an evaluation of a typical reinforced concrete flat plate floor slab for strength at the Riverview Condominium building located at 221 Mt. Auburn St., Cambridge, MA. We performed representative ground penetrating radar ("GPR") scans on the 7th Floor within Unit 705 to determine the as-built slab thickness and steel reinforcement ("rebar") depths and spacings. We also reviewed material testing data and GPR scan data obtained by Simpson Gumpertz & Heger ("SGH"). Additionally, we gathered structural information from existing structural drawings entitled "Riverview Apartments," dated 12/14/1961, and prepared by Nichols, Norton and Zaldastani Consulting Engineers. To TT's knowledge, existing rebar shop drawings are not available for review. Finally, we performed structural analysis to determine the flexural and punching shear demand-to-capacity ratios ("DCRs") of a representative area of the 7th Floor slab within Unit 705 considering the as-built slab conditions. Our analysis covered the floor slab at the exterior balcony and the interior column bays (Figure 1). Our code-based evaluation determined that the slab has adequate flexural strength but inadequate punching shear strength considering the as-built slab conditions (excluding the as-built concrete compressive strength).

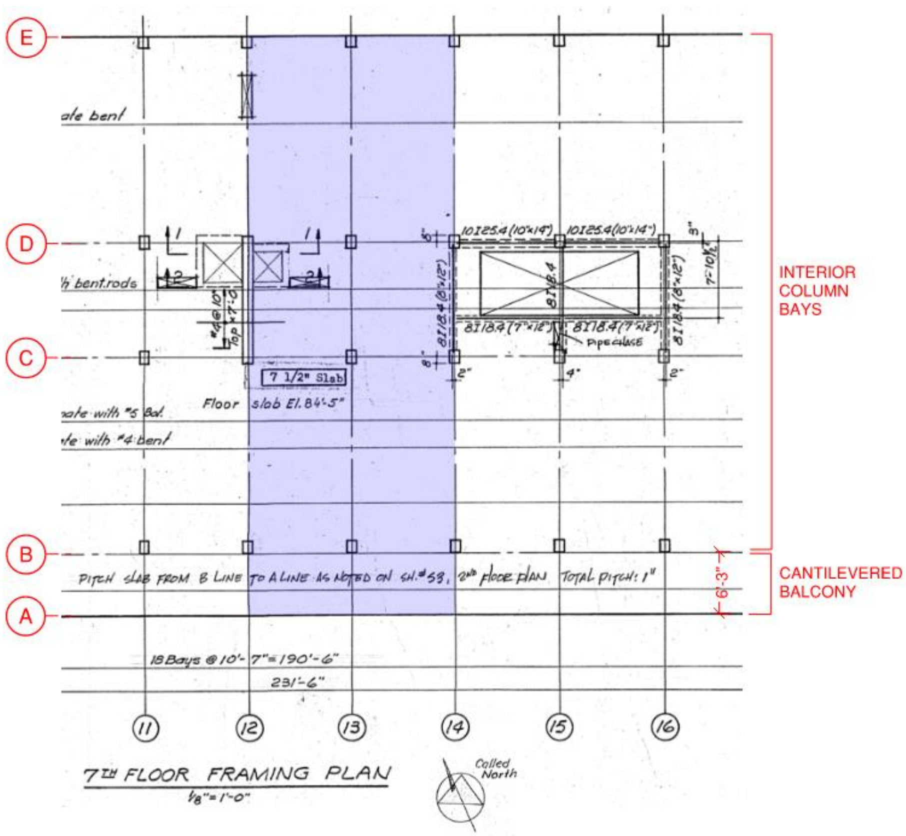


Figure 1. Part plan view of 7th Floor slab indicating representative area evaluated for strength.

BUILDING DESCRIPTION

The Riverview Condominium building is an 8-story, reinforced concrete structure with flat plate floor slabs, constructed circa 1961. The existing structural drawings indicate that the building is 52'-6" long x 231'-6" wide x 101'-9" tall. Gravity loads are carried by the flat plate floor slabs through two-way action to the rectangular reinforced concrete columns, which transfer the load down to the foundations. The foundations consist of reinforced concrete pile caps on piles. The lateral force resisting system ("LFRS") in the north-south (short) direction consists of reinforced concrete shear walls. The existing structural drawings suggest that the LFRS in the east-west (long) direction relies on frame-action between the floor slabs and columns.

The existing structural drawings indicate that the elevated floor slabs have thicknesses ranging between 5 ½" - 7 ½" thick. Each elevated floor, excluding the roof slab, has a 6'-3" long cantilevered balcony slab extending out from the south face of the building along Grid Line B. The top and bottom slab rebar is specified as either #4 or #5 rebar with horizontal spacing ranging between 5"-12".

The 7th Floor slab is specified to be 7 ½" thick. The drawings indicate that the top of the balcony slab linearly pitches away from the south building face such that the slab thickness reduces to 6 ½" at the balcony edge along Grid Line A.

The existing structural drawings indicate that the typical rectangular concrete columns are 16" deep x 12" wide. The columns along the north face of the building along Grid Line E are 14" deep x 12" wide. The drawings do not indicate any drop panels where the floor slabs connect to the columns.

GPR SCANS

TT conducted a site visit on 12/06/2023 to visually review the general floor slab conditions and to observe the overall building characteristics. We implemented multiple GPR line scans along the topside of the 7th Floor slab. Representative photos from our site visit are shown in **Appendix A**. The general locations and results of our GPR scans are summarized in **Appendix B**.

Our scans indicate that the interior slab thickness ranges between 7 1/8" – 7 ½". We field measured the edge of balcony slab to be 7 ½" thick instead of 6 ½" thick, suggesting that the as-built condition of the balcony slab does not have a pitched surface per the existing drawings.

The scans show that the top and bottom rebar horizontal spacing ranges between 5" - 12", and is generally in conformance with the existing structural drawings. GPR does not allow for detecting the rebar diameter; therefore, we relied on the existing structural drawings for the rebar size.

We identified several areas where the as-built top rebar depths are set lower than specified in the existing structural drawings. The existing drawings specify slab rebar to have 1" minimum protective cover. This implies that the top rebar should be nominally set at a depth of 1" plus half the rebar diameter with respect to the top of slab. Based on our scan data, the as-built top rebar depths are set much lower, between 3"-4" deep. The placement of the top rebar is essential for the slab strength at negative bending moment regions in the vicinity of columns. The depth of the top rebar also effects the slab punching shear strength at the columns. Top rebar that is set lower in the slab generally reduces both the slab negative flexural strength and the punching shear strength.

Our scans show that the bottom rebar depths are set between 5 ½" – 6 ¼" deep with respect to the top of slab. The existing structural drawings imply that the bottom rebar should be nominally set at a depth of 1" plus half the rebar diameter with respect to the bottom of slab. The as-built bottom rebar depths would be generally acceptable for the bottom rebar placement. The placement of the bottom rebar is important for the slab strength at positive bending moment regions between columns.

The as-built slab conditions are summarized in Figure 2 for a typical section of the 7th Floor slab.

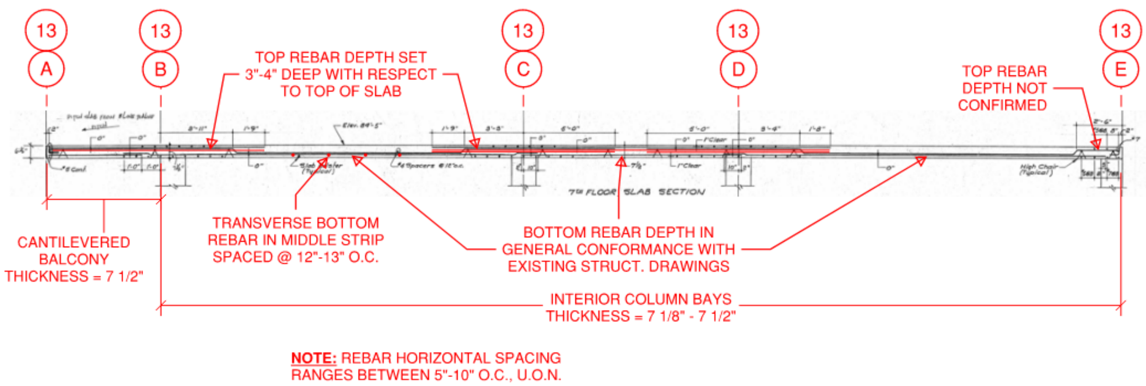


Figure 2. Typical section of 7th Floor slab indicating as-built conditions.

Our GPR scan data, including slab thickness, rebar depth, and rebar spacing, is generally in agreement with the GPR scan data obtained by SGH at the 7th Floor exterior balcony slab.

SLAB ANALYSIS

TT performed structural analysis of the 7th Floor slab within Unit 705 based on the existing structural drawings and the as-built slab conditions obtained from our GPR scans and SGH's material testing data. We performed the analysis in accordance with Massachusetts Building Code, CMR 780, 9th Edition adapting the 2015 *International Building Code* ("IBC") and ACI 318-14 *Building Code Requirements for Structural Concrete*, as this is the current code for Massachusetts as of the date of this letter report. We performed the analysis following the Load and Resistance Factor Design ("LRFD") methodology.

Elastic strip methods for two-way slabs are conventionally used to analyze the lower-bound slab capacity. It is known that elastic strip methods are inherently conservative for capturing the true behavior of two-way slabs. This is because elastic methods do not fully account for the two-way redistribution of internal slab forces from negative moment regions to positive moment regions as the slab begins to crack and exhibits stiffness reductions at the negative moment regions. The slab will form plastic hinges (yield lines) along a pattern that tends to minimize the potential energy of the slab as the gravity loading on a two-way slab approaches the maximum possible magnitude (i.e., ultimate load). This plastic slab behavior can be evaluated using yield line analysis to determine the upper-bound slab capacity.

Elastic Analysis Methods

We first analyzed the slab in accordance with two versions of elastic strip methods for analyzing two-way slabs. We evaluated a representative column strip along Grid Line 13 by modeling the column strip as a 2-D frame using the structural analysis software CSi SAP2000 in the first method. (See **Appendix C**). Next, we evaluated a representative slab area bounded by Grid Lines A-E:11-16 by modeling the slab using the structural analysis software CSi SAFE in the second method. (See **Appendix D**). We obtained the slab internal flexural and shear forces for comparison to the slab flexural and shear strength, respectively, from both methods (DCRs). The slab strength was calculated following ACI 318-14, the existing structural drawings, and the as-built slab conditions.

Table 1 below lists the service-level gravity loads assumed in our analysis.

Table 1. Gravity loads acting on the 7th Floor slab.

Gravity Load	***Magnitude***
Superimposed dead load (SDL)	15 psf
Concrete slab self-weight (DL)	150 pcf
Live load (LL)[1]	60 psf

1. Based on existing structural drawings “Riverview Apartments,” dated 12/14/1961, and prepared by Nichols, Norton and Zaldastani Consulting Engineers.

The analysis considered the following LRFD gravity load combinations following ACI 318-14, Section 5.3:

(1) 1.4DL
(2) 1.2DL + 1.6LL

Table 2 below lists the slab structural properties used in our analysis. Testing data for the as-built concrete compressive strength was unavailable. As a result, we considered the 28-day concrete compressive strength specified in the existing structural drawings. We used the rebar steel yield strength obtained from SGH’s material testing data.

Table 2. Structural properties for 7th Floor slab.

Parameter	***Value***
Concrete compressive strength[1], f'_c	3,000 psi
Rebar steel yield strength[2], F_y	40 ksi
Rebar size[1]	#4 or #5
Rebar spacing[3]	5” or 10”
Slab thickness[3]	7 1/8” – 7 ½”

1. Based on existing structural drawings “Riverview Apartments,” dated 12/14/1961, and prepared by Nichols, Norton and Zaldastani Consulting Engineers.
2. Based on SGH’s rebar testing data, dated 06/27/2023.
3. Based on TT’s GPR scan data (**Appendix B**).

The elastic strip methods determined that the slab has inadequate negative flexural strength at three locations along the representative column strip when considering the as-built slab conditions (Figure 3). Of note, the cantilevered balcony slab has sufficient negative flexural strength where the slab joins the south column line along Grid Line B.

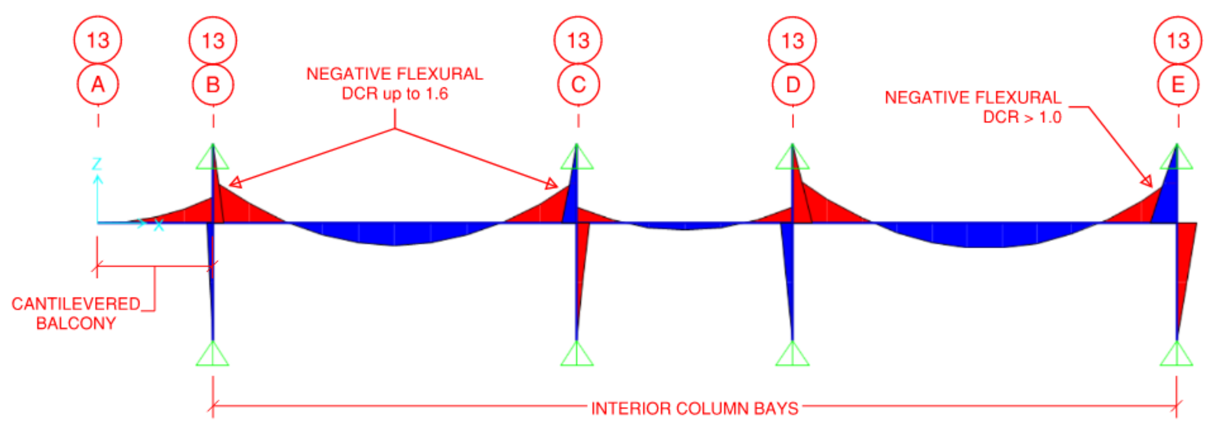


Figure 3. Moment diagram of column strip along Grid Line 13 at 7th Floor showing locations with inadequate negative flexural strength.

Punching shear refers to the tendency of a slab to fail by shearing in a radial pattern around the column supports due to the applied loads. Per ACI 318-14, punching shear for a rectangular column must be checked at the critical section located *d/2* from the face of the column, where *d* is the depth of the rebar from the bottom of slab. A fraction of unbalanced slab moment is resisted at the columns by flexure and the rest is resisted by shear action.

TT evaluated the punching shear DCRs using CSi SAFE for both the as-designed condition according to the existing structural drawings and the as-built condition. Figure 4 indicates the slab-to-column connections having inadequate punching shear strength for the as-designed condition. All slab-to-column connections were found to have insufficient punching shear strength for the as-built condition.

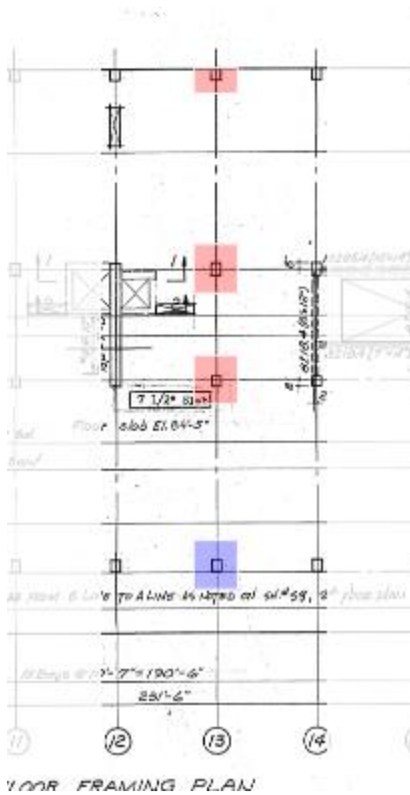


Figure 4. Part plan view of representative area of 7th Floor slab indicating slab-to-column connections with inadequate punching shear strength for as-designed condition (highlighted in red). All slab-to-column connections fail in punching shear for the as-built condition.

Yield Line Analysis

Next, we analyzed the slab using yield line analysis. Per ACI 318-14, commentary R8.2-1, the design of slabs may be achieved by yield line analysis after evaluating the stress conditions for shear, torsion, and flexure around the supports. The rebar will yield first in regions of high moment in yield line analysis for slabs failing in flexure. When that occurs, the yielded slab portion behaves as a plastic hinge and resists the force only equal to its plastic strength. The additional load is then redistributed to the adjacent regions, resulting in yielding of those sections. The bands in which yielding has occurred are known as yield lines. Figure 5 below shows the yield line mechanism for a typical interior column bay. The perimeter lines (AB, AC, CD, and BD) represent negative moment yield lines and the rest of the lines represent positive moment yield lines. The slab portions between the yield lines behave as a rigid plate.

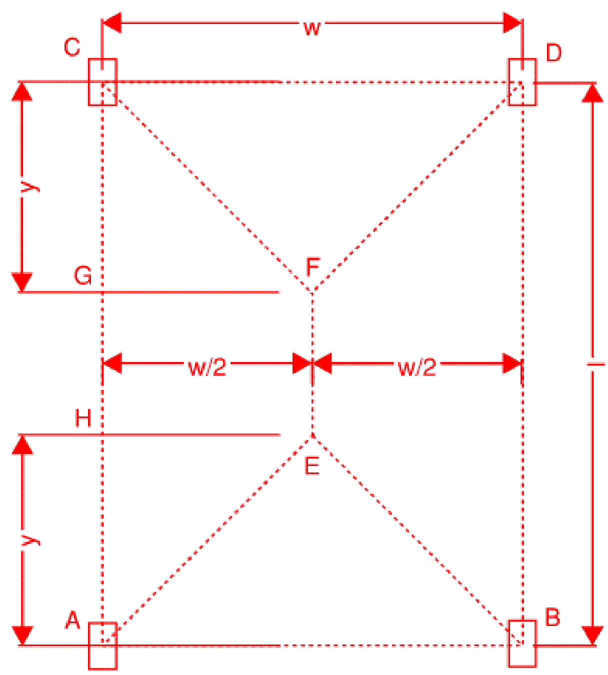


Figure 5. Representative column bay evaluated with yield line analysis.

Note that a yield line analysis uses rigid plastic theory to compute the ultimate failure loads. The deflections at the ultimate capacity of the slab may be higher than the serviceability requirements and excessive cracks may form at this stage.

TT conducted a yield line analysis for representative interior column bays of the 7th Floor slab within Unit 705 (Figure 1) and determined that the slab has adequate capacity for the assumed gravity loads.

Rebar Development Length at Cantilevered Balcony Slab

The cantilevered balcony slab has adequate negative flexural strength using the elastic strip methods. The balcony slab top rebar must have continuity through the columns along Grid Line B, or otherwise have sufficient development length into the column line. The interior column bays have sufficient capacity while allowing for the formation of a plastic hinge along the critical section at the north face of the columns along Grid Line B using the yield line analysis. TT calculated the development length for deformed #4 rebar having steel yield strength of 45 ksi following ACI 318-14, Section 25.4. We determined that the balcony slab top rebar has sufficient development length between the critical section at the south face of the columns along Grid Line B and the critical section at the north face of the columns where a plastic hinge is allowed at the interior bays

(Figure 6). This demonstrates that there is a load path for the cantilevered balcony slab to remain stable while allowing for the interior column bays to form plastic hinges.

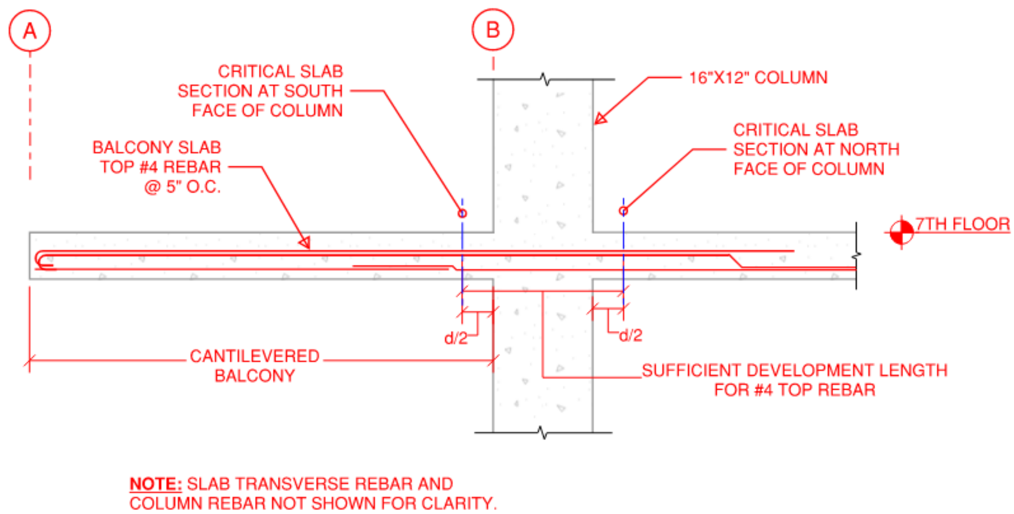


Figure 6. Development length of balcony slab top rebar passing through column line along Grid Line B.

CONCLUSIONS AND RECOMMENDATIONS

Our code-based evaluation of a representative area of the 7th Floor slab required us to employ plastic analysis methods (yield line analysis) to supplement conventional elastic strip methods for two-way slabs to demonstrate the adequacy of the slab flexural strength. While the slab has sufficient flexural capacity, increased cracking and deflections may be expected.

The slab's as-built punching shear strength is inadequate using elastic analysis using the concrete compressive strength specified in the existing structural drawings. In-situ concrete can increase slightly in compressive strength over time. Therefore, TT recommends testing concrete cores extracted from the slab for compressive strength. An increase in compressive strength will also increase the punching shear strength. TT recommends the following next steps once the in-situ compressive strength is determined via testing:

- Perform higher-order nonlinear punching shear evaluation considering redistributed unbalanced moments from plastic analysis.
- Perform evaluation of representative columns along Grid Line B to ensure that the columns have sufficient strength to resist redistributed unbalanced moment from the cantilevered balcony slab should yield lines form at the interior column bays.
- Perform deflection analysis (short-term and long-term deflection) for representative areas of the floor slabs. Perform moment-curvature analysis to determine the magnitude of

rotation that would occur should yield lines form in the slab. These two tasks are relevant for serviceability and not life-safety.

Please feel free to reach out should you have any questions or concerns.

Very truly yours,

THORNTON TOMASETTI, INC.

Lisa A. Davey

Lisa A. Davey, P.E.
Senior Principal

Sebastian B. Mendes

Sebastian Mendes, P.E.
Associate

APPENDIX A

Photographs taken by Thornton Tomasetti

Thornton Tomasetti

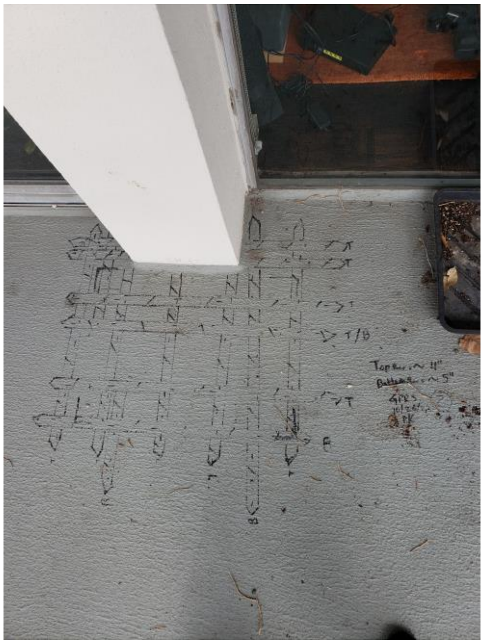


Photo 1. 7th Floor balcony slab near column B-14 (GPR markings by SGH).

Photo 2. Underside of 8th Floor balcony slab near column B-14 (GPR markings by SGH).

Photo 3. 7th Floor balcony slab between along Grid Line B.

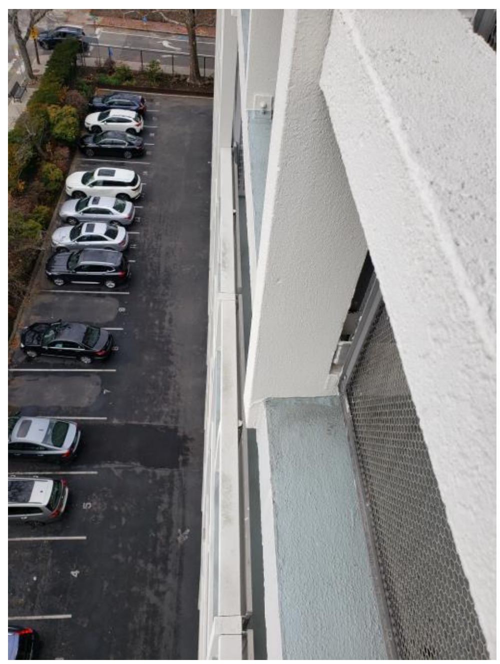


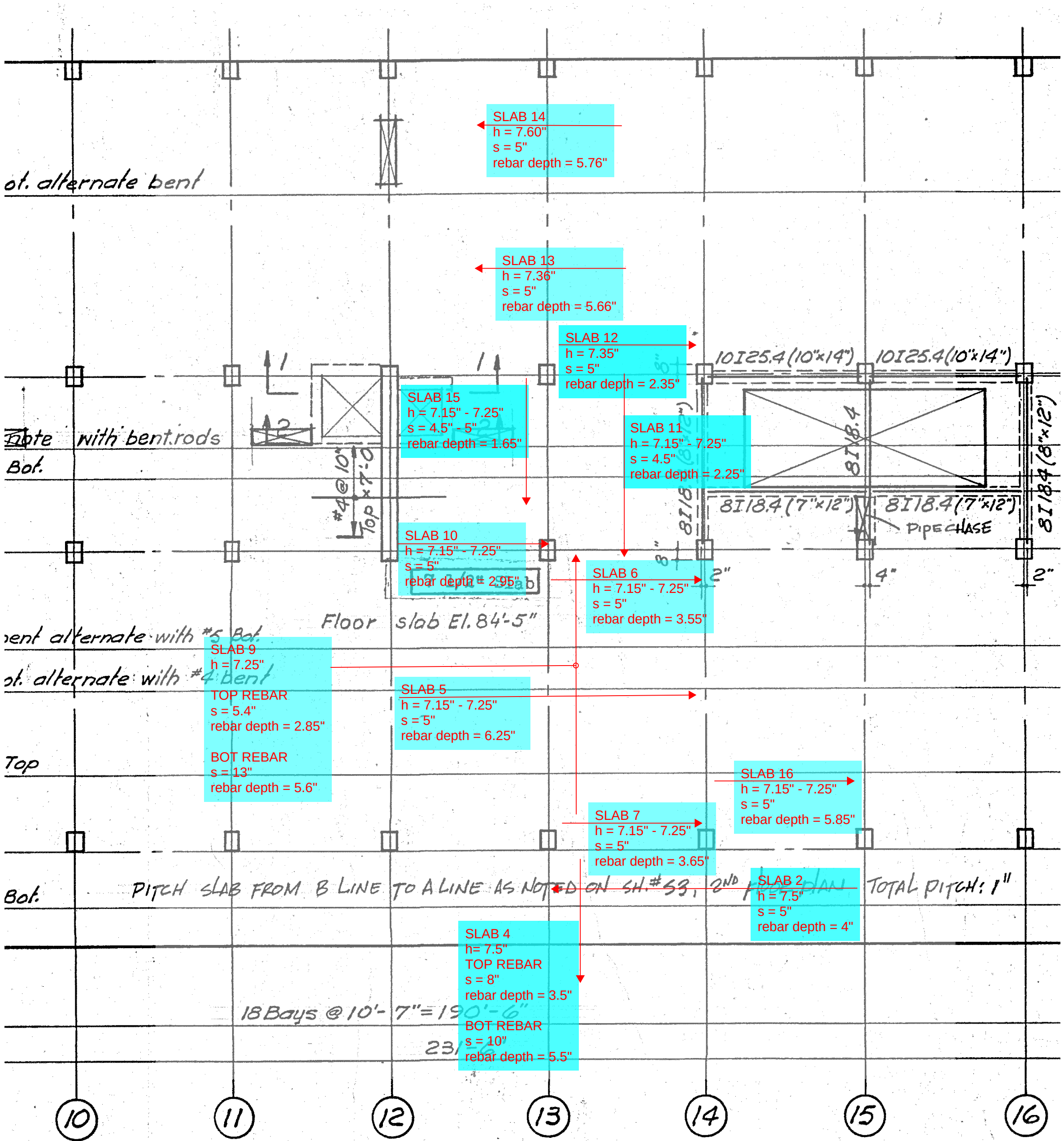
Photo 4. View of balcony slab edge from 7th Floor.

Photo 5. View of thin floor finish inside Unit 705 with exposed concrete floor slab.

Photo 6. GPR scanning inside Unit 705.

APPENDIX B

GPR SCANS



7TH FLOOR FRAMING PLAN

INDICATES GPR LINE SCAN

CALIBRATED CONCRETE DIALECTIC CONSTANT = 6.8

FLOOR FINISHES:

WOOD FLOOR = 1/4" THICK

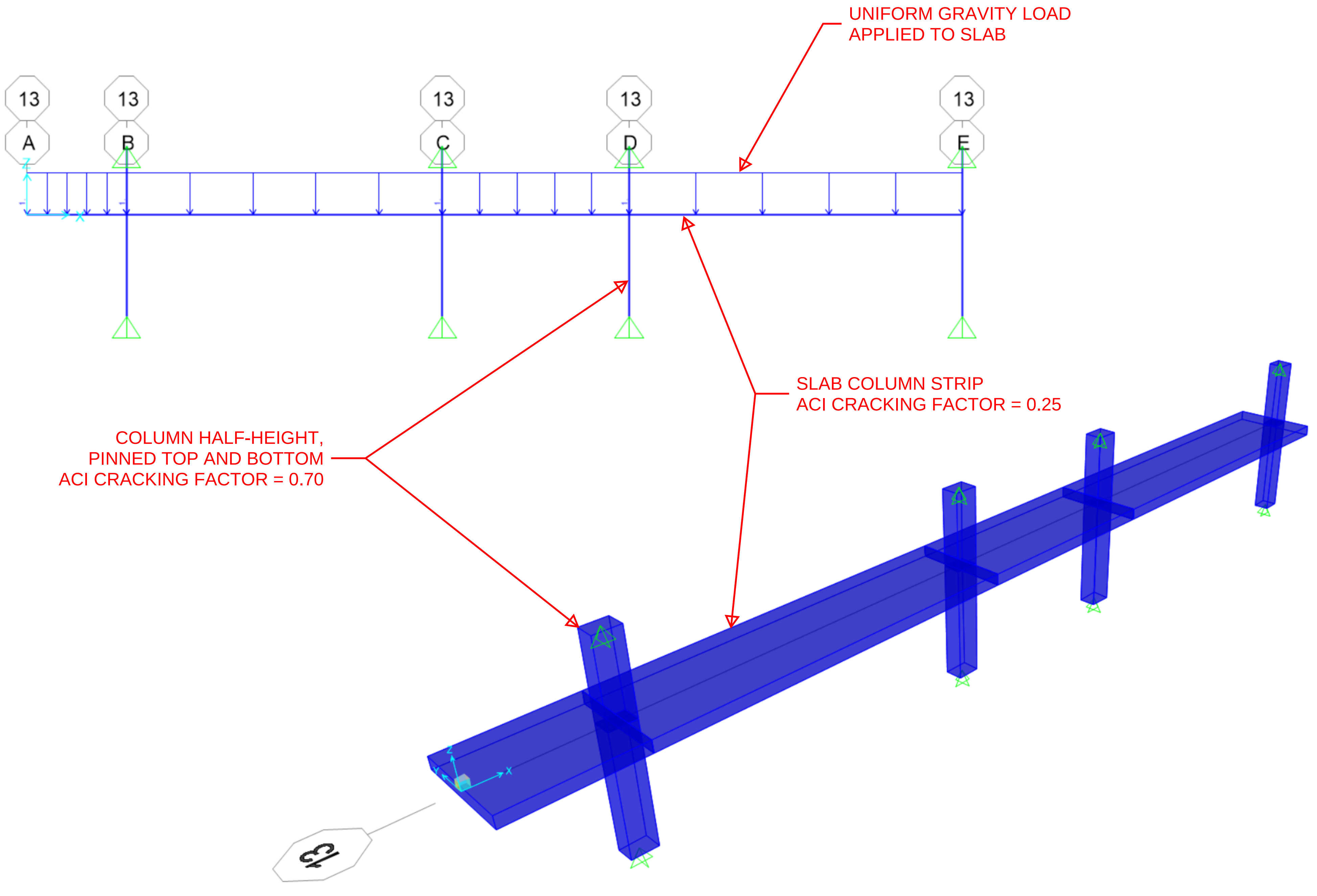
CARPET = 3/16"

Thornton Tomasetti

PROJECT:	FLOOR SLAB GPR AND STRENGTH REVIEW - RIVERVIEW CONDOMINIUMS
SUBJECT:	GPR SCANS
REFERENCE:	
BY: TT	DATE: 12/22/2023

APPENDIX C

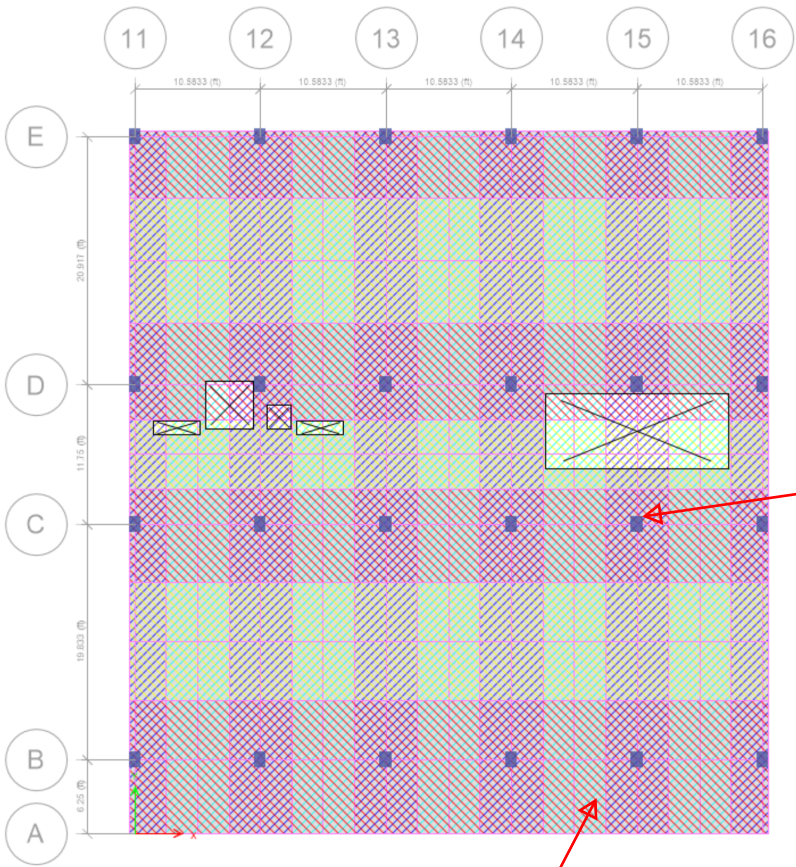
SLAB ANALYSIS WITH CSi SAP2000

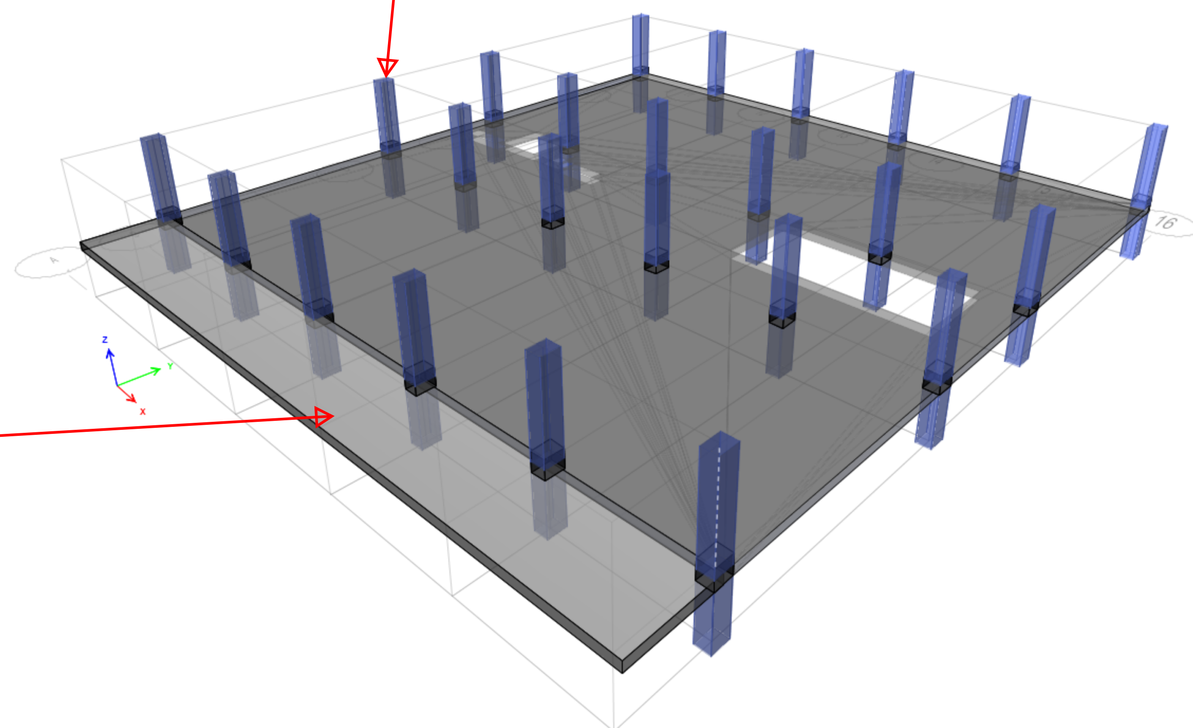


Thornton Tomasetti	PROJECT: FLOOR SLAB GPR AND STRENGTH REVIEW - RIVERVIEW CONDOMINIUMS	
	SUBJECT: CSi SAP2000 FRAME MODEL	
BY: TT / DATE: 12/21/2023	REFERENCE:	

APPENDIX D

SLAB ANALYSIS WITH CSi SAFE





Thornton Tomasetti	PROJECT: FLOOR SLAB GPR AND STRENGTH REVIEW - RIVERVIEW CONDOMINIUMS
	SUBJECT: CSi SAFE SLAB MODEL
BY: TT DATE: 12/21/2023	REFERENCE:

Appendix B: Photographs Taken by Thornton Tomasetti


Photo 1: View of Unit 705 exterior balcony.


Photo 2: View of column B13 outside Unit 705.


Photo 3: Interior view of a typical column along Grid Line B.

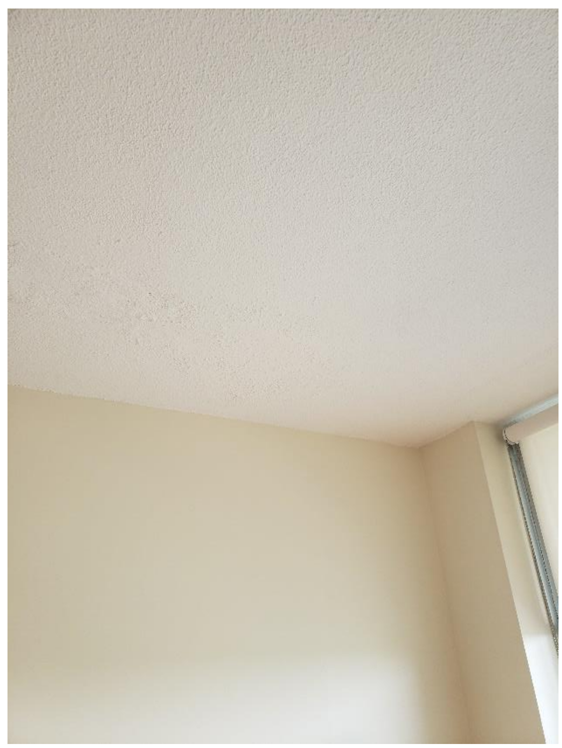


Photo 4: Typical interior wall and ceiling finishes with no cracking or deformation evident.

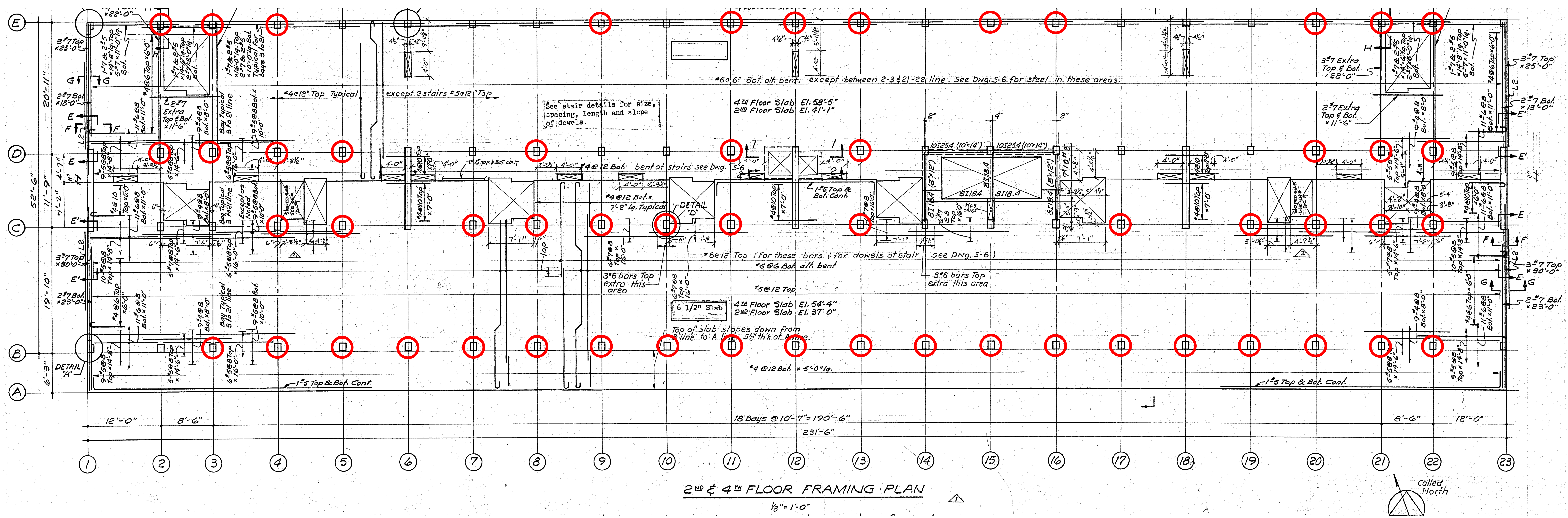
Photo 5: Concrete spalls and rebar corrosion near slab edge along Grid Line E.

Photo 6: Hairline slab cracks in vicinity of a column along Grid Line E.

Photo 7: Typical slab-to-column connection exhibiting core holes and cracks around a column.

Photo 8: Slab-to-column connection exhibiting three core holes clustered together directly adjacent to the column face.

Appendix C: Punching Shear Classification



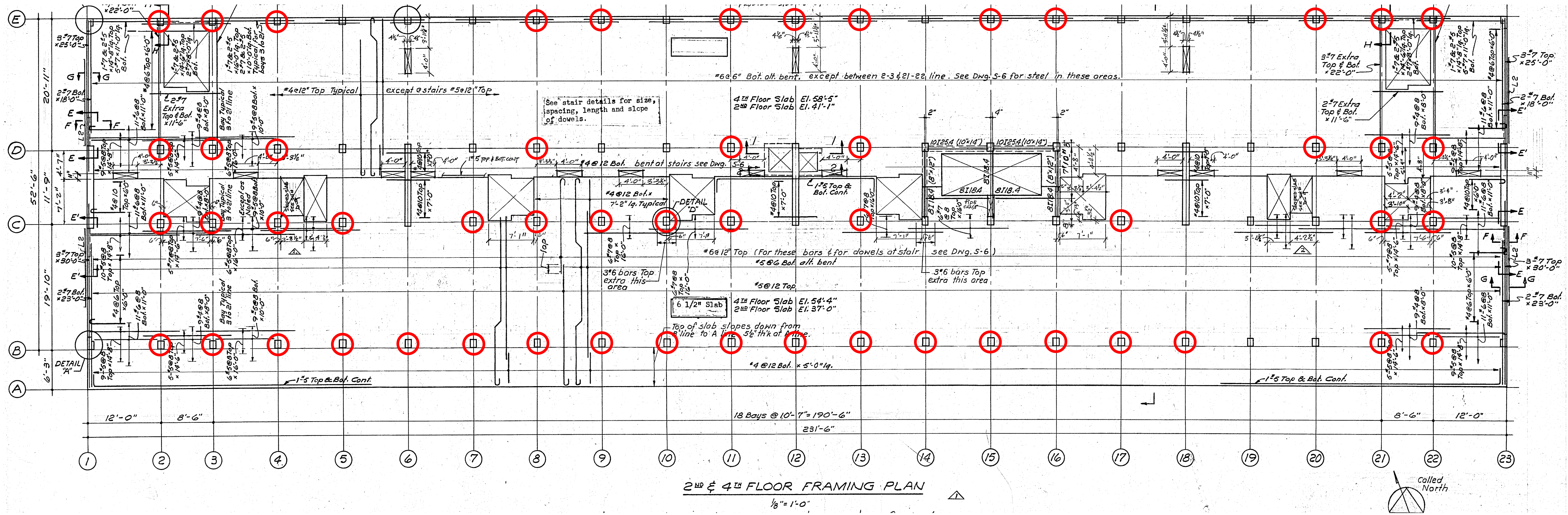
LEVEL 2

LEGEND

○	CATEGORY 1	DCR > 1.05
○	CATEGORY 2	DCR ≤1.05

NOTE: UNCIRCLED GRID INTERSECTIONS INDICATE AREAS WITH NO GPR SCANNING DATA PROVIDED BY SGH.

Thornton Tomasetti	PROJECT: RIVERVIEW CONDOMINIUMS PUNCHING SHEAR REVIEW	SUBJECT: SLAB-TO-COLUMN CONNECTION BUCKET CATEGORIES	SSK-01A
	REFERENCE: BY: TT DATE: 02/11/2025		



LEVEL 4

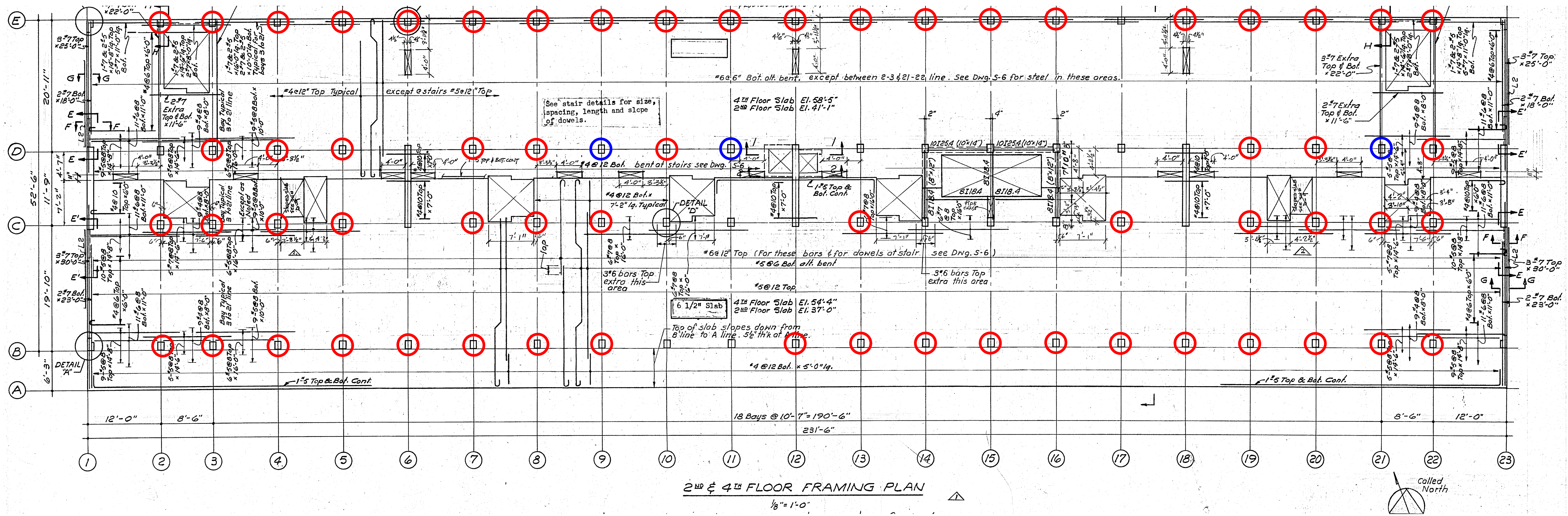
LEGEND

○ (red)	CATEGORY 1	DCR > 1.05
○ (blue)	CATEGORY 2	DCR ≤1.05

NOTE: UNCIRCLED GRID INTERSECTIONS INDICATE AREAS WITH NO GPR SCANNING DATA PROVIDED BY SGH.

Thornton Tomasetti

PROJECT: RIVERVIEW CONDOMINIUMS PUNCHING SHEAR REVIEW	SUBJECT: SLAB-TO-COLUMN CONNECTION BUCKET CATEGORIES
REFERENCE:	BY: TT
DATE: 02/11/2025	SSK-01B



LEVEL 6

LEGEND

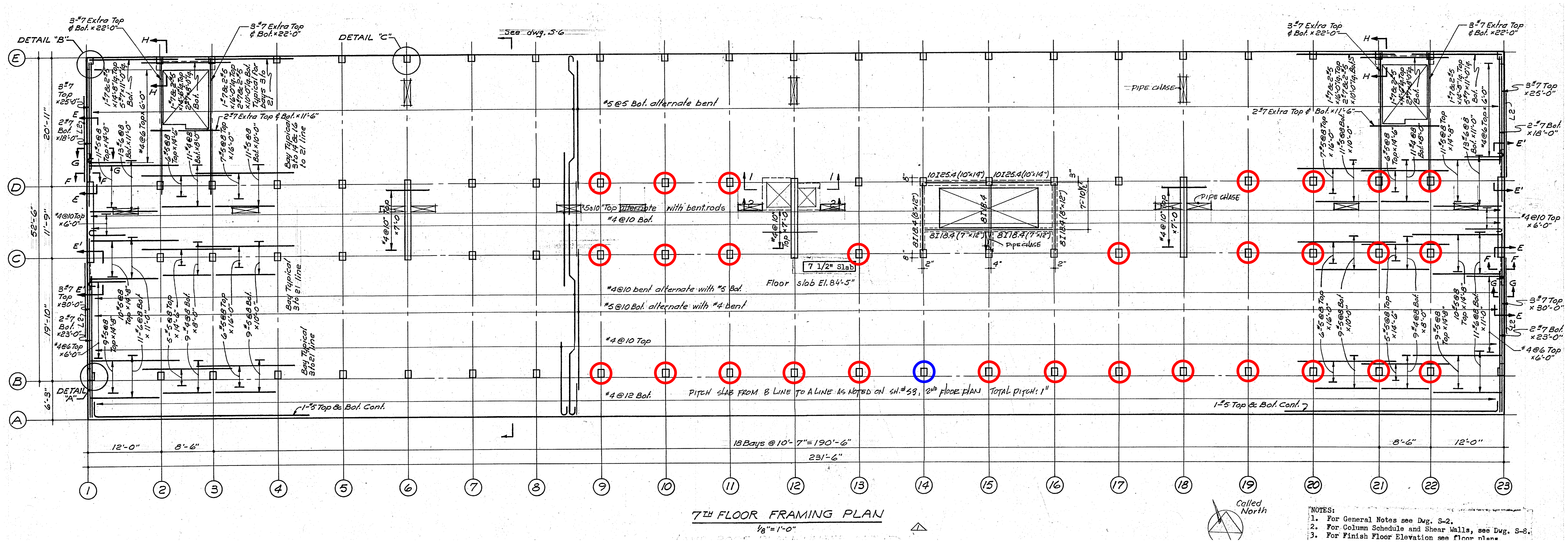
Red circle	CATEGORY 1	DCR > 1.05
Blue circle	CATEGORY 2	DCR ≤1.05

NOTE: UNCIRCLED GRID INTERSECTIONS INDICATE AREAS WITH NO GPR SCANNING DATA PROVIDED BY SGH.

Thornton Tomasetti

PROJECT:	RIVERVIEW CONDOMINIUMS PUNCHING SHEAR REVIEW
REFERENCE:	
BY:	TT
DATE:	02/11/2025
SUBJECT:	SLAB-TO-COLUMN CONNECTION BUCKET CATEGORIES

SSK-01C



LEVEL 7

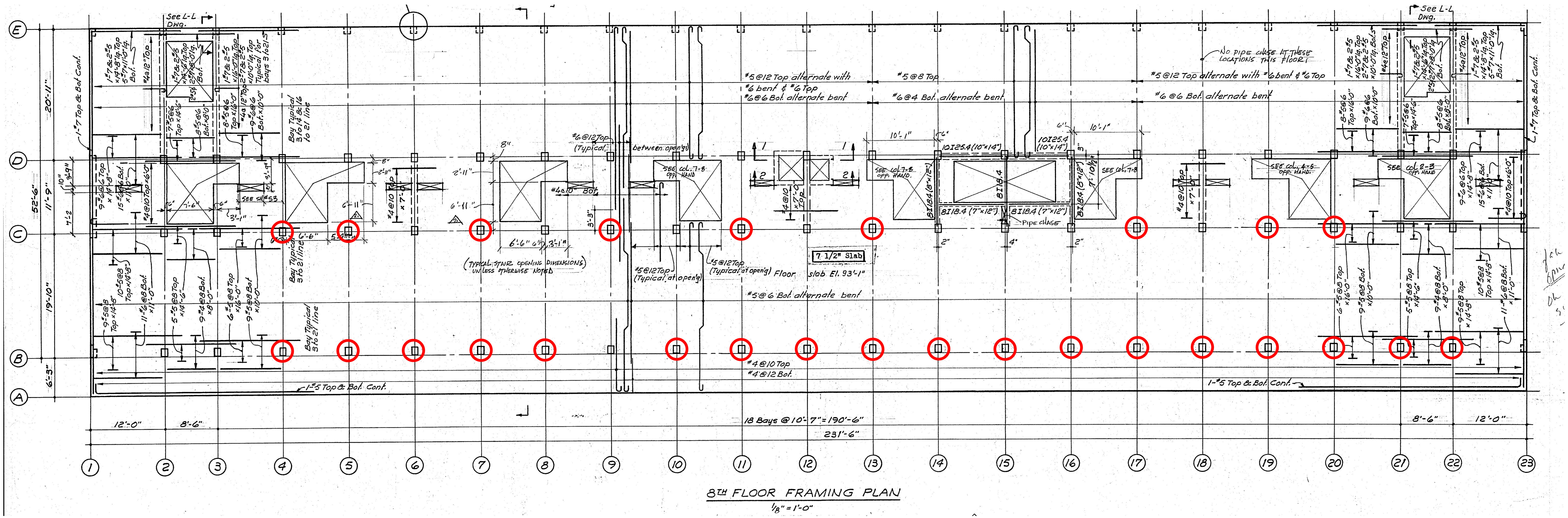
LEGEND

○ (red)	CATEGORY 1	DCR > 1.05
○ (blue)	CATEGORY 2	DCR ≤1.05

NOTE: UNCIRCLED GRID INTERSECTIONS INDICATE AREAS WITH NO GPR SCANNING DATA PROVIDED BY SGH.

Thornton Tomasetti

PROJECT: RIVERVIEW CONDOMINIUMS PUNCHING SHEAR REVIEW		SUBJECT: SLAB-TO-COLUMN CONNECTION BUCKET CATEGORIES	SSK-01D
REFERENCE:	BY: TT	DATE: 02/11/2025	



LEVEL 8

LEGEND

○	CATEGORY 1	DCR > 1.05
○	CATEGORY 2	DCR ≤1.05

NOTE: UNCIRCLED GRID INTERSECTIONS INDICATE AREAS WITH NO GPR SCANNING DATA PROVIDED BY SGH.

Thornton Tomasetti	PROJECT: RIVERVIEW CONDOMINIUMS PUNCHING SHEAR REVIEW REFERENCE: BY: TT DATE: 02/11/2025	SUBJECT: SLAB-TO-COLUMN CONNECTION BUCKET CATEGORIES	SSK-01E