Date: January 29, 2025

To: John Pete – FFA Architecture

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Re: Gearhart Public Safety Building – Seismic/Tsunami Assessment

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The existing combined Fire and Police Facility is a single-story structure with CMU (concrete masonry unit) exterior walls and a wood-framed roof. The building is located approximately half a mile from the shoreline at 670 Pacific Way and is on a flat site. The building was originally constructed in 1958, and the two western apparatus bays were added in 1966. As-built construction documents are not currently available; therefore, assumptions regarding the existing construction details are based on limited onsite observations and experience with similar buildings from the same era.

Executive Summary

Expected Building Performance

The building is expected to perform poorly and pose a significant hazard to life safety during a moderate earthquake, as well as during a code-level seismic event and subsequent tsunami, due to significant deficiencies in its lateral force resisting system and poor subgrade/foundation conditions. Under seismic loading, at least 80% of CMU shear walls are likely to be overstressed under in-plane shear loads, with the potential for shear failure. The exterior walls are unlikely to be adequately anchored to the roof framing, posing a risk of losing gravity support for the roof framing and leading to potential building collapse. Additionally, the roof diaphragm likely lacks the necessary capacity to effectively transfer lateral forces to the shear walls, which could result in overstressing of the roof framing. Significant geotechnical hazards are present at the site including liquefaction and lateral spreading, resulting in the loss of foundation support in a seismic event. Furthermore, a design-level tsunami would result in water levels above the existing roof elevation and exerting lateral forces well exceeding those experienced during the code level seismic event.

Building Retrofit Requirements

Retrofits required to mitigate the seismic deficiencies would be extensive and disruptive to operation of the facility. The existing foundations would require new deep foundations or ground improvement to mitigate the liquefaction and lateral spread hazards. Installation of new foundation support at existing interior footings is costly and disruptive to the interior of the building, likely requiring removal of most interior walls and MEP systems. All CMU walls would require new strongback bracing to brace the walls for out-of-plane seismic loads and new ties into the roof sheathing. Strongbacks would consist of new steel posts spaced approximately 4 feet to 8 feet on center. Strengthening of the CMU walls for in-plane shear loads is likely required for most of the walls. This could consist of adding new reinforced shotcrete facing on the existing CMU walls, or new fully grouted CMU walls, to increase the in-plane shear strength. All of the wood sill plates require additional anchorage into the existing CMU and/or retrofitted walls to transfer diaphragm forces into the shear walls, and this work will likely require removing roof sheathing along all of the existing CMU walls to gain access from above. Additionally, all of the roof sheathing would likely require additional nailing to transfer the seismic lateral forces to the CMU shear walls, requiring all roofing to be removed and replaced.

The scope of construction needed to retrofit the building is similar to that required for new construction – foundations, walls, roof framing and connections – without providing any added space or modernization to the facility. The remaining building would contain many elements that are already more than half way through their anticipated useable life. We would anticipate the cost of a retrofit to be more than half of the structural cost for a newly constructed building on the same site. Furthermore, these retrofits are suitable only for upgrading the building for seismic forces and mitigating geotechnical hazards. Retrofitting for tsunami hazards would likely be impractical as it would necessitate substantial new construction and would make reuse of the existing building unnecessary.

Site Observations

The building has an L-shaped configuration, measuring approximately 96’ x 84’, with a 50’ x 54’ reentrant corner on the northeast side. The total roof area is approximately 5,364 square feet. The perimeter walls are constructed of CMU (concrete masonry unit) blocks, and the roof framing consists of wood 2x roof joists supporting a plywood deck. The roof joists are typically supported by bearing plates on the top face of the CMU walls.

Figure 1: Overall Exterior

Overall, the building appears to be nearing the end of its expected service life. There are localized areas of structural deterioration observed in the exterior wall and roof framing. Erosion is evident on numerous exterior CMU blocks – as shown in Figure 2 below. A crack is present on the northwest corner of the building, extending approximately half the height of the wall. At this location, in some sections of the crack, the joint mortar appears to have eroded – as shown in Figure 3 below.

Figure 2: CMU Block Erosion

Figure 3: Crack in Block and Missing Joint Mortar

Extensive paint peeling is visible around the building's perimeter, though it does not appear to have significantly impacted the condition of the CMU blocks. The fascia board shows significant weathering in several areas and appears to be nearing the end of its service life.

Figure 4: West Wall Peeling Paint

The existing wood roof framing is visible in limited locations inside the building and appears to be in generally good condition, with some evidence of past water intrusion in isolated areas. However, the intrusion does not appear to be extensive, and the wood appears to be structurally sound. The sill plates atop the CMU walls, while not pressure-treated, also show signs of past water exposure but do not appear to be structurally compromised when observed from the interior.

Figure 5: Roof Framing Sill Plate

A recent windstorm removed a section of facia board at the southwest corner of the building, exposing the rafter ends and rim board as shown in Figure 6. The exposed rafter ends were observed to be decayed and have significant deterioration, and there appears to be decay in the original rim board. Since the roof flashing detail is likely the same around the entire perimeter of the building, we would expect a large portion of the rafter ends to have at least some decay.

Figure 6: Exterior view of Framing

In the building's reentrant corner on the northeast, two small wood-framed canopies are attached to the exterior CMU walls. These canopies are bolted to the wall faces using ledger boards and are supported by posts on the side opposite the walls. Based on their construction details, these canopies are unlikely to have been designed for seismic loads. The canopies do not appear to have been constructed with pressure-treated wood and exhibit extensive water damage. Based on their age and exposure, some decay and loss of wood density is expected. Additionally, many metal connectors on the canopies show significant corrosion.

Figure 7: Exterior Canopy Framing

Figure 8: Exterior Canopy Framing

At the Northeast corner of the site there is a small wood framed storage building. The building is approximately 35'x 40', one story, and has a gabled roof. The building is framed with three bays of wood trusses supported on wood posts. Spanning between the roof trusses are 2x purlins supporting metal roof panels. The walls consist of metal roof panels spanning vertically, and are braced by flatwise wind girts. There is no slab on grade in the building, and no foundations are visible.

Figure 9: Storage Building Framing

The roof framing does not appear to be framed with pressure treated wood and there appears to be some water damage in some of the roof framing purlins and trusses. The wood posts do appear to be pressure treated lumber; however, they are in contact with grade and may have some deterioration at the base – as shown in Figure 10.

Figure 10: Wood Post in Contact with Grade

The metal roof panels that make up the roof and exterior walls of the storage building are corroded significantly on the roof, and in many locations on the walls – as shown in Figure 11. Additionally, many of the screws that fasten the metal panels to the wood framing are corroded.

Figure 11: Metal Panel Corrosion

Structural Evaluation

The structure has been evaluated for potential deficiencies under seismic and tsunami loading. The following structural deficiencies are identified based on the Tier 1 analysis procedure outlined in the American Society of Civil Engineers Seismic Evaluation of Existing Buildings (ASCE 41-17). As a Risk Category IV structure – defined by the building code as an “Essential Facility” - the Basic Performance Objective for existing structures (BPOE) required for this type of structure is Immediate Occupancy for the BSE-1E earthquake (225yr) and Life Safety for the BSE-2E earthquake (975yr). For tsunami loads the structure has been evaluated under using ASCE 7-22 (Minimum Loads and Associated Criteria for Buildings and other Structures) for a Risk Category IV structure. The 2022 Oregon Structural Specialty Code (OSSC) requires that all Risk Category III and IV buildings be designed per chapter 6 of ASCE 7. This chapter requires that all Risk Category IV structures are designed to meet the immediate occupancy performance objective under the Maximum Considered Tsunami (MCT) which is a 2,475yr event.

Identified Deficiencies and Hazards

Geotechnical Hazards

Potential geotechnical hazards at the site consist of ground shaking, liquefaction, and lateral spread. During the design earthquake significant ground shaking at the site is expected and may last for several minutes. Ground shaking is expected to generate building inertial forces greater than 1g. Preliminary site borings provided by the geotechnical engineer, and available Oregon DOGAMI hazards maps, indicate the site is located in a zone with a high potential of soil liquefaction. Liquefaction could cause significant vertical differential settlement and seriously damage walls and foundations. Additionally, based on preliminary discussions with the geotechnical engineer significant lateral spread is expected at this site, which could lead to several feet of differential horizontal translation between foundations.

Seismic Force Resisting System

The seismic force resisting system consists of CMU shear walls around the perimeter of the building and wood framed diaphragm roof. The extent of grouting and reinforcing in the CMU is unknown; however, we would expect a building of this era to be lightly reinforced with partially grouted CMU. The Fire Department has drilled numerous walls over the life of the building and the Fire Chief has indicated that they have not been able to locate any grouted cells. Based on the layout of windows and doors we have estimated the North and East walls are approximately 50% available to resist lateral loads. The west wall was observed to be 100% solid with no window or door openings. The south wall is mostly openings for the apparatus bay doors, and we have estimated only 12 feet of wall available to resist shear. For seismic loading the North, South and East walls the shear stress checks on the CMU walls indicate that the shear stresses exceed acceptable limits and would require retrofits. Additionally, tsunami loads in the East West direction are 5x greater than the seismic base shear.

Diaphragms

The diaphragm appears to consist of wood structural panels and blocking over the CMU shear walls. The roof sheathing appears to have blocking at panel edges, as observed in Figure 12, but may not be consistent throughout the building. Transfer of seismic forces to from the diaphragm to the CMU wall appears to be through the blocking and top plate then into the CMU wall using top plate anchor bolts. Nailing of the diaphragm blocking is unknown; however, it was observed onsite that at some locations the nailing missed the blocking, thus is not effective. No anchors from the plate into the wall were observed during our site observation; however, we would assume some anchor bolts to be present based on the era of construction. Regardless, the number of anchor bolts from sill plate into the top of the CMU wall is less than we would expect to be needed to adequately transfer lateral forces. Therefore, we do not believe there is adequate diaphragm capacity to transfer lateral forces into the shear walls.

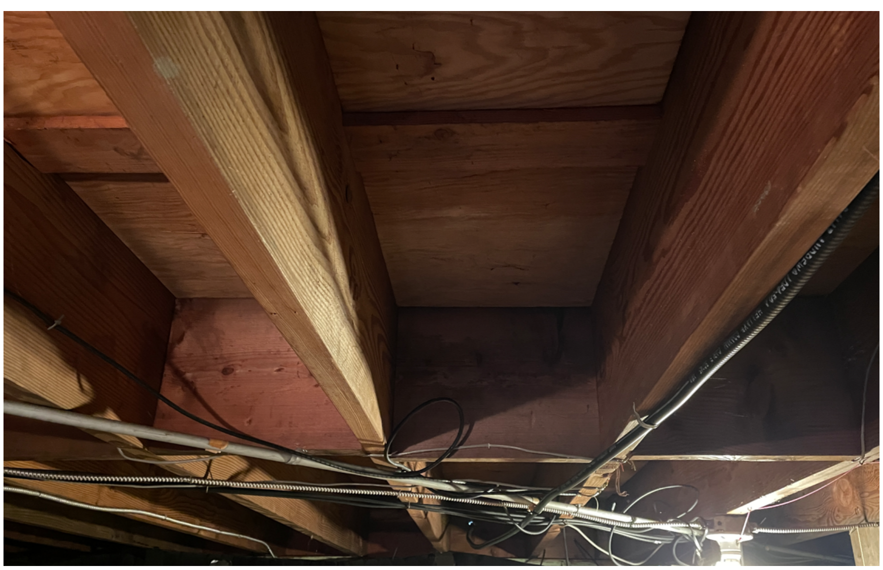

Figure 12: Potential Roof Sheathing Blocking

The plan “L” shape of the building creates a reentrant corner irregularity in the roof diaphragm. The diaphragm likely does not have adequate capacity to transfer forces around the reentrant corner. This could result in localized damage of the diaphragm around the reentrant corner. Based on the construction date of the building we would not expect there to be adequate diaphragm chords or cross ties between chords. Without these connections the diaphragm may not be able to transfer seismic forces to the shear walls.

Based on site observations there does not appear to be anchorage of the exterior walls to the roof diaphragm beyond toe nails. This will result in the wall pulling away from the roof joists and the loss of gravity support for the roof joists and potential collapse of the wall. Retrofit of this condition would require steel straps tying the walls into the roof to prevent the wall from detaching from the roof.

Canopy Structures

The exterior wood-framed canopies lack an adequate lateral system in both directions. For lateral loads perpendicular to the CMU walls, there are no lateral ties connecting to the main wood roof diaphragm, and lateral forces would be resisted through cross-grain bending of the wood ledger. For lateral forces parallel to the CMU walls, wood-framed knee braces are used; however, there do not appear to be sufficient fasteners to effectively transfer lateral forces through these braces. These canopies would likely collapse during the design earthquake and could block exit paths out of the building. Additionally, the canopy ledgers are below the main roof elevation and will impart additional out of plane loading on the CMU walls. The CMU walls are likely not adequate to resist additional out of plane seismic loading from the canopies. This could lead to an out-of-plane wall failure and loss of gravity support for the interior roof framing and canopy framing.

Foundations

For a building of this size, type, and era of construction we would expect the foundations to consist of shallow strip footings supporting the CMU walls and shallow spread footings supporting the columns. For this evaluation we have assumed the building is supported on shallow strip footings and spread footings. These types of foundations are typically only able to accommodate small amounts of differential settlement, typically less than 1”, and are likely not adequate to accommodate large differential settlement associated with liquefaction. In addition to vertical differential settlement, lateral spread at the site may lead to several feet of differential horizontal translation between footings, and could cause a partial or total collapse of the building.

Storage Building

The storage building lacks a well-defined seismic lateral force resisting system. Any resistance to lateral shear forces relies primarily on the metal roof panels, which serve as both siding and roofing. However, the building's south side features a large sliding barn door that effectively reduces the available shear-resisting length by approximately 50%.

The roof diaphragm comprises metal panel roofing, but a ridge vent or joint along the roof ridge introduces a discontinuity, diminishing its capacity to transfer lateral forces to the exterior walls. Additionally, the fasteners used to secure the metal panels to the framing appear inadequately spaced to provide significant shear resistance. Observations also indicate that these fasteners are corroded, which likely further reduces their effectiveness in resisting shear forces.

Overall, the storage building exhibits minimal lateral shear resistance, and its structural performance during a seismic event is expected to be poor.

<u>Tsunami Hazards</u>
Based on the site location, the building is within the tsunami inundation zone as defined by the 2022 OSSC. The building is a Risk Category IV "Essential Facility" structure, so design for tsunami loading is required per the OSSC and IBC. The site is located approximately 2,770 feet inland from the mean high-water line at a post-subsidence elevation of approximately 20ft – as shown in Figure 13. In the event of the design tsunami, the anticipated inundation depth is approximately 21ft feet above the post-subsidence grade, with a flow velocity of approximately 28 feet per second (19 MPH).

These conditions create significant uplift (buoyancy) and lateral loading (hydrodynamic drag) that the building is not capable of resisting in its current condition. The effective design tsunami base shear is approximately 5x the seismic base shear in the east west direction. The lateral force resisting system is not capable of resisting the net lateral load and the CMU walls are inadequate to resist the out of plane loads.

The building is required to resist waterborne debris carried in the tsunami flow. These debris strikes would also likely trigger failures in the exterior structural elements that could lead to loss of gravity carrying capacity. In addition, due to the depth and velocity of the inundation the spread footings would see significant scour which would undermine their gravity carrying capacity of these foundation elements.

The storage building does not have adequate strength to resist the hydrodynamic drag forces or the buoyancy created due to the water elevation. Due to its limited shear resistance, we would expect the building to become waterborne debris early in a tsunami event and post substantial risk to other structures.

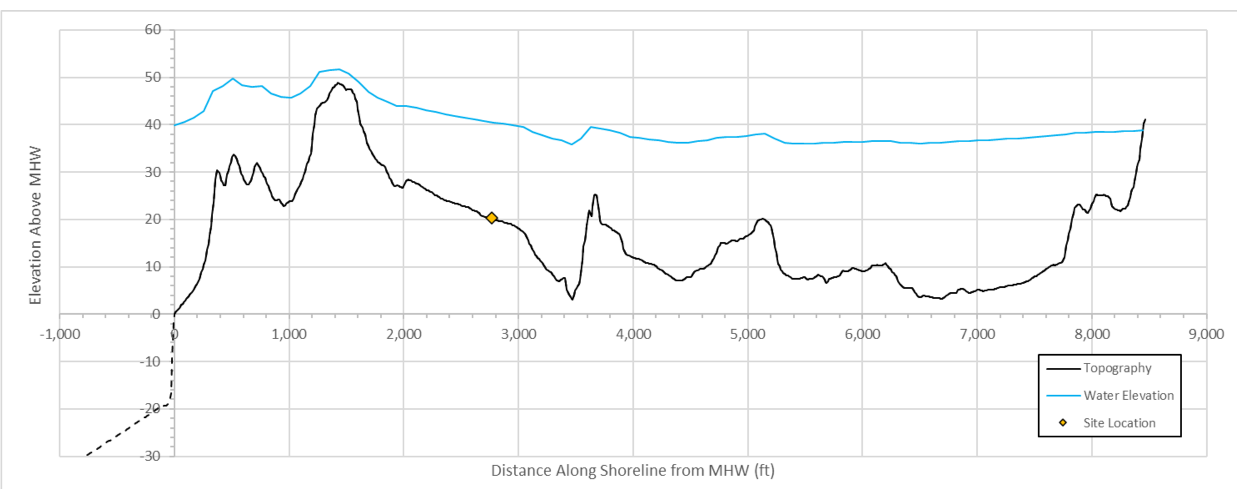

Figure 13: Tsunami Water Level Analysis