

CITY OF GLENDALE, CA  
PARKING STRUCTURE EVALUATION

# City of Glendale Parking Structure Evaluation Report



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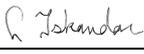
**PARKING STRUCTURE EVALUATION  
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**ATTACHMENTS**

APPENDIX 1 - SUPPORTING PICTURES

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## **1. SUMMARY**

Glendale City parking garage is located at 650 E Wilson Avenue, Glendale, California (see Figure 1 in Appendix 1 for an aerial view). It is positioned directly north of the City of Glendale Planning Division building. The main entrance to and exit from the parking garage is from Wilson Avenue to the North. There is an additional entrance on the west side from the adjacent Police Station parking area. From the available drawings it is presumed that the garage structure was originally built in 1982 based on Uniform Building Code 1982 (UBC 1982) requirements. The original structure had three parking levels - P1, P2 and P3. These included ground level (Level P1) parking at the street level. The structure occupies an approximate foot print of 245.5 feet by 192 feet.

A partial 4<sup>th</sup> level (level P4) was presumably added to the existing structure in 1987. Some architectural and structural drawings for this addition are made available to Worley for the evaluation of the existing structure. Erection and fabrication drawings for some items for the original structure have also been recovered. However, not all the fabrication drawings for the precast double T beams, inverted L & T girders and miscellaneous precast columns utilized at different floor levels have been available at the time of this evaluation.

Worley has been tasked to evaluate the existing structure at its current condition and determine if additional solar panels and associated supports can be attached on top of the existing parking structure. Design calculation and construction specifications for the original (1982) or upgraded (1987) structure are not made available to aid the evaluation. Therefore, conservative engineering assumptions were made during the evaluation of the existing structure. As a part of this task Worley evaluated the gravity load and lateral load of the existing structure and determined how much more vertical and lateral loads could be added without triggering new structural modifications to the existing structure as required by Chapter 4 of 2016 California Existing Building Code.

Based on the evaluation results in Section 4 and recommendations in Section 5, it is concluded that only limited amount of future installations in terms of gravity loads can be allowed at the roof level at selected locations without any major modifications to the existing structure. These future installations are allowed only on top of existing shear walls on Building Grids A, C, D, 1 and 11, and on top of certain interior and exterior columns. Minor repairs to the existing structure as identified in Section 4 and 5 should be completed prior to the addition of any new installation to maintain safety and integrity of the structure.

It is also recommended that the existing structure should be upgraded to meet current California Building Code (CBC 2016) requirements in case the gravity and lateral loads from the new installation exceed the limits identified in Section 5 of this report.

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## **2. INTRODUCTION**

The existing parking structure is approximately 244'-6" long in the north/south direction and 192'-0" wide in the east/west direction. The structure has four (4) parking levels - P1, P2, P3 and P4 with P1 level being located at the ground floor elevation. The elevation difference between two adjacent parking levels is approximately 12'-3". The total height of the structure along with the perimeter spandrel beams is approximately 40'-0".

The structure is constructed of both precast and cast-in-place concrete elements. All the beams, girders and columns are comprised of precast (possibly prestressed) sections whereas all the shear walls and foundations are constructed of cast-in-place reinforced concrete. There are cast-in-place reinforced concrete slabs on grade at level P1 and topping slabs with wire mesh on top of precast double T beams on the elevated floors (levels P2, P3 and P4). Only north shear wall on Building Grid 1 from level P2 and above is constructed of reinforced masonry

The perimeter girders (precast inverted L sections) at all parking levels are furnished with 5 inches thick by 7'-10" deep precast spandrel beams to act as guard rails as well as to attach precast concrete planters at selected locations for architectural purpose.

The load paths for the vertical and lateral loads on the structure are as follows:

- A. Dead and live loads at P1 level are supported on reinforced concrete slabs on grade. Dead and live loads at elevated floors (levels P2, P3 and P4) are supported on precast double T beams topped with 3.5 inches thick concrete topping slabs. Double T beam depths vary from floor to floor depending on their location. These beams carry the vertical loads on to either the shear walls or the precast concrete girders comprised of inverted T or L sections depending on their location. The precast girders finally transfer the load onto the precast concrete corbels attached to the columns which carry the loads down to the foundations. The columns are supported on reinforced concrete spread footings and the shear walls are supported on individual reinforced concrete strip footings.
- B. The lateral load caused by wind or seismic events in the north/south direction is resisted primarily by three (3) shear walls located on Building Grids A, C and D between Grids 3 & 5. Refer to existing Roof Plan in Appendix 3 for Building Grid locations. There are a few smaller shear walls surrounding three stair cases at the corners of the structure; however, these are not considered as part of lateral load resisting system for simplicity. Transfer of the lateral load to the shear walls is accomplished via floor diaphragms comprised of precast double T beams topped with 3.5 inches concrete slab. The thickness of the shear walls on Grids A and D is 18 inches whereas that of the wall on Grid C is 24 inches. These three shear walls are constructed of cast-in-place

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reinforced concrete and they carry the lateral load down to the foundation constructed of reinforced concrete strip footings.

- C. The lateral load caused by wind or seismic events in the east/west direction is resisted primarily by two (2) shear walls located on Building Grids 1, and 11 between Grids A.3 & C.3 and B & C respectively. As discussed previously, transfer of lateral load is accomplished via floor diaphragms comprised of precast double T beams topped with 3.5 inches concrete slab. The thickness of the shear wall on Grid 11 is 18 inches whereas that of the wall on Grid 1 is 12 inches. The shear wall on Grid 11 is constructed of cast-in-place reinforced concrete. The shear wall on Grid 1 is constructed of reinforced concrete up to level P2 and from level P2 to P4 it is constructed of reinforced masonry. These two shear walls finally carry the lateral load down to the foundation constructed of reinforced concrete strip footings.

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### **3. EVALUATION**

The intent of this structural evaluation for the current phase is to compute the distribution of gravity and lateral load in the different parts of the existing structure including the foundations. This evaluation will be utilized to determine how much additional vertical and lateral loads can be imposed on the existing structural elements by the future solar panel installation without jeopardizing the safety and stability of the existing parking garage structure.

To initiate the evaluation Hans Nepf of Worley performed a field walk down and on-site assessment of the existing parking structure on June 18, 2019. Mr. Nepf acquired several pictures of the existing structure, some of which are attached in Appendix 1 (Figures 2 thru 14). Existing concrete and masonry soundness were checked by visual observation. Condition of supporting arrangements of the double T beams at different levels were also visually examined for soundness. Subsequently, Andi Setiawan of Worley visited the site on July 19, 2019 to obtain field measurements of the existing structural elements. These dimensions were missing from the available design drawings. The markups of all these measurements are depicted on Sketches 1 thru 4 in Appendix 2.

In order to obtain a good representation of the dead and live load distribution, a three-dimensional finite element model was generated using Staad/Pro software for only a portion of the structure between Building Grids A & D in the east/west direction and Grids 3 & 7 in the north/south direction. Refer to the existing Roof Plan in Appendix 3 for Building Grid locations. The dimensional and material properties of the existing structural elements were obtained from available drawings listed in References (Section 6) and field sketches shown in Appendix 2. In Staad/Pro model, beams, girders and columns were modeled as beam elements with appropriate end releases. Beams and girders were assumed to be pinned supported at the ends. Beam elements were modeled with prismatic properties generated in RISA Section software using either the information obtained from design drawings, when available, or field measured dimensions (see Appendix 2). Walls and floor toppings were modeled as plate elements with actual thicknesses obtained from design drawings. The connections between columns and shear walls to the foundations were conservatively modeled as pinned. Footings were not included in the model for simplicity.

Dead loads were calculated based on the self-weight of the members (beam elements and plate elements) plus weight of any appurtenances such as spandrel beams, planters, handrails etc. supported by the structural members. Information on these items were obtained from available design drawings and fabrication drawings from pre-cast contractor. Self-weight calculation was based on the concrete density of 150 pounds per cubic feet.

Live load rating at different parking levels was taken as 50 pounds per square feet as noted on the available design drawing (specifically Hugh Brooks Associates, Inc. Drawing S1). Live load reduction on

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columns and corresponding foundations was considered as permitted by Section 2306 of UBC 1982. The calculation revealed that the maximum permissible live load reduction of 60% can be achieved for the existing columns and walls supporting live loads from multiple levels. This is due to large tributary area supported by each column or wall.

Lateral wind loads were calculated based on the requirements of Sections 2311 of UBC 1982. Method 2 (Projected Area Method) was utilized for wind base shear and overturning moment calculation.

Lateral seismic loads were calculated following the requirements of Section 2312 of UBC 1982. An equivalent lateral force procedure was used to generate seismic base shear and associated overturning moment. The effect of torsion on the shear walls caused by the lateral seismic force was also considered. Evaluation of lateral loads using MathCad template revealed that seismic load controlled the lateral load resisting system over wind load for the parking structure.

Following design parameters were used for lateral load calculation:

Basic Wind Speed (V)	70 mph
Exposure Category	C
Wind Importance Factor ( $I_w$ )	1.0
Seismic Zone	4
Seismic Importance Factor ( $I_E$ )	1.0

The results from Staad/Pro model were used to calculate the gravity load effects. The building dead weight from Staad/Pro results were extrapolated to include the entire structure based on the ratio of actual building size to modeled building size. This is an acceptable approximation in lieu of modeling the entire structure in Staad/Pro. Staad/Pro analysis results were exported to MS Excel templates to compile the vertical loads on column and wall footings for different basic load cases and load combinations.

Using the dead weight generated as described above the evaluation of lateral load on shear walls was performed in MathCad template using standard engineering principles and UBC 1982 requirements. Wall capacities were evaluated at the base of the shear walls and compared with the corresponding demands to determine adequacy. The shear walls at upper levels were not checked since they had identical thicknesses with less amount story shears.

MathCad template was also utilized to evaluate selected footings for existing loads and load combinations per applicable UBC 1982 requirements. Since no original design calculations were available, it was assumed that the concrete design code for the existing structure is ACI 318-77 based on the time of

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construction. Therefore, the existing condition of the structure was evaluated per the requirements of ACI 318-77. The design strengths of cast-in-place concrete for the walls and footings were taken as 4000 and 3000 pounds per square inches respectively as indicated on the design drawing (Hugh Brooks Associates, Inc. Drawing S1).

Allowable soil bearing pressure of 6000 pounds per square feet at service level was used as indicated on the design drawing (Hugh Brooks Associates, Inc. Drawing S1) to check the adequacy of the existing foundations. In the absence of any geotechnical report, allowable bearing pressure was conservatively taken as gross allowable and no increase in allowable soil bearing pressure was assumed for load combinations associated with wind or seismic loads.

After the evaluation of the parking structure for existing loading conditions, allowable future addition of gravity and lateral loads due to solar panel installation was determined using the requirements from Chapter 4 of 2016 California Existing Building Code. Since the future gravity load addition will be concentrated on the roof (level 4) of the structure, 5% of roof dead weight limit was used as recommended by 2016 California Existing Building Code for future installation. Similarly, the lateral load from future installation was limited to a quantity that would not increase the existing stress ratio in the lateral load resisting elements by more than 10%. These computations were performed using MathCad template.

The results and conclusions of the evaluation are described in Section 4 of the report.

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**4. RESULTS AND CONCLUSIONS**

Following are the results of the evaluation described in previous section:

- A. Structural capacities of existing reinforced concrete shear walls and corresponding foundations are adequate for current loading condition and are deemed capable of carrying additional gravity and lateral loads from future solar panel installation
- B. Existing external column footings on Grid A (specifically Grid A-6.3 and A-7.3) are at or above their design capacities for the current loading condition. No additional gravity load should be placed on top of or at locations adjacent to these columns. See existing Roof Plan in Appendix 3 for Building Grid locations.
- C. Existing internal column footings on Grid B (specifically Grid B-3, B-4, B-5, B-6 and B-7) are close to their design capacities. No additional gravity loads are recommended to be placed on top of or at locations adjacent to these columns. See existing Roof Plan in Appendix 3 for Building Grid locations.
- D. Structural capacities of existing precast (and possibly prestressed) inverted L & T girders, and double T floor beams cannot be determined because fabrication drawings for most of these items are not currently available. Therefore, any addition of future gravity loads on these structural items are not recommended.
- E. In the absence of fabrication drawings for the existing double T floor beams on the roof (level 4), no drilled in attachments are recommended on the roof for future installation of solar panels and accessories to avoid accidental damaging of prestressing tendons in floor beams unless the existing rebars and prestressing tendons are precisely located.
- F. Based on the requirements of Chapter 4 of 2016 California Existing Building Code, the gravity load introduced by new installation on the roof of the parking structure is limited to 5% of the existing gravity load on the roof. The increase in lateral load due to new installation is limited such that the stress ratio on the existing lateral load resisting system is not increased by more than 10% above the existing stress ratio. This will ensure that the existing structure need not meet design and detailing requirement of current building code.
- G. There are visible degradations and local failures of existing parking structure elements at various locations. Some of these are captured during field visit and are shown on Figures 9 thru 14 in Appendix 1. All necessary repairs to the existing structure should be completed before any new loads from solar panel installations are introduced.

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## **5. RECOMMENDATION**

Based on the findings identified during the structural evaluation described previously, it is concluded that the installation of solar panels and their appurtenances is feasible provided the recommendations listed below are followed:

- A. Any addition of future gravity load due to solar panel installation on the roof level should be located on top of the shear walls on Building Grids 1, 11, A, C and D. In addition, top of the columns located at Building Grids D6, C7, D7, B8, B9, C8, C9, D8 and D9 can be utilized for attaching new installations. See existing Roof Plan in Appendix 3 for Building Grid locations.
- B. The total weight of future installation should be evenly distributed amongst the structural members identified in Item A.
- C. Magnitude of total weight of new installation should not exceed 320 kips based on the requirements of Chapter 4 of 2016 California Existing Building Code.
- D. Magnitude of total lateral load (due to wind or seismic events) caused by new installation should not exceed 90 kips based on the requirements of Chapter 4 of 2016 California Existing Building Code.
- E. Magnitude of total overturning moment with respect to base at the bottom of the shear walls (due to wind or seismic events) caused by new installation should not exceed 3580 kip. ft.
- F. Repairs to the existing structure at the locations identified in Section 4 should be completed before installation of new solar panels and associated accessories on the roof.
- G. If the recommendations listed in items A thru F are met, the parking structure should be evaluated using the reactions from the solar panel installation on the roof structure once the solar panel layout is finalized.
- H. In the case where addition of either vertical or lateral load exceed the limits indicated in items C, D and E above, following recommendations shall be met:
  - 1. Thorough inspection of the existing structure should be performed to determine suitability for the required structural upgrades.
  - 2. The structure should be analyzed, designed and upgraded to meet the requirements of current California Building Code (CBC 2016).

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3. All the existing structural connection details should be inspected, evaluated and upgraded, as required, to meet current building code requirements for strength and ductility.
4. Core samples of existing reinforced concrete shear walls and footings, and footings for columns can be tested for in-situ strengths which could be higher than the original design values to benefit evaluation and possible reduction in required upgrades.

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**6. REFERENCES**

A. Codes, Standards and Publications:

1. Uniform Building Code – 1982
2. Building Code Requirements for Reinforced Concrete – ACI 318-77
3. PCI Design Handbook – Third Edition – 1985
4. 2016 California Building Code
5. 2016 California Existing Building Code

B. Design Drawings:

1. Hugh Brooks Associates, Inc. Dwg No. A 2 Dated April 7, 1982 – P-1 LEVEL FLOOR PLAN
2. Hugh Brooks Associates, Inc. Dwg No. A 3 Dated April 7, 1982 – P-2 LEVEL FLOOR PLAN
3. Hugh Brooks Associates, Inc. Dwg No. A 4 Dated April 7, 1982 – P-3 LEVEL FLOOR PLAN
4. Hugh Brooks Associates, Inc. Dwg No. A 5 Dated April 7, 1982 – BUILDING ELEVATIONS AND SECTIONS
5. Hugh Brooks Associates, Inc. Dwg No. A 6 Dated April 7, 1982 – WALL SECTIONS
6. Hugh Brooks Associates, Inc. Dwg No. A 7 Dated April 7, 1982 – STAIRWAY SECTIONS AND DETAILS
7. Hugh Brooks Associates, Inc. Dwg No. A 8 Dated April 7, 1982 – SPANDREL AND PLANTER DETAILS
8. Hugh Brooks Associates, Inc. Dwg No. S 1 Dated April 7, 1982 – FOUNDATION PLAN

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9. Hugh Brooks Associates, Inc. Dwg No. S 2 Dated April 7, 1982 – P-2 LEVEL FRAMING PLAN
10. Hugh Brooks Associates, Inc. Dwg No. S 3 Dated April 7, 1982 – P-3 LEVEL FRAMING PLAN
11. Hugh Brooks Associates, Inc. Dwg No. S 4 Dated April 7, 1982 – SHEAR WALL ELEVATIONS
12. Hugh Brooks Associates, Inc. Dwg No. S 4 Dated April 7, 1982 – SHEAR WALL SECTIONS
13. Nelson Behrend AIA Architect Dwg No. A 2 Dated June 9, 1987 – SITE PLAN
14. Nelson Behrend AIA Architect Dwg No. A 3 Dated June 2, 1987 – P-3 LEVEL DEMOLITION PLAN
15. Nelson Behrend AIA Architect Dwg No. A 3 Dated June 2, 1987 – P-3 LEVEL PARKING PLAN
16. Nelson Behrend AIA Architect Dwg No. A 4.1 Dated June 2, 1987 – ROOF P-4 LEVEL PARKING PLAN
17. Nelson Behrend AIA Architect Dwg No. A 5 Dated June 2, 1987 – EXTERIOR ELEVATIONS BUILDING SECTIONS
18. Nelson Behrend AIA Architect Dwg No. A 6 Dated June 2, 1987 – WALL SECTIONS
19. Nelson Behrend AIA Architect Dwg No. A 8 Dated June 2, 1987 – DETAILS
20. Nelson Behrend AIA Architect Dwg No. S 1 Dated June 3, 1987 – P-3 LEVEL FRAMING PLAN
21. Nelson Behrend AIA Architect Dwg No. S 2 Dated June 3, 1987 – P-4 LEVEL FRAMING PLAN
22. Nelson Behrend AIA Architect Dwg No. S 3 Dated June 3, 1987 – SHEAR WALL ELEVATIONS

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23. Nelson Behrend AIA Architect Dwg No. S 4 Dated June 3, 1987 – SHEAR WALL ELEVATIONS
  24. Nelson Behrend AIA Architect Dwg No. S 5 Dated June 3, 1987 – SHEAR WALL SECTIONS
- C. Fabrication Drawings:
1. The Tanner Companies Drawings – A1 to A8, C1 to C4, C4A, C5, C5A to C5F, SB1, SB1-1, SB1A, SB1B, SB1C, SB1C-1, SB1D, SB1E, SB1E-1, SB1F, SB1G, SB1H, SB1H-1, SB1J, SB1K, SB1K-1, SB1L, SB1M, SB1M-1, SB1N, SB1P, SB1P-1, SB1Q, SB1R, SB1R-X, SB1T, SB1U, SB1X, SB1X-1, SB1W, P1 to P21, P23, P23-1, E-1 to E-19.

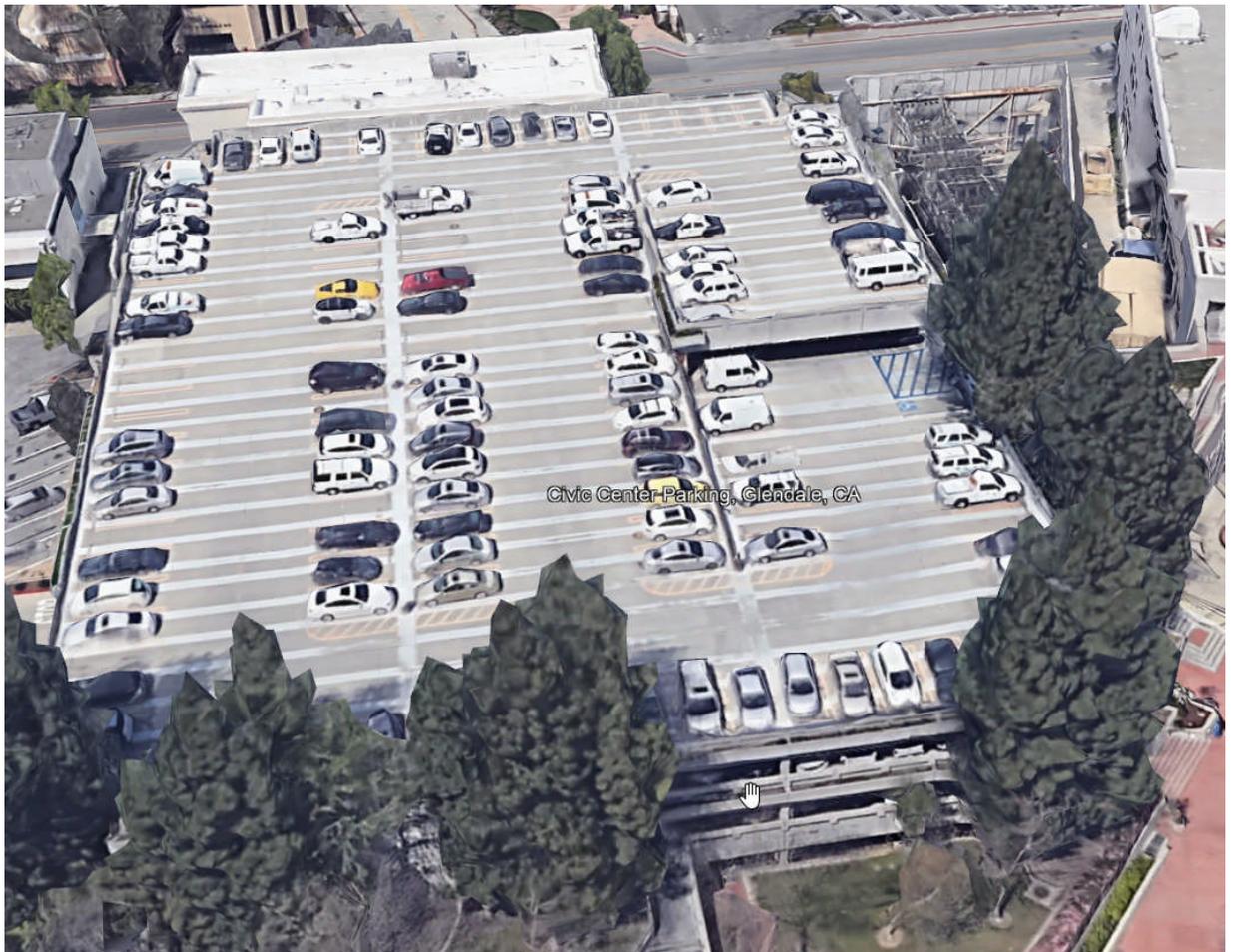
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**Appendix 1 - Supporting Pictures**

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**Figure 1: Aerial View of Civic Center Parking Structure**

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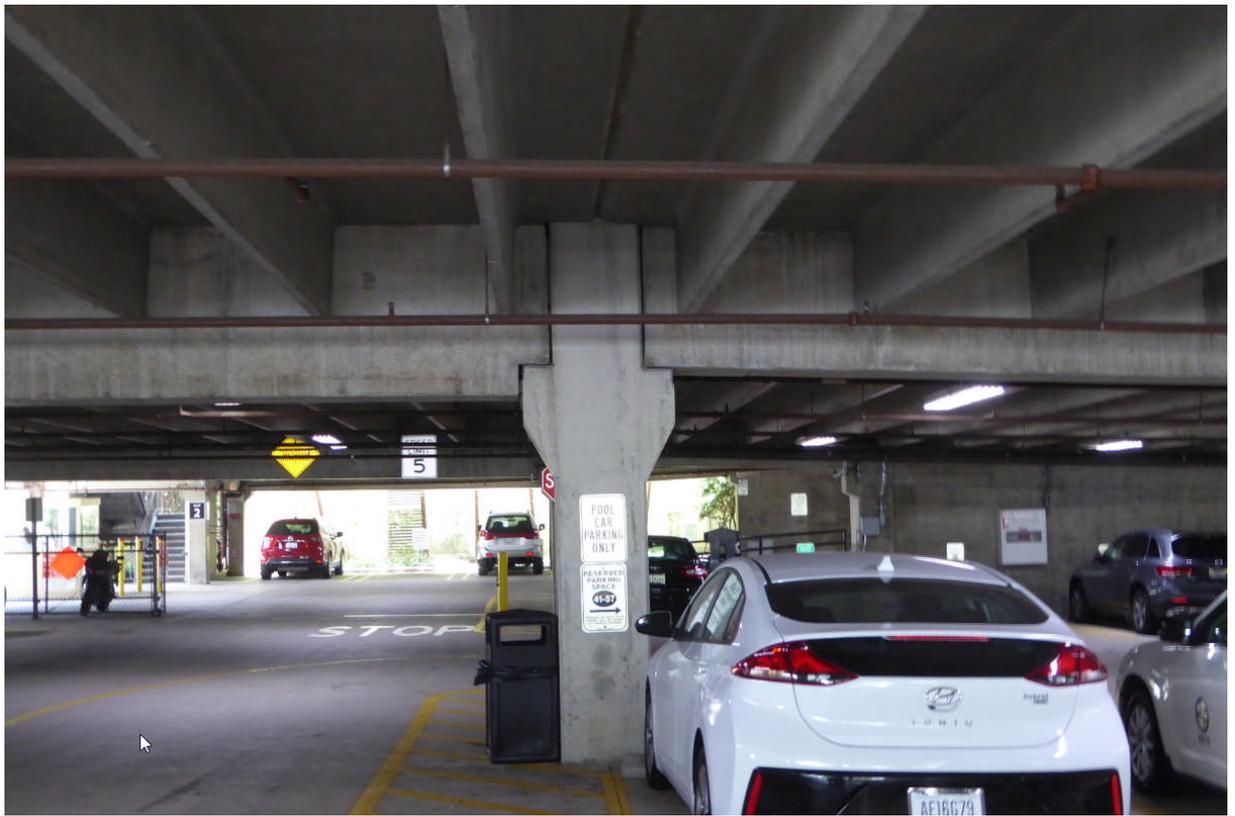
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**Figure 2: View of Parking Level 4 from Level 3**

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**Figure 3: View of Parking Level 2**

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**Figure 4: View of Stair Case**

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**Figure 5: View of Precast Inverted T Beam**

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**Figure 6: View of Precast Inverted L Beam**

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**Figure 7: Existing Double T Beam Supports**

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**Figure 8: View of Spandrel Beam & Planter**

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**Figure 9: Local Degradation of Concrete Floor**

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**Figure 10: Local Degradation of Concrete Girder Support**

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**Figure 11: Local Degradation of Double T Beams**

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**Figure 12: Local Degradation of Double T Beam Support**

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**Figure 13: Local Degradation of Concrete Shear Wall**

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**Figure 14: Local Degradation of Masonry Shear Wall**

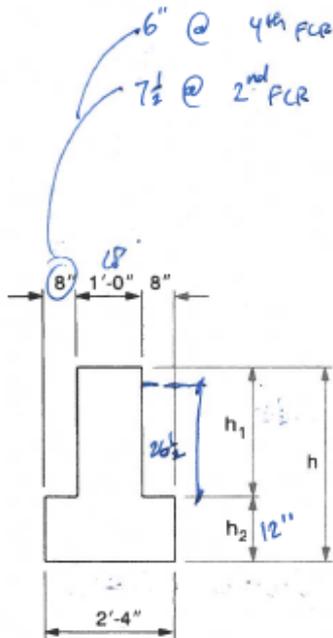
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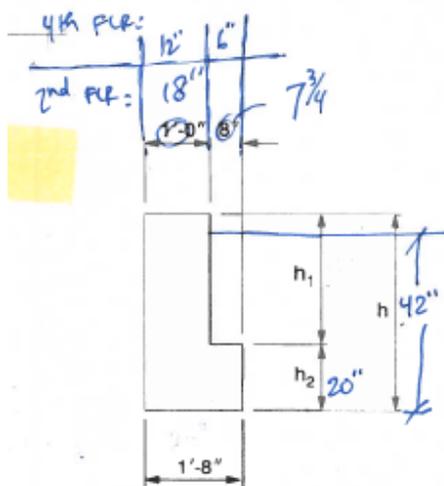
**Appendix 2 – Field Sketches**



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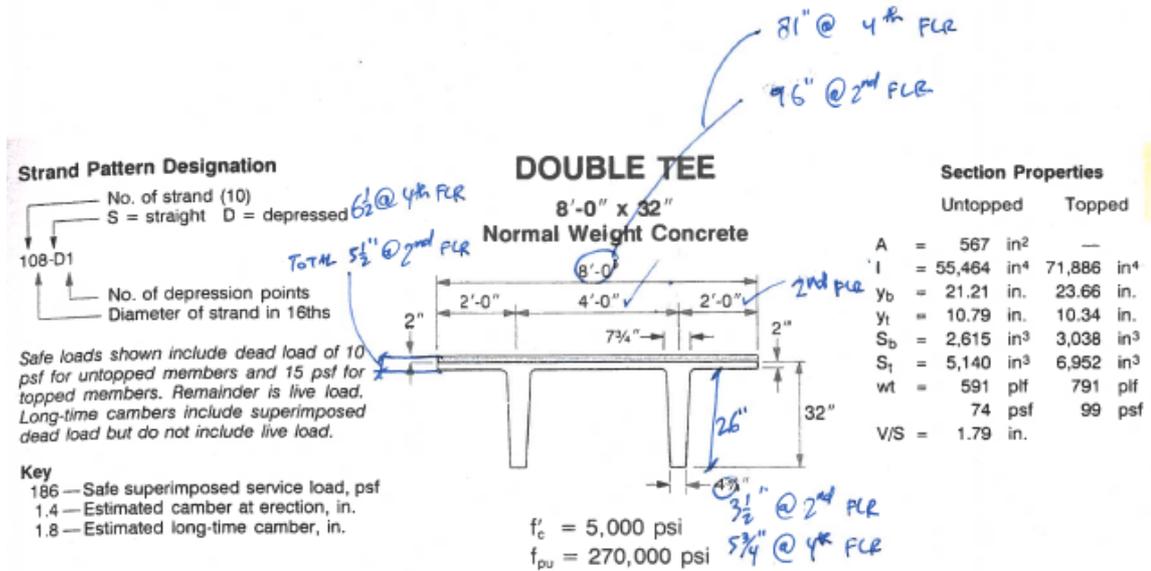


**Sketch 2: Field Measurements for Inverted T Girders**



**Sketch 3: Field Measurements for Inverted L Girders**

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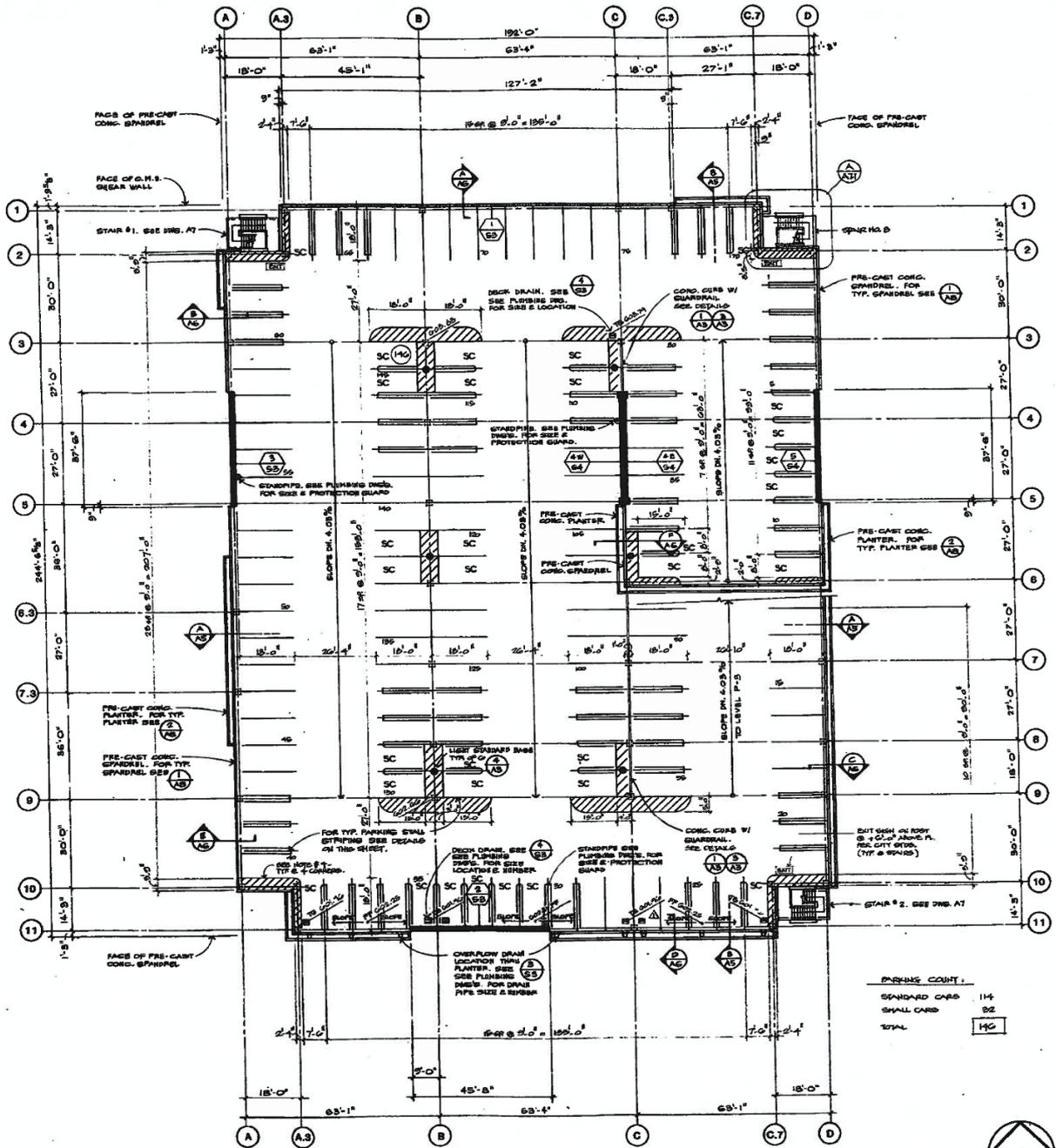
**Sketch 4: Field Measurements for Double T Beams**

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**Appendix 3 – Roof Plan**

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**ROOF P-4 LEVEL PARKING PLAN**

