

Structural Report

Structural Assessment of Existing Building



Hardwick, VT Town Garage

David Upson, Town Manager, Hardwick, VT



Existing Building

Date of Assessment:	10/28/25
Date of Report:	10/31/25
Property Address:	Hardwick, VT Town Garage
Client Name:	David Upson
Engineer Name:	Ricky McLain, PE, SE
Engineering Company:	Mass Timber Advisors

1. Introduction

As requested, Ricky McLain of Mass Timber Advisors (MTA) performed a limited structural condition assessment of the town garage located on Creamery Street in Hardwick, VT on October 28, 2025. David Upson, Town Manager, was present during the assessment.

The purpose of this assessment was to review the overall structural health and stability of the existing garage, identify conditions that may present near-term risk, and provide clear recommendations to keep the building serviceable until its planned replacement. The Town indicated the garage is approximately 52 years old, has known structural concerns, and is slated for future replacement; this report focuses on risk-based mitigation and service-life extension measures appropriate to an end-of-life facility.

2. Existing Building Summary

The garage is a single-story, monoslope-roof structure with a plan footprint of approximately 90 ft (longitudinal) × 40 ft (transverse). The primary use is interior town truck storage with six 12 ft × 12 ft overhead doors along the front (high roof eave). A small office/break room/bathroom block with limited mezzanine storage exists in the eastern corner of the building.

2.1 Primary structure

- **Foundations / slab:** Concrete slab-on-grade; cast-in-place concrete perimeter walls: approximately 2 ft stem exposure at gable ends; approximately 5 ft-7 in exposed wall at the rear (low-eave) side; front wall at slab elevation along the overhead doors.
- **Roof framing:** Wood rafters approximately 1-3/4 in × 9-5/8 in at 24 in on center (o.c.) forming an approx. 2.5:12 monoslope from front (high) to rear (low)*. Rafters span side-to-side and are spliced or jointed over interior beam-and-column lines.
- **Interior beam and column lines:** Built-up beams (four-ply 2 in × 11-1/2 in wood) bearing over W6x20 steel columns. Wood beams are split half on each side of each column, with dual 4x4 steel angles at the head of each column acting as beam bearing support. Two interior columns per line; interior lines spaced about 13 ft-4 in longitudinally and ≈12 ft-6 in from each front/rear wall (i.e., a total of 12 interior steel columns, with two interior per bearing line and a total of six interior bearing lines).
- **Exterior Walls:** Although most of the exterior wall framing was concealed due to the presence of finishes, a cursory review suggests that structure includes vertical wall studs and steel columns within the walls, aligned with interior columns, supporting the high and low eave ends of the noted roof beams.
- **Ceilings:** Mixed 2×4 framing and wood strapping; rear half sloped to follow roof; front half flat.

See Figures 1 and 2 below for existing roof framing and foundation plans, respectively.

*Slope direction and eave naming follow field observation: roof sheds to the rear; rear wall is low eave.

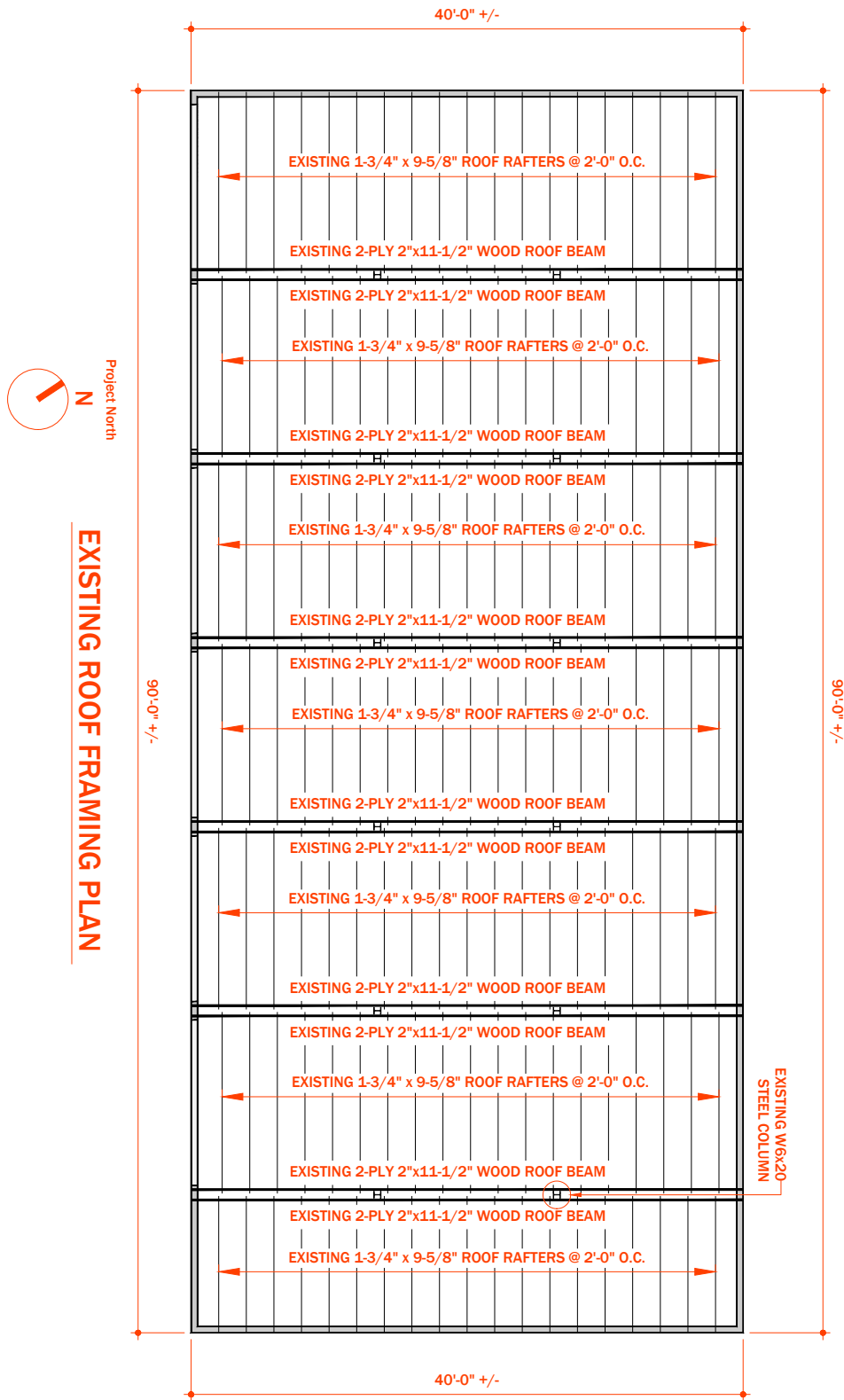


Figure 1: Existing Schematic Roof Framing Plan

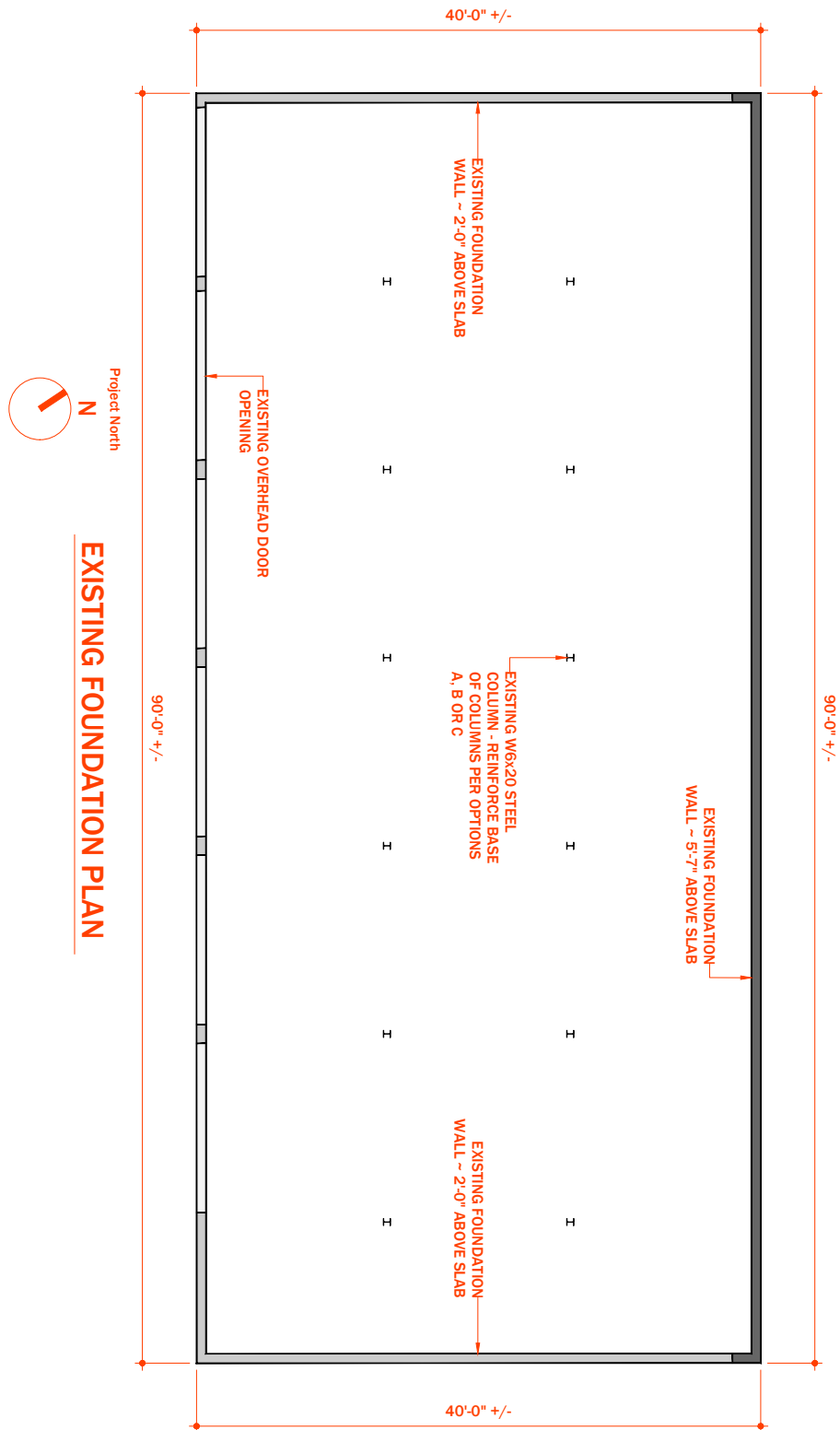


Figure 2: Existing Schematic Foundation Plan

3. Scope, Method, and Limitations

- **Scope:** Visual, non-destructive review of accessible foundations, slab, interior steel columns and beam seats, visible roof/ceiling interfaces (through limited areas of temporarily removed ceiling and insulation), and selected door jambs. One between-door steel jamb column base was exposed for inspection. No finishes or exterior soils were removed except as noted.
- **Not included:** No selective demolition beyond limited column-base and roof framing exposure; no probe openings at roof or wall framing beyond what was already open; no material testing or survey; no MEP (mechanical, electrical, plumbing) review. The existing exterior shed roof at the northeast corner of the building was not included in our assessment.
- **Access limits:** Drop ceilings concealed much of the roof framing; most between-door jamb columns were not exposed. Exterior wall backfill and winter snow storage patterns inferred from site context.
- **Use of this report:** Risk-focused guidance to inform near-term safety and life-extension measures pending replacement.

4. Existing Conditions — Observations

4.1 Interior Steel Columns and Built-Up Beams (Priority 1)

- **General steel corrosion:** W6 interior steel columns show advanced corrosion, particularly at bases where meltwater/track-in and wetting cycles occur. Corrosion and deterioration may be worsened due to any presence of road salts on the slab in winter months. Multiple column bases exhibit section loss and complete or near-complete perforation of flanges/webs at and just above the slab line.
- **Anchorage loss:** Several columns have no visible mechanical anchorage remaining to the concrete slab (baseplates/anchors are either absent, buried in corrosion products, or deteriorated to the point of ineffectiveness) due to rust and corrosion. Other columns show the original anchorage consisted of anchor bolts attaching a base plate below the columns to the top of the slab. It is not known if the slab is thickened at these column bases and/or if there are separate, isolated concrete footings below each of the interior footings.
- **Impact damage:** The bases of some columns are twisted/torqued, with local flange/web buckling, consistent with vehicle or vehicle appendage impact in tight bays and misalignment during maneuvering (especially with snowplows attached).
- **Steel columns between overhead doors:** Matching steel columns at overhead-door jambs (centered between doors) likely act as gravity posts for roof beam support and door jamb stiffeners for wind loading support. The one exposed jamb column base was heavily deteriorated (perforated steel; no visible or reliable anchorage to the foundation below). During exposure, accumulated debris (rust scale, sand, organic fines) spilled from the base pocket, indicating moisture trapping against this column base and chronic wetting/drying.

- **Roof Beams:** Built-up 4-ply 2×11-1/2 wood roof beams bear on 4x4 steel angles attached to the steel column caps. Bearing appears generally intact where visible, but long-term column settlement/rotation risk exists due to base deterioration.

Implication: The primary vertical load path (roof → built-up beams → steel columns → slab/foundation) is compromised at multiple column bases. Loss of anchorage and section at the base reduces axial and bending capacity, diminishes buckling resistance, and introduces disproportionate-collapse risk if a deteriorated column fails under snow load or impact.

4.2 Foundations / Rear Tall Wall (Priority 2)

4.2.1 Moisture infiltration: Widespread efflorescence on interior faces of side and rear walls; damp areas align with exterior wetting and roof shed toward the rear (low eave). The lack of roof gutters is contributing to the presence of water along this rear foundation wall, and this moisture's detrimental impact on the foundation.

- Efflorescence is a salt residue (white in appearance) left behind when water migrates through the foundation walls and evaporates on the inside surface. This is not merely cosmetic: it is evidence of ongoing moisture intrusion from the surrounding soils.
- Moisture intrusion through the foundation is also an indication that moisture is readily present in the soils on the exterior side of the foundation wall, and that these soils likely do not have a method or diverting the moisture away from the foundation (i.e. drainage plane, footing drain). This creates two potential concerns: hydrostatic pressure and frost action.

Hydrostatic Pressure

Moisture present in the soils against the foundation wall increases the inward loading of the soils on the foundation wall due to hydrostatic pressure — the lateral force exerted by water-saturated soil. Unlike static soil pressure (which increases linearly with depth), hydrostatic pressure exerts a uniform force across the wall height and can be significantly higher in magnitude. Most foundation walls are not designed to account for the increased lateral soil loading associated with hydrostatic pressure and therefore a wall which is subject to these loads is at higher risk of inward deflection and related structural failure mechanisms. Although the impacts of hydrostatic pressure on this rear foundation likely do exist, the differential elevation from exterior grade to interior slab is not particularly significant, less than 2 feet, and no obvious signs of foundation wall leaning or deflection were noted. There were however ample signs of foundation wall cracking as explained below.

Frost Action

The presence of moisture ingress through the existing foundation walls also increases the risk of moisture entrapment and freeze-thaw deterioration of the foundation walls, particularly near grade where winter temperatures can induce freezing within the wall cross-section. Over time, this can cause structural issues with the foundation. For example: water enters the foundation wall, then freezes, expands, causes isolated cracking. At the next water ingress cycle, moisture can penetrate more rapidly and with more volume due to the increased cracking, which leads to more freeze-thaw cycling, more foundation impact, and the cycle continues.

Additionally, when moisture in the soil directly adjacent to the foundation freezes, it expands and if conditions allow, can cause the wall to heave upward. As the frozen, saturated soil thaws in the spring months, the expanded soils are compacted, causing settlement.

4.2.2 Other Foundation Conditions Noted

- **Wall Cracking:** At least eight (8) vertical cracks on the rear wall; many originate or terminate at window openings (weakened regions). Typical widths 0.10–0.20 inches; several up to ~7/16 in. Concrete spalling has occurred on both interior and exterior faces at some cracks. Several cracks had loose concrete which could be removed by hand or with limited screwdriver effort during the assessment.
- **Exposed reinforcement:** One region with exposed, corroding rebar and localized spall; corrosion products indicate ongoing moisture ingress and concrete cover distress.
- **Control joints absent:** The ~90-ft long rear foundation wall has no control joints. It appears that the above noted cracking likely started as shrinkage cracking but has expanded and widened with time due to moisture infiltration and freeze-thaw effects. It is typically recommended that foundation walls have control joints at spacings of approximately 20 to 25 feet.
 - Control joints create an intentionally thinner section of wall at discrete locations (with reduced reinforcing steel), forcing the wall to crack controllably as shrinkage occurs. Shrinkage cracks also often develop at the weakest points in walls such as re-entrant corners (i.e. window openings) or near the mid-length of walls. In walls without control joints that are as long as the rear wall in this building (90 ft long), it is not uncommon to have several vertical shrinkage cracks.
- **Frost susceptibility:** With no insulation at interior/exterior faces and a wet wall, freeze-thaw in crack networks is accelerating deterioration.
- **Backfill / grade:** Exterior grade at the rear is estimated as less than 2 ft above interior slab elevation—i.e., low retained height; global soil lateral load is not the governing issue. The concern is wall section integrity and ability to support gravity/lateral loads from the roof and wall amid progressive cracking and reinforcement corrosion.

Implication. The rear wall's serviceability (water-tightness, durability) is poor, and localized strength is reduced near wide cracks and spalled rebar areas. Without mitigation, progressive freeze-thaw and corrosion will worsen.

4.3 Roof / Ceiling System

- **Configuration:** Rafters (~1-3/4 × 9-5/8, 24 in o.c.) span side-to-side approximately 13'-4" between gable end walls and interior beam lines. Ceiling framing masks most connections; only limited bearing zones were visible.
- **Moisture exposure history:** The rear low eave experiences roof-shed snow/rain, with snowbanks and splashback contributing to wall wetting. Interior humidity and roof condensation potential (with sloped/flat mixed ceiling) elevate corrosion risk at steel bases.

- **Condition:** No generalized roof member distress was observed in accessible areas; however, given deferred maintenance and concealment, the true condition of rafter bearings, beam splices, and connections remains unknown.

4.4 Slab-on-Grade / Interior Environment

- **Wet-use conditions:** Evidence of standing water/wet tracks typical of a plow/truck garage. Standing water was present on the slab at several areas along the rear foundation during the assessment. No global slab settlement was documented during this limited visit, but persistent wetting at column bases is confirmed.
- **Durability:** Wet deicing salts likely present; this environment accelerates steel corrosion.

The following images provide an overview of the existing conditions.



Figure 3: Existing Roof Framing



Figure 4: Steel Corrosion and Deterioration at Base of Interior Steel Column



Figure 5: Steel Corrosion and Impact Warping at Base of Interior Steel Column



Figure 6: Steel Corrosion and Deterioration at Base of Interior Steel Column



Figure 7: Steel Corrosion and Debris Accumulation at Base of Steel Column at Overhead Door



Figure 8: Spalled Concrete and Exposed/Corroding Reinforcing Steel at Rear Foundation Wall



Figure 9: Efflorescence at West Foundation Wall



Figure 10: Active Moisture and Cracking at Rear Foundation Wall



Figure 11: Vertical Cracking at Rear Foundation Wall



Figure 12: Vertical Cracking at Rear Foundation Wall



Figure 13: Vertical Cracking at Rear Foundation Wall



Figure 14: Vertical Cracking and Spalling at Rear Foundation Wall



Figure 15: Vertical Crack Width Measurement (7/16\") at Rear Foundation Wall

5. Building Codes and Structural Loading Criteria

5.1 Applicable Building Codes and Standards

The existing roof framing conditions and structural components were analyzed using common structural engineering principles and following the requirements of the applicable building code and referenced standards including:

- *ASCE 7-10 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*
- *National Design Specification for Wood Construction (NDS)*
- *American Institute of Steel Construction (AISC) standards*
- 2015 International Building Code, which is currently adopted by the 2015 Vermont Fire and Building Safety Code.

The following design loads have been used to perform our structural analysis of existing roof framing conditions.

5.2 Design Dead & Live Loads

Component	Design Dead Load	Design Live Load
Roofs	20 psf	20 psf + Snow Load (see below)

The allowable live load deflection criteria for roof members is Span/240 for live loading conditions and Span/180 for total loading conditions, in accordance with IBC 2015 Table 1604.3.

5.3 Design Snow Loads

Hardwick, Vermont ground snow load = 60 pounds per square foot (PSF) per the minimum ground snow loads map, published by the State of Vermont Division of Fire Safety, available [here](#). The ground snow load is adjusted by the following factors:

- Exposure factor, $C_e = 1.0$, (determined in accordance with ASCE 7-10 Table 7-2) partially exposed, terrain category C
- Thermal factor, $C_t = 1.1$, structures with cold, ventilated roofs (determined in accordance with ASCE 7-10 Table 7-3)
- Importance factor, $I = 1.0$ using Risk Category = II (determined in accordance with ASCE 7-10 Tables 1.5-1 and 1.5-2)

The flat roof snow load is calculated as:

$$P_f = (0.7)(1.0)(1.1)(1.0)(60) = 46.2 \text{ psf.}$$

6. Roof Framing Structural Analysis Results

The existing roof framing was analyzed using the span and loading conditions noted above. Wood roof framing members (rafters, beams) were analyzed assuming Spruce-Pine-Fir #2 (species and grade), which is the most commonly available and used softwood species and grade in Vermont. The existing steel columns were analyzed assuming $F_y = 36$ ksi steel.

All the existing roof framing was determined to be structurally adequate for the present dead, snow and roof live load conditions. Note that this conclusion is based only on having structural members which are fully intact and without structural degradation (rusting, corrosion, etc.). As discussed, the condition of the bases of the existing steel columns is such that their vertical and lateral load carrying capacity has been significantly reduced and should be addressed through reinforcement or replacement.

A summary of the structural analysis results of the existing roof framing is presented in the table below.

Framing Component	Actual Snow Load Capacity	Required Snow Load Capacity
Roof Rafters	50.4 PSF	46.2 PSF
Roof Beams	45.3 PSF	46.2 PSF
Interior Steel Columns	400 PSF	46.2 PSF

7. Risk Assessment (Prioritized)

1. Immediate life-safety risk — steel column base failures

Multiple interior (and at least one south wall jamb) columns have severe base deterioration and no reliable anchorage. Loss of a column during peak snow or due to vehicle impact could precipitate progressive framing distress along the line of support.

2. Near-term durability/structural risk — rear foundation wall cracking

Wide vertical cracks, exposed rebar, and active moisture at the uninsulated rear wall create a freeze-thaw and corrosion cycle that is degrading section capacity locally and increasing water intrusion into the space.

3. Serviceability and secondary risk — continued wetting at interior column line

Without improved drainage/housekeeping, deicing salts and wetting will continue to attack column repairs, shortening their life.

8. Recommendations

8.1 Steel Column Repair (Priority 1)

Prior to significant snow fall, it is highly recommended that either option A, option B or option C presented below is accomplished. Option A is intended to be a temporary solution while options B and C will gain several additional years of service life for the building.

Option A. Shore and restrict use around compromised columns

- Install adjustable steel shores (or timber posts with screw jacks) directly adjacent to any column base showing >25% section loss, perforation, or no visible anchorage. Provide continuous bearing at top and bottom; shim tight.
- Document (photo log) each column ID and condition state for tracking.

Temporary corrosion arrest and housekeeping.

- Dry sweep and remove debris from all column base pockets; keep areas dry when practicable.
- Apply rust converter/primer to accessible steel as an interim measure (this is not a structural fix).

Option B: Column Base Repairs & Impact Protection

Replace column base segment and install new baseplate/anchors.

- Cut-back existing steel columns to sound steel above corrosion line; shop-fabricate a replacement lower stub (matching W6 size) with full-penetration weld splice above splash zone.
- Provide new baseplates with (4) adhesive anchors into slab (using a two-part epoxy such as Simpson SET 3G). Grout under baseplate as needed to provide uniform and level bearing of base plate on existing slab; add galvanized sleeve or protective coating at splash zone.
- Optional: Install curb-style concrete pedestals (e.g., 6–8 in high) to elevate the new base above wet slab.

Option C: Concrete piers and elevated steel base anchorage

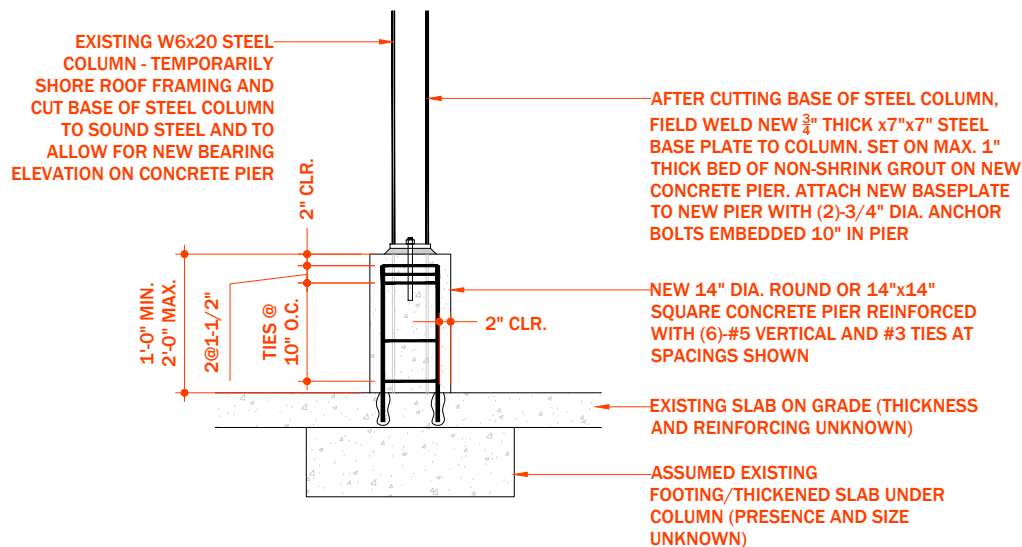
- Form and pour a 1–2 ft high reinforced concrete pier under the base of each column (cut base of steel column to sound steel, new concrete pier to be dimensioned to clear truck/plow geometry), doweled and keyed into the slab. New, elevated base of steel column to have mechanical anchorage to top of new concrete pier. See Figure 16 for a detail of this option.
- **Note:** This option reduces accidental impact risk but still requires sound steel at the top of the concrete pier. Severely wasted steel must be cut out and replaced prior to new concrete piers are installed.

Door-jamb columns (between overhead doors).

- Repeat the same base repair/anchorage strategy. Where jamb columns share door hardware loads, verify lateral bracing and provide new base connections capable of combined gravity and door loads.

Finish & protection.

- After repairs, apply zinc-rich primer and industrial epoxy topcoat at bases; consider gasketed base wraps to shed splash.



1 STEEL COLUMN PIER DETAIL

Figure 16: New Concrete Pier / Steel Base Plate Detail (Option C)

8.2 Rear Wall Moisture & Structural Repairs (Priority 2)

Exterior water management.

- Add continuous gutters at the roof edge with downspouts discharging to grade-away leaders or to daylight; provide splash protection to prevent direct wetting of the wall.
- Regrade the rear swale for positive drainage; avoid winter snow storage piled directly against the rear wall.

Crack and spall repair.

- Map and number all cracks; record widths.
- For cracks $\leq 1/8$ in: Epoxy injection to restore continuity and water-tightness.

- For cracks $>1/8$ in to $\sim 7/16$ in: Route-and-seal with semi-rigid epoxy or polyurethane after cleaning
- Exposed rebar/spalls: Remove loose concrete, abrade/rust-convert steel, apply corrosion-inhibiting primer, then patch with polymer-modified structural repair mortar; cure and seal.

Thermal/freeze–thaw mitigation.

- Add exterior insulation (e.g., rigid mineral wool or XPS with protective coating) and waterproofing at the rear wall to reduce freeze–thaw within the section. Seal all patch transitions.

8.3 Operational & Housekeeping Measures (Ongoing)

- **Keep column bases dry:** squeegee/dry-vac after storms; deploy rubber mats or sacrificial curbs to divert meltwater.
- **Deicing salts:** Minimize accumulation near columns; rinse zone during thaws when safe to do so.
- **Impact controls:** Add high-visibility guards (curbs/bollards) and painted alignment marks on slab to guide trucks. Confirm clear bay widths with plow geometry before constructing encasements.

9. Conclusions

- The building shows significant deferred maintenance. The most critical deficiency is advanced corrosion and anchorage loss at all interior steel column bases (including at least one between-door jamb column). This condition directly endangers the primary gravity load path and must be addressed first with shoring and base repair/replacement (and impact protection).
- The rear concrete wall exhibits widespread vertical cracking, efflorescence, and localized rebar corrosion consistent with long-term wetting and freeze–thaw in an uninsulated wall. This primarily affects durability and localized section strength and should be repaired and protected (water management, crack/spall repair, optional insulation/waterproofing).
- The roof rafter sizing appears adequate for balanced snow given the observed multi-span support, but reserve capacity is not generous, and localized load cases (rear-eave accumulation) plus column risk dictate caution.
- With the Priority 1 and Priority 2 measures implemented, we believe the garage can be stabilized and maintained for limited continued service as an end-of-life facility until the replacement building is realized.

10. Limitations

The conclusions, analysis, and opinions presented in this report have been prepared with a reasonable degree of engineering certainty, based on visual observations and the best information available at the time of the inspection. This evaluation is limited to the condition of the structure at the time of inspection and is based on visible and accessible areas only, with no destructive or invasive testing performed. Since the building was complete and partially finished, with some structural systems concealed, and since we were not expected to perform demolition/excavation, this was a limited observation based upon evidence readily available at the time of the site visit. A limited observation cannot reveal every detail, defect or deficiency of a structure that might be of interest. It is important to note that this inspection is not exhaustive and does not constitute a comprehensive engineering evaluation; it is intended to provide a professional opinion based on observed evidence. The contents of this report are intended solely for the use of the designated client and its representatives. Any misuse of this information by others is not the engineer's responsibility.

I trust that this report adequately addresses your questions. Should you have any further questions or require additional assistance, please do not hesitate to contact me. Thank you for the opportunity to assist in this matter.

Sincerely,



Richard McLain, PE, SE
Mass Timber Advisors

