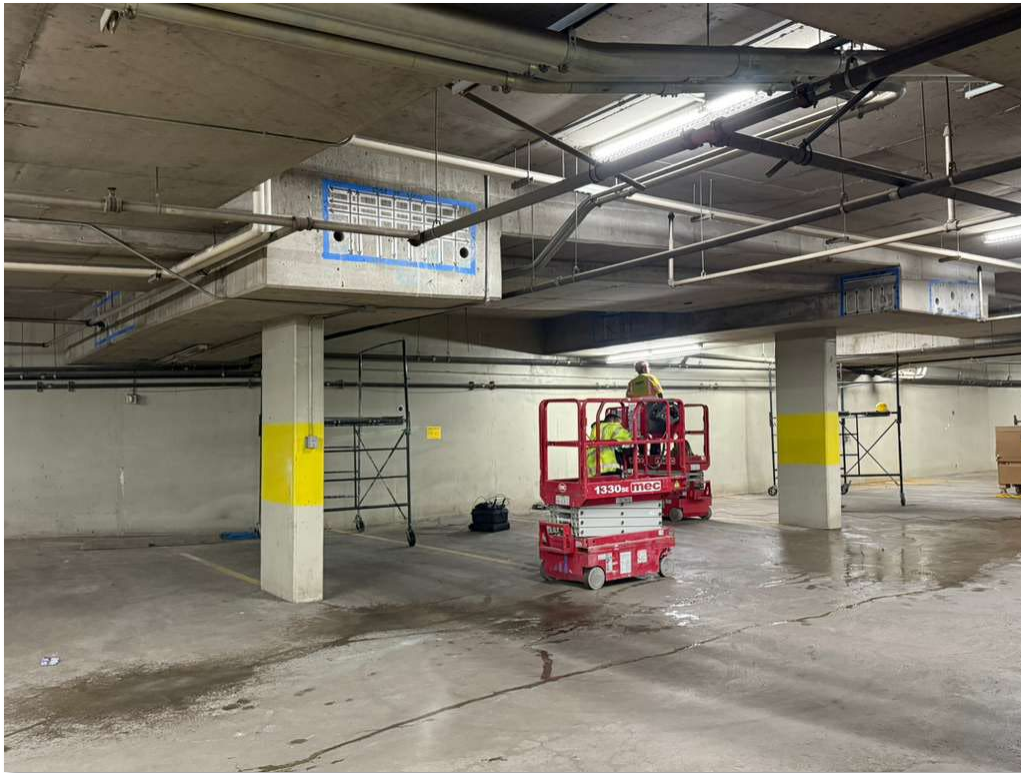


REPAIR DESIGN SUMMARY REPORT

Repairs to Existing Beams and Ramp

Project: Building 5, Parking Garage

Address: The Village at Palisades, Olympic Valley, CA 94146



Prepared by:



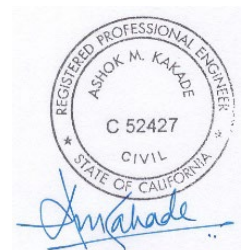
Concrete Science, Inc.
INVESTIGATION | TESTING | ENGINEERING

1534 B Street, Hayward, CA 94541-3018

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September 15, 2026

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Index

Sr.	SUBJECT	PAGE
1	Project information	3
2	Scope of work	3
3	Applicable codes & standards	3
4	Existing structure information	4
5	Existing condition summary	8
6	Material properties	13
7	Design load Combinations (ASCE 7)	14-15
8	Demand-capacity checks	16
9	Existing beam capacity	17
10	Strengthening design	18
11	Discussion	20
11	Assumptions & limitations	21
12	Engineer's certification	21



1. PROJECT INFORMATION

- **Project Name:** RC Beam and Slab strengthening using Steel plates and FRP
- **Project Address:** Vehicular Ramp Beams at Upper Garage Level slab, Village at Palisades, Building 5, Olympic Valley, CA
- **Owner:** Palisades Tahoe
- **Prepared By:** Concrete Science Inc.
- **Date:** 08/05/2026

2. SCOPE OF WORK

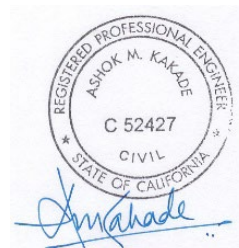
This report presents the structural calculations, design, and details for strengthening existing reinforced concrete (RC) beams and slabs using epoxy injection, cementitious grouting, externally bonded steel plates, and externally applied fiber-reinforced polymer (FRP) systems to provide structural strengthening and the requisite fire protection as per CBC 2025.

Strengthening is proposed due to the prolonged presence of cracks. The strengthening is proposed to mitigate any further duress and restore the intended design capacity.

3. APPLICABLE CODES & STANDARDS

Design is based on:

- California Building Code 2025
- ACI 318
- ACI 440.2R
- AISC 360
- ASCE 7



4. EXISTING STRUCTURE INFORMATION

- Beam ID (s): existing beam cross-section drawings with reinforcement called out showing the zone with stirrups and the zone without stirrups.
- Beam size: B1 and B2 (84" × 34")
- Beam length: B1 ±25' and B2 ±28' (measured from retaining wall to the cantilever tip)
- Concrete strength (f'_c): 5000 psi
- Reinforcement (existing): Refer to drawings below
- Steel yield strength (f_y): 60,000 psi
- Cover: 1 ½"

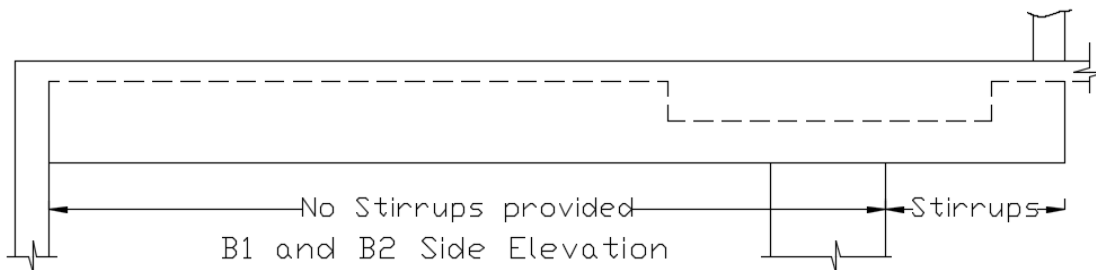
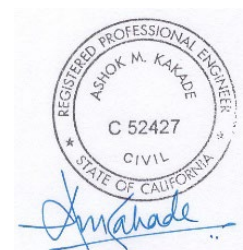


Figure 1. Top main reinforcement is curtailed per the as-built drawings: 30-#10 is provided till for 18' from the tip of the cantilever (refer to sketch 5 on page 11 and as-built drawing S-209).



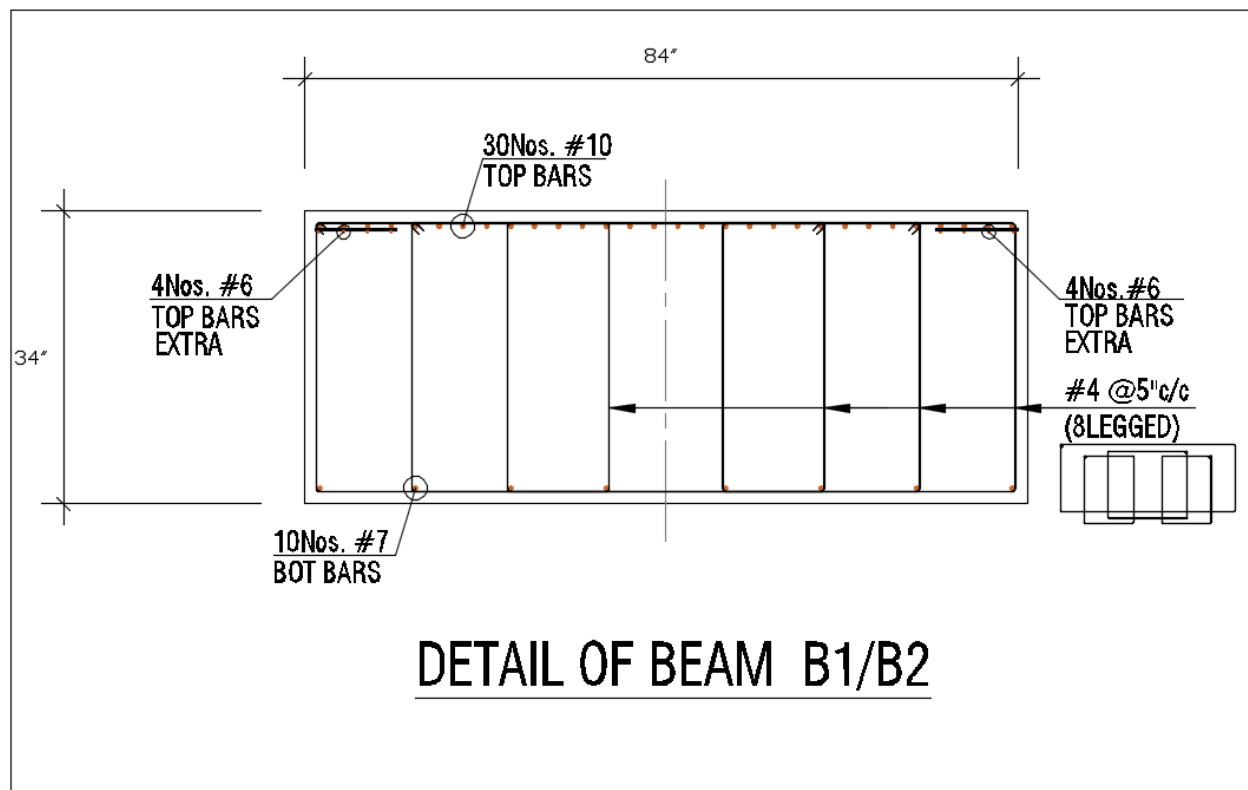
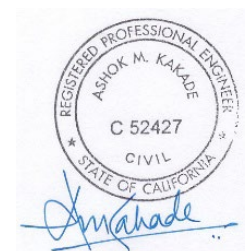


Figure 2. The shear reinforcement -stirrups are provided only along the beam cantilever span, stirrups are not provided in the span from column towards the retaining wall. From column to the wall, there are no stirrups as per the as-built drawings and no stirrups were detected in field during the rebar scanning as well.



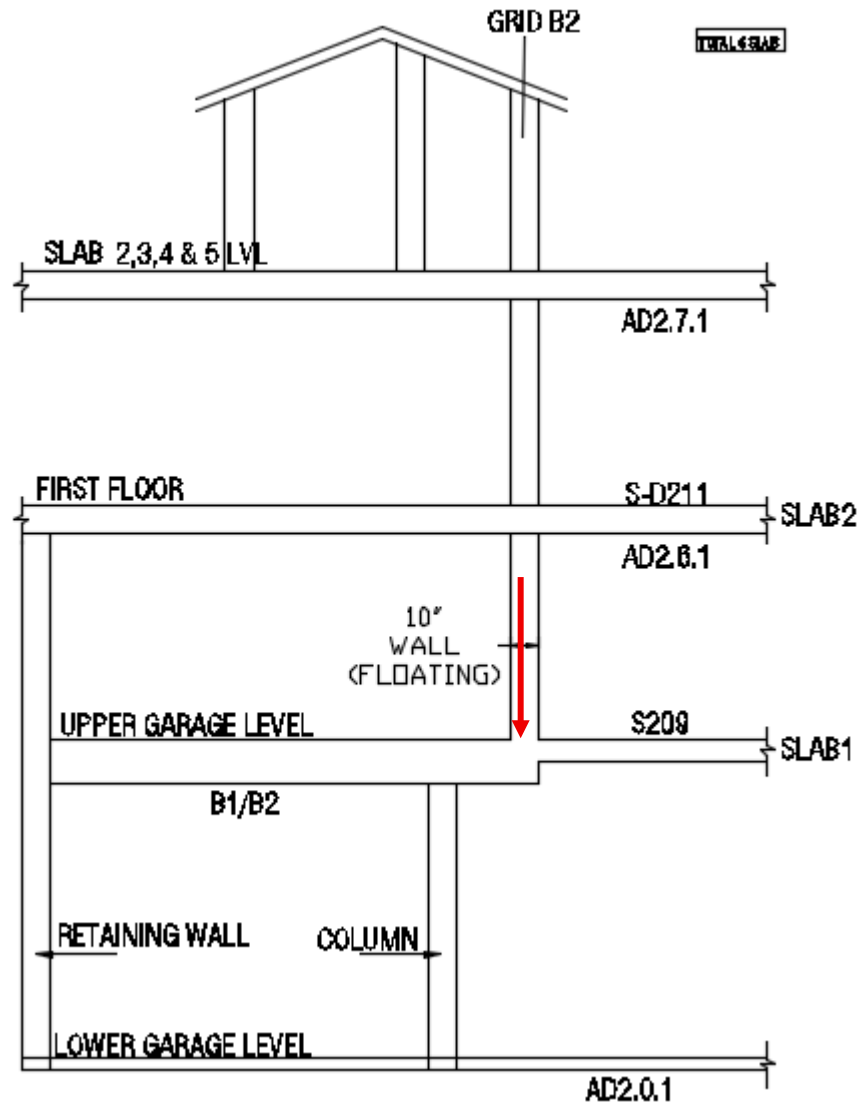
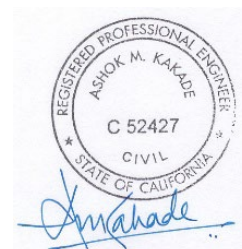


Figure 3. The schematic section shows a floating type of arrangement for a 10" structural wall. Transferring the loads to the cantilever ends of B1 and B2.



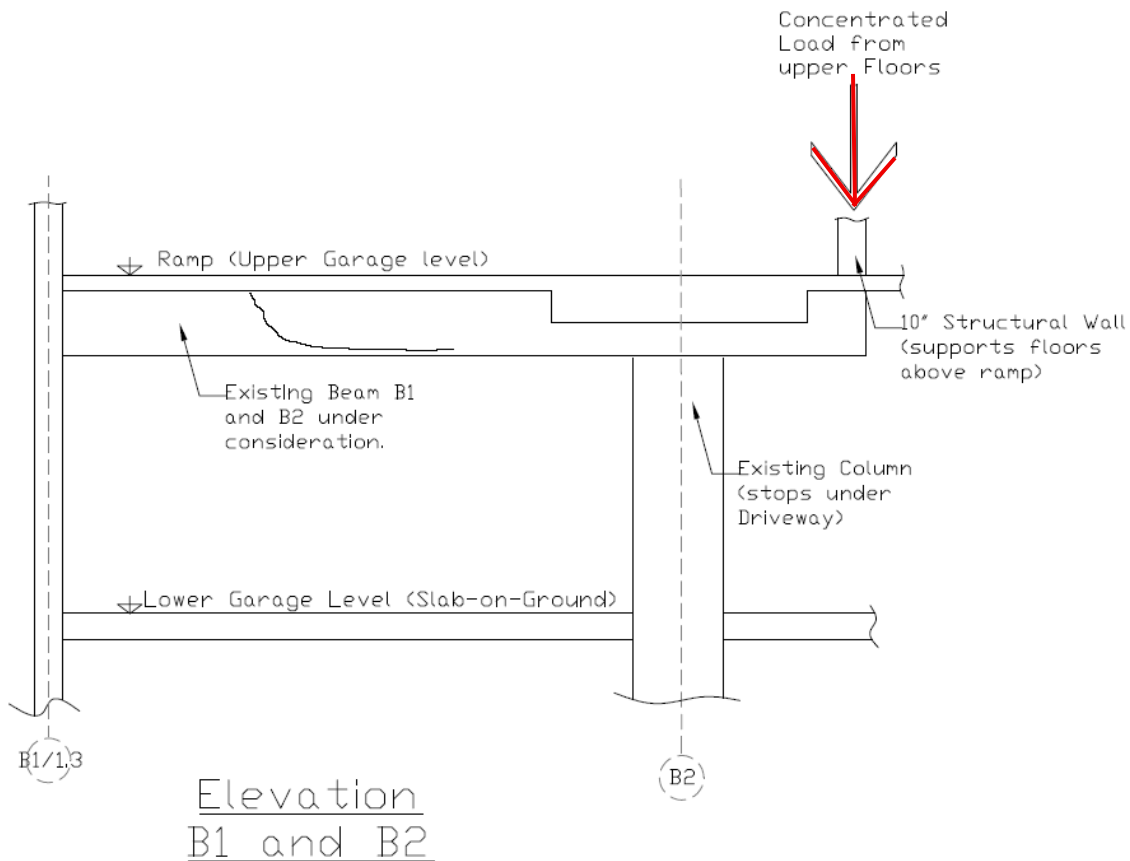
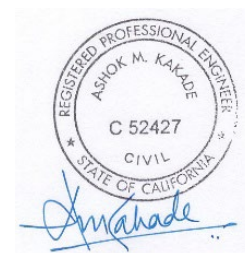
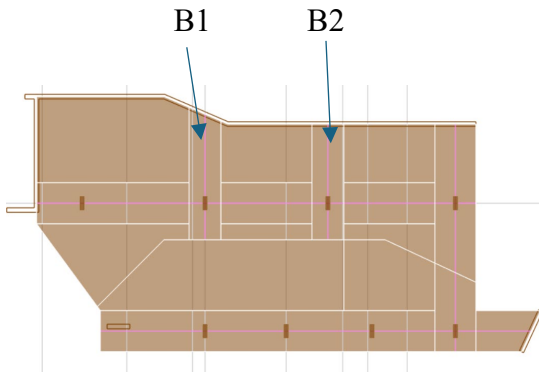


Figure 4. Blow-up of the load path, with a heavy point load supported on the cantilever ends of B1 and B2.



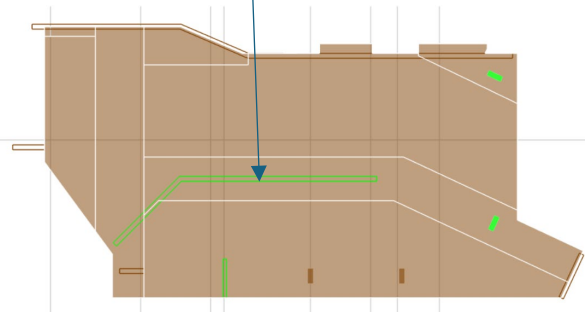
Upper Garage Level – ETABS Plan



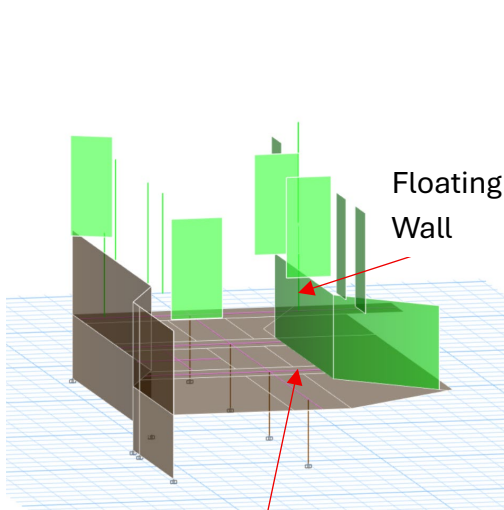
First floor level – ETABS Plan

The wall in green sits on cantilever beam

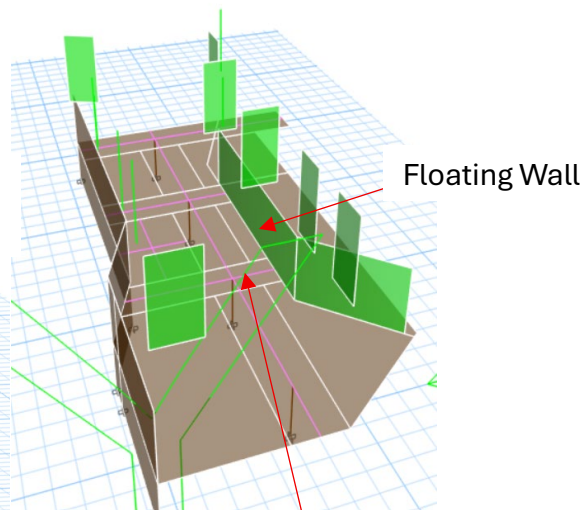
Tip B1 and B2 at Upper garage level



Plans from ETABS software, showing B1 B2 and the floating wall above it (green color)



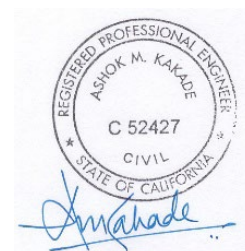
Cantilever tips of B1 and B2

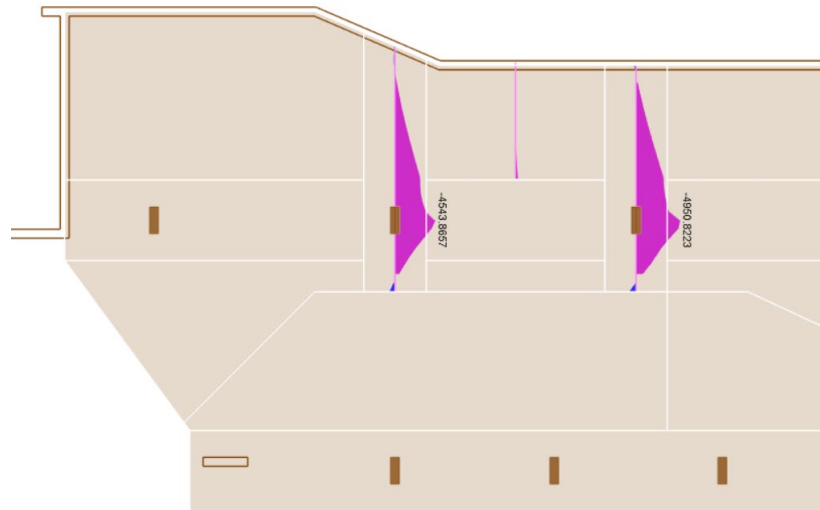


Cantilever tips of B1 and B2

3D Model views from ETABS showing floating walls supported on cantilever ends of B1 and B2

Sketch 1. Plans and 3D model screenshots from ETABS software

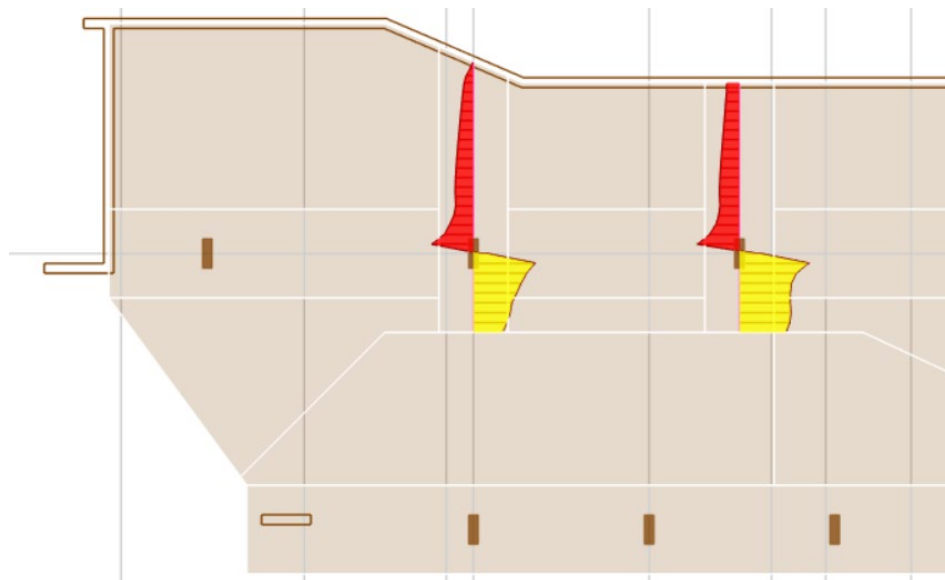




4543 Kip-ft

4950 Kip ft

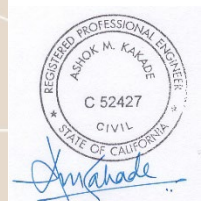
Image of Bending Moment Diagram for 1.2D + 1.6L case.



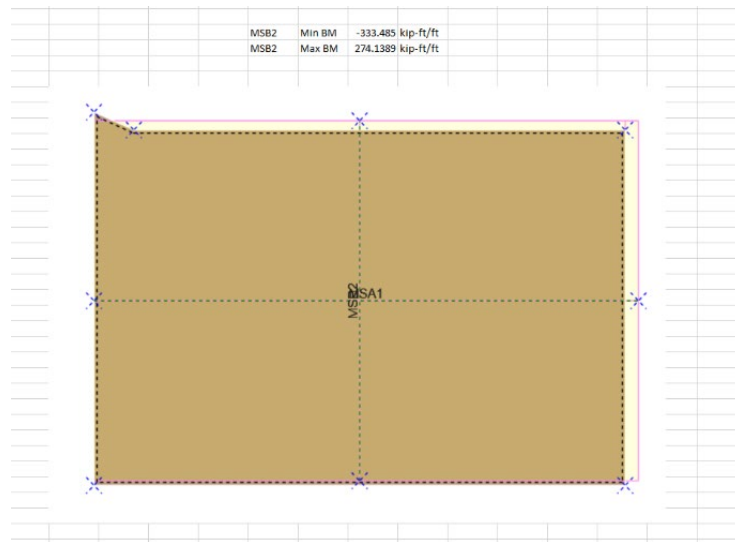
928 Kip

1041 Kip

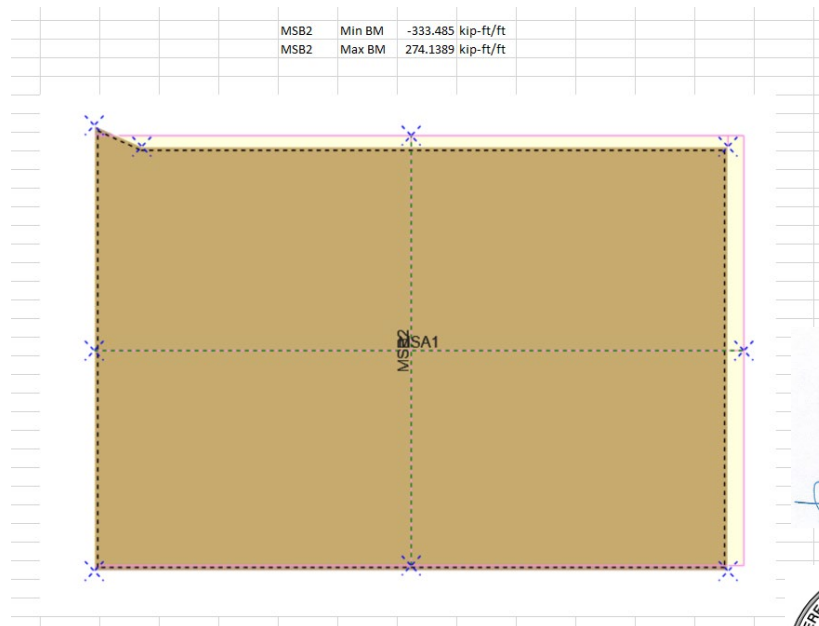
Image of Shear Force Diagram for 1.2D + 1.6L case.



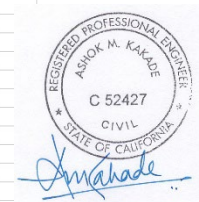
Sketch 2. The beams' bending moments and shear force images from ETABS



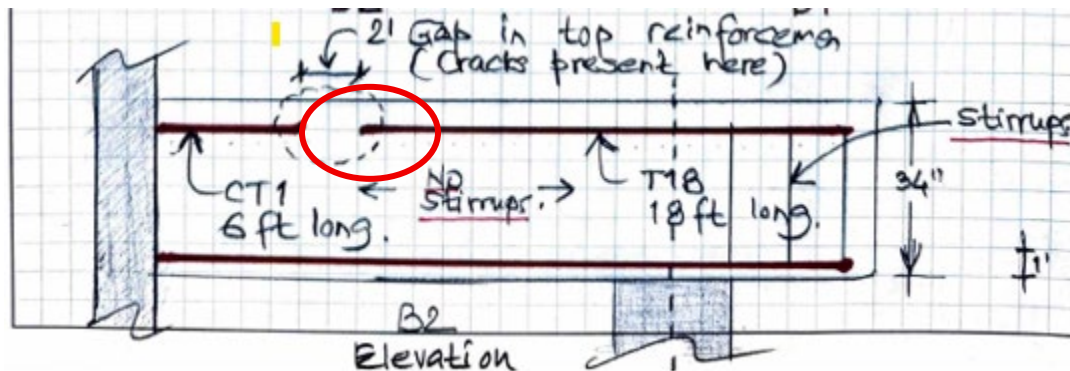
MSA1 is assigned for the X-direction strip



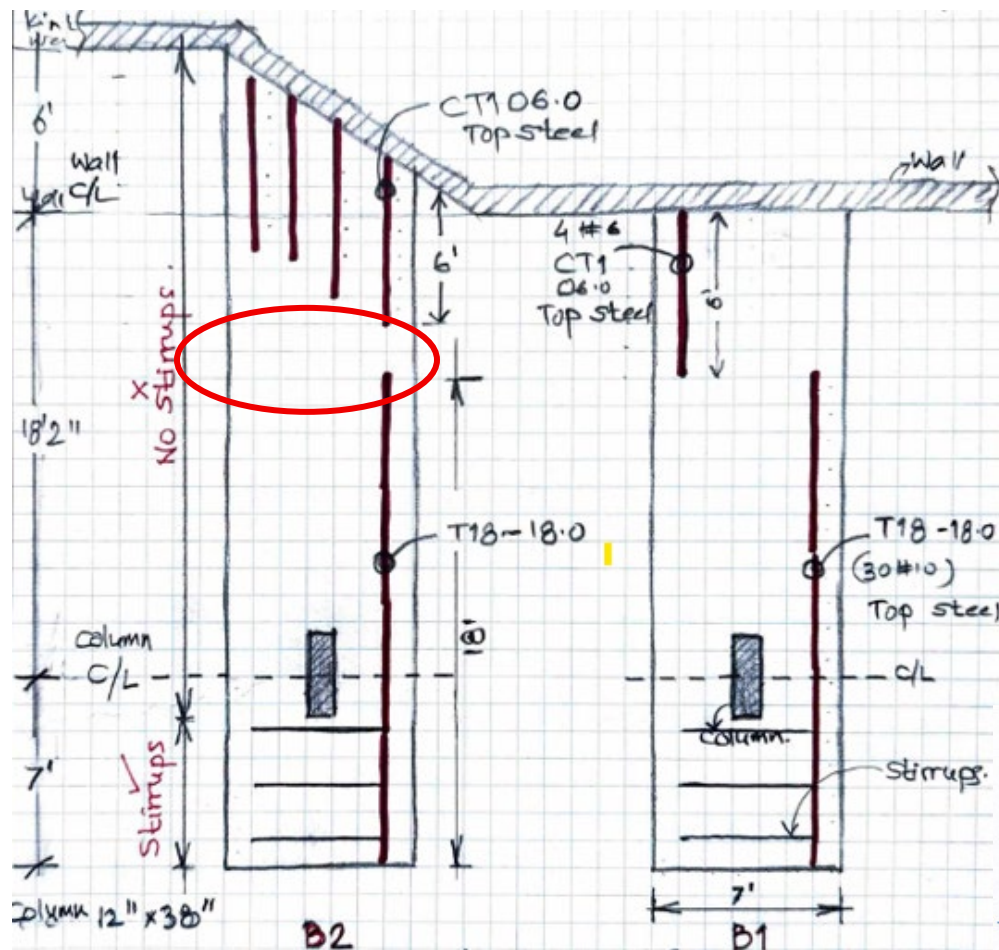
MSB2 is assigned for the Y-direction strip.



Sketch 3. The slab bending moments between the two beams have been considered using design strips, with MSA1 assigned to the X-direction strip and MSB2 assigned to the Y-direction strip.



Sketch 4. The elevational sketch of the Slab Band (Beam) shows a gap in the top tension reinforcement.



Sketch 5. The plan view of the Slab Band (Beams) shows a gap in the top tension reinforcement. The red circle shows the gap in the tension reinforcement.

SUMMARY OF STRUCTURAL ANALYSIS

6. MATERIAL PROPERTIES

Concrete

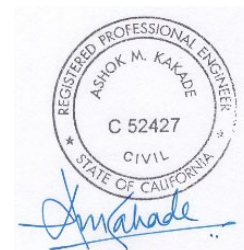
- $f'_c = 5000$ psi

Steel Plates

- Grade: 50 (e.g., ASTM A36/A572)
- $f_y = 50$ ksi

7. DESIGN LOADS

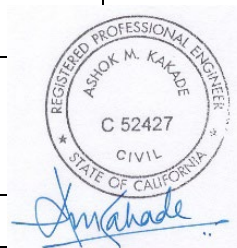
- Self-weight
- Live load 250 psf for driveway
- Imposed Dead Load (finishes) 5 psf



LOAD COMBINATIONS: Per ASCE 7;

Table 1. ASCE 7 Load Combination as defined in computer software, ETABS

ASCE 7 Load Combination			
Load Combination	Load type	Multiplier	Combination
DSlbS1	Dead	1.4	Dead [Strength]
DSlbS2	Dead	1.2	Dead + Live [Strength]
DSlbS2	Live	1.6	
DSlbS3	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DSlbS3	Live	1	
DSlbS3	wind x	1	
DSlbS4	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DSlbS4	Live	1	
DSlbS4	wind x	-1	
DSlbS5	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DSlbS5	Live	1	
DSlbS5	wind y	1	
DSlbS6	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DSlbS6	Live	1	
DSlbS6	wind y	-1	
DSlbS7	Dead	0.9	Dead (min) + Wind [Strength]
DSlbS7	wind x	1	
DSlbS8	Dead	0.9	Dead (min) - Wind [Strength]
DSlbS8	wind x	-1	
DSlbS9	Dead	0.9	Dead (min) + Wind [Strength]
DSlbS9	wind y	1	
DSlbS10	Dead	0.9	Dead (min) - Wind [Strength]
DSlbS10	wind y	-1	
DSlbS11	Dead	1.3	Dead + Live + Static Earthquake [Strength]
DSlbS11	Live	1	
DSlbS11	Eqx	1	



LOAD COMBINATIONS: Per ASCE 7;

Table 1 (Cont.) ASCE 7 Load Combination as defined in computer software, ETABS

DSlbS12	Dead	1.3	Dead + Live - Static Earthquake [Strength]
DSlbS12	Live	1	
DSlbS12	Eqx	-1	
DSlbS13	Dead	1.3	Dead + Live + Static Earthquake [Strength]
DSlbS13	Live	1	
DSlbS13	Eqy	1	
DSlbS14	Dead	1.3	Dead + Live - Static Earthquake [Strength]
DSlbS14	Live	1	
DSlbS14	Eqy	-1	
DSlbS15	Dead	0.8	Dead (min) + Static Earthquake [Strength]
DSlbS15	Eqx	1	
DSlbS16	Dead	0.8	Dead (min) - Static Earthquake [Strength]
DSlbS16	Eqx	-1	
DSlbS17	Dead	0.8	Dead (min) + Static Earthquake [Strength]
DSlbS17	Eqy	1	
DSlbS18	Dead	0.8	Dead (min) - Static Earthquake [Strength]
DSlbS18	Eqy	-1	
DSlbS19	Dead	1.3	Dead + Live + Response Spectrum [Strength]
DSlbS19	Live	1	
DSlbS19	Resp-x	1	
DSlbS20	Dead	1.3	Dead + Live + Response Spectrum [Strength]
DSlbS20	Live	1	
DSlbS20	Resp-y	1	
DSlbS21	Dead	0.8	Dead (min) + Response Spectrum [Strength]
DSlbS21	Resp-x	1	
DSlbS22	Dead	0.8	Dead (min) + Response Spectrum [Strength]
DSlbS22	Resp-y	1	



8. EXISTING BEAM CAPACITY

Flexural Capacity

- $\phi M_n = 4215$ kip-ft top tension

(Refer Sketch 5: As per as built drawing length of top longitudinal bars are curtailed , therefore top tension moment capacity is negligible after the rebar gets curtailed.)

- $\phi M_n = 922$ kip-ft Bottom tension
- (Refer Sketch 5: As per as built drawing bottom longitudinal rebars are not curtailed, provided full length as per as built drawings)

Demand

Moment

$M_u = 5319$ kip – ft (top tension) kips Load Case DSlbs20 (1.3 D + 1 LL + 1.0 RESP-y)

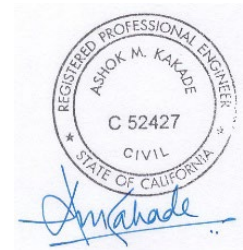
Deficiency Identified: Yes - $M_u 5319$ kip-ft > $\phi M_n 4215$ kip-ft existing capacity is deficient)

$M_u = 1653$ Kip- ft (bottom tension) Load case (DSlbs20)

Deficiency Identified: Yes - $M_u 1635$ kip-ft > $\phi M_n 922$ kip-ft (existing capacity is deficient)

Shear Capacity (existing)

- $V_c = 380$ kips
- $V_s = 614$ kips (stirrups missing for main length of beam)
- $\phi V_n = 0.75 \times (614 + 380) = 745.5$ kips
- (Refer to Sketch 5: Stirrups are provided only on the projecting portion from the column towards the outer tip of the beam. From the column toward the retaining wall, even the minimum stirrups are not provided; therefore, the calculated shear capacity cannot be considered the existing capacity over the entire length.)



Shear

- $V_u = 1085$ kips Load case Dslbs20 (1.3 D + 1.0 L + 1.0 Resp-y)
- Deficiency identified: Yes $V_u = 1085$ kips > 380 kips (Concrete Capacity only, as no stirrups for main length)

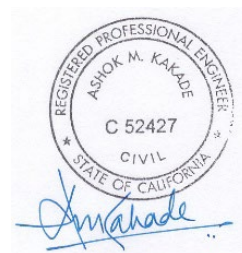
- Demand for Steel plates = $V_u - V_c$ (Stirrups ignored) = 1085 kip - 380 kip = 705 kips---- “A”

8.1.2 Steel Plate Strengthening (Shear) side plates

- Plate size: (width $\times$ thickness) $\frac{3}{4}$ " $\times$ 24 in for full length on both sides
- For a Single Plate -
- ϕV_n (plates) = $0.75 \times 0.6 \times 50 \text{ ksi} \times A_w = 0.6 \times 50 \times 24" \times 0.75" = 405 \text{ kips}$
- For both sides plates = $405 \times 2 = 810 \text{ kips} > 705 \text{ kips}$ as per “A” above
- Therefore, provide both sides 24" $\times$ $\frac{3}{4}$ " 50 ksi steel plates for full length.
- Force per foot one side of plate = $1085 \text{ kips} / (18 \text{ feet}) = 31.9 \text{ kips}$
- Force per foot with plate on both sides = $31.9 \text{ kips} / 2 = 15.95 \text{ kips / foot}$
- Provide: 6 nos. $\frac{3}{4}$ in diameter bolts @ 12" OC
- (HILTI Kwik Bolt Wedge Anchor, checked for conservative 3000 psi)
- Shear per bolt = $15.95 / 6 = 2.65 \text{ kips / bolt} < 10.76 \text{ kips/bolt}$ (As per HILTI Profis)

UPV values fluctuate, and there are long and through cracks and large delamination areas at soffits. Bolt installation values cannot be fully relied upon in the filled-up cracks; therefore, a conservative bolt configuration is proposed.

- Edge distance ≈ 3 in minimum
- **OK in steel shear**
- Refer HILTI Profis Software Calculation as appendix ____
- **Plate Bearing Check**
- Plate thickness: $t = \frac{3}{4}$ in
- Bolt bearing stress: $\sigma_b = \frac{16.2}{1.25 \times 0.75} = 17.2 \text{ ksi}$
- Acceptable for A36/A572 plates.



9. STRENGTHENING DESIGN

9.1.1 Steel Plate Strengthening (Flexure/ Top moment) tension at top

- Plate size: (width $\times$ thickness) 60 inch $\times$ 1 inch
- Plate length: $24 + 4 = 28 \text{ ft}$ (Considering for longest beams)
- Location: top of beam

Moment Contribution

- Steel plate force = $60 \times 1 \times 50 = 3000$ kips
- Lever arm = 34 in (top and bottom steel plates, considered as a steel beam)
- $M_n(\text{plate}) = 34/12 \times 3000 = 8500$ kip-ft
- $M_n(\text{plate}) = 8500$ kip-ft > 5319 kip-ft
- As the rebars are curtailed, top reinforcement cannot be relied upon for the full moment demand.

Composite Capacity

- Total $M_n = \text{Existing} + \text{Plate} = 4683 + 8500 = 13183$ kip-ft
- $\phi M_n = 0.75 \times 8500 = 6375$ kip-ft
- $\phi M_n = 6375$ kip-ft $\geq M_u 5319$ kip-ft ✓

Anchorage Design

- Anchor type: HILTI HIS-N type $\frac{3}{4}$ " anchor bolts, 9 per row
 - Anchor spacing: 12 inches OC
 - Total shear = $5319 / (34/12) = 1877$ kips
 - Shear per Bolt = $(1877 \text{ kips}) / (25 \text{ rows} \times 9 \text{ per row}) = 8.34$ kips
- (Assume it has the same capacity as other HILTI bolts **10.76 kips** OK)

9.1.1 Steel Plate Strengthening (Flexure) Tension at Bottom

- Plate size: (width $\times$ thickness) 14-inch $\times$ $\frac{1}{2}$ inch 2 nos
- Plate length: $24 + 4 = 28$ ft
- Location: bottom of beam



Moment Contribution

- Steel plate force = $28 \times \frac{1}{2} \times 50 = 700$ kips
- Lever arm = 34 in

- $M_n(\text{plate}) = 700 \times (34/12) = 1983 \text{ kip ft}$

Composite Capacity

- Total $M_n = \text{Existing} + \text{Plate} = 1983 + 922 = 2905 \text{ kip-ft}$
- (full reinforcement present at bottom therefore rebar capacity is also considered)
- $\phi M_n = 0.75 \times 2905 = 2178 \text{ kip-ft} > M_u \text{ } 922 \text{ kip-ft} \checkmark$

Anchorage Design

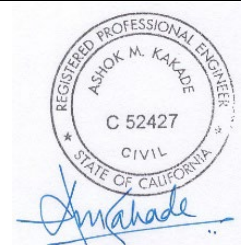
- Anchor type: HILTI expansion type anchor bolts 6 per row
- Anchor spacing: 12 inches OC
- Shear Per Bolt = $(700 \text{ kips}) / (18 \text{ rows} \times 6 \text{ bolt per row}) = 6.48 \text{ kips}$
- (Assume it has the same capacity as other HILTI bolts **10.76 kips OK**)



Table 2. The demand-capacity of the beams B1 and B2.

Demand Capacity Table					
Conservatively designing for the maximum values					
Design for FLEXURE					
	Demand Mu	New Capacity ØMn	D/C Ratio	Status	Additive Steel Plates
Maximum Positive Moment, MU+	922 Kip- ft	2178 kip-ft	0.42	Adequate	Bottom plate ½" thick, 28 inches wide, and full length of the beam.
Maximum Negative Moment, Mu-	5319 Kip-ft	6375 kip-ft	0.83	Adequate	Top plate 1" thick, 60 inches wide, and full length of the beam.

Design for SHEAR					
	Demand Vu	New Capacity ØVn	D/C Ratio	Status	Additive Steel Plates
Column face Vu max	705 kip (Vu1085-Vc 380)	810	0.87	Adequate	Side plates Plates on both vertical faces. ¾" thick, 24" tall, and full length of the beam.
d from column face	340 kips (Vc 720 - Vc 380)	810	0.41	Adequate	Side plates Plates on both vertical faces. ¾" thick, 24" tall, and full length of the beam.



10. DISCUSSIONS

This discussion explains our reasoning for the recommended repair.

The beams (called Slab Bands in the as-built drawings) are partly detailed as a beam for the cantilever portion with stirrups; however, minimum stirrups have not been provided in the portion of the beams that span from the column to the retaining wall.

Also, as per the as-built drawings, the top rebars do not fully extend to the wall; reinforcement has a length of 18' called out on the drawings from the cantilever tip. Thus, a portion of the slab band has no top reinforcement and no stirrups (Sketches 4 & 5). There is another set of top reinforcement in the Slab Band that comes from Wall 4, 6 bars for a length of 6' (Sketch 5), but these rebars do not lap with the remaining top reinforcement; hence, they are inadequate.

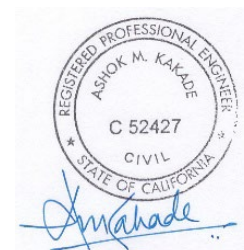
However, the analysis shows that a negative moment persists to the end of the Slab Band. Refer to the Bending Moment diagram from the computer analysis. This has created tensile stresses at the top of the Slab Band (beam), where there is no top reinforcement to resist them. This is most likely the cause of the development of tension cracks at the top.

As the beam was cast and additional floors were constructed, loads continued to accumulate at the cantilever tips of B1 and B2. Due to inadequate reinforcement, a tension crack likely initiated in the tension zone, and the absence of stirrups and side-face (skin) reinforcement further aggravated it. These ACI 318 provisions are intended to control and limit crack propagation and wide cracking in the beam web. Steel reinforcement curtailment in the tension zone likely caused the initiation. The crack was recorded in 2009; however, we suspect it is older, having developed as the structure was fully loaded with dead, superimposed, and live loads.

The proposed strengthening is based on 3D modeling and accounts for the structural configuration and behavior of the members in question. It also addresses the gap in the tensile reinforcement at the top of the slab band and the absence of

11. ASSUMPTIONS & LIMITATIONS

- Based on available drawings and limited field verification
- Reinforcement assumed where not exposed / reference made to as-built drawings
- No change in structural system
- Valid only for stated loads



12. ENGINEER'S CERTIFICATION

I hereby certify that this design complies with the requirements of the California Building Code and applicable standards.



Ashok Kakade, PE
Principal Engineer
Concrete Science, Inc.



Brett Ferrari, SE
Structural Engineer
Ferrari Moe, LLP