ENGINEERING VENTURES PC

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January 22, 2015

Town of Plainfield, NH EV #15006
Attn.: Michael O'Leary
Cc: Steve Halleran
110 Main St.
Meriden, NH 03770 DRAFT
mol03766@tds.net
Cc: plainfield.ta@plainfieldnh.org

Re: Town Highway Garage Building Structural Evaluation
Plainfield, NH

Dear Michael:

The following is a brief summary of my observations/assumptions and recommendations from my June 24th 2014 and January 12th 2015 site visits, review of existing project documents as they relate to the structure, and structural analysis work. The intent of this report is to assist the Town in long-term planning implementation of roof-mounted solar equipment on a portion of the building. Because I have already evaluated the roof trusses and reported on their condition as part of earlier work relating to this building, this portion of the structure will be excluded from this report and I would direct the Town to reference my August 19, 2014 report for information regarding the trusses.

Observations and Assumptions:

General:

1. This report addresses the structural aspects of the building design only. Other aspects of the building design, including but not limited to architectural design, mechanical design, electrical design are not included in the scope of this document. Investigation for the presence of hazardous materials is also excluded from the scope of this report.
2. It is understood that no licensed design professionals were involved in the design of this building. No building plans are in existence. All information contained within this report is based on visual observations obtained at the site, an informal specification document from Madeira Construction Co., Inc. dated August 3, 1983, and a letter from Timothy Buzzell & Associates December 2, 1983.
3. No destructive testing was included as part of our scope of work.
4. The building was analyzed under current code-required loadings.
5. According to project document provided by the Town, the highway garage was constructed in 1983.
6. The building measures approximately 50 ft x 100 ft in plan view.

Roof:

1. The roof is framed with prefabricated gable-style wood trusses, which span 50 ft. and are spaced at 2'-0" on center, (o.c.).
2. The roofing consists of metal roof panels and it is the original roofing. Beneath the roofing is a layer of asphalt paper, and beneath that are 1 in. x 4 in. wood strapping members spaced apart at 1ft. on center, (o.c.).
3. Corrosion of the roofing was observed in places.
4. While not structural in nature, condensation was observed beneath the asphalt paper near the gable ends of the building, (Figure 1). This is indicative of inadequate ventilation of the attic space. Vents were observed on the gable ends of the building but asphalt paper appeared to be obstructing the ridge area of the roof, (Figure 2). This obstruction would interfere with the proper functioning of a ridge vent system. I was not able to observe if a ridge vent was in place on this building. Proper vents were observed along the building eaves. However, I was not able to observe them closely and thus could not determine whether or not they were obstructed with insulation.
5. I was not able to gain access to the roof eave in order to observe attachment of the truss ends to the wall below. Hurricane ties should be installed as part of future repair work to the roof if none are in place. An example of this type of product which is commonly used in construction is the H2.5T hurricane tie manufactured by Simpson Strong-Tie.
6. The gable end trusses (trusses at the east and west end walls) were not designed for or braced to withstand out-of–plane wind pressure. The framing between top and bottom chords consisted of 2x4 wood studs laid in the flatwise orientation, (Figure 3), which is structurally not adequate, particularly closer to the ridge of the roof. Additionally, the bottom chords of the gable end trusses, (the horizontal members spanning across the building at the ceiling level) were not observed to be properly braced back to the roof framing to support out of plane wind loads. Some truss erection bracing was observed to be tied into these gable ends but this is not sufficient to meet current code requirements. This bracing is important because the top of the building wall framing ties into the underside of this bottom chord and the effect of not having the bracing of this cord is to create a hinge point at the bottom chord.
7. In its current configuration, the metal roofing and wood strapping system serves as a de facto roof diaphragm. The purpose of the diaphragm is to transfer lateral forces, (ex., wind, seismic loads) into shear walls, (which are essentially walls of the structure which have been specially designed to resist shear and overturning forces generated from wind and seismic loads). This system will not meet current building codes or building codes in place at the time of construction of the building.

Walls:

1. The walls are 16 ft in overall height, consisting of a concrete foundation wall projecting 4 ft above grade and topped with a 12 ft tall 2x6 wood stud wall framing system. The wall studs were observed to be spaced apart at 16" o.c. They were framed from rough-sawn lumber which was planed on the depth to produce the 5.5"

actual depth of a standard dressed 2x6. The width was left in a rough-sawn configuration and was found to vary between 1 5/8. in and 1 3/4 in.

2. Being in the rough-sawn configuration lumber grade stamps were not observed on the wall studs. They were likely obtained from a local saw mill and are likely either eastern white pine or hemlock.
3. 2x4 Diagonal wood bracing was observed to have been let in to the wall studs, (Figure 4). This diagonal bracing was possibly intended to prevent racking of the wall assembly during erection. The bracing members have an actual width of 1 1/2 in. and current building codes permit a maximum notch depth of 1 3/8 in. into wall studs. Thus any wall studs which have been notched do not comply with current building codes. In the un-notched configuration and assuming a lumber grade of Eastern Hemlock – balsam fir No. 2, the wall studs would be structurally adequate to support current code-required loads. The extent of the wall studs which have been improperly notched could not be determined as the wall was concealed by finishes.
4. Though it could not be observed, it is likely that the wall framing and member end connections in the area adjacent to the overhead doors on the east and west ends of the building is deficient. The structure in this area is important because it carries additional lateral wind loads because the structure at each edge of the opening is responsible for supporting lateral wind loads imparted onto half of the width of the overhead doors.
5. The bottom plate of the wall system consists of a non-pressure treated 2x6 sill plate, (Figure 4). Current building codes require that wood members which are in contact with concrete be pressure treated. Only minimal anchorage of the sill plate appearing to consist of powder actuated fasteners was observed during the January 12 site visit, (Figure 4). This configuration is insufficient for current code-required loadings as well as code-required loadings in place at the time of construction. The code also requires that the sill plate be attached to the foundation with anchor bolts or code-approved straps. Due to the relatively small portion of framing observed, it could not be conclusively determined that this was the foundation anchorage which was carried around the entire perimeter of the structure. Additional anchorage provisions are required at the edges of shear wall sections. It is assumed that this anchorage is not in place.
6. The exterior walls are sheathed with 4 ft x 8 ft sheets of 5/8" gypsum wall board on the interior face and T1-11 plywood siding on the exterior face. The thickness of the T1-11 siding is believed to be 5/8 in. but this could not be confirmed as it was concealed by finishes along its edges and exterior face. No wood blocking was observed to have been installed between the studs to provide a surface for sheathing panel edge fastening. This blocking is required for proper shear wall design and function, (as noted earlier in this report, shear wall assemblies are specially designed walls which are capable of keeping the building upright when subjected to wind and seismic forces). The east wall of the structure is the most structurally deficient for shear wall design, (due to the large percentage of unsheathed wall area due to door and window openings) followed by the west end wall, south wall, and the north wall.
7. The interior bearing walls supporting the mezzanine are framed with 2x6 studs. This framing was concealed by finishes but similar to the exterior walls the studs likely do

not include blocking at the wall sheathing panel edges, thus compromising their effectiveness for use as shear walls.

8. The concrete foundation wall consists of a 10 in. thick wall. A significant portion of the wall on the south edge of the building was concealed from view. However, cracks were observed at two locations along the portion of the wall on the north edge of the building, (Figure 5). The cracks extended from the top of the wall to the floor level. There was no differential movement observed at the crack locations and the foundation wall condition below the floor level was concealed from view.

<u>Mezzanine:</u>

1. The mezzanine floor framing consists of wood joists at 16 in. o.c. spacing. An office area is framed with 2x10 joists and a kitchen/break area is framed with 2x8 joists. The current code-required design floor live loads for the office and kitchen/break area are 50 pounds per square foot, (psf) and 100 psf, respectively. Under these loadings, the office floor joists are structurally adequate. The kitchen/break room floor joists are adequate if the joists are two-span joists, (span continuously over three bearing walls). For the single span condition, (spanning between two walls) they are overstressed. The office floor joists frame into a ledger on one end and to a double 2x10 header on the opposite end. Both the double 2x10 header and the joist connection to it, (Figure 6) are structurally deficient. In Figure 6 it can be observed that the floor joist end has pulled away from the header.

<u>Foundation and Floor Slab:</u>

1. The foundation was concealed from view so details such as size and depth of footings and reinforcing configuration could not be determined. However, it did not appear that the building had foundation issues such as differential settlement. The floor slab appeared to be in good condition.

Discussion and Recommendations:

<u>General:</u>

1. The building contains a fairly long list of structural deficiencies which are indicative of the lack of utilization of design professionals in the building design process and a lack of familiarity of the contractor with some building code requirements at that time.
2. The following recommendations highlight changes that would need to be made to the structure to bring it into compliance with current building code requirements. These requirements may be triggered should the Town wish to renovate or alter the structure which would either increase the loading on existing elements or weaken them beyond current code-mandated threshold values.
3. Design of any of the structural upgrades discussed in this report is not included in our scope of work. However, a proposal for structural engineering services can be provided upon request.

Roof:

1. As per the Engineering Ventures report dated August 19, 2014, the existing metal roof is at the end of its design life and consideration should be given to its replacement. The trusses themselves should also be upgraded as per this report.
2. The new roof system should include plywood sheathing for diaphragm action and be designed with proper insulation and ventilation systems so as to eliminate condensation issues.
3. Hurricane ties should be installed as part of the roof replacement work is none are found to be currently in place.
4. The gable end trusses should be reinforced to support out of plane wind loads and a proper lateral bracing system for the gable end truss bottom chords should be provided.

Walls:

1. Aside from the vertical cracks observed along the north wall, the visible portion of the concrete walls appeared to be in good condition.
2. Due to the implementation of 2x diagonal bracing during construction, the wood studs have been notched to a depth which exceeds current code requirements. Any notched studs would have to be reinforced or replaced.
3. A pressure treated sill plate would need to be provided.
4. Sill plate anchorage would need to be designed and installed. This could possibly consist of epoxy anchor bolts embedded into the top of the foundation wall. The minimum prescriptive sill plate anchorage is ½" dia. Anchor bolts spaced not more than 6 ft. apart, (IBC 2009 2308.3.3), but more substantial anchorage may be needed for this application.
5. Wood blocking would need to be provided at sheathing panel edges to permit proper functioning of the walls as shear walls. Additional fastening of the sheathing to the framing and blocking would also be needed in order to achieve published shear wall design values. The east portion of the structure may require plywood sheathing on both sides in order to achieve the required shear resistance. Additionally, additional wood studs and special hold-down/anchorage requirements would be needed at the edges of shear wall elements.

Mezzanine:

1. Floor joists in the single span configuration beneath the kitchen/break area should be reinforced.
2. Face-mount joist hangers should be provided at the ends of the joists beneath the office area which frame into the side of the (2) – 2x10 header.

Thank you for the opportunity to be of assistance. Please feel free to contact me with any questions regarding the above.

Sincerely,

Miles Stetson, PE
Project Engineer
Engineering Ventures, PC