Alaska Department of Environmental Conservation

# Structural Assessment for Remedial Design

# The Buckner Building

### Hazard ID: 4151

## Whittier, Alaska

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in conjunction with

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**APPENDICES**

## EXECUTIVE SUMMARY

The Buckner Building, located in Whittier, Alaska, was constructed by the U.S. Military in 1951-1953 as a "Composite Bachelor Housing, Service and Recreation Center." The Buckner Building is a cast-in-place concrete structure, six stories tall, with approximately 275,000 square feet of floor space. In 1960, the Whittier Army Port was closed and the Buckner Building has been vacant since. Coffman Engineers, Inc., teamed with Shannon & Wilson, Inc., has performed a structural assessment of the Buckner Building. Shannon & Wilson previously performed a Property Assessment Cleanup Plan (PACP) and found the Buckner Building to contain hazardous materials (e.g. asbestos, polychlorinated biphenyls, lead, etc.).

The onsite structural assessment occurred on the 4th, 5th and 11th of November 2015. The structure's condition varied. Some portions of the structure appeared in good condition, while other areas had experienced such significant deterioration that the onsite structural engineers did not feel safe entering certain rooms. Nearly all of the Buckner Building's exterior doors and windows were missing as well as flashing at seismic joints. This has compromised the building envelope, exposing much of the structure to the elements. Rain leaders from roof drains were also damaged, which added to the already large water infiltration issue. Water infiltration, coupled with freeze-thaw cycles over decades of seasonal changes, had taken its toll on the concrete structure. Concrete degradation such as spalling, cracking, and corroded rebar was present throughout much of the structure. Standing water was common on floor slabs. Infiltrated water had often leached through concrete, particularly at crack locations, and formed stalactites (icicle shaped formations created by calcium hydroxide being dissolved out of the concrete). Stalactites were present on the underside of the majority of the elevated floor slabs, floor beams, roof slabs and roof beams, at times in large concentrations. At stalactite locations, water had penetrated the pervious concrete and has potentially corroded reinforcement, even if the corrosion had not yet created a visible concrete surface defect.

In addition to the onsite structural condition assessment, a Tier 1 seismic evaluation was performed in accordance with ASCE 41-13 *Seismic Evaluation and Retrofit of Existing Buildings*. Several items on the Tier 1 checklists were found to be noncompliant, suggesting a less than desirable seismic performance level. Concrete detailing of structures in high seismic regions has changed significantly since the 1950s to improve seismic performance. Tier 1 evaluations are intended to be a "screening" phase that identify potential deficiencies and items in need of further evaluation. Sometimes, additional evaluation done during a Tier II effort can find a checklist item adequate that was previously identified as noncompliant during the screening phase. However, considering the Buckner Building's generally poor condition, it would not be a good use of resources to further evaluate its seismic performance level.

Overall, the Buckner Building's structure was in poor condition with widespread defects including concrete cracking, spalling, corroded reinforcement, excessive deflections, and collapse of canopy structures. The north side of the facility, closest to the ocean, was in the worst condition. Stairwell cores, which were typically less exposed to weather, faired the best. The southern half of the eastern most wing (Unit G) was in relatively better condition than other building wings, however even this portion of the structure would require significant retrofit work. Retrofit would be needed to address general deterioration as well as reinforcement to improve seismic performance. A cost estimate is not part of the assessment team's scope of work. However, based on engineering judgment alone, retrofit of any sizeable portion of the Buckner Building would exceed the cost of new construction for an equivalent square footage. Retrofit of the entire facility is not feasible. While almost any facility can be repaired with an infinite budget, from a practical perspective, it is unlikely that any significant portion of the Buckner Building can be rehabilitated for occupancy.

## BACKGROUND

The Buckner Building, located in Whittier, Alaska, was constructed by the U.S. Military in 1951-1953 as a "Composite Bachelor Housing, Service and Recreation Center." In 1960, the Whittier Army Port was closed and the Buckner Building was vacated. The Buckner Building has been vacant for more than 50 years and because the majority of the windows and doors are now either broken or missing, the facility has been exposed to the elements for decades. After an extended period of time exposed to weather and vandals, the Buckner Building has sustained large amounts of damage. The building has had several owners since the U.S. Military; the most recent private owner forfeited the building to the City of Whittier in a property tax foreclosure. Now that the City of Whittier has acquired the Buckner Building, the question has been raised of what the city should do with the property. The facility is a large cast-in-place concrete structure, six stories stall, with approximately 275,000 square feet. The vast size of the Buckner Building poses a challenge for pursuing any type of rehabilitation or demolition.

## SCOPE OF WORK

The City of Whittier and the Alaska Department of Environmental Conservation (ADEC) have requested a structural assessment of the Buckner Building to determine the feasibility of reuse. Coffman Engineers, Inc. (Coffman), teamed with Shannon & Wilson, Inc. to perform the assessment. Shannon & Wilson previously performed a Property Assessment Cleanup Plan (PACP) and found the Buckner Building to contain hazardous materials (e.g. hydrocarbons, polychlorinated biphenyls, asbestos, lead, etc.). Further evaluation of hazardous materials is not part of the structural assessment scope of work.

Coffman performed an onsite condition assessment, taking note of deteriorated structural members and the structure's ability to resist gravity (dead, live and snow) loads and lateral (seismic and wind) loads. In addition to the general condition assessment, a Tier 1 seismic evaluation was performed in accordance with the American Society of Civil Engineers (ASCE) 41-13 *Seismic Evaluation and Retrofit of Existing Buildings*. The intent of the structural evaluation and condition assessment is to provide recommendations for demolishing and/or refurbishing the structure, and to what extent. Design of structural retrofit or repair is not part of this scope of work. The following report describes the structural assessment and recommendations for the building's potential use.

## PRE-SITE VISIT WORK

As mentioned above, the Buckner Building contains hazardous materials. After discussion with EHS-Alaska, Inc. (EHS) and Environmental Management, Inc. (EMI), it was determined that the structural assessment team could safely complete the onsite structural assessment if first properly trained. Coffman and Shannon & Wilson had the site visit team attend a full day asbestos awareness training class to learn about general asbestos safety, personal protective equipment (PPE), hazards specific to the Buckner Building, and decontamination procedures. EMI had been in the Buckner Building recently and was aware of the Bucker Building site-specific hazards; the training course was tailored specifically for this scope of work. Each site visit team member also underwent a respirator physical and fit test.

Prior the site visit, a cursory review of the existing drawings was performed. The cursory review was to familiarize the team with the facility layout and to identify specific areas of potential concern that warranted extra attention during the site investigation. Important plan sheets were printed and hardcopies taken to the field. The ASCE 41-13 Tier 1 checklists were also printed for field use. However, the majority of the structural calculations were performed after field work was complete.

## ONSITE CONDITION ASSESSMENT

The onsite structural assessment occurred on the 4th, 5th and 11th of November 2015. The team commuted from Anchorage to Whittier, typically taking the 8:30am tunnel from Bear Valley to Whittier and the 5:00pm tunnel from Whittier to Bear Valley. Prior to beginning field work on the first day, the team met with Scott Korbe (City of Whittier, Public Works) for a brief kickoff meeting. During the kickoff meeting the team obtained the gate key for site access, discussed the tentative schedule, the Buckner Building hazards, and how the team was going to safely perform the work. The main hazard was friable asbestos. Therefore, the following PPE was used: full face Powered Air Purifying Respirators (PAPRs), low volume air monitoring pumps, Tyvek overalls, gloves, disposable clothing base layers, and rubber hard toe boots. Other field equipment included: headlamps, flashlights, digital cameras, tape measurers, and hammers. As a safety precaution and to increase the thoroughness of the condition assessment, a minimum of two engineers were together on the site. After each day of field work, the engineers performed a decontamination procedure to remove asbestos fibers. Decontamination equipment included a HEPA Vac, baby wipes, a spray bottle with cleaner, paper towels and trash bags. All contaminated clothing and equipment was transported back to Anchorage for disposal.

The first day of field work the team took advantage of decent weather and walked the perimeter of the building's exterior. After photographing much of the exterior, the team donned PPE and entered the facility. The first day, the Coffman structural engineers were joined by a Shannon & Wilson geologist to observe the building's foundation and supporting soils/rock. To complete Shannon & Wilson's geotechnical scope of work, much of the first day was spent in the basement and crawlspace areas. The following day, Coffman structural engineers covered the majority of the building's interior, from the ground floor to the fourth. On the final day, stair towers, roof tops, and ancillary structures were assessed along with miscellaneous areas that were previously overlooked or needed additional photo documentation. The following describes the condition of the Buckner Building; descriptions are organized by building area. Refer to Appendix A for photographs of the conditions described below.

### Building Exterior

Cladding was not present on the exterior walls, so the concrete structure was exposed to view. The building exterior was assessed from the ground level; neither ladders nor man lifts were used for detailed inspection at higher elevations. The exterior walls were cast-in-place concrete shear walls that were typically perforated with regularly spaced windows [Figure A1]. The north façade (ocean side) had concrete columns and spandrel beams with concrete masonry unit (CMU) infill adjacent to window openings [Figure A2]. As previously stated, essentially all doors and windows were broken and/or missing. Door and window openings at lower levels were typically boarded closed to discourage public access.

Exterior concrete walls often exhibited significant moss growth. Vegetation was scraped off the exterior walls in several locations to inspect the concrete surface. It appeared everywhere moss had grown on the concrete walls, a concrete surface defect (e.g. cracking or spalling) was present. Typically, the surface defect was a small crack, or shallow spalling (i.e. flaking, peeling, or breakout of the concrete surface) that exposed the aggregate and provided a rough surface for the vegetation to purchase on the vertical wall surface [Figures A3 & A4]. Walls that maintained their smooth formed surface and were relatively free of surface defects did not appear to experience moss growth. The theater had a large continuous crack for the full length of each exterior bearing wall [Figure A5]. These cracks appeared to be from a poor connection detail that cast the theater's roof joist bottom chords into the exterior wall, therefore the joist bending moment created a flexural crack in the wall. Occasionally, there were diagonal cracks propagating from window corners, indicative of a shear wall that has undergone a significant lateral load event [Figure A6]. Numerous locations had large enough surface defects that corroded rebar was exposed [Figure A7]. Corroded rebar usually was not present throughout wall field locations, but was located at wall corners and columns, where concrete cover was reduced at stirrup and ties [Figure A8]. More significant areas of deterioration at exterior walls were present at locations where the roof had overflowed with rain water (or melted snow) and drained onto the structure below [Figure A9].

The end of each building wing was equipped with an egress stair. It appears the egress stairs were at one point enclosed with a glass curtain wall. The steel egress stairs were exposed to weather and corroded because glass was missing from the curtain walls [Figure A10].

Cantilevered concrete canopies were typical features of the original construction that covered exterior doorways and even some ground floor windows. On the north façade, several of the cantilevered canopies had collapsed [Figure A11]. The edge beam that supported these collapsed canopies had exposed rebar and had experienced significant degradation [Figure A12]. The remaining canopies ranged from poor condition and questionable safety, to extremely poor condition that posed an immediate safety threat [Figure A13]. While the majority of the Buckner Building was surrounded by fencing and had openings boarded to prevent public entrance, much of the north building elevation was not within the fenced boundary and the canopies in question could be accessed by the public. Coffman recommends that the city of Whittier performs one or more of the following: demolish the canopies on the north side of the building, install fencing around the unsafe canopies, and/or install shoring to prevent collapse of the canopies. It is difficult to say when the remaining canopies will fail. However, it is not out of the question that they could collapse this winter if snow loads are high enough, particularly the remaining canopy at wing "C" which was in the worst condition. Refer to the annotated ground floor plans in Appendix E for canopy locations.

## Crawlspace and Foundation

The crawlspace was accessed on the west side of the facility through stair #1. As anticipated, the building was founded on rock. Several spread footings were as-built and appeared to conform to the existing foundation drawings. Columns were supported by spread footings [Figures A14 & A15]. However, walls were often cast directly on rock without a strip footing; it appears wall vertical rebar may have been installed in holes drilled into the bedrock [Figure A18]. Refer to Shannon & Wilson's geotechnical report, located in Appendix B, for additional information regarding the building's foundation and supporting soils.

The crawlspace was only under a portion of the building's footprint; there were large portions of the facility with slab on grade. Crawlspace access was also limited because some areas were only accessible through a confined space, which the assessment team could not enter safely. Areas of the crawlspace that were accessed appeared in fair condition. Water infiltration was less than that of other building levels and because weather does not have as much opportunity to degrade the concrete below grade. Stalactites (icicle shaped formations created by calcium hydroxide being dissolved out of the concrete) were present, but not in the high concentrations seen in other areas of the building. Some deficiencies were noted, such as a shored basement floor beam with poor concrete quality [Figures A16 & A17]. The utilidor tunnel appeared in good condition [Figure A19]. Only the building side of the tunnel was accessed; the full length of the tunnel was not inspected.

**Basement**

The basement level was a daylight basement and at grade elevation facing Blackstone Road (north side of the facility). The south basement wall was below grade. The basement level had sustained large amounts of damage from vandalism and degradation from weather. In general, the north side of the basement, was in worse condition than the southern portion of the basement. The typical condition at the basement level included: large amounts of debris; flooded floor slabs; vegetation growth; stalactites suspended from the majority of floor slabs and beams; and numerous, isolated locations of reinforcement corrosion visible on concrete surfaces [Figure A20]. There was not a large amount of severely corroded rebar, but lots of smaller areas of visible rust scattered throughout the basement. The basement housed large rooms such as the bowling alley and two rifle ranges [Figures A21 & A22], which were in similar condition to the remainder of the basement. The bowling alley floor was in particularly poor condition; near the west end there were holes through the slab to the crawlspace below [Figure A23]. While stalactites were present throughout the basement, some rooms had large concentrations [Figures A24 & A25]. The quantity of stalactites on the underside of a slab was typically proportional to the quantity of standing water on the top side of the slab. Since the north side of the basement was exposed to sunlight, vegetation was able to grow [Figure A26]. Water infiltration not only promoted vegetation growth on the structure, but created spalling on the top surface of concrete slabs [Figure A27]. CMU infill was also damaged in some locations [Figure A28].

**Ground Level**

Although this floor was named “ground level” in the original drawings, this level was an elevated slab over the basement. The elevated slab was at or near the grade elevation on the south side of the facility and is one story above grade on the north side of the facility. The ground level was in similar condition as the basement, with more severe floor slab deterioration and more visible corroded rebar. Debris, water infiltration and vegetation growth was more uniform at the ground level since window openings were present on both the north and south building elevations. Some rooms had large amounts of standing water and moss as thick as one inch [Figure A30]. The top of floor slabs often had concrete spalling [Figure A31], at times so severe the concrete resembled a fine gravel mixture that could easily be scraped away by hand [Figure A32]. In addition to water infiltration from weather entering the window openings, pipes embedded in floor slabs had frozen and burst in some locations [Figure A33]. Corroded rebar was visible at numerous column, beam and slab locations [Figures A34-A36]. Similar to the basement level, stalactites were present throughout, with certain areas having large concentrations [Figure A37]. Since flashing at seismic joints was mostly missing, the structure on each side of the seismic joint often had severe damage from water infiltration [Figures A38-A39]. At some seismic joints, CMU faces had eroded to the extent the wall looked like pea gravel aggregate held together by mortar joints. CMU walls were not isolated from the concrete superstructure. In some locations, CMU had been damaged by interaction with concrete members [Figure A40].

The theater was located at the ground level. The theater was a separate two-story structure, isolated from the other building units by a seismic joint. The theater was similar to the remainder of the facility in that it had concrete bearing and shear walls. However, the roof structure was open web steel joists [Figure A41]. The roof joists were visibly corroded [Figure A42]. The ceiling was too high for close inspection, but from below it appeared the reinforcing mat in the theater's concrete roof deck was visible, which would indicate large amounts of spalling.

**Floor Levels 1-4**

Similar concrete defects were found on upper levels of the facility as were found in the basement and ground level. The primary issue continued to be water infiltration combined with freeze thaw cycles that corroded reinforcing steel and cracked concrete members. Numerous rooms were flooded with standing water. Stalactites were plentiful in most areas, with occasional stalagmites [Figure A47]. Moss growth was also present in rooms with flooding and adequate sunlight. The top surface of flooded floor slabs often had significant spalling [Figure A48]. CMU walls at seismic joints were continuously degraded at all floor levels [Figures A50, A54 & A62]. A few different Squad Rooms on the north side of building units had floor beams so severely deteriorated that the rooms were unsafe to enter [Figures A52 & A53]. Rain leaders had broken in a couple Squad Rooms; rain water was actively flowing through the roof drain pipe onto the 4$^{th}$ floor slab [Figures A64-A65]. Not only did this flood the room, but the slab area directly beneath the falling water was extremely eroded. The quantity of remaining architectural finishes increased with ascending floor levels. Although, even if finishes were intact, they were still typically damaged by water infiltration, vandalism, or both. Refer to the photo log in Appendix A, specifically figures 43-68, for photo documentation on the reoccurring deficiencies noted above for floor levels 1-4.

**Penthouse and Roof Levels**

It appears nearly all roof drains have failed; rain water has accumulated on the lower roofs [Figures A69 & A70]. Internal roof drains for the main, upper roof also seem nonfunctioning as rain water had frozen and created a substantial ice layer on top of the roof the day of the site visit [Figures A71 & A72]. The mechanical penthouses experienced similar types of surface defects as the remainder of the exterior walls: minor cracking, minor spalling, exposed aggregate and moss growth [Figures A75 & A76]. The inside of penthouses were at times in fair condition, but had stalactites in some locations [Figure A77]. All mechanical penthouses were accessed via dedicated stairwell towers. Stairwells were less exposed to weather and were in better condition than remainder of the structure, but were not free of defects [Figure A78].

**Ancillary Structures and Nonstructural Components**

Concrete canopies have been previously described (refer to the Building Exterior subsection of this report), and are in poor condition. Egress stairs were also briefly mentioned. In addition to the corrosion issue, egress stairs were too steep (9-inch rise and 9-inch run) and too narrow (24 inches) to meet current International Building Code egress requirements [Figures A79 & A80]. Egress stair guardrails were also noncompliant.

While the primary objective for this scope of work is evaluation of the building's main superstructure and foundation, it worth noting that essentially all architectural, mechanical and electrical systems appeared to be in complete disarray [Figure A81]. Nonstructural components had either been damaged from water infiltration, corrosion, vandalism or a combination of these events.

## STRUCTURAL EVALUATION – GRAVITY & WIND LOADS

The following building codes are currently adopted by the State of Alaska:

- American Concrete Institute (ACI) 318-08

- ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures*
- International Building Code (IBC) 2009

The Buckner Building's structural general notes sheet states design loads, which were in accordance with the code at the time, the 1949 UBC (Uniformed Building Code). Some of the design live loads listed on the general notes sheet, such as 100psf (pounds per square foot) used for the Mess Hall, Day Rooms and other public spaces is consistent with current code requirements. However, other areas were designed for live loads less than current code would require. For example, Squad Rooms and Officer's Quarters have a design live load of 30psf; current code would require a minimum of 40psf. Corridors were designed for 40psf or 60psf, depending on the area. Current code requires first floor corridors be designed for 100psf and corridors above the first floor be designed for 80psf.

Roof live load (i.e. snow load) is listed as 100psf on the general structural notes. Whittier has one of the highest snow loads in the State of Alaska. Current code stipulates a ground snow load of 300psf, which translates into a flat roof snow load of 210psf (can vary with code prescribed factors). Snow drift at low roof locations could add as much as 150psf additional snow load to the flat roof snow load mentioned above.

Slab and beam capacity have been analyzed in a few select locations [Appendix C]. The typical roof slab and roof beam analyzed do not meet code for support of the 210psf snow load mentioned above; the roof appears to have been designed for the 100psf load stated in the general notes. The floor slab and floor beam analyzed appear to have marginal but adequate flexural capacity for the 100psf live load required for the first floor Day Room. There is a minimum area of steel code requirement the existing beams do not meet, but they are not necessarily capacity driven requirements and may not have been a requirement of the 1949 UBC. The calculations were performed assuming the structure is in good condition, which is not the case. In reality, most slabs and beams have reduced capacity because of reduced rebar cross section (corrosion), or a cracked concrete section. Additional gravity analysis has not been performed because the condition assessment revealed a typically poor structural condition, so structural analysis based on the original construction drawings is moot.

Wind load is listed as 20psf on the general structural notes. Current code would require a design wind pressure of 27psf to 37psf, increasing with the building height, for the main wind force resisting system. The wind load for the main force resisting system is not critical as lateral design in regions of high seismicity is typically controlled by seismic forces, especially for concrete superstructures with large seismic mass, like the Buckner Building. The current components and cladding design wind pressure varies from 55psf to 75psf, increasing with the building height. The component and cladding wind load can be reduced for effective wind areas in excess of 10 square feet. But, even with effective area reduction, the ASCE 7-05 components and cladding pressures will still be larger than the original design wind load of 20psf. This could affect out-of-plane bending design of CMU infill walls.

## SEISMIC EVALUATION (ASCE 41-13)

A Tier 1 seismic evaluation was performed in accordance with ASCE 41-13 *Seismic Evaluation and Retrofit of Existing Buildings*. The Tier 1 checklists and supporting calculations can be found in Appendix D. The "Life Safety" Tier 1 checklists were used. Since the building is currently vacant, it did not seem appropriate to use the more stringent "Immediate Occupancy" Tier 1 checklists. The Buckner Building was regularly shaped because individual wings were isolated by seismic joints, creating individual rectangular structures. Regular building configurations are preferred for seismic performance. Despite the building's regular configuration, it is "noncompliant" for several Tier 1 checklist items, suggesting a less than desirable seismic performance level. Concrete detailing of structures in high seismic regions has changed significantly since the 1950s. Current building codes place a lot of emphasis on redundancy and detailing for ductility, which is sometimes difficult to prove existing buildings can achieve.

The first checklist completed was *Life Safety Basic Configuration Checklist*. Since building units (i.e. A, B, C, D, E, F & G) were separated by seismic joints, they were treated somewhat separately for the seismic evaluation. On this checklist, all building units were noncompliant for the "adjacent building" check because the separation between building units at seismic joints was too small. Building Unit A was noncompliant for vertical irregularities because of discontinuity in shear walls between floor levels. Building units D and F had multiple noncompliant items because shear walls layout was not uniform, which created a change in stiffness and strength between floor levels. Building Unit F had the most noncompliant checklist items because discontinuous shear walls made the load path unclear.

The second checklist completed was *Life Safety Structural Checklist for Building Types C2: Concrete Shear Walls with Stiff Diaphragms*. Two of the main noncompliant checklist items was a lack of redundancy and shear wall stress. Other noncompliant checklist items include flat slab reinforcement at column joints and coupling beam reinforcement. Another feature of the Buckner Building that is discouraged for seismic detailing is the CMU infill. If CMU is used in modern construction in high seismic regions, it would be isolated from the main seismic force resisting system to prevent overloading the CMU and to prevent adversely affecting the main seismic force resisting system's stiffness.

In addition to the checklists for the main seismic force resisting system, the *Nonstructural Checklist* was also completed. As previously stated, nonstructural components in the Buckner Building were corroded, damaged, missing, vandalized or otherwise in an unserviceable condition, so many of the Tier 1 nonstructural checklist items were found "not applicable."

Another part of the ASCE 41 effort was the evaluation of ground failures. The Buckner Building was founded on rock, so the potential for ground failures such as liquefaction, and ground fault rupture are highly unlikely. Refer to Shannon & Wilson's geotechnical report in Appendix B for additional information.

Tier 1 evaluations are intended to be a "screening" phase that identify potential deficiencies and items in need of further evaluation. Sometimes, additional evaluation done during a Tier II effort can find a checklist item adequate that was previously identified as noncompliant during the screening phase. However, considering the Buckner Building's generally poor condition, it would not be a good use of resources to further evaluate its seismic performance level.

## CONCLUSION

The Buckner Building's structure was in poor condition. Once the building envelope was compromised (i.e. windows broken, flashing removed, etc.), large amounts of water infiltrated a structure that was not designed for that type of exposure. The building was no longer heated so concrete degradation from water infiltration was greatly exacerbated by freeze-thaw cycles. Current concrete code would require a minimum concrete compressive strength of 4500 pounds per square inch (psi) for concrete exposed to moisture and freeze thaw cycles for durability reasons (in addition to other mix design requirements). The Buckner Building's superstructure is constructed from much less durable, 2500psi concrete.

During onsite assessment, many concrete structural members had visible defects such as cracking, spalling, or corroded reinforcement. Stalactites were present on the underside of the vast majority of the elevated floor slabs, floor beams, roof slabs and roof beams, at times in large concentrations. Everywhere there were stalactites, water had penetrated the pervious concrete and has potentially corroded reinforcement, even if the corrosion had not yet created a visible concrete surface defect. Defects were widespread, not isolated to a certain portion or portions of the structure.

In addition to the structure's poor condition, the original loads used for design are substantially less than loads required by current code. Load criteria has been developed based on additional research since the 1949 UBC, and current code load requirements are more representative of loads the structure should be designed to resist. The roof design live load was less than half the code required design snow load. Portions of floor structures were designed for live loads below current code requirements. Depending on building use, the floor capacity would be exceeded. For example, any retail, office, or assembly type function would exceed the 30psf and 40psf design live loads used in the living quarters areas of the existing facility. The design wind load was also a fraction of the ASCE 7-05 required wind load. Seismic performance was also questionable per the ASCE 41-13 Tier 1 process. Seismic retrofit would likely be required to comply with ASCE 41, which could include, for example, installation of additional shear walls.

The north side of the facility, closest to the ocean, was in the worst condition. Stairwell cores, which were typically less exposed to weather, faired the best. The southern half of the eastern most wing (Unit G) was in relatively better condition than other building wings. However, even this portion of the structure would require significant retrofit work. Retrofit would be needed to address general deterioration as well as reinforcement to improve seismic performance. A cost estimate is not part of the assessment team's scope of work. However, based on engineering judgment alone, retrofit of any sizeable portion of the Buckner Building would exceed the cost of new construction for an equivalent square footage. Retrofit of the entire facility is not feasible. While almost any facility can be repaired with an infinite budget, from a practical perspective, it is unlikely that any significant portion of the Buckner Building can be rehabilitated for occupancy.

For these reasons, Coffman recommends demolition of the Buckner Building. We also suggest that plans for demolition begin soon. We understand that the Buckner Building contains large quantities of asbestos, among other hazards. The Buckner Building's current condition allows for an abatement team to safely work in most areas of the facility, with some exceptions. As the Buckner Building's structure continues to deteriorate, it is only a matter of time before it will be unsafe for an abatement crew, or any other contractor not trained to address the structural hazards, to enter the facility.

Sincerely,

**COFFMAN ENGINEERS, INC.**

Matthew Stielstra, PE, SE
Civil/Structural Engineer

Will Veelman, PE, SE
Principal, Civil/Structural Engineering

# Appendix A

PHOTO LOG

**Figure A1 – Typical southern building elevation with perforated concrete shear walls and missing windows**

**Figure A2 – Typical northern building elevation with concrete columns, concrete spandrel beams and CMU infill.**

**Figure A3 – Example of moss growth on exterior wall.**

**Figure A4 – Example of surface defect beneath moss growth on exterior wall**

**Figure A5 – East wall of theatre with large crack for the full wall length at approximate location of the roof joist bottom chord attachment.**


**Figure A6 – Concrete shear wall with diagonal crack protruding from upper left and lower right window corners.**

**Figure A7 – Example of exposed exterior wall rebar at egress stair landing.**

**Figure A8 – Example of exterior concrete column with exposed, corroded spiral ties.**

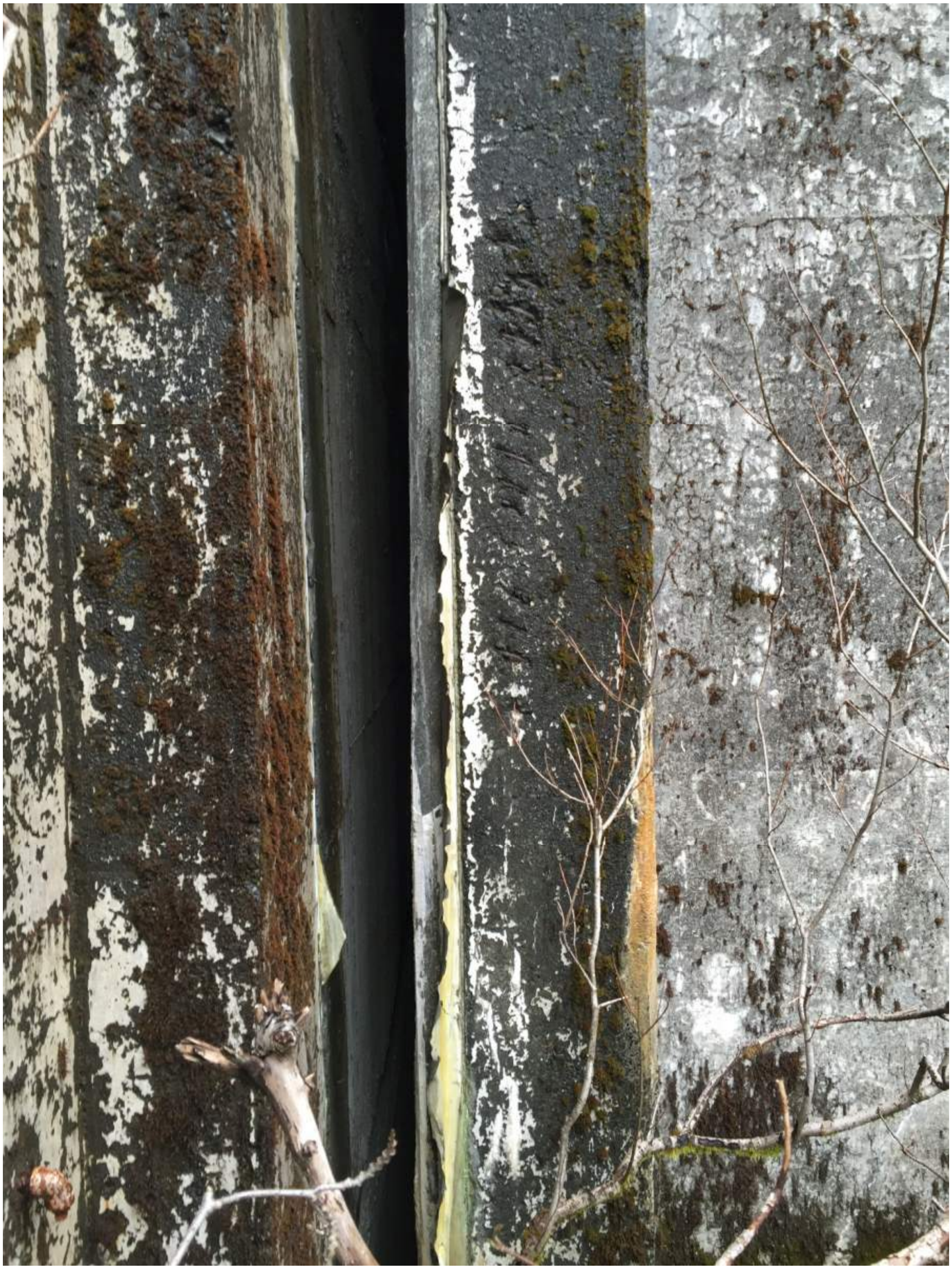


**Figure A9 – Exposed column spiral ties, moss growth and other concrete defects at seismic joint.**

**Figure A10 – Typical egress stair where curtain wall glass is gone and steel exposed to weather has corroded.**

**Figure A11 – Collapsed portion of concrete canopy on north façade.**

**Figure A12 – Edge beam with rebar protruding from collapsed canopy.**

**Figure A13 – Large crack at concrete wall / canopy attachment. Failure appears imminent.**

**Figure A14 – Example of isolated column footing in crawlspace.**

**Figure A15 – Column pedestal cast directly on rock in crawlspace.**

**Figure A16 – Crawlspace floor beam with shoring.**

**Figure A17 – Crawlspace floor beam with voids on underside and exposed rebar.**

**Figure A18 – Wall in crawlspace with vertical rebar doweled directly to rock.**

**Figure A19 – Utilidor tunnel.**

**Figure A20 – Typical basement room with large amount of debris and stalactites hanging from floor beams and underside of floor slab above.**

**Figure A21 – Rifle range with flooded floor.**

**Figure A22 – Bowling alley.**

**Figure A23 – Hole in bowling alley floor to crawlspace below.**

**Figure A24 – Basement ceiling with large stalactite concentrations.**

**Figure A25 – Long stalactite on underside of floor beam.**

**Figure A26 – Daylight basement with debris and significant moss growth on floor.**

**Figure A27– Close up view of basement floor with moss growth and spalled concrete slab surface.**

**Figure A28 – Basement floor beam with stalactites and damaged CMU wall.**

**Figure A29 – Mess hall at the north side of the ground floor.**

**Figure A30 – Typical ground floor office with floor flooded from water infiltration and heavy moss growth on slab.**

**Figure A31 – Ground floor, elevated slab with severe spalling.**

**Figure A32 – Ground floor, close up of severely deteriorated slab surface.**

**Figure A33 – Pipes embedded ground floor slab that have ruptured from frozen water and damaged the floor slab.**

**Figure A34 – Ground floor interior column with exposed ties.**

**Figure A35 – Ground floor, exposed slab reinforcement.**

**Figure A36 – Ground floor beam/column connection with exposed beam stirrups and stalactites.**

**Figure A37 – Example of concentrated stalactites at ground floor.**

**Figure A38 – Ground floor column at seismic joint with exposed spiral ties.**

**Figure A39 – Ground floor CMU, extremely deteriorated at seismic joint.**

**Figure A40 – Cracked CMU ground floor corridor wall at floor beam penetration.**


**Figure A41 – Theatre**

**Figure A42 – Corroded steel open web roof joists at the theatre.**

**Figure A43 – Typical 1st floor Day Room.**

**Figure A44 – Typical 1st floor Squad Room.**

**Figure A45 – 1st floor day room with beam each beam line extremely deteriorated.**

**Figure A46 – Flooded 1st floor room.**

**Figure A47 – Beam/column grid at 1st floor Day Room with stalactites suspended from beam and stalagmites beginning to form on the floor slab.**

**Figure A48 – Severely deteriorated 1st floor slab adjacent to north facing windows.**

**Figure A49 – Close up of stalactites and calcium carbonite forming on the side of the concrete beam, through large horizontal cracks.**

**Figure A50 – Severely deteriorated CMU at 1st floor, with holes eroded complete through.**

**Figure A51 – Squad Room on southern end of Unit G's 2nd floor. This portion of the building is generally in the best condition.**

**Figure A51 – 2nd floor beam with large horizontal cracks.**


**Figure A52 – Floor beam at 2nd level Squad Room, northern half of Unit G. Beam is approaching collapse.**

**Figure A53 – Floor beam at 2nd level Squad Room, northern half of Unit E. Beam is approaching collapse.**

**Figure A54 – Deteriorated CMU wall at the 2nd floor level.**

**Figure A55 – Squad Room on southern end of Unit G's 3rd floor. This portion of the building is generally in the best condition.**

**Figure A56 – Seismic joint with 3rd floor beams each side. Stalactites present on each beam and flashing is missing at joint.**

**Figure A57 – 3rd floor Squad Room on northern half of unit. Stalactites and stalagmites are present along beam line.**

**Figure A58 – Flooded 3rd floor Squad Room. Stalagmites forming on floor along the beam line.**

**Figure A59 – 3rd floor lounge/game room with flooded floor.**

**Figure A60 – 3rd floor beam with horizontal cracking along beam face.**


**Figure A61 – 3rd floor barracks with architectural finishes still intact (although damaged).**

**Figure A62 – Severely deteriorated CMU wall at 3rd floor level.**

**Figure A63 – Squad Room on southern end of Unit G's 4^nd floor. This portion of the building is generally in the best condition.**

**Figure A64 – Flooded squad room on norther side of the building.**

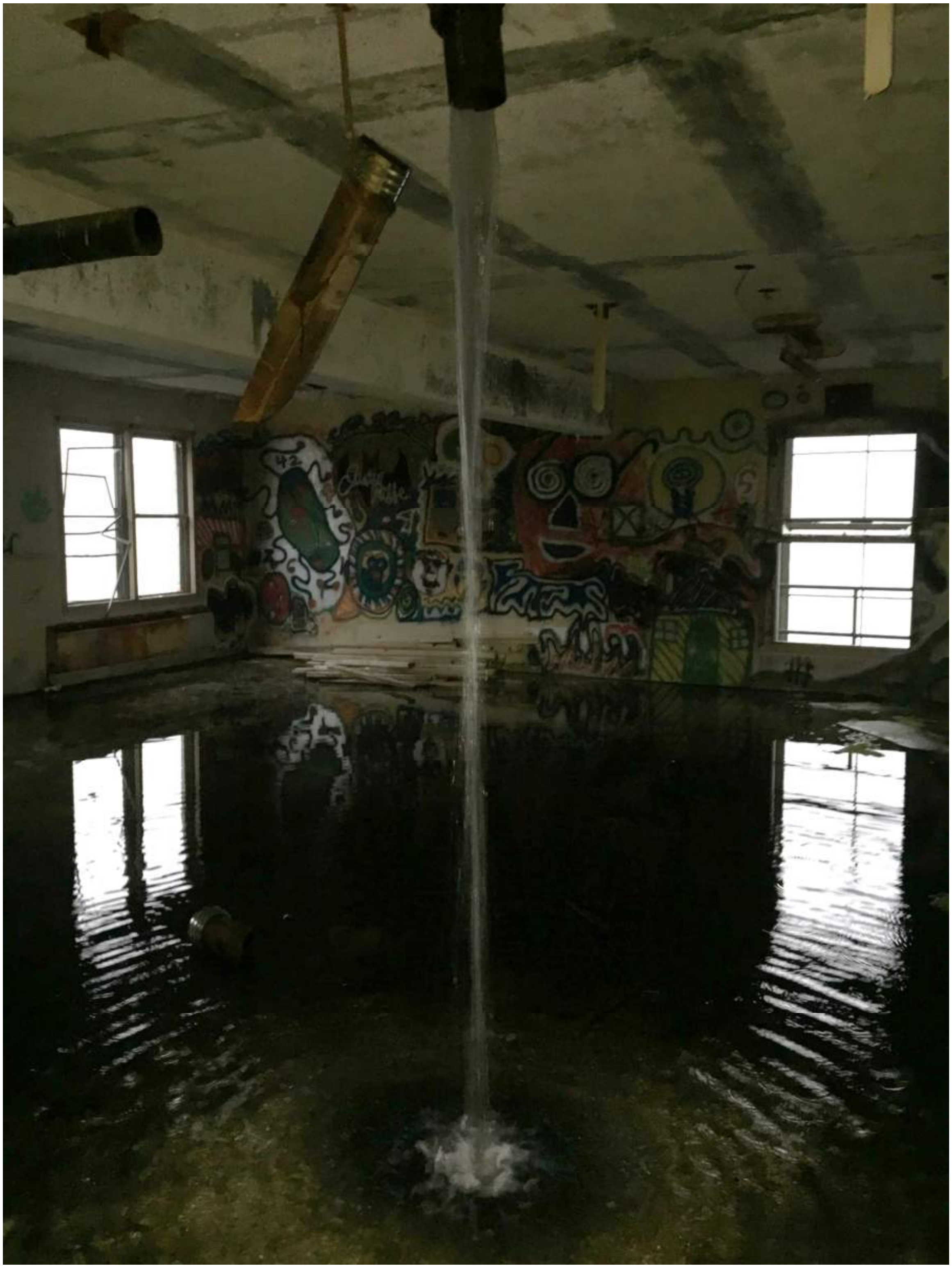


**Figure A65 – Broken rain leader draining directly onto floor slab.**

**Figure A66 – Close up of deteriorating roof beam with exposed stirrups.**

**Figure A67 – Cracking in roof slab above 4th floor barracks.**

**Figure A68 – 4th floor corridor showing more architectural finishes still in place.**

**Figure A69 – Low roof over Mess Hall and Kitchen.**

**Figure A70 – Low roof over the theatre.**

**Figure A71 – Penthouse fan rooms for Units A, C and E.**


**Figure A72 – Smaller penthouse fan room at Unit A.**


**Figure A73 – Interior of Unit A fan room, accessed via stair 2.**

**Figure A74 – Typical penthouse fan room.**

**Figure A75 – Shallow surface defects of concrete penthouses, typical.**

**Figure A76 – Shallow surface defects of concrete penthouses, typical.**

**Figure A77 – Typical penthouse mechanical mezzanine. Stalactites visible from the ceiling slab.**

**Figure A78 – Stairwells to for roof access were typically in fair condition, but not free of defects.**

**Figure A79 – Egress stair rise and run.**

**Figure A80 – Egress stair width.**

**Figure A81 – Example of typical condition of nonstructural components.**

# Appendix B

SHANNON & WILSON GEOTECHNICAL REPORT

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

ALASKA
CALIFORNIA
COLORADO
FLORIDA
MISSOURI
OREGON
WASHINGTON
WISCONSIN

December 15, 2015

Alaska Department of Environmental Conservation
555 Cordova Street
Anchorage, Alaska 99501

Attn: Mr. Bill O'Connell

**RE: STRUCTURAL ASSESSMENT FOR REMEDIAL DESIGN, GEOTECHNICAL EVALUATION, BUCKNER BUILDING, WHITTIER, ALASKA; ADEC HAZARD ID 4151**

We are pleased to provide this letter presenting the results of our site visit and geotechnical evaluation conducted by our firm to support a Tier 1 structural assessment for the Buckner Building in Whittier, Alaska. The assessment is being conducted by Coffman Engineers in accordance with the American Society of Civil Engineers (ASCE) 41-13, *Seismic Evaluation and Retrofit of Existing Buildings*. In addition to the structural considerations, the Tier 1 assessment requires an evaluation of seismic-induced geological hazards. To accomplish this, we visited the site and made observations of the soil and rock at the site, observed several foundation elements, and reviewed available data and building construction drawings. Presented in this letter is a description of our observations, our interpretation of the site conditions, and the results of our geotechnical evaluation.

The work was performed for the Alaska Department of Environmental Conservation (ADEC) Division of Spill Prevention and Response under Term Contract 18-8036-03. The scope of work was based on the ADEC's June 29, 2015 request for proposal and performed in material accordance with Shannon & Wilson's August 3, 2015 proposal. Authorization to proceed with this work was provided by the ADEC in the form a Notice to Proceed (NTP) 18-8036-03-023, dated August 12, 2015. The NTP was modified in consultation with the ADEC project manager on September 17, 2015 and October 21, 2015 to include attendance of site specific asbestos awareness training, purchase additional respiratory personal protective equipment (PPE), and air monitoring supplies that were recommended during the training to protect against inhalation of asbestos containing materials (ACM).

5430 FAIRBANKS STREET
ANCHORAGE, ALASKA 99518-1263
907-561-2120 FAX: 907-561-4483
TDD 1-800-833-6388
www.shannonwilson.com

SHANNON & WILSON, INC.

## SITE VISIT AND OBSERVATIONS

A Shannon & Wilson representative (Ryan Collins), accompanied by representatives from Coffman Engineers (Matthew Stielstra and Peter Hewko), visited the site on November 4, 2015 to make general observations of the site, building foundations (where exposed) as well as the soil and rock conditions in the footing area. Observations generally consisted of attempting to identify the presence and type of bedrock and whether the footings appeared to be founded on the rock. Our representative also measured several footings for comparison against construction drawings. It is important to note that the conditions below the ground surface discussed in the report text are inferred from observations of the conditions exposed at the ground surface at the time of our site visit.

**Excavated rock slope behind the northeast corner of the building. Looking southwest.**

The building is situated on northwest facing slopes of the ridge located east of Whittier. A vicinity map, included as Figure 1 shows the general building location. Based on construction drawings, the building appears to have been constructed on a bench that was excavated or blasted into the bedrock. This is consistent with our observations at the site as cut slopes with exposed bedrock are visible to the north, east, and south of the building. These slopes range in height from about 10 to 20 feet. Bedrock exposed in the cut appears to consist of slightly weathered, dark gray, phyllite or slate. The rock mass contains prominent bedding or cleavage in approximately 1/2 to 1 inch thick planes. The bedding planes dip at about 70 to 80 degrees to the northwest, which corresponds to the general direction of the slopes in the area. In addition to this bedding, at least one joint set, which is oriented roughly perpendicular to the bedding, is visible. Spacing in this joint set is relatively wide with typical spacings on the order of about 3 to 8 feet.

During our site visit, our representative attempted to make observations of the footings, soil fills, and rock on the building interior. A site plan, included as Figure 2, shows the general building layout and other prominent site features. Most of the footings were inaccessible due to floor slabs or other coverings. However, we were able to make observations of footings exposed in the south end of Unit A and relatively short sections of stem walls exposed in utility tunnels in the northwest building corner and in the mechanical/boiler room in Unit C.

**Typical footing observed in the sub-basement area of Unit A. Note potential in-situ bedrock exposed in foreground.**

The footings exposed in the south end of Unit A consisted of spread column footings with dimensions roughly matching those shown on the construction drawings. The ground surface around the footings was generally obscured by shot rock fills that were likely placed after the footings were constructed and the footing bottoms were typically not visible. At several of the observed footings, the bottom edge of the footing was visible and appeared to be cast onto bedrock. We also observed limited bedrock exposures where the stem walls were observed near the northwest building corner and Unit C mechanical/boiler room. In general, it appeared that the stem walls were supported directly on the bedrock, although our representative noted some areas where the rock, or potentially shot rock backfill, had raveled from beneath the wall. Test pit logs shown on the construction drawings indicated that bedrock was encountered at elevations ranging between 58.5 and 97.1 feet. In general, it appears that the bottom of footing elevations shown on the construction drawings are consistent with the bedrock depths shown on the test pit logs.


**Stem wall observed in a utility tunnel near northwest corner of Unit G.**

The footing areas observed were typically dry and, based on our observations, groundwater seepage does not appear to be present in the footing (sub-basement) and basement areas. Occasional puddles were observed but it is likely that the water came from leakage through openings in the building above. Construction drawings show that a footing drain was to be installed around the building perimeter.

## SEISMIC CONDITIONS

The project area is located in a zone of active seismicity from both shallow crustal events and deep-seated subduction zone earthquakes. In 1964, Southcentral Alaska experienced the largest recorded earthquake in North America, the Great Alaskan Earthquake, with a Moment Magnitude of 9.2. The earthquake occurred in the northeast section of the Aleutian Megathrust which resulted in an estimated 100,000 square mile area of surface deformation. According to available maps, Whittier is located within the area encompassed by the 1964 rupture zone. A 1965 report authored by the United States Geological Survey following the earthquake, reported that structural damage to the Buckner Building was reported as "not significant". Other reports indicated that there may have been damage to a stairwell at the east end of the structure and that construction joints in exterior walls showed evidence of movement as "each joint was clearly visible and had fresh mortar spalls."

The Tier 1 assessment requires an evaluation of seismic-induced geologic hazards at the site. These hazards largely include seismically induced ground failure (ie. surface rupture, faulting, lateral spreading, liquefaction, and landslides) and tsunami. Based on the available site data and our site visit observations, it is our opinion that seismically-induced ground failure, liquefaction, and surface rupture are unlikely at this site and the site class according to the 2012 International Building Code (IBC 2012) will be B for a soil profile containing rock. Excluding the underlying Aleutian Megathrust, the closest mapped fault runs along the western edge of Blackstone Bay, approximately 2 miles from the Buckner Building. The fault appears to be inactive according to limited information available. Tsunami inundation mapping conducted by the Alaska Department of Natural Resources, Division of Geological & Geophysical Surveys for Whittier in 2011 indicates the maximum tsunami inundation line is well below the Buckner Building bottom floor elevation.

Based on the ground motions in Figures 1613.3.1(4) and 1613.3.1(5), IBC 2012, the mapped spectral accelerations for short-period ($S_s$) and 1-second period ($S_1$) were estimated at 1.500 and 0.734 times the gravitational coefficient (g), respectively. For Site Class B, site specific modifying coefficients for the spectral response accelerations are $F_A = 1.0$ and $F_v = 1.0$ for the short and long periods, respectively. Consequently, $S_{MS}$ and $S_{M1}$ for site class D were calculated to be 1.500 and 0.734 g, respectively, and the corresponding $S_{DS}$ and $S_{D1}$ are 1.000 and 0.489 g, respectively.

We conducted a brief seismic hazard analysis of the site using software developed by the United States Geological Survey (USGS) to calculate the peak ground acceleration (PGA). According to this software, the calculated PGA for the site is 0.68 g. This value is roughly equivalent to what would be calculated using probabilistic estimates of ground motions with a 2 percent probability of exceedance in 50 years (2,475-year return period). The corresponding earthquake magnitude (M) is M9.2.

## ALLOWABLE BEARING CAPACITY

Based on our brief review of the construction drawings and our observations at the site it appears that the building foundations are likely cast on slate or phyllite bedrock beneath the building. The construction drawings also give an allowable "soil bearing capacity" of 10,000 pounds per square foot (psf) for the footings. Assuming the footings were cast directly on bedrock, it is our opinion that the bearing capacity provided is appropriate for the assumed site conditions, although the bedrock could likely provide significantly higher bearing capacity. This bearing value may be increased by 1/3 for short-term wind and seismic loading.

## CLOSURE AND LIMITATIONS

This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions and recommendations contained in this report are based on information provided from the limited research and field observations that we conducted. Our observations were limited to areas that were readily accessible and the analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist. It is assumed that the conditions observed near the surface and nearby rock exposures are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by our observations.

Unanticipated soil and rock conditions are commonly encountered and cannot fully be determined by mere surface observations. If conditions different from the assumed conditions are observed or appear to be present, Shannon & Wilson, Inc. should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. Shannon & Wilson has prepared the document in Attachment A *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact the undersigned.

We appreciate this opportunity to be of service. Please contact the undersigned at (907) 561-2120 with questions or comments concerning the contents of this report.

Sincerely,

**SHANNON & WILSON, INC.**

Ryan Collins
Senior Geotechnical Professional

RDC:SJG

STATE OF ALASKA
49TH
Stafford J. Glashan
CE - 10365
12/15/15
REGISTERED PROFESSIONAL ENGINEER

Stafford Glashan, P.E.
Vice President

Enc: Figure 1 – Vicinity Map
Figure 2 – Site Plan
Attachment A - Important Information About Your Geotechnical/Environmental Report


## NOTES

1. Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.

BUCKNER BUILDING STRUCTURAL ASSESSMENT
WHITTIER, ALASKA

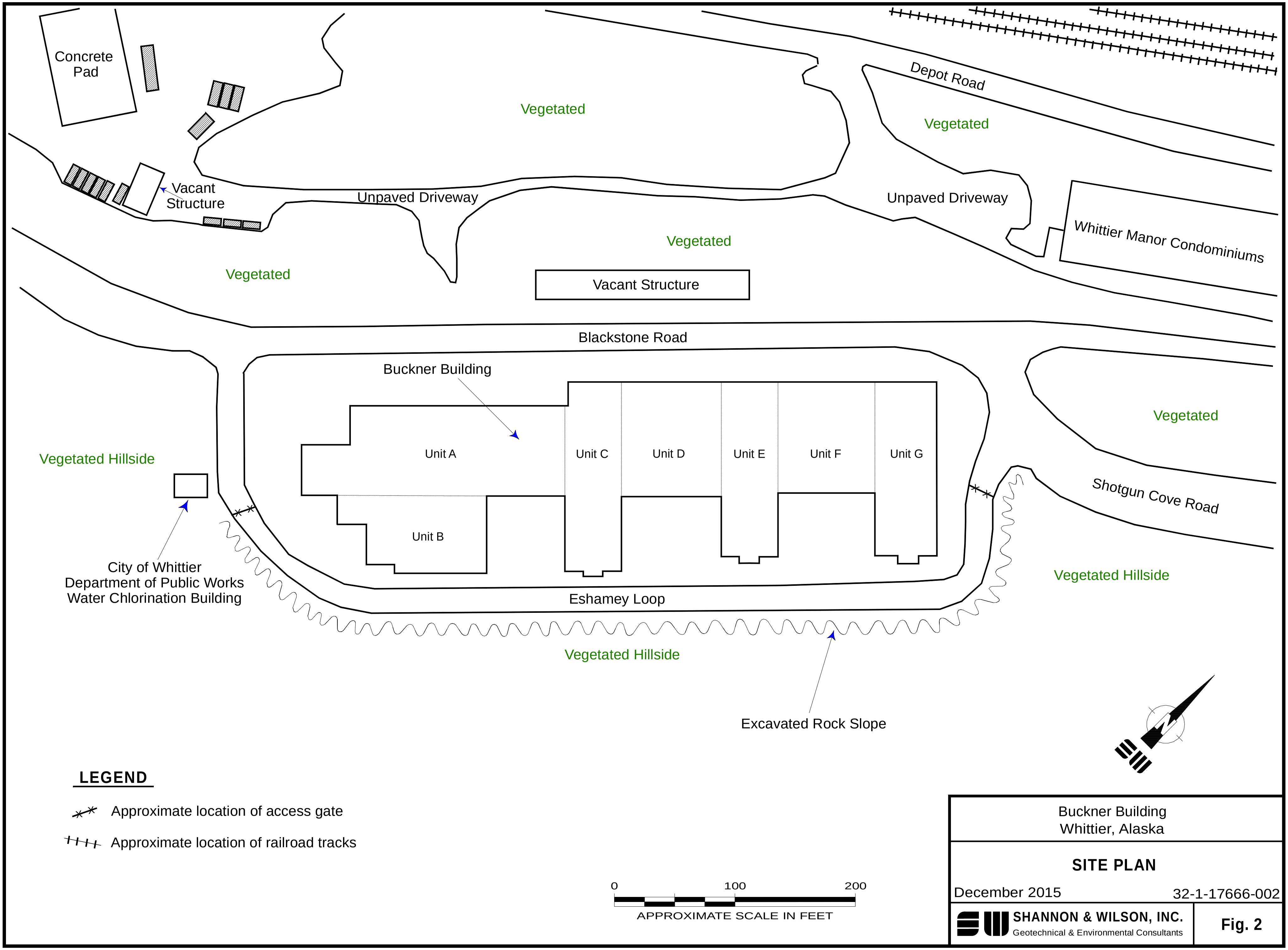
VICINITY MAP

DECEMBER 2015

32-1-17666-002

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

FIG. 1


Concrete Pad
Vacant Structure
Vegetated
Unpaved Driveway
Depot Road
Vegetated
Unpaved Driveway
Whittier Manor Condominiums
Vegetated
Vegetated
Vacant Structure
Blackstone Road
Buckner Building
Unit A
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
Vegetated
Shotgun Cove Road
Vegetated Hillside
City of Whittier
Department of Public Works
Water Chlorination Building
Eshamey Loop
Vegetated Hillside
Vegetated Hillside
Excavated Rock Slope
LEGEND
Approximate location of access gate
Approximate location of railroad tracks
0
100
200
APPROXIMATE SCALE IN FEET
Buckner Building
Whittier, Alaska
SITE PLAN
December 2015
32-1-17666-002
SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants
Fig. 2

Date: December 2015
To: ADEC
Re: Buckner Building Tier 1 Structural Assessment, Whittier, Alaska

# Important Information About Your Geotechnical/Environmental Report

## CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

## THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

## SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

## MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

**A REPORT'S CONCLUSIONS ARE PRELIMINARY.**

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

**THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.**

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

**BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.**

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

**READ RESPONSIBILITY CLAUSES CLOSELY.**

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

# Appendix C

STRUCTURAL CALCULATIONS – GRAVITY & WIND LOADS

## SNOW LOAD CRITERIA

SNOW LOAD CALCULATIONS ARE IN ACCORDANCE WITH THE 2006 INTERNATIONAL BUILDING CODE (IBC) AND ASCE 7-05 AS ADOPTED BY THE UNIFIED FACILITIES CRITERIA.

### GENERAL SNOW LOAD CRITERIA

| | | | |
|---|---|---|---|
| OCCUPANCY CATEGORY | | II | |
| IMPORTANCE FACTOR | Is = | 1 | |
| GROUND SNOW LOAD | Pg = | 300 | PSF |

### FLAT ROOF SNOW LOAD

| | | | |
|---|---|---|---|
| TERRAIN CATEGORY | | C | |
| EXPOSURE | | PARTIALLY EXPOSED | |
| EXPOSURE FACTOR | Ce = | 1.0 | |
| THERMAL CONDITION | | ALL OTHER STRUCTURES | |
| THERMAL FACTOR | Ct = | 1.0 | |
| FLAT ROOF SNOW LOAD | Pf = | 210.0 | PSF |
| MINIMUM FLAT ROOF SNOW LOAD | Pf > | 20 | PSF |
| RAIN-ON-SNOW SURCHARGE | | N.A. | |
| DESIGN FLAT ROOF SNOW LOAD | Pf = | 210 | PSF |

### SLOPED ROOF SNOW LOAD

| | | | |
|---|---|---|---|
| ROOF TYPE | | WARM (Ct <= 1.0) | |
| SURFACE TYPE | | SLIPPERY - MEMBRANE | |
| ROOF SLOPE | | 1 | DEGREES |
| ROOF SLOPE FACTOR | Cs = | 1.00 | |
| SLOPED ROOF SNOW LOAD | Ps = | N.A. | |

### SNOW DENSITY

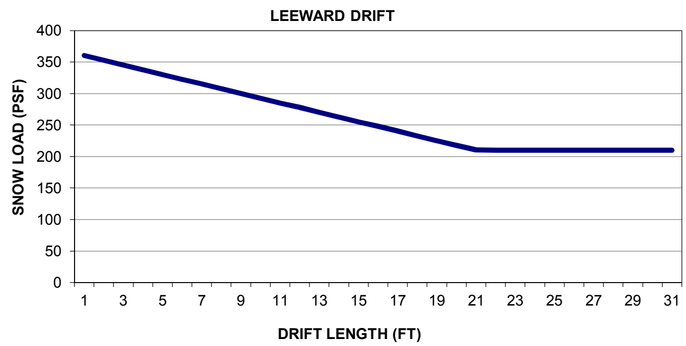
| | | | |
|---|---|---|---|
| SNOW DENSITY | γ = | 30.0 | PCF |
| BALANCED SNOW HEIGHT | Hb = | 7.00 | FT |

## SNOW DRIFT DESCRIPTION / LOCATION

LOW ROOF DRIFT OVER MESS HALL AND ENTRY.

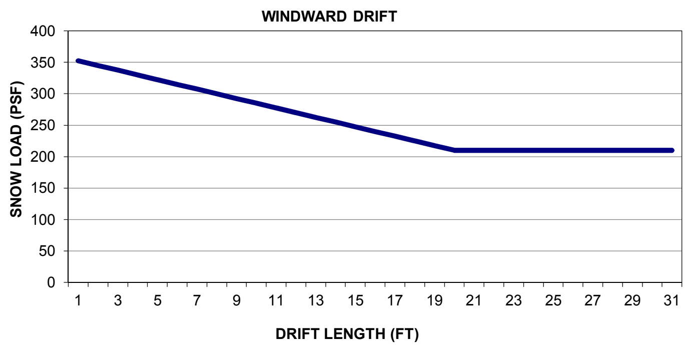
### LEEWARD DRIFT

| | | | |
|---|---|---|---|
| CLEAR HEIGHT | Hc = | 30 | FT |
| | Hc/Hb = | 4.29 | OK |
| "UPPER" ROOF LENGTH | Lu = | 47 | FT |
| THEORETICAL DRIFT HEIGHT | Hd = | 5.01 | FT |
| ACTUAL DRIFT HEIGHT | Hd = | 5.01 | FT |
| DRIFT WIDTH | w = | 20.046 | FT |
| MAX DRIFT LOAD | Pd = | 150 | PSF |




### WINDWARD DRIFT

| | | | |
|---|---|---|---|
| CLEAR HEIGHT | Hc = | 30 | FT |
| | Hc/Hb = | 4.29 | OK |
| "UPPER" ROOF LENGTH | Lu = | 81.5 | FT |
| THEORETICAL DRIFT HEIGHT | Hd = | 4.74 | FT |
| ACTUAL DRIFT HEIGHT | Hd = | 4.74 | FT |
| DRIFT WIDTH | w = | 18.968 | FT |
| MAX DRIFT LOAD | Pd = | 142 | PSF |




COFFMAN ENGINEERS

| | | | | |
|---|---|---|---|---|
| project | **BUCKNER BUILDING ASSESSMENT** | by | **MCS** | sheet no. |
| location | **WHITTIER, AK** | date | **12/30/2015** | |
| client | **ADEC / SHANNON & WILSON** | rev | | job no. |
| topic | **SNOW LOAD** | rev | | **150859** |


ASCE 7 Windspeed | ASCE 7 Ground Snow Load | Related Resources | Sponsors | About ATC | Contact

# Search Results


**Query Date:** Mon Dec 28 2015
**Latitude:** 60.7747
**Longitude:** -148.6750

**ASCE 7-10 Windspeeds**
**(3-sec peak gust in mph*):**

**Risk Category I:** 133
**Risk Category II:** 142
**Risk Category III-IV:** 154
**MRI** 10-Year:** 98
**MRI** 25-Year:** 108
**MRI** 50-Year:** 121
**MRI** 100-Year:** 120

**ASCE 7-05 Windspeed:**
121 (3-sec peak gust in mph)
**ASCE 7-93 Windspeed:**
85 (fastest mile in mph)

*Miles per hour
**Mean Recurrence Interval

Users should consult with local building officials to determine if there are community-specific wind speed requirements that govern.

Print your results

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**WIND LOAD CRITERIA - MWFRS**

WIND LOAD CALCULATIONS ARE IN ACCORDANCE WITH THE 2006 INTERNATIONAL BUILDING CODE (IBC) AND ASCE 7-05 AS ADOPTED BY THE UNIFIED FACILITIES CRITERIA. ANALYSIS PROCEDURE IS METHOD 2 - ANALYTICAL PROCEDURE - RIGID BUILDINGS OF ALL HEIGHTS.

**VELOCITY PRESSURES**

| | | | |
|---|---|---|---|
| OCCUPANCY CATEGORY | | II | |
| IMPORTANCE FACTOR | Iw = | 1.00 | |
| BASIC WIND SPEED | V = | 121 | MPH |
| DIRECTIONALITY FACTOR | Kd = | 0.85 | |
| TOPOGRAPHIC FACTOR | Kzt = | 1.00 | |
| EXPOSURE CATEGORY | | C | |

| HEIGHT, Z (FT) | PRESSURE COEFFICIENT, Kz | VELOCITY PRESSURE, Qz (PSF) |
|---|---|---|
| 0 | 0.85 | 27.04 |
| 15 | 0.85 | 27.04 |
| 20 | 0.90 | 28.73 |
| 25 | 0.95 | 30.12 |
| 30 | 0.98 | 31.29 |
| 40 | 1.04 | 33.25 |
| 50 | 1.09 | 34.85 |
| 60 | 1.14 | 36.21 |
| 70 | 1.17 | 37.40 |
| 80 | 1.21 | 38.47 |
| 90 | 1.24 | 39.44 |
| 100 | 1.27 | 40.32 |

**WIND DIRECTION / DESCRIPTION**

NORTH/SOUTH WIND DIRECTION - TRANSVERSE TO MAIN BUILDING DIMENSION.

**WALL DESIGN WIND PRESSURES**

| | | | |
|---|---|---|---|
| GUST EFFECT FACTOR | G = | 0.85 | |
| ENCLOSURE CLASSIFICATION | | ENCLOSED | |
| INTERNAL PRESSURE COEFF. | GCpi = | 0.18 | +/- |
| PLAN DIM., PARALLEL TO WIND | L = | 573 | FT |
| PLAN DIM., NORMAL TO WIND | B = | 154 | FT |
| ROOF SLOPE | θ = | 1 | DEGREES |
| WINDWARD PRESSURE COEFF. | Cp = | 0.80 | USE W/ Qz |
| LEEWARD PRESSURE COEFF. | Cp = | -0.21 | USE W/ Qh |
| SIDE WALL PRESSURE COEFF. | Cp = | -0.70 | USE W/ Qh |
| LEEWARD PRESSURE, +GCpi | P = | -13.28 | PSF |
| LEEWARD PRESSURE, -GCpi | P = | -0.07 | PSF |
| SIDE WALL PRESSURE, +GCpi | P = | -28.45 | PSF |
| SIDE WALL PRESSURE, -GCpi | P = | -15.23 | PSF |

| | WINDWARD WALL DESIGN PRESSURES (PSF) | | NET DESIGN WALL PRESSURES, WINDWARD + LEEWARD (PSF) | |
|---|---|---|---|---|
| HEIGHT, Z (FT) | +GCpi | -GCpi | NET, + GCpi | NET, - GCpi |
| 0 | 11.78 | 25.00 | 25.07 | 25.07 |
| 15 | 11.78 | 25.00 | 25.07 | 25.07 |
| 20 | 12.93 | 26.15 | 26.21 | 26.21 |
| 25 | 13.87 | 27.09 | 27.15 | 27.15 |
| 30 | 14.67 | 27.89 | 27.95 | 27.95 |
| 40 | 16.00 | 29.22 | 29.28 | 29.28 |
| 50 | 17.09 | 30.30 | 30.37 | 30.37 |
| 60 | 18.02 | 31.23 | 31.30 | 31.30 |
| 70 | 18.83 | 32.04 | 32.11 | 32.11 |
| 80 | 19.55 | 32.77 | 32.84 | 32.84 |
| 90 | 20.21 | 33.42 | 33.49 | 33.49 |
| 100 | 20.81 | 34.03 | 34.09 | 34.09 |

COFFMAN ENGINEERS

| | | | | |
|---|---|---|---|---|
| project | **BUCKNER BUILDING ASSESSMENT** | by | **MCS** | sheet no. |
| location | **WHITTIER, AK** | date | **12/30/2015** | |
| client | **ADEC / SHANNON & WILSON** | rev | | job no. |
| topic | **WIND LOAD - MWFRS** | rev | | **150859** |

**WIND LOAD CRITERIA - COMPONENTS & CLADDING**

WIND LOAD CALCULATIONS ARE IN ACCORDANCE WITH THE 2006 INTERNATIONAL BUILDING CODE (IBC) AND ASCE 7-05 AS ADOPTED BY THE UNIFIED FACILITIES CRITERIA. ANALYSIS PROCEDURE IS METHOD 2 - ANALYTICAL PROCEDURE - BUILDINGS WITH HEIGHT GREATER THAN 60 FT.

**VELOCITY PRESSURES**

| | | | |
|---|---|---|---|
| OCCUPANCY CATEGORY | | II | |
| IMPORTANCE FACTOR | Iw = | 1.00 | |
| BASIC WIND SPEED | V = | 121 | MPH |
| DIRECTIONALITY FACTOR | Kd = | 0.85 | |
| TOPOGRAPHIC FACTOR | Kzt = | 1.00 | |
| EXPOSURE CATEGORY | | C | |

| HEIGHT, Z (FT) | PRESSURE COEFFICIENT, Kz | VELOCITY PRESSURE, Qz (PSF) |
|---|---|---|
| 0 | 0.85 | 27.04 |
| 15 | 0.85 | 27.04 |
| 20 | 0.90 | 28.73 |
| 25 | 0.95 | 30.12 |
| 30 | 0.98 | 31.29 |
| 40 | 1.04 | 33.25 |
| 50 | 1.09 | 34.85 |
| 60 | 1.14 | 36.21 |
| 70 | 1.17 | 37.40 |
| 80 | 1.21 | 38.47 |
| 90 | 1.24 | 39.44 |
| 100 | 1.27 | 40.32 |

**WIND DIRECTION / DESCRIPTION**

WIND DIRECTION FROM NORTH TO SOUTH. TRANSVERSE TO THE BUILDING'S LONG DIMENION.

**PRESSURE COEFFICIENTS**

| | | | |
|---|---|---|---|
| ENCLOSURE CLASSIFICATION | | ENCLOSED | |
| INTERNAL PRESSURE COEFF. | GCpi = | 0.18 | +/- |
| ROOF TYPE | | [6-17] LOW SLOPE θ<10° | |
| ROOF OVERHANG? | | NO OVERHANG | |

| | | |
|---|---|---|
| (1) ROOF FIELD | +GCp = | N.A. |
| (1) ROOF FIELD | -GCp = | -1.40 |
| (1) ROOF FIELD, OVERHANG | GCp = | N.A. |
| (2) ROOF EDGE | +GCp = | N.A. |
| (2) ROOF EDGE | -GCp = | -2.30 |
| (2) ROOF EDGE, OVERHANG | GCp = | N.A. |
| (3) ROOF CORNER | +GCp = | N.A. |
| (3) ROOF CORNER | -GCp = | -3.20 |
| (3) ROOF CORNER, OVERHANG | GCp = | N.A. |
| (4) WALL FIELD | +GCp = | 0.90 |
| (4) WALL FIELD | -GCp = | -0.90 |
| (5) WALL EDGE | +GCp = | 0.90 |
| (5) WALL EDGE | -GCp = | -1.80 |

**WALL DESIGN WIND PRESSURES**

| | | | |
|---|---|---|---|
| ROOF SLOPE | θ = | 1.00 | DEGREES |
| BUILDING WIDTH | W = | 154 | FT |
| WIDTH OF PRESS. COEFF. ZONE | a = | 15.4 | FT |

| HEIGHT, Z (FT) | MAX. POSITIVE WALL FIELD PRESS., p (PSF) | MAX. NEGATIVE WALL FIELD PRESS., p (PSF) | MAX. POSITIVE WALL EDGE PRESS., p (PSF) | MAX. NEGATIVE WALL EDGE PRESS., p (PSF) |
|---|---|---|---|---|
| 0 | 30.9 | -30.9 | 30.9 | -55.3 |
| 15 | 30.9 | -30.9 | 30.9 | -55.3 |
| 20 | 32.5 | -32.5 | 32.5 | -58.3 |
| 25 | 33.7 | -33.7 | 33.7 | -60.8 |
| 30 | 34.8 | -34.8 | 34.8 | -62.9 |
| 40 | 36.5 | -36.5 | 36.5 | -66.5 |
| 50 | 38.0 | -38.0 | 38.0 | -69.3 |
| 60 | 39.2 | -39.2 | 39.2 | -71.8 |
| 70 | 40.3 | -40.3 | 40.3 | -73.9 |
| 80 | 41.2 | -41.2 | 41.2 | -75.9 |
| 90 | 42.1 | -42.1 | 42.1 | -77.6 |
| 100 | 42.9 | -42.9 | 42.9 | -79.2 |

**ROOF DESIGN WIND PRESSURES**

| BUILDING SURFACE | MAX. POSITIVE DESIGN PRESSURES, p (PSF) | MAX. NEGATIVE DESIGN PRESSURES, p (PSF) |
|---|---|---|
| (1) ROOF FIELD | 0.0 | -58.0 |
| (1) ROOF FIELD, OVERHANG | N.A. | N.A. |
| (2) ROOF EDGE | 0.0 | -91.0 |
| (2) ROOF EDGE, OVERHANG | N.A. | N.A. |
| (3) ROOF CORNER | 0.0 | -124.1 |
| (3) ROOF CORNER, OVERHANG | N.A. | N.A. |

# Concrete Beam



Description : typ roof slab (M)

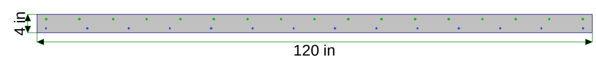
## CODE REFERENCES

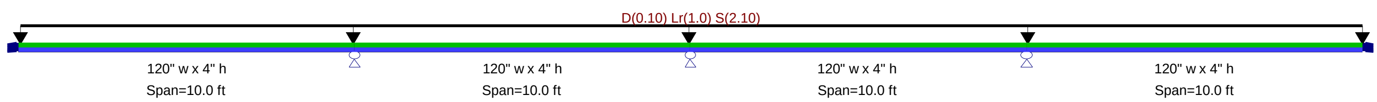
Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combination Set : ASCE 7-05

## Material Properties

| | | | | | | |
|---|---|---|---|---|---|---|
| f'c | = | 2.50 ksi | ϕ Phi Values | Flexure : | 0.90 | |
| fr = $f'c^{1/2}$ * 7.50 | = | 375.0 psi | | Shear : | 0.750 | |
| ψ Density | = | 145.0 pcf | $\beta_1$ | = | 0.850 | |
| λ LtWt Factor | = | 1.0 | | | | |
| Elastic Modulus | = | 3,122.0 ksi | Fy - Stirrups | | 40.0 ksi | |
| fy - Main Rebar | = | 40.0 ksi | E - Stirrups | = | 29,000.0 ksi | |
| E - Main Rebar | = | 29,000.0 ksi | Stirrup Bar Size # = | # | 3 | |
| | | | Number of Resisting Legs Per Stirrup = | | 2 | |

Load Combination : ASCE 7-05




## Cross Section & Reinforcing Details

Rectangular Section, Width = 120.0 in, Height = 4.0 in

Span #1 Reinforcing....
14-#3 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span
17-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span

Span #2 Reinforcing....
14-#3 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span
17-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span

Span #3 Reinforcing....
14-#3 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span
17-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span

Span #4 Reinforcing....
14-#3 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span
17-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span

## Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.010, Lr = 0.10, S = 0.210
Uniform Load on ALL spans : D = 0.010, Lr = 0.10, S = 0.210 ksf, Tributary Width = 10.0 ft

## DESIGN SUMMARY

**Design N.G.**

| | | | | | |
|---|---|---|---|---|---|
| Maximum Bending Stress Ratio = | **1.169** : 1 | Maximum Deflection | | | |
| Section used for this span | **Typical Section** | Max Downward Transient Deflection | 0.066 in | Ratio = | 1819 |
| Mu : Applied | -33.833 k-ft | Max Upward Transient Deflection | 0.000 in | Ratio = | 0 <360 |
| Mn * Phi : Allowable | 28.935 k-ft | Max Downward Total Deflection | 0.107 in | Ratio = | 1126 |
| | | Max Upward Total Deflection | 0.000 in | Ratio = | 999 <180 |
| Load Combination | +1.20D+0.50L+1.60S | | | | |
| Location of maximum on span | 0.000 ft | | | | |
| Span # where maximum occurs | Span # 4 | | | | |

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 | Support 4 | Support 5 |
|---|---|---|---|---|---|
| Overall MAXimum | 13.417 | 26.833 | 26.833 | 26.833 | 13.417 |
| Overall MINimum | 1.750 | 3.500 | 3.500 | 3.500 | 1.750 |
| D Only | 2.917 | 5.833 | 5.833 | 5.833 | 2.917 |
| +D+L+H | 2.917 | 5.833 | 5.833 | 5.833 | 2.917 |
| +D+Lr+H | 7.917 | 15.833 | 15.833 | 15.833 | 7.917 |
| +D+S+H | 13.417 | 26.833 | 26.833 | 26.833 | 13.417 |

# Concrete Beam



Description : typ roof slab (M)

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 | Support 4 | Support 5 |
|---|---|---|---|---|---|
| +D+0.750Lr+0.750L+H | 6.667 | 13.333 | 13.333 | 13.333 | 6.667 |
| +D+0.750L+0.750S+H | 10.792 | 21.583 | 21.583 | 21.583 | 10.792 |
| +D+W+H | 2.917 | 5.833 | 5.833 | 5.833 | 2.917 |
| +D+0.70E+H | 2.917 | 5.833 | 5.833 | 5.833 | 2.917 |
| +D+0.750Lr+0.750L+0.750W+H | 6.667 | 13.333 | 13.333 | 13.333 | 6.667 |
| +D+0.750L+0.750S+0.750W+H | 10.792 | 21.583 | 21.583 | 21.583 | 10.792 |
| +D+0.750Lr+0.750L+0.5250E+H | 6.667 | 13.333 | 13.333 | 13.333 | 6.667 |
| +D+0.750L+0.750S+0.5250E+H | 10.792 | 21.583 | 21.583 | 21.583 | 10.792 |
| +0.60D+W+H | 1.750 | 3.500 | 3.500 | 3.500 | 1.750 |
| +0.60D+0.70E+H | 1.750 | 3.500 | 3.500 | 3.500 | 1.750 |
| D Only | 2.917 | 5.833 | 5.833 | 5.833 | 2.917 |
| Lr Only | 5.000 | 10.000 | 10.000 | 10.000 | 5.000 |
| L Only | | | | | |
| S Only | 10.500 | 21.000 | 21.000 | 21.000 | 10.500 |
| W Only | | | | | |
| E Only | | | | | |
| H Only | | | | | |

## Shear Stirrup Requirements

Entire Beam Span Length : PhiVc/2 < Vu <= PhiVc, Req'd Vs = Ht<=10", Not Reqd, use stirrups spaced at 0.000 in

## Maximum Forces & Stresses for Load Combinations

| Load Combination / Segment Length | Span # | Location (ft) in Span | Bending Stress Results ( k-ft ) Mu : Max | Phi*Mnx | Stress Ratio |
|---|---|---|---|---|---|
| MAXimum BENDING Envelope | | | | | |
| Span # 1 | 1 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 2 | 2 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 3 | 3 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 4 | 4 | 10.000 | -33.83 | 28.94 | 1.17 |
| +1.40D | | | | | |
| Span # 1 | 1 | 10.000 | -6.81 | 28.94 | 0.24 |
| Span # 2 | 2 | 10.000 | -6.81 | 28.94 | 0.24 |
| Span # 3 | 3 | 10.000 | -6.81 | 28.94 | 0.24 |
| Span # 4 | 4 | 10.000 | -6.81 | 28.94 | 0.24 |
| +1.20D+0.50Lr+1.60L+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -10.00 | 28.94 | 0.35 |
| Span # 2 | 2 | 10.000 | -10.00 | 28.94 | 0.35 |
| Span # 3 | 3 | 10.000 | -10.00 | 28.94 | 0.35 |
| Span # 4 | 4 | 10.000 | -10.00 | 28.94 | 0.35 |
| +1.20D+1.60L+0.50S+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -14.58 | 28.94 | 0.50 |
| Span # 2 | 2 | 10.000 | -14.58 | 28.94 | 0.50 |
| Span # 3 | 3 | 10.000 | -14.58 | 28.94 | 0.50 |
| Span # 4 | 4 | 10.000 | -14.58 | 28.94 | 0.50 |
| +1.20D+1.60Lr+0.50L | | | | | |
| Span # 1 | 1 | 10.000 | -19.17 | 28.94 | 0.66 |
| Span # 2 | 2 | 10.000 | -19.17 | 28.94 | 0.66 |
| Span # 3 | 3 | 10.000 | -19.17 | 28.94 | 0.66 |
| Span # 4 | 4 | 10.000 | -19.17 | 28.94 | 0.66 |
| +1.20D+1.60Lr+0.80W | | | | | |
| Span # 1 | 1 | 10.000 | -19.17 | 28.94 | 0.66 |
| Span # 2 | 2 | 10.000 | -19.17 | 28.94 | 0.66 |
| Span # 3 | 3 | 10.000 | -19.17 | 28.94 | 0.66 |
| Span # 4 | 4 | 10.000 | -19.17 | 28.94 | 0.66 |
| +1.20D+0.50L+1.60S | | | | | |
| Span # 1 | 1 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 2 | 2 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 3 | 3 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 4 | 4 | 10.000 | -33.83 | 28.94 | 1.17 |
| +1.20D+1.60S+0.80W | | | | | |
| Span # 1 | 1 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 2 | 2 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 3 | 3 | 10.000 | -33.83 | 28.94 | 1.17 |
| Span # 4 | 4 | 10.000 | -33.83 | 28.94 | 1.17 |
| +1.20D+0.50Lr+0.50L+1.60W | | | | | |
| Span # 1 | 1 | 10.000 | -10.00 | 28.94 | 0.35 |
| Span # 2 | 2 | 10.000 | -10.00 | 28.94 | 0.35 |
| Span # 3 | 3 | 10.000 | -10.00 | 28.94 | 0.35 |

## Concrete Beam



Description : typ roof slab (M)

| Load Combination / Segment Length | Span # | Location (ft) in Span | Bending Stress Results ( k-ft ) Mu : Max | Phi*Mnx | Stress Ratio |
|---|---|---|---|---|---|
| Span # 4 | 4 | 10.000 | -10.00 | 28.94 | 0.35 |
| +1.20D+0.50L+0.50S+1.60W | | | | | |
| Span # 1 | 1 | 10.000 | -14.58 | 28.94 | 0.50 |
| Span # 2 | 2 | 10.000 | -14.58 | 28.94 | 0.50 |
| Span # 3 | 3 | 10.000 | -14.58 | 28.94 | 0.50 |
| Span # 4 | 4 | 10.000 | -14.58 | 28.94 | 0.50 |
| +1.20D+0.50L+0.20S+E | | | | | |
| Span # 1 | 1 | 10.000 | -9.33 | 28.94 | 0.32 |
| Span # 2 | 2 | 10.000 | -9.33 | 28.94 | 0.32 |
| Span # 3 | 3 | 10.000 | -9.33 | 28.94 | 0.32 |
| Span # 4 | 4 | 10.000 | -9.33 | 28.94 | 0.32 |
| +0.90D+1.60W+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -4.37 | 28.94 | 0.15 |
| Span # 2 | 2 | 10.000 | -4.38 | 28.94 | 0.15 |
| Span # 3 | 3 | 10.000 | -4.37 | 28.94 | 0.15 |
| Span # 4 | 4 | 10.000 | -4.38 | 28.94 | 0.15 |
| +0.90D+E+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -4.37 | 28.94 | 0.15 |
| Span # 2 | 2 | 10.000 | -4.38 | 28.94 | 0.15 |
| Span # 3 | 3 | 10.000 | -4.37 | 28.94 | 0.15 |
| Span # 4 | 4 | 10.000 | -4.38 | 28.94 | 0.15 |

### Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+S+H | 1 | 0.1065 | 5.000 | | 0.0000 | 0.000 |
| +D+S+H | 2 | 0.1065 | 5.000 | | 0.0000 | 0.000 |
| +D+S+H | 3 | 0.1065 | 5.000 | | 0.0000 | 0.000 |
| +D+S+H | 4 | 0.1065 | 5.000 | | 0.0000 | 0.000 |

Title Block Line 1
You can change this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 28 DEC 2015, 10:00PM

# Concrete Beam

File = P:\Anc\15Jobs\150859~1\56E1B~1.0PR\GRAVIT~1.EC6
ENERCALC, INC. 1983-2015, Build:6.15.12.4, Ver:6.15.12.4

**Lic. # : KW-06003501** **Licensee : COFFMAN ENGINEERS**

Description : typ roof beam (B15/B16)

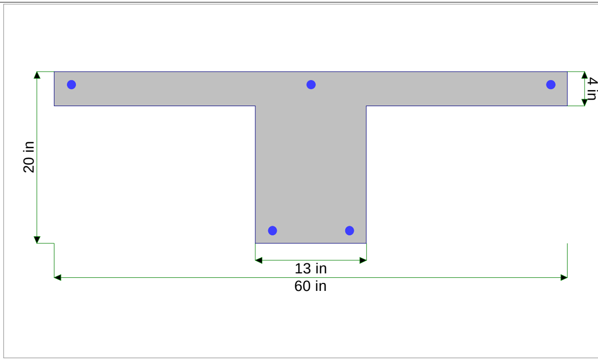
## *CODE REFERENCES*

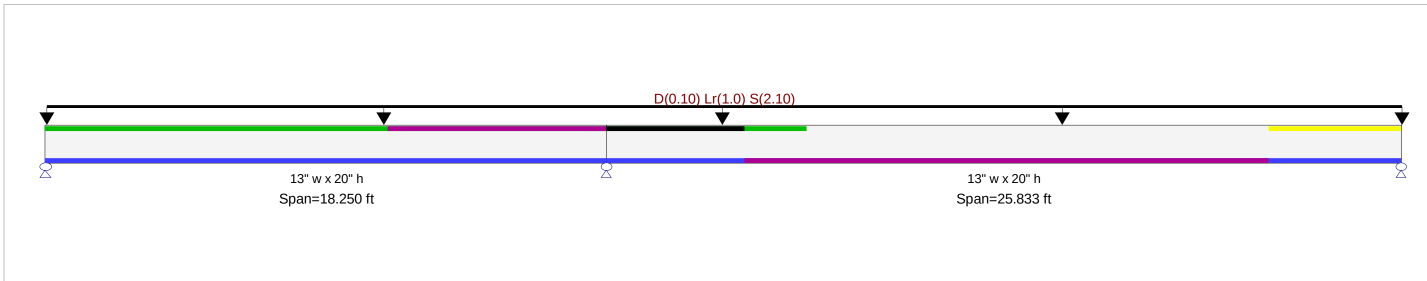
Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combination Set : ASCE 7-05

## Material Properties

| | | | | | | |
|---|---|---|---|---|---|---|
| f'c | = | 2.50 ksi | $\phi$ Phi Values | Flexure : | | 0.90 |
| fr = $f'c^{1/2}$ * 7.50 | = | 375.0 psi | | Shear : | | 0.750 |
| $\psi$ Density | = | 145.0 pcf | $\beta_1$ | = | | 0.850 |
| $\lambda$ LtWt Factor | = | 1.0 | | | | |
| Elastic Modulus | = | 3,122.0 ksi | Fy - Stirrups | | | 40.0 ksi |
| fy - Main Rebar | = | 40.0 ksi | E - Stirrups | = | | 29,000.0 ksi |
| E - Main Rebar | = | 29,000.0 ksi | Stirrup Bar Size # = | | # | 3 |
| | | | Number of Resisting Legs Per Stirrup = | | | 2 |

Load Combination : ASCE 7-05




## Cross Section & Reinforcing Details

Tee Section, Stem Width = 13.0 in, Total Height = 20.0 in, Top Flange Width = 60.0 in, Flange Thickness = 4.0 in

Span #1 Reinforcing....
- 2-#7 at 1.50 in from Bottom, from 0.0 to 18.250 ft in this span
- 2-#7 at 1.50 in from Top, from 0.0 to 18.250 ft in this span
- 1-#8 at 1.50 in from Top, from 11.125 to 18.250 ft in this span

Span #2 Reinforcing....
- 2-#8 at 1.50 in from Bottom, from 0.0 to 25.833 ft in this span
- 2-#7 at 1.50 in from Top, from 0.0 to 6.50 ft in this span
- 2-#8 at 1.50 in from Bottom, from 4.50 to 21.50 ft in this span
- 3-#8 at 1.50 in from Top, from 0.0 to 4.50 ft in this span
- 2-#8 at 1.50 in from Top, from 21.50 to 25.833 ft in this span

## Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.010, Lr = 0.10, S = 0.210
Uniform Load on ALL spans : D = 0.010, Lr = 0.10, S = 0.210 ksf, Tributary Width = 10.0 ft

## *DESIGN SUMMARY*

**Design N.G.**

| | |
|---|---|
| Maximum Bending Stress Ratio = | **2.527** : 1 |
| Section used for this span | **Typical Section** |
| Mu : Applied | -262.080 k-ft |
| Mn * Phi : Allowable | 103.718 k-ft |
| Load Combination | +1.20D+0.50L+1.60S |
| Location of maximum on span | 18.176 ft |
| Span # where maximum occurs | Span # 1 |

| Maximum Deflection | | |
|---|---|---|
| Max Downward Transient Deflection | 0.522 in | Ratio = 594 |
| Max Upward Transient Deflection | -0.036 in | Ratio = 6149 |
| Max Downward Total Deflection | 0.682 in | Ratio = 454 |
| Max Upward Total Deflection | -0.050 in | Ratio = 4362 |

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| Overall MAXimum | 16.086 | 72.267 | 28.517 |
| Overall MINimum | 1.819 | 9.333 | 3.425 |
| D Only | 3.032 | 15.555 | 5.708 |
| +D+L+H | 3.032 | 15.555 | 5.708 |
| +D+Lr+H | 9.282 | 42.502 | 16.593 |
| +D+S+H | 16.086 | 72.267 | 28.517 |
| +D+0.750Lr+0.750L+H | 7.707 | 35.787 | 13.863 |

# Concrete Beam

**Lic. # : KW-06003501** **Licensee : COFFMAN ENGINEERS**

Description : typ roof beam (B15/B16)

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| +D+0.750L+0.750S+H | 12.864 | 58.018 | 22.844 |
| +D+W+H | 3.032 | 15.555 | 5.708 |
| +D+0.70E+H | 3.032 | 15.555 | 5.708 |
| +D+0.750Lr+0.750L+0.750W+H | 7.707 | 35.787 | 13.863 |
| +D+0.750L+0.750S+0.750W+H | 12.864 | 58.018 | 22.844 |
| +D+0.750Lr+0.750L+0.5250E+H | 7.707 | 35.787 | 13.863 |
| +D+0.750L+0.750S+0.5250E+H | 12.864 | 58.018 | 22.844 |
| +0.60D+W+H | 1.819 | 9.333 | 3.425 |
| +0.60D+0.70E+H | 1.819 | 9.333 | 3.425 |
| D Only | 3.032 | 15.555 | 5.708 |
| Lr Only | 5.812 | 27.696 | 10.576 |
| L Only | | | |
| S Only | 12.702 | 57.311 | 22.561 |
| W Only | | | |
| E Only | | | |
| H Only | | | |

## Shear Stirrup Requirements

Between 0.00 to 3.28 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.6.1, use stirrups spaced at 9.000 in
Between 3.35 to 7.67 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 11.4.6.1, use stirrups spaced at 0.000 in
Between 7.75 to 10.28 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.6.1, use stirrups spaced at 9.000 in
Between 10.35 to 18.18 ft, PhiVc < Vu, Req'd Vs = 33.154, use stirrups spaced at 3.000 in
Between 18.25 to 19.62 ft, Vs>(4bdf'c^.5) ACI 11.5.5.3, Req'd Vs = 36.411, use stirrups spaced at 2.000 in
Between 19.73 to 29.32 ft, PhiVc < Vu, Req'd Vs = 0.1193, use stirrups spaced at 3.000 in
Between 29.43 to 31.54 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.6.1, use stirrups spaced at 9.000 in
Between 31.64 to 35.86 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 11.4.6.1, use stirrups spaced at 0.000 in
Between 35.96 to 38.07 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.6.1, use stirrups spaced at 9.000 in
Between 38.18 to 43.98 ft, PhiVc < Vu, Req'd Vs = 21.551, use stirrups spaced at 5.000 in

## Maximum Forces & Stresses for Load Combinations

| Load Combination | | Location (ft) | Bending Stress Results ( k-ft ) | | |
|---|---|---|---|---|---|
| Segment Length | Span # | in Span | Mu : Max | Phi*Mnx | Stress Ratio |
| MAXimum BENDING Envelope | | | | | |
| Span # 1 | 1 | 18.250 | -262.08 | 103.72 | 2.53 |
| Span # 2 | 2 | 25.833 | 142.47 | 89.71 | 1.59 |
| +1.40D | | | | | |
| Span # 1 | 1 | 18.250 | -50.28 | 103.72 | 0.48 |
| Span # 2 | 2 | 25.833 | 27.34 | 89.71 | 0.30 |
| +1.20D+0.50Lr+1.60L+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -75.69 | 103.72 | 0.73 |
| Span # 2 | 2 | 25.833 | 41.15 | 89.71 | 0.46 |
| +1.20D+1.60L+0.50S+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -111.53 | 103.72 | 1.08 |
| Span # 2 | 2 | 25.833 | 60.63 | 89.71 | 0.68 |
| +1.20D+1.60Lr+0.50L | | | | | |
| Span # 1 | 1 | 18.250 | -147.38 | 103.72 | 1.42 |
| Span # 2 | 2 | 25.833 | 80.12 | 89.71 | 0.89 |
| +1.20D+1.60Lr+0.80W | | | | | |
| Span # 1 | 1 | 18.250 | -147.38 | 103.72 | 1.42 |
| Span # 2 | 2 | 25.833 | 80.12 | 89.71 | 0.89 |
| +1.20D+0.50L+1.60S | | | | | |
| Span # 1 | 1 | 18.250 | -262.08 | 103.72 | 2.53 |
| Span # 2 | 2 | 25.833 | 142.47 | 89.71 | 1.59 |
| +1.20D+1.60S+0.80W | | | | | |
| Span # 1 | 1 | 18.250 | -262.08 | 103.72 | 2.53 |
| Span # 2 | 2 | 25.833 | 142.47 | 89.71 | 1.59 |
| +1.20D+0.50Lr+0.50L+1.60W | | | | | |
| Span # 1 | 1 | 18.250 | -75.69 | 103.72 | 0.73 |
| Span # 2 | 2 | 25.833 | 41.15 | 89.71 | 0.46 |
| +1.20D+0.50L+0.50S+1.60W | | | | | |
| Span # 1 | 1 | 18.250 | -111.53 | 103.72 | 1.08 |
| Span # 2 | 2 | 25.833 | 60.63 | 89.71 | 0.68 |
| +1.20D+0.50L+0.20S+E | | | | | |
| Span # 1 | 1 | 18.250 | -70.47 | 103.72 | 0.68 |
| Span # 2 | 2 | 25.833 | 38.31 | 89.71 | 0.43 |
| +0.90D+1.60W+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -32.33 | 103.72 | 0.31 |
| Span # 2 | 2 | 25.833 | 17.57 | 89.71 | 0.20 |

# Concrete Beam



**Lic. # : KW-06003501** **Licensee : COFFMAN ENGINEERS**

Description : typ roof beam (B15/B16)

| Load Combination Segment Length | Span # | Location (ft) in Span | Bending Stress Results ( k-ft ) Mu : Max | Phi*Mnx | Stress Ratio |
|---|---|---|---|---|---|
| +0.90D+E+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -32.33 | 103.72 | 0.31 |
| Span # 2 | 2 | 25.833 | 17.57 | 89.71 | 0.20 |

## Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+S+H | 1 | 0.0167 | 18.619 | +D+S+H | -0.0502 | 15.382 |
| +D+S+H | 2 | 0.6818 | 14.393 | | 0.0000 | 15.382 |

# Concrete Beam



Description : typ floor slab (G)

## CODE REFERENCES

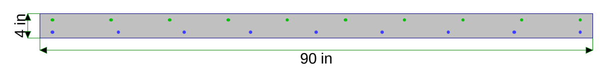
Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combination Set : ASCE 7-05

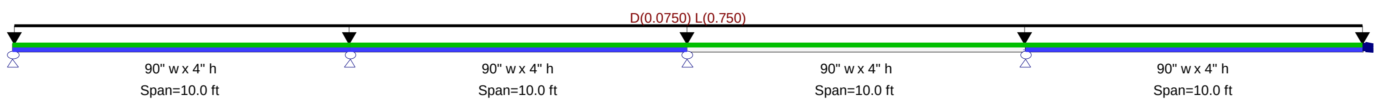
## Material Properties

f'c = 2.50 ksi
$fr = f'c^{1/2} * 7.50$ = 375.0 psi
$\psi$ Density = 145.0 pcf
$\lambda$ LtWt Factor = 1.0
Elastic Modulus = 3,122.0 ksi
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

$\phi$ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850
Fy - Stirrups 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # = # 3
Number of Resisting Legs Per Stirrup = 2

Load Combination : ASCE 7-05




## Cross Section & Reinforcing Details

Rectangular Section, Width = 90.0 in, Height = 4.0 in
Span #1 Reinforcing....
9-#4 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span — 10-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span
Span #2 Reinforcing....
9-#4 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span — 10-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span
Span #3 Reinforcing....
9-#4 at 1.0 in from Bottom, from 10.0 to 10.0 ft in this span — 10-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span
Span #4 Reinforcing....
9-#4 at 1.0 in from Bottom, from 0.0 to 10.0 ft in this span — 10-#4 at 1.0 in from Top, from 0.0 to 10.0 ft in this span

## Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.010, L = 0.10
Uniform Load on ALL spans : D = 0.010, L = 0.10 ksf, Tributary Width = 7.50 ft

## DESIGN SUMMARY

**Design N.G.**

Maximum Bending Stress Ratio = **0.994** : 1
Section used for this span **Typical Section**
Mu : Applied -18.228 k-ft
Mn * Phi : Allowable 18.332 k-ft
Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 2

Maximum Deflection
Max Downward Transient Deflection 0.056 in Ratio = 2128
Max Upward Transient Deflection -0.002 in Ratio = 70134
Max Downward Total Deflection 0.138 in Ratio = 869
Max Upward Total Deflection -0.002 in Ratio = 57480

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 | Support 4 | Support 5 |
|---|---|---|---|---|---|
| Overall MAXimum | 4.826 | 13.147 | 11.724 | 11.845 | 5.959 |
| Overall MINimum | 1.035 | 2.977 | 2.530 | 2.652 | 1.306 |
| D Only | 1.725 | 4.961 | 4.217 | 4.420 | 2.176 |
| +D+L+H | 4.826 | 13.147 | 11.724 | 11.845 | 5.959 |
| +D+Lr+H | 1.725 | 4.961 | 4.217 | 4.420 | 2.176 |
| +D+S+H | 1.725 | 4.961 | 4.217 | 4.420 | 2.176 |

# Concrete Beam



Description : typ floor slab (G)

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 | Support 4 | Support 5 |
|---|---|---|---|---|---|
| +D+0.750Lr+0.750L+H | 4.039 | 11.124 | 9.791 | 10.061 | 4.985 |
| +D+0.750L+0.750S+H | 4.039 | 11.124 | 9.791 | 10.061 | 4.985 |
| +D+W+H | 1.725 | 4.961 | 4.217 | 4.420 | 2.176 |
| +D+0.70E+H | 1.725 | 4.961 | 4.217 | 4.420 | 2.176 |
| +D+0.750Lr+0.750L+0.750W+H | 4.039 | 11.124 | 9.791 | 10.061 | 4.985 |
| +D+0.750L+0.750S+0.750W+H | 4.039 | 11.124 | 9.791 | 10.061 | 4.985 |
| +D+0.750Lr+0.750L+0.5250E+H | 4.039 | 11.124 | 9.791 | 10.061 | 4.985 |
| +D+0.750L+0.750S+0.5250E+H | 4.039 | 11.124 | 9.791 | 10.061 | 4.985 |
| +0.60D+W+H | 1.035 | 2.977 | 2.530 | 2.652 | 1.306 |
| +0.60D+0.70E+H | 1.035 | 2.977 | 2.530 | 2.652 | 1.306 |
| D Only | 1.725 | 4.961 | 4.217 | 4.420 | 2.176 |
| Lr Only | | | | | |
| L Only | 2.957 | 8.505 | 7.229 | 7.577 | 3.731 |
| S Only | | | | | |
| W Only | | | | | |
| E Only | | | | | |
| H Only | | | | | |

## Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 11.4.6.1, use stirrups spaced at 0.000 in

## Maximum Forces & Stresses for Load Combinations

| Load Combination / Segment Length | Span # | Location (ft) in Span | Bending Stress Results ( k-ft ) Mu : Max | Phi*Mnx | Stress Ratio |
|---|---|---|---|---|---|
| MAXimum BENDING Envelope | | | | | |
| Span # 1 | 1 | 10.000 | -17.14 | 18.33 | 0.93 |
| Span # 2 | 2 | 10.000 | -18.23 | 18.33 | 0.99 |
| Span # 3 | 3 | 10.000 | -13.76 | 16.75 | 0.82 |
| Span # 4 | 4 | 10.000 | -14.67 | 18.33 | 0.80 |
| +1.40D | | | | | |
| Span # 1 | 1 | 10.000 | -6.09 | 18.33 | 0.33 |
| Span # 2 | 2 | 10.000 | -6.47 | 18.33 | 0.35 |
| Span # 3 | 3 | 10.000 | -4.89 | 16.75 | 0.29 |
| Span # 4 | 4 | 10.000 | -5.21 | 18.33 | 0.28 |
| +1.20D+0.50Lr+1.60L+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -17.14 | 18.33 | 0.93 |
| Span # 2 | 2 | 10.000 | -18.23 | 18.33 | 0.99 |
| Span # 3 | 3 | 10.000 | -13.76 | 16.75 | 0.82 |
| Span # 4 | 4 | 10.000 | -14.67 | 18.33 | 0.80 |
| +1.20D+1.60L+0.50S+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -17.14 | 18.33 | 0.93 |
| Span # 2 | 2 | 10.000 | -18.23 | 18.33 | 0.99 |
| Span # 3 | 3 | 10.000 | -13.76 | 16.75 | 0.82 |
| Span # 4 | 4 | 10.000 | -14.67 | 18.33 | 0.80 |
| +1.20D+1.60Lr+0.50L | | | | | |
| Span # 1 | 1 | 10.000 | -8.94 | 18.33 | 0.49 |
| Span # 2 | 2 | 10.000 | -9.51 | 18.33 | 0.52 |
| Span # 3 | 3 | 10.000 | -7.18 | 16.75 | 0.43 |
| Span # 4 | 4 | 10.000 | -7.65 | 18.33 | 0.42 |
| +1.20D+1.60Lr+0.80W | | | | | |
| Span # 1 | 1 | 10.000 | -5.22 | 18.33 | 0.28 |
| Span # 2 | 2 | 10.000 | -5.55 | 18.33 | 0.30 |
| Span # 3 | 3 | 10.000 | -4.19 | 16.75 | 0.25 |
| Span # 4 | 4 | 10.000 | -4.47 | 18.33 | 0.24 |
| +1.20D+0.50L+1.60S | | | | | |
| Span # 1 | 1 | 10.000 | -8.94 | 18.33 | 0.49 |
| Span # 2 | 2 | 10.000 | -9.51 | 18.33 | 0.52 |
| Span # 3 | 3 | 10.000 | -7.18 | 16.75 | 0.43 |
| Span # 4 | 4 | 10.000 | -7.65 | 18.33 | 0.42 |
| +1.20D+1.60S+0.80W | | | | | |
| Span # 1 | 1 | 10.000 | -5.22 | 18.33 | 0.28 |
| Span # 2 | 2 | 10.000 | -5.55 | 18.33 | 0.30 |
| Span # 3 | 3 | 10.000 | -4.19 | 16.75 | 0.25 |
| Span # 4 | 4 | 10.000 | -4.47 | 18.33 | 0.24 |
| +1.20D+0.50Lr+0.50L+1.60W | | | | | |
| Span # 1 | 1 | 10.000 | -8.94 | 18.33 | 0.49 |
| Span # 2 | 2 | 10.000 | -9.51 | 18.33 | 0.52 |
| Span # 3 | 3 | 10.000 | -7.18 | 16.75 | 0.43 |

# Concrete Beam



Description : typ floor slab (G)

| Load Combination / Segment Length | Span # | Location (ft) in Span | Bending Stress Results ( k-ft ) Mu : Max | Phi*Mnx | Stress Ratio |
|---|---|---|---|---|---|
| Span # 4 | 4 | 10.000 | -7.65 | 18.33 | 0.42 |
| +1.20D+0.50L+0.50S+1.60W | | | | | |
| Span # 1 | 1 | 10.000 | -8.94 | 18.33 | 0.49 |
| Span # 2 | 2 | 10.000 | -9.51 | 18.33 | 0.52 |
| Span # 3 | 3 | 10.000 | -7.18 | 16.75 | 0.43 |
| Span # 4 | 4 | 10.000 | -7.65 | 18.33 | 0.42 |
| +1.20D+0.50L+0.20S+E | | | | | |
| Span # 1 | 1 | 10.000 | -8.94 | 18.33 | 0.49 |
| Span # 2 | 2 | 10.000 | -9.51 | 18.33 | 0.52 |
| Span # 3 | 3 | 10.000 | -7.18 | 16.75 | 0.43 |
| Span # 4 | 4 | 10.000 | -7.65 | 18.33 | 0.42 |
| +0.90D+1.60W+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -3.91 | 18.33 | 0.21 |
| Span # 2 | 2 | 10.000 | -4.16 | 18.33 | 0.23 |
| Span # 3 | 3 | 10.000 | -3.14 | 16.75 | 0.19 |
| Span # 4 | 4 | 10.000 | -3.35 | 18.33 | 0.18 |
| +0.90D+E+1.60H | | | | | |
| Span # 1 | 1 | 10.000 | -3.91 | 18.33 | 0.21 |
| Span # 2 | 2 | 10.000 | -4.16 | 18.33 | 0.23 |
| Span # 3 | 3 | 10.000 | -3.14 | 16.75 | 0.19 |
| Span # 4 | 4 | 10.000 | -3.35 | 18.33 | 0.18 |

## Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L+H | 1 | 0.1381 | 4.474 | +D+L+H | -0.0016 | 10.263 |
| +D+L+H | 2 | 0.0278 | 5.000 | +D+L+H | -0.0021 | 0.789 |
| +D+L+H | 3 | 0.0411 | 5.000 | | 0.0000 | 0.789 |
| +D+L+H | 4 | 0.0400 | 5.000 | | 0.0000 | 0.789 |

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Project Descr:

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# Concrete Beam

**Lic. # : KW-06003501** **Licensee : COFFMAN ENGINEERS**

Description : typ floor beam (B32/B33)

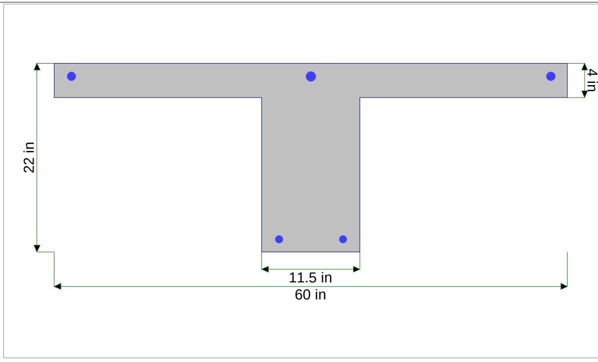
## CODE REFERENCES

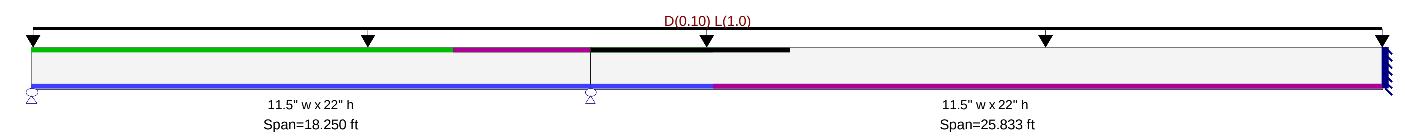
Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05
Load Combination Set : ASCE 7-05

## Material Properties

| | | | | | |
|---|---|---|---|---|---|
| f'c | = | 2.50 ksi | $\phi$ Phi Values | Flexure : | 0.90 |
| fr = f'c$^{1/2}$ * 7.50 | = | 375.0 psi | | Shear : | 0.750 |
| $\psi$ Density | = | 145.0 pcf | $\beta_1$ | = | 0.850 |
| $\lambda$ LtWt Factor | = | 1.0 | | | |
| Elastic Modulus | = | 3,122.0 ksi | Fy - Stirrups | | 40.0 ksi |
| fy - Main Rebar | = | 40.0 ksi | E - Stirrups | = | 29,000.0 ksi |
| E - Main Rebar | = | 29,000.0 ksi | Stirrup Bar Size # = | # | 3 |
| | | | Number of Resisting Legs Per Stirrup = | | 2 |

Load Combination :ASCE 7-05




## Cross Section & Reinforcing Details

Tee Section, Stem Width = 11.50 in, Total Height = 22.0 in, Top Flange Width = 60.0 in, Flange Thickness = 4.0 in

Span #1 Reinforcing....
- 2-#7 at 1.50 in from Bottom, from 0.0 to 18.250 ft in this span
- 2-#8 at 1.50 in from Top, from 0.0 to 18.250 ft in this span
- 1-#9 at 1.50 in from Top, from 13.750 to 18.250 ft in this span

Span #2 Reinforcing....
- 2-#8 at 1.50 in from Bottom, from 0.0 to 25.833 ft in this span
- 1-#9 at 1.50 in from Top, from 0.0 to 4.0 ft in this span
- 1-#9 at 1.50 in from Bottom, from 4.0 to 25.833 ft in this span
- 2-#8 at 1.50 in from Top, from 0.0 to 6.50 ft in this span

## Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.010, L = 0.10
Uniform Load on ALL spans : D = 0.010, L = 0.10 ksf, Tributary Width = 10.0 ft

## DESIGN SUMMARY

**Design N.G.**

Maximum Bending Stress Ratio = **0.742** : 1

| | |
|---|---|
| Section used for this span | **Typical Section** |
| Mu : Applied | -110.365 k-ft |
| Mn * Phi : Allowable | 148.779 k-ft |
| Load Combination | +1.20D+0.50Lr+1.60L+1.60H |
| Location of maximum on span | 0.000 ft |
| Span # where maximum occurs | Span # 2 |

Maximum Deflection

| | | |
|---|---|---|
| Max Downward Transient Deflection | 0.047 in | Ratio = 6611 |
| Max Upward Transient Deflection | 0.000 in | Ratio = 0 <360 |
| Max Downward Total Deflection | 0.128 in | Ratio = 2419 |
| Max Upward Total Deflection | -0.003 in | Ratio = 66740 |

## Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| Overall MAXimum | 9.943 | 39.889 | 18.501 |
| Overall MINimum | 2.129 | 8.028 | 4.393 |
| D Only | 3.548 | 13.380 | 7.322 |
| +D+L+H | 9.943 | 39.889 | 18.501 |
| +D+Lr+H | 3.548 | 13.380 | 7.322 |
| +D+S+H | 3.548 | 13.380 | 7.322 |
| +D+0.750Lr+0.750L+H | 8.317 | 33.055 | 15.941 |
| +D+0.750L+0.750S+H | 8.317 | 33.055 | 15.941 |
| +D+W+H | 3.548 | 13.380 | 7.322 |

## Concrete Beam



Description : typ floor beam (B32/B33)

### Vertical Reactions

Support notation : Far left is #1

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| +D+0.70E+H | 3.548 | 13.380 | 7.322 |
| +D+0.750Lr+0.750L+0.750W+H | 8.317 | 33.055 | 15.941 |
| +D+0.750L+0.750S+0.750W+H | 8.317 | 33.055 | 15.941 |
| +D+0.750Lr+0.750L+0.5250E+H | 8.317 | 33.055 | 15.941 |
| +D+0.750L+0.750S+0.5250E+H | 8.317 | 33.055 | 15.941 |
| +0.60D+W+H | 2.129 | 8.028 | 4.393 |
| +0.60D+0.70E+H | 2.129 | 8.028 | 4.393 |
| D Only | 3.548 | 13.380 | 7.322 |
| Lr Only | | | |
| L Only | 6.352 | 24.894 | 12.837 |
| S Only | | | |
| W Only | | | |
| E Only | | | |
| H Only | | | |

### Shear Stirrup Requirements

Between 0.00 to 2.46 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.6.1, use stirrups spaced at 10.000 in
Between 2.53 to 10.35 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 11.4.6.1, use stirrups spaced at 0.000 in
Between 10.43 to 26.79 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.6.1, use stirrups spaced at 10.000 in
Between 26.90 to 34.59 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 11.4.6.1, use stirrups spaced at 0.000 in
Between 34.70 to 43.98 ft, PhiVc < Vu, Req'd Vs = 11.421, use stirrups spaced at 10.000 in

### Maximum Forces & Stresses for Load Combinations

| Load Combination / Segment Length | Span # | Location (ft) in Span | Bending Stress Results ( k-ft ) Mu : Max | Phi*Mnx | Stress Ratio |
|---|---|---|---|---|---|
| MAXimum BENDING Envelope | | | | | |
| Span # 1 | 1 | 18.250 | -108.38 | 148.44 | 0.73 |
| Span # 2 | 2 | 25.833 | -110.37 | 148.78 | 0.74 |
| +1.40D | | | | | |
| Span # 1 | 1 | 18.250 | -36.93 | 148.44 | 0.25 |
| Span # 2 | 2 | 25.833 | -37.61 | 148.78 | 0.25 |
| +1.20D+0.50Lr+1.60L+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -108.38 | 148.44 | 0.73 |
| Span # 2 | 2 | 25.833 | -110.37 | 148.78 | 0.74 |
| +1.20D+1.60L+0.50S+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -108.38 | 148.44 | 0.73 |
| Span # 2 | 2 | 25.833 | -110.37 | 148.78 | 0.74 |
| +1.20D+1.60Lr+0.50L | | | | | |
| Span # 1 | 1 | 18.250 | -55.63 | 148.44 | 0.37 |
| Span # 2 | 2 | 25.833 | -56.65 | 148.78 | 0.38 |
| +1.20D+1.60Lr+0.80W | | | | | |
| Span # 1 | 1 | 18.250 | -31.66 | 148.44 | 0.21 |
| Span # 2 | 2 | 25.833 | -32.23 | 148.78 | 0.22 |
| +1.20D+0.50L+1.60S | | | | | |
| Span # 1 | 1 | 18.250 | -55.63 | 148.44 | 0.37 |
| Span # 2 | 2 | 25.833 | -56.65 | 148.78 | 0.38 |
| +1.20D+1.60S+0.80W | | | | | |
| Span # 1 | 1 | 18.250 | -31.66 | 148.44 | 0.21 |
| Span # 2 | 2 | 25.833 | -32.23 | 148.78 | 0.22 |
| +1.20D+0.50Lr+0.50L+1.60W | | | | | |
| Span # 1 | 1 | 18.250 | -55.63 | 148.44 | 0.37 |
| Span # 2 | 2 | 25.833 | -56.65 | 148.78 | 0.38 |
| +1.20D+0.50L+0.50S+1.60W | | | | | |
| Span # 1 | 1 | 18.250 | -55.63 | 148.44 | 0.37 |
| Span # 2 | 2 | 25.833 | -56.65 | 148.78 | 0.38 |
| +1.20D+0.50L+0.20S+E | | | | | |
| Span # 1 | 1 | 18.250 | -55.63 | 148.44 | 0.37 |
| Span # 2 | 2 | 25.833 | -56.65 | 148.78 | 0.38 |
| +0.90D+1.60W+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -23.74 | 148.44 | 0.16 |
| Span # 2 | 2 | 25.833 | -24.18 | 148.78 | 0.16 |
| +0.90D+E+1.60H | | | | | |
| Span # 1 | 1 | 18.250 | -23.74 | 148.44 | 0.16 |
| Span # 2 | 2 | 25.833 | -24.18 | 148.78 | 0.16 |

### Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L+H | 1 | 0.0190 | 7.039 | +D+L+H | -0.0033 | 16.946 |

# Concrete Beam



Description : typ floor beam (B32/B33)

## Overall Maximum Deflections

| Load Combination | Span | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| +D+L+H | 2 | 0.1281 | 12.917 | | 0.0000 | 16.946 |

# Appendix D

ASCE 41-13 CHECKLISTS AND CALCULATIONS

# ASCE 41-13 Evaluation

## Tier 1 Evaluation

Basic Performance Objective = Life-Safety (Table 2-1)

$S_{DS} > 0.50g \longrightarrow$ High Seismicity (Table 2-5)

Building Type = C2 (Table 3-1)

# of stories = 6 < 8 ✓ (Table 3-2)

Required Checklists: (High, LS) (Table 4-7)

- Basic Configuration
- Life Safety Structural
- Life Safety Non-Structural

## Example Calculations: Bldg A

$T = C_t h_n^{\beta} = 0.02(64ft)^{0.75} = 0.453\ s$ (4-5)

$S_a = \dfrac{S_{x1}}{T} = \dfrac{0.344}{0.453} = 0.759 \leq S_{xs} = 0.774$ ✓ (4-4)

controls

$C = 1.0$ (Table 4-8)

$V = CS_aW = 1.0(0.759)(7139\ k) = 5418.5\ k$ (4-1)

$M_s = 4.0$ (Table 4-9)

$v_j^{avg} = \dfrac{1}{M_s}\left(\dfrac{V_j}{A_w}\right)$ (See spreadsheet) (4-9)

COFFMAN ENGINEERS

| | | | |
|---|---|---|---|
| project Buckner Building | by BJW | sheet no. | |
| location | date 1/22/16 | | |
| client | checked | job no. | |
| | date | 150859 | |

## Basic Configuration Checklist

### Adjacent Buildings

Clear distance = 8 in $< 0.04h_n = 0.04(64ft)(12^{in}/ft) = 30.7in$

$\therefore$ NC

### Overturning - Bldg D, F

$b/h = 18.33ft/64ft = 0.286 < 0.6S_a = 0.6(0.759) = 0.455$

$\therefore$ NC

### Overturning - Bldg A, C, E, G

$b/h = 45.67ft/64ft = 0.714 > 0.6S_a$

$\therefore$ C (Bldg B good by inspection)

| | | | |
|---|---|---|---|
| COFFMAN ENGINEERS | project Buckner Building | by BJW | sheet no. |
| | location | date 1/22/16 | |
| | client | checked | job no. 150859 |
| | | date | |

# Design Maps Detailed Report

## ASCE 41-13 Retrofit Standard, BSE-1E (60.7747°N, 148.675°W)

Site Class B – “Rock”

### Section 2.4.1 – General Procedure for Hazard Due to Ground Shaking

20%/50-year maximum direction spectral response acceleration for 0.2s and 1.0s periods, respectively:

| | |
|---|---|
| **From Section 2.4.1.4** | $S_{S,20/50}$ = 0.774 g |
| **From Section 2.4.1.4** | $S_{1,20/50}$ = 0.344 g |

### Section 2.4.1.6 – Adjustment for Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class B, based on the site soil properties in accordance with Section 2.4.1.6.1.

| SITE CLASS | SOIL PROFILE NAME | Soil shear wave velocity, $\bar{v}_s$, (ft/s) | Standard penetration resistance, $\bar{N}$ | Soil undrained shear strength, $\bar{s}_u$, (psf) |
|---|---|---|---|---|
| A | Hard rock | $\bar{v}_S > 5{,}000$ | N/A | N/A |
| B | Rock | $2{,}500 < \bar{v}_S \le 5{,}000$ | N/A | N/A |
| C | Very dense soil and soft rock | $1{,}200 < \bar{v}_S \le 2{,}500$ | $\bar{N} > 50$ | >2,000 psf |
| D | Stiff soil profile | $600 \le \bar{v}_S < 1{,}200$ | $15 \le \bar{N} \le 50$ | 1,000 to 2,000 psf |
| E | Stiff soil profile | $\bar{v}_S < 600$ | $\bar{N} < 15$ | <1,000 psf |
| E | — | Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf | | |
| F | — | Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where $H$ = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet) | | |

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Table 2–3. Values of $F_a$ as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration $S_s$

| Site Class | Mapped Spectral Acceleration at Short-Period $S_s$ | | | | |
|---|---|---|---|---|---|
| | $S_S \leq 0.25$ | $S_S = 0.50$ | $S_S = 0.75$ | $S_S = 1.00$ | $S_S \geq 1.25$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | Site-specific geotechnical and dynamic site response analyses shall be performed | | | | |

Note: Use straight–line interpolation for intermediate values of $S_S$

**For Site Class = B and $S_s$ = 0.774 g, $F_a$ = 1.000**

Table 2–4. Values of $F_v$ as a Function of Site Class and Mapped Spectral Response Acceleration at 1 s Period $S_1$

| Site Class | Mapped Spectral Acceleration at 1 s Period $S_1$ | | | | |
|---|---|---|---|---|---|
| | $S_1 \leq 0.10$ | $S_1 = 0.20$ | $S_1 = 0.30$ | $S_1 = 0.40$ | $S_1 \geq 0.50$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.2 | 2.8 | 2.4 | 2.4 |
| F | Site-specific geotechnical and dynamic site response analyses shall be performed | | | | |

Note: Use straight–line interpolation for intermediate values of $S_1$

**For Site Class = B and $S_1$ = 0.344 g, $F_v$ = 1.000**

**Provided as a reference for Equation (2-4):** $F_aS_{S,20/50}$ = 1.000 x 0.774 g = 0.774 g

**Provided as a reference for Equation (2-5):** $F_vS_{1,20/50}$ = 1.000 x 0.344 g = 0.344 g

**Provided as a reference for Equation (2-4):** $S_{XS,BSE-1N}$ = ⅔ x $S_{XS,BSE-2N}$ = ⅔ x $F_aS_{S,BSE-2N}$ = 1.000 g

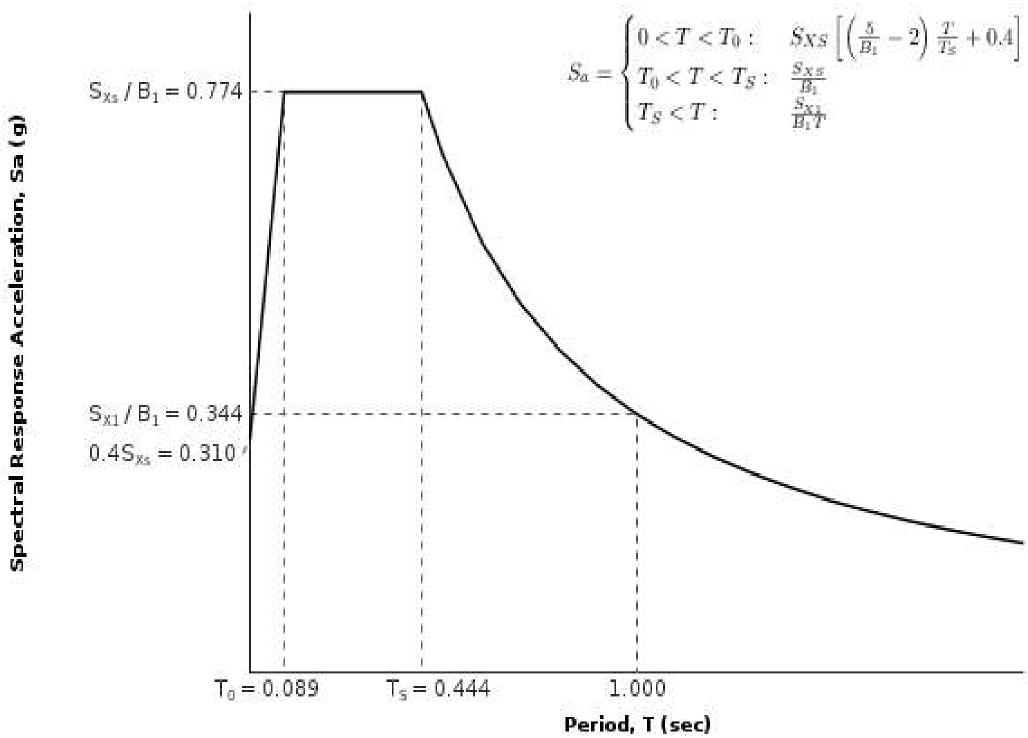
**Provided as a reference for Equation (2-5):** $S_{X1,BSE-1N}$ = ⅔ x $S_{X1,BSE-2N}$ = ⅔ x $F_vS_{1,BSE-2N}$ = 0.489 g

**Equation (2–4):** $S_{XS,BSE-1E}$ = MIN[$F_aS_{S,20/50}$, $S_{XS,BSE-1N}$] = MIN[0.774g, 1.000g] = 0.774g

**Equation (2–5):** $S_{X1,BSE-1E}$ = MIN[$F_vS_{S,20/50}$, $S_{X1,BSE-1N}$] = MIN[0.344g, 0.489g] = 0.344g

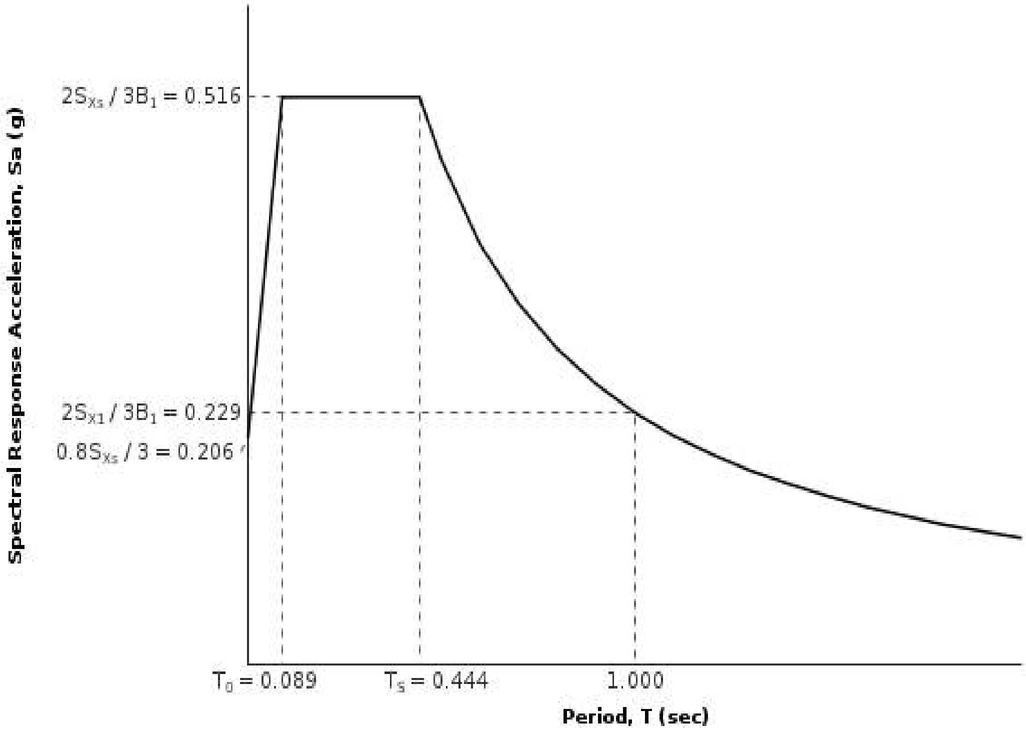
## Section 2.4.1.7.1 — General Horizontal Response Spectrum

Figure 2-1. General Horizontal Response Spectrum



# Section 2.4.1.7.2 — General Vertical Response Spectrum

The General Vertical Response Spectrum is determined by multiplying the General Horizontal Response Spectrum by ⅔.



Project: Buckner Building

Location: Whittier, AK

Completed by: Brian Walkenhauer

Date:

## 16.1.2LS LIFE SAFETY BASIC CONFIGURATION CHECKLIST

Building Units A, B, C, E & G

### Low Seismicity

### Building System

*General*

C NC N/A U LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

C NC N/A U MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

*Building Configuration*

C NC N/A U WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)

C NC N/A U SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

units A, E & G

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

C NC N/A U GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

unit A

C NC N/A U MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

### Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

### Geologic Site Hazards

C NC N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)

C NC N/A U SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

### High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

### Foundation Configuration

C NC N/A U OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

C NC N/A U TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

Project: Buckner Building  Location: Whittier, AK

Completed by: Brian Walkenhauer  Date: 

## 16.1.2LS LIFE SAFETY BASIC CONFIGURATION CHECKLIST

Building Units D & F

**Low Seismicity**

**Building System**

*General*

unit F

C NC N/A U LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

C NC N/A U MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

*Building Configuration*

unit F

C NC N/A U WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)

unit F

C NC N/A U SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

C NC N/A U GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

C NC N/A U MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

**Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.**

**Geologic Site Hazards**

C NC N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)

C NC N/A U SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

**High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.**

**Foundation Configuration**

C NC N/A U OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

C NC N/A U TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

Project: Buckner Building

Location: Whittier, AK

Completed by: Peter Hewko & Brian Walkenhauer

Date:

## 16.10LS LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES C2: CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS AND C2A: CONCRETE SHEAR WALLS WITH FLEXIBLE DIAPHRAGMS

**Low and Moderate Seismicity**

**Seismic-Force-Resisting System**

C NC N/A U COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1)

C NC N/A U REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)

C NC N/A U SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 lb/in.$^2$ or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)

C NC N/A U REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)

**Connections**

C NC N/A U WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

C NC N/A U TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)

C NC N/A U FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)

**High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.**

**Seismic-Force-Resisting System**

C NC N/A U DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)

C NC N/A U FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)

C NC N/A U COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than $d/2$ and are anchored into the confined core of the beam with hooks of 135 degrees or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)

**Connections**

C NC N/A U UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)

**Diaphragms (Flexible or Stiff)**

C NC N/A U DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)

C NC N/A U OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)

unit A

**Flexible Diaphragms**

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

Buckner Building - Seismic Dead Weight

**A WING**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 20% Flat Roof Snow Load | 218.50 | 44.17 | 227 | | | 438162 |

| Component | Width (ft) | Height (ft) | Length (ft) | Quantity (ea) | Volume (cf) | Weight (lbs) |
|---|---|---|---|---|---|---|
| Roof | | | | | | |
| P.H. Roof Slab | 32.50 | 0.44 | 19.00 | 1 | 270 | 40523 |
| P.H. Roof Beam "B5" | 1.25 | 2.00 | 28.00 | 2 | 140 | 21000 |
| P.H. Walls | 0.67 | 17.00 | 47.00 | 2 | 1065 | 159800 |
| | 0.67 | 17.00 | 29.50 | 2 | 669 | 100300 |
| machine room platform | 15.50 | 19.00 | 0.33 | 1 | 98 | 14725 |
| platform walls | 0.67 | 8.00 | 30.00 | 1 | 160 | 24000 |
| Roof Slab | 46.17 | 0.44 | 218.50 | 1 | 4413 | 661987 |
| | | | | | | |
| Roof Beams | | | | | | |
| B15 | 0.58 | 1.67 | 46.17 | 2 | 90 | 13466 |
| B16 | 1.42 | 1.67 | 18.33 | 1 | 43 | 6492 |
| B17 | 1.42 | 1.67 | 18.33 | 10 | 433 | 64919 |
| B18 | 1.42 | 1.67 | 25.83 | 11 | 671 | 100641 |
| B19 | 1.42 | 1.67 | 25.83 | 2 | 122 | 18298 |
| B20 | 1.42 | 1.67 | 25.00 | 1 | 59 | 8854 |
| B21 | 0.79 | 1.00 | 11.67 | 1 | 9 | 1386 |
| | | | | | | |
| Total | | | | | 8243 | **1674553** |

| | | | | | | |
|---|---|---|---|---|---|---|
| 4th Floor Walls | 0.67 | 10.00 | 218.50 | 2 | 2913 | 437000 |
| | 0.67 | 10.00 | 45.67 | 3 | 913 | 137010 |
| 4th Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 43 | 968 | 145125 |
| 18x24 | 1.50 | 10.00 | 2.00 | 3 | 90 | 13500 |
| 4th Floor Slab | 45.67 | 0.42 | 218.50 | 1 | 4158 | 623681 |
| 4th Floor Beams | | | | | | |
| B10 | 1.08 | 1.67 | 18.33 | 1 | 33 | 4964 |
| B12 | 1.25 | 1.67 | 18.33 | 2 | 76 | 11456 |
| B13 | 1.25 | 1.67 | 25.83 | 2 | 108 | 16146 |
| B23 | 1.25 | 1.67 | 18.33 | 10 | 382 | 57281 |
| B24 | 1.25 | 1.67 | 25.83 | 11 | 592 | 88801 |
| | | | | | | |
| total | | | | | 10233 | **1534964** |

Buckner Building - Seismic Dead Weight

**A WING**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 3rd Floor Walls | 0.67 | 10.00 | 218.50 | 2 | 2913 | 437000 |
| | 0.67 | 10.00 | 45.67 | 3 | 913 | 137010 |
| 3rd Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 43 | 968 | 145125 |
| 18x24 | 1.50 | 10.00 | 2.00 | 3 | 90 | 13500 |
| 3rd Floor Slab | 45.67 | 0.42 | 218.50 | 1 | 4158 | 623681 |
| 3rd Floor Beams | | | | | | |
| B10 | 1.08 | 1.67 | 18.33 | 1 | | |
| B12 | 1.25 | 1.67 | 18.33 | 2 | 76 | 11456 |
| B13 | 1.25 | 1.67 | 25.83 | 2 | 108 | 16146 |
| B23 | 1.25 | 1.67 | 18.33 | 10 | 382 | 57281 |
| B24 | 1.25 | 1.67 | 25.83 | 5 | 269 | 40364 |
| B25 | 1.25 | 1.67 | 25.83 | 1 | 54 | 8073 |
| B26 | 1.25 | 1.67 | 25.83 | 4 | 215 | 32291 |
| B27 | 1.25 | 1.67 | 25.83 | 1 | 54 | 8073 |
| | | | | | | |
| total | | | | | 10200 | **1530000** |

| | | | | | | |
|---|---|---|---|---|---|---|
| 2nd Floor Walls | 0.67 | 10.00 | 218.50 | 2 | 2913 | 437000 |
| | 0.67 | 10.00 | 45.67 | 3 | 913 | 137010 |
| 2nd Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 43 | 968 | 145125 |
| 18x24 | 1.50 | 10.00 | 2.00 | 3 | 90 | 13500 |
| 2nd Floor Slab | 45.67 | 0.42 | 218.50 | 1 | 4158 | 623681 |
| G.L. 26-37 & B-C Slab | 24.00 | 0.50 | 124.00 | 1 | 1488 | 223200 |
| 2nd Floor Beams | | | | | | |
| B23 | 1.25 | 1.67 | 18.33 | 3 | 115 | 17184 |
| B24 | 1.25 | 1.67 | 25.83 | 3 | 161 | 24216 |
| B25 | 1.25 | 1.67 | 24.00 | 2 | 100 | 15000 |
| B26 | 1.25 | 1.67 | 18.33 | 2 | 76 | 11456 |
| B28 | 1.25 | 1.67 | 24.00 | 1 | 50 | 7500 |
| B29 | 1.25 | 1.67 | 18.33 | 1 | 38 | 5728 |
| B30 | 1.25 | 1.67 | 25.83 | 1 | 54 | 8073 |
| B31 | 1.42 | 1.50 | 24.33 | 2 | 103 | 15510 |
| B32 | 1.42 | 0.86 | 25.83 | 1 | 31 | 4705 |
| B33 | 1.58 | 1.50 | 18.33 | 1 | 44 | 6530 |
| B34 | 1.58 | 1.50 | 25.83 | 1 | 61 | 9202 |
| B35 | 1.42 | 1.75 | 25.83 | 1 | 64 | 9606 |
| B36 | 1.25 | 1.67 | 18.33 | 3 | 115 | 17184 |
| B37 | 1.25 | 1.67 | 25.83 | 3 | 161 | 24216 |
| B41 | 1.25 | 1.67 | 24.00 | 3 | 150 | 22500 |
| B42 | 1.25 | 1.67 | 18.33 | 1 | 38 | 5728 |
| B43 | 1.25 | 1.67 | 25.83 | 1 | 54 | 8072 |
| | | | | | | |
| total | | | | | 11946 | **1791926** |

Buckner Building - Seismic Dead Weight

**A WING**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 1st Floor Walls | 0.67 | 10.00 | 46.50 | 4 | 1240 | 186000 |
| | 0.67 | 10.00 | 45.67 | 3 | 913 | 137010 |
| | 0.67 | 10.00 | 124.00 | 1 | 831 | 124620 |
| | 0.67 | 10.00 | 24.00 | 2 | 322 | 48240 |
| 1st Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 47 | 1058 | 158625 |
| 18x24 | 1.50 | 10.00 | 2.00 | 10 | 300 | 45000 |
| 1st Floor Slab | 45.67 | 0.46 | 218.50 | 1 | 4574 | 686049 |
| | 24.00 | 0.46 | 124.00 | 1 | 1364 | 204600 |
| 1st Floor Beams | | | | | | |
| B12 | 1.25 | 1.67 | 18.33 | 2 | 76 | 11456 |
| B13 | 1.25 | 1.67 | 25.83 | 3 | 161 | 24218 |
| B23 | 1.25 | 1.67 | 18.33 | 6 | 229 | 34369 |
| B24 | 1.25 | 1.67 | 25.83 | 3 | 161 | 24218 |
| B27 | 1.08 | 1.67 | 18.33 | 1 | 33 | 4964 |
| B28 | 1.25 | 1.67 | 24.00 | 5 | 250 | 37500 |
| B29 | 1.25 | 1.67 | 18.33 | 4 | 153 | 22913 |
| B30 | 1.25 | 1.67 | 25.83 | 5 | 269 | 40364 |
| B31 | 1.25 | 1.67 | 25.83 | 3 | 161 | 24218 |
| | | | | | | |
| total | | | | | 12096 | **1814365** |

| | | | | | | |
|---|---|---|---|---|---|---|
| Ground Floor Walls | 0.67 | 12.00 | 46.50 | 4 | 1488 | 223200 |
| | 0.67 | 12.00 | 45.67 | 3 | 1096 | 164412 |
| | 0.67 | 12.00 | 124.00 | 1 | 997 | 149544 |
| | 0.67 | 12.00 | 24.00 | 2 | 386 | 57888 |
| Ground Floor Columns | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 11.00 | 297 | 44550 |
| 20x20 | 1.67 | 12.00 | 1.67 | 36 | 1200 | 180000 |
| 18x24 | 1.50 | 12.00 | 2.00 | 10.00 | 360 | 54000 |
| 22x22 | 1.83 | 12.00 | 1.83 | 1 | 40 | 6050 |
| Ground Floor Slab | 45.67 | 0.46 | 218.50 | 2 | 9147 | 1372098 |
| | 24.00 | 0.46 | 124.00 | 1 | 1364 | 204600 |
| Ground Floor Beams | | | | | | |
| B14 | 1.25 | 1.67 | 18.33 | 1 | 38 | 5728 |
| B15 | 1.08 | 1.67 | 24.00 | 1 | 43 | 6500 |
| B16 | 1.08 | 1.67 | 24.00 | 1 | 43 | 6500 |
| B18 | 1.08 | 1.67 | 24.00 | 1 | 43 | 6500 |
| B20 | 1.25 | 1.67 | 18.33 | 1 | 38 | 5728 |
| B21 | 1.25 | 1.67 | 18.33 | 2 | 76 | 11456 |
| B22 | 1.25 | 1.67 | 25.83 | 3 | 161 | 24216 |
| B23 | 0.67 | 4.83 | 18.33 | 3 | 177 | 26579 |
| B24 | 0.96 | 1.75 | 25.83 | 1 | 43 | 6498 |
| B25 | 1.08 | 1.75 | 24.00 | 2 | 91 | 13650 |
| B28 | 1.08 | 1.75 | 24.00 | 5 | 228 | 34125 |
| B29 | 1.25 | 1.75 | 18.33 | 4 | 160 | 24058 |
| B30 | 1.25 | 1.75 | 25.83 | 6 | 339 | 50859 |
| B31 | 1.08 | 1.75 | 25.83 | 1 | 49 | 7345 |
| B36 | 1.08 | 1.75 | 18.33 | 1 | 35 | 5213 |
| | | | | | 0 | 0 |
| total | | | | | 17942 | **2691296** |

Buckner Building - Seismic Dead Weight

**A WING**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| Basement Floor Walls | 0.67 | 12.00 | 255.00 | 1 | 2040 | 306000 |
| Basement Floor Columns | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 7 | 189 | 28350 |
| 20x20 | 1.67 | 12.00 | 1.67 | 17 | 567 | 85000 |
| Basement Floor Slab | 24.00 | 0.38 | 62.00 | 1 | 558 | 83700 |
| | 46.50 | 0.46 | 44.50 | 1 | 948 | 142261 |
| Basement Floor Beams | | | | | | |
| B1 | 0.96 | 1.17 | 15.50 | 2 | 35 | 5199 |
| B2 | 0.96 | 1.17 | 15.50 | 2 | 35 | 5199 |
| B3 | 1.25 | 1.67 | 18.33 | 1 | 38 | 5728 |
| B4 | 1.25 | 1.67 | 25.83 | 1 | 54 | 8072 |
| B5 | 1.25 | 1.67 | 8.00 | 1 | 17 | 2500 |
| B6 | 1.25 | 1.67 | 25.83 | 1 | 54 | 8072 |
| | | | | | | |
| total | | | | | 4534 | **680081** |

Buckner Building - Seismic Dead Weight

**B WING**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 20% Flat Roof Snow Load | 96.50 | 100 | 227 | | | 438110 |

| Component | Width (ft) | Height (ft) | Length (ft) | Quantity (ea) | Volume (cf) | Weight (lbs) |
|---|---|---|---|---|---|---|
| 2nd Floor Roof | | | | | | |
| G.L. 27-30 Steel Joists | | | 50.00 | 30 | | 75000 |
| G.L. 27-30 Concrete Deck | 46.50 | 0.25 | 77.75 | 1 | 904 | 135577 |
| G.L. 30-33 Concrete Slab | 46.50 | 0.50 | 77.75 | 1 | 1808 | 271153 |
| 2nd Floor Beams | | | | | | |
| B38 | 1.08 | 1.67 | 20.00 | 2 | 72 | 10833 |
| B39 | 1.08 | 1.67 | 20.00 | 2 | 72 | 10833 |
| B40 | 1.08 | 1.67 | 20.00 | 2 | 72 | 10833 |
| B44 | 0.96 | 1.17 | 15.50 | 1 | 17 | 2599 |
| B45 | 0.96 | 1.17 | 15.50 | 1 | 17 | 2599 |
| B46 | 0.96 | 1.17 | 15.50 | 1 | 17 | 2599 |
| total | | | | | 2980 | **960138** |
| 1st Floor Walls | 0.67 | 10.00 | 96.33 | 2 | 1284 | 192660 |
| | 0.67 | 10.00 | 77.75 | 3 | 1555 | 233250 |
| 1st Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 14 | 315 | 47250 |
| 20x20 | 1.67 | 10.00 | 1.67 | 8 | 222 | 33333 |
| 1st Floor Slab | 45.50 | 0.46 | 77.75 | 1 | 1621 | 243212 |
| 1st Floor Beams | | | | | | |
| B34 | 1.08 | 1.67 | 20.00 | 4 | 144 | 21667 |
| B35 | 1.08 | 1.67 | 20.00 | 2 | 72 | 10833 |
| B36 | 1.08 | 1.33 | 15.50 | 2 | 45 | 6717 |
| total | | | | | 5259 | **788922** |
| Ground Floor Walls | 0.67 | 12.00 | 96.33 | 1 | 771 | 115596 |
| | 0.67 | 12.00 | 65.00 | 1 | 520 | 78000 |
| | 0.67 | 12.00 | 77.75 | 3 | 1866 | 279900 |
| Ground Floor Columns | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 14 | 378 | 56700 |
| 20x20 | 1.67 | 12.00 | 1.67 | 8 | 267 | 40000 |
| Ground Floor Slab | 45.50 | 0.46 | 77.75 | 2 | 3243 | 486423 |
| | 50.00 | 0.46 | 20.00 | 1 | 458 | 68750 |
| Ground Floor Beams | | | | | | |
| B34 | 1.08 | 1.67 | 20.00 | 4 | 144 | 21667 |
| B35 | 1.08 | 1.67 | 20.00 | 2 | 72 | 10833 |
| B36 | 1.08 | 1.33 | 15.50 | 2 | 45 | 6717 |
| | | | | | 0 | 0 |
| total | | | | | 7764 | **1164586** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 20% Flat Roof Snow Load | 47.00 | 153.83 | 227 | | | 328242 |

| Component | Width (ft) | Height (ft) | Length (ft) | Quantity (ea) | Volume (cf) | Weight (lbs) |
|---|---|---|---|---|---|---|
| Roof | | | | | | |
| P.H. Roof Slab | 47.00 | 0.44 | 29.50 | 1 | 607 | 90989 |
| P.H. Roof Beam "B20" | 1.25 | 2.00 | 28.00 | 2 | 140 | 21000 |
| P.H. Walls | 0.67 | 17.00 | 47.00 | 2 | 1065 | 159800 |
| | 0.67 | 17.00 | 29.50 | 2 | 669 | 100300 |
| P.H. Columns - 18x18 | 1.50 | 17.00 | 1.50 | 8 | 306 | 45900 |
| machine room platform | 19.33 | 19.83 | 0.42 | 1 | 160 | 23957 |
| platform walls | 0.67 | 8.00 | 55.00 | 1 | 293 | 44000 |
| Roof Slab | 47.50 | 0.44 | 153.00 | 1 | 3180 | 476930 |
| | | | | | | |
| Roof Beams | | | | | | |
| B1 | 1.08 | 2.00 | 26.50 | 2 | 115 | 17225 |
| B2 | 1.08 | 2.00 | 26.50 | 2 | 115 | 17225 |
| B3 | 1.08 | 2.00 | 18.33 | 1 | 40 | 5957 |
| B4 | 1.08 | 2.00 | 25.83 | 1 | 56 | 8395 |
| B5 | 1.08 | 2.00 | 26.50 | 1 | 57 | 8613 |
| B6 | 1.08 | 2.00 | 26.50 | 3 | 172 | 25838 |
| B7 | 1.08 | 2.00 | 18.33 | 1 | 40 | 5957 |
| B8 | 0.79 | 1.17 | 8.83 | 1 | 8 | 1223 |
| B9 | 0.79 | 1.17 | 10.50 | 1 | 10 | 1455 |
| B10 | 0.63 | 1.00 | 15.33 | 2 | 19 | 2874 |
| B11 | 0.96 | 1.17 | 17.00 | 1 | 19 | 2851 |
| B12 | 0.79 | 1.00 | 11.00 | 1 | 9 | 1306 |
| B13 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| | | | | | | |
| Total | | | | | 7109 | **1394648** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 4th Floor Walls | 0.67 | 10.00 | 100.00 | 2 | 1333 | 200000 |
| | 0.67 | 10.00 | 47.50 | 3 | 950 | 142500 |
| | 0.67 | 10.00 | 46.83 | 1 | 312 | 46830 |
| 4th Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 18 | 405 | 60750 |
| 18x24 | 1.50 | 10.00 | 2.00 | 2 | 60 | 9000 |
| 4th Floor Slab | 47.00 | 0.35 | 53.75 | 2 | 1789 | 268414 |
| | 47.00 | 0.42 | 46.33 | 1 | 907 | 136094 |
| 4th Floor Beams | | | | | | |
| B1 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B2 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B3 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B4 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B5 | 1.08 | 1.83 | 26.50 | 1 | 53 | 7895 |
| B6 | 1.08 | 1.83 | 26.50 | 3 | 158 | 23684 |
| B7 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B8 | 0.79 | 1.17 | 8.33 | 1 | 8 | 1154 |
| B9 | 0.79 | 1.17 | 10.75 | 1 | 10 | 1489 |
| B10 | 0.63 | 1.00 | 13.50 | 1 | 8 | 1266 |
| B11 | 0.63 | 1.00 | 13.50 | 1 | 8 | 1266 |
| B12 | 0.96 | 1.17 | 15.67 | 1 | 18 | 2628 |
| B15 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B16 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| B17 | 0.96 | 1.75 | 8.75 | 1 | 15 | 2201 |
| B18 | 0.79 | 1.00 | 11.00 | 1 | 9 | 1306 |
| B19 | 0.67 | 1.67 | 6.00 | 1 | 7 | 1000 |
| | | | | | | |
| total | | | | | 6467 | **969980** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 3rd Floor Walls | 0.67 | 10.00 | 100.00 | 2 | 1333 | 200000 |
| | 0.67 | 10.00 | 47.50 | 3 | 950 | 142500 |
| | 0.67 | 10.00 | 46.83 | 1 | 312 | 46830 |
| 3rd Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 18 | 405 | 60750 |
| 18x24 | 1.50 | 10.00 | 2.00 | 2 | 60 | 9000 |
| 3rd Floor Slab | 47.00 | 0.35 | 53.75 | 2 | 1789 | 268414 |
| | 47.00 | 0.42 | 46.33 | 1 | 907 | 136094 |
| 3rd Floor Beams | | | | | | |
| B1 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B2 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B3 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B4 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B5 | 1.08 | 1.83 | 26.50 | 1 | 53 | 7895 |
| B6 | 1.08 | 1.83 | 26.50 | 3 | 158 | 23684 |
| B7 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B8 | 0.79 | 1.17 | 8.33 | 1 | 8 | 1154 |
| B9 | 0.79 | 1.17 | 10.75 | 1 | 10 | 1489 |
| B10 | 0.63 | 1.00 | 13.50 | 1 | 8 | 1266 |
| B11 | 0.63 | 1.00 | 13.50 | 1 | 8 | 1266 |
| B12 | 0.96 | 1.17 | 15.67 | 1 | 18 | 2628 |
| B15 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B16 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| B17 | 0.96 | 1.75 | 8.75 | 1 | 15 | 2201 |
| B18 | 0.79 | 1.00 | 11.00 | 1 | 9 | 1306 |
| B19 | 0.67 | 1.67 | 6.00 | 1 | 7 | 1000 |
| | | | | | | |
| total | | | | | 6467 | **969980** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 2nd Floor Walls | 0.67 | 10.00 | 100.00 | 2 | 1333 | 200000 |
| | 0.67 | 10.00 | 47.50 | 3 | 950 | 142500 |
| | 0.67 | 10.00 | 46.83 | 1 | 312 | 46830 |
| 2nd Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 18 | 405 | 60750 |
| 18x24 | 1.50 | 10.00 | 2.00 | 2 | 60 | 9000 |
| 2nd Floor Slab | 47.00 | 0.35 | 53.75 | 2 | 1789 | 268414 |
| | 47.00 | 0.42 | 46.33 | 1 | 907 | 136094 |
| 2nd Floor Beams | | | | | | |
| B1 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B2 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B3 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B4 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B5 | 1.08 | 1.83 | 26.50 | 1 | 53 | 7895 |
| B6 | 1.08 | 1.83 | 26.50 | 3 | 158 | 23684 |
| B7 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B8 | 0.79 | 1.17 | 8.33 | 1 | 8 | 1154 |
| B9 | 0.79 | 1.17 | 10.75 | 1 | 10 | 1489 |
| B10 | 0.63 | 1.00 | 13.50 | 1 | 8 | 1266 |
| B11 | 0.63 | 1.00 | 13.50 | 1 | 8 | 1266 |
| B12 | 0.96 | 1.17 | 15.67 | 1 | 18 | 2628 |
| B15 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B16 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| B17 | 0.96 | 1.75 | 8.75 | 1 | 15 | 2201 |
| B18 | 0.79 | 1.00 | 11.00 | 1 | 9 | 1306 |
| B19 | 0.67 | 1.67 | 6.00 | 1 | 7 | 1000 |
| | | | | | | |
| total | | | | | 6467 | **969980** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 1st Floor Walls | 0.67 | 10.00 | 53.00 | 3 | 1060 | 159000 |
| | 0.67 | 10.00 | 44.00 | 2 | 587 | 88000 |
| | 0.67 | 10.00 | 46.33 | 1 | 309 | 46330 |
| | 0.58 | 10.00 | 18.33 | 1 | 107 | 16039 |
| 1st Floor Columns | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 24 | 540 | 81000 |
| 20x20 | 1.67 | 10.00 | 1.67 | 1 | 28 | 4167 |
| 18x24 | 1.50 | 10.00 | 2.00 | 3 | 90 | 13500 |
| 1st Floor Slab | 47.00 | 0.35 | 53.75 | 4 | 3579 | 536828 |
| | 47.00 | 0.42 | 46.33 | 1 | 907 | 136094 |
| | 47.00 | 0.42 | 18.33 | 1 | 359 | 53844 |
| 1st Floor Beams | | | | | | |
| B5 | 1.08 | 1.83 | 26.50 | 1 | 53 | 7895 |
| B6 | 1.08 | 1.83 | 26.50 | 3 | 158 | 23684 |
| B9 | 1.08 | 1.83 | 10.75 | 1 | 21 | 3203 |
| B12 | 0.96 | 1.17 | 15.50 | 1 | 17 | 2599 |
| B18 | 0.67 | 1.00 | 5.42 | 1 | 4 | 542 |
| B19 | 1.08 | 1.83 | 26.50 | 2 | 105 | 15790 |
| B20 | 1.08 | 1.83 | 8.75 | 2 | 35 | 5214 |
| B21 | 1.08 | 1.83 | 9.58 | 2 | 38 | 5708 |
| B22 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B23 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B24 | 1.08 | 1.83 | 18.33 | 1 | 36 | 5461 |
| B25 | 0.79 | 1.17 | 8.75 | 1 | 8 | 1212 |
| B26 | 0.67 | 3.17 | 8.75 | 1 | 18 | 2771 |
| B27 | 1.08 | 1.83 | 25.83 | 1 | 51 | 7695 |
| B43 | 0.96 | 4.00 | 26.50 | 2 | 203 | 30475 |
| total | | | | | 8401 | **1260207** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| Ground Floor Walls | 0.67 | 12.00 | 100.00 | 2 | 1600 | 240000 |
| | 0.67 | 12.00 | 47.50 | 3 | 1140 | 171000 |
| | 0.67 | 12.00 | 46.83 | 1 | 375 | 56196 |
| Ground Floor Columns | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 13 | 351 | 52650 |
| 20x20 | 1.67 | 12.00 | 1.67 | 11 | 367 | 55000 |
| 22x22 | 1.83 | 12.00 | 1.83 | 1 | 40 | 6050 |
| 18x24 | 1.50 | 12.00 | 2.00 | 3 | 108 | 16200 |
| | | | | | | |
| Ground Floor Slab | 18.33 | 0.42 | 44.00 | 1 | 336 | 50408 |
| | 26.50 | 0.44 | 30.50 | 1 | 354 | 53041 |
| | 24.25 | 0.44 | 28.00 | 1 | 297 | 44559 |
| | 53.00 | 0.50 | 15.00 | 1 | 398 | 59625 |
| | | | | | | |
| Ground Floor Beams | | | | | | |
| B1 | 1.42 | 1.88 | 26.50 | 1 | 70 | 10559 |
| B2 | 1.42 | 1.88 | 8.75 | 1 | 23 | 3486 |
| B3 | 0.79 | 1.17 | 9.58 | 1 | 9 | 1327 |
| B4 | 1.50 | 1.71 | 18.33 | 1 | 47 | 7046 |
| B5 | 1.08 | 1.79 | 15.00 | 1 | 29 | 4367 |
| B6 | 1.08 | 1.42 | 15.50 | 1 | 24 | 3568 |
| B7 | 1.08 | 1.42 | 15.00 | 1 | 23 | 3453 |
| B8 | 0.79 | 1.00 | 18.33 | 1 | 15 | 2177 |
| B9 | 0.79 | 1.00 | 8.75 | 1 | 7 | 1039 |
| B10 | 0.67 | 1.00 | 5.42 | 1 | 4 | 542 |
| B11 | 1.08 | 1.42 | 15.00 | 1 | 23 | 3453 |
| B12 | 0.96 | 1.17 | 15.50 | 1 | 17 | 2599 |
| B13 | 0.96 | 1.54 | 15.00 | 1 | 22 | 3324 |
| B14 | 1.42 | 1.75 | 26.50 | 1 | 66 | 9855 |
| B15 | 1.42 | 1.75 | 8.75 | 1 | 22 | 3254 |
| B16 | 0.79 | 1.17 | 9.58 | 1 | 9 | 1327 |
| B17 | 0.96 | 1.75 | 26.50 | 1 | 44 | 6666 |
| B18 | 0.96 | 1.75 | 26.50 | 1 | 44 | 6666 |
| B19 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| B20 | 0.96 | 1.75 | 28.00 | 1 | 47 | 7044 |
| B21 | 0.96 | 1.75 | 28.00 | 1 | 47 | 7044 |
| | | | | | | |
| total | | | | | 5988 | **898137** |

Buckner Building - Seismic Dead Weight

**C, E, G WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| Basement Floor Walls | 0.67 | 12.00 | 53.00 | 3 | 1272 | 190800 |
| | 0.67 | 12.00 | 45.50 | 2 | 728 | 109200 |
| | 0.67 | 12.00 | 46.33 | 1 | 371 | 55596 |
| | 0.50 | 12.00 | 27.58 | 1 | 165 | 24822 |
| | 0.50 | 12.00 | 15.00 | 1 | 90 | 13500 |
| | 0.50 | 12.00 | 26.50 | 1 | 159 | 23850 |
| | 0.58 | 12.00 | 8.17 | 3 | 172 | 25736 |
| | 0.58 | 12.00 | 24.29 | 2 | 340 | 51009 |
| | | | | | | |
| Basement Floor Columns | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 13 | 351 | 52650 |
| 20x20 | 1.67 | 12.00 | 1.67 | 11 | 367 | 55000 |
| 22x22 | 1.83 | 12.00 | 1.83 | 1 | 40 | 6050 |
| 18x24 | 1.50 | 12.00 | 2.00 | 3 | 108 | 16200 |
| Basement Floor Slab | 15.83 | 0.33 | 10.00 | 1 | 53 | 7915 |
| | 14.25 | 0.33 | 10.00 | 1 | 48 | 7125 |
| | 21.92 | 0.29 | 10.00 | 1 | 64 | 9590 |
| Basement Floor Beams | | | | | | |
| B7 | 0.79 | 1.17 | 26.50 | 1 | 24 | 3671 |
| B8 | 1.50 | 3.92 | 26.50 | 1 | 156 | 23353 |
| | | | | | | |
| total | | | | | 4507 | **676067** |

Buckner Building - Seismic Dead Weight

**D, F WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| 20% Flat Roof Snow Load | 44.17 | 80 | 227 | | | 160425 |

Roof

| Component | Width (ft) | Height (ft) | Length (ft) | Quantity (ea) | Volume (cf) | Weight (lbs) |
|---|---|---|---|---|---|---|
| **Roof Walls** | 0.67 | 1.42 | 80.00 | 2 | 151 | 22667 |
| | 0.67 | 1.42 | 44.17 | 2 | 83 | 12515 |
| | | | | | | |
| **Roof Slab** | 44.17 | 0.33 | 80.00 | 1 | 1178 | 176680 |
| | | | | | | |
| **Roof Beams** | | | | | | |
| B14 | 1.08 | 2.00 | 25.83 | 2 | 112 | 16790 |
| B15 | 1.08 | 1.67 | 18.33 | 7 | 232 | 34751 |
| B16 | 1.08 | 1.67 | 25.83 | 7 | 326 | 48969 |
| SB2 | 0.67 | 1.75 | 20.00 | 2 | 47 | 7000 |
| SB3 | 0.67 | 1.75 | 20.00 | 2 | 47 | 7000 |
| | | | | | | |
| Total | | | | | 2176 | **486796** |

| | | | | | | |
|---|---|---|---|---|---|---|
| **4th Floor Walls** | 0.67 | 10.00 | 80.00 | 2 | 1067 | 160000 |
| | 0.58 | 10.00 | 18.33 | 2 | 214 | 32078 |
| | | | | | | |
| **4th Floor Columns** | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 15 | 338 | 50625 |
| | | | | | | |
| **4th Floor Slab** | 44.17 | 0.29 | 80.00 | 1 | 1031 | 154595 |
| | | | | | | |
| **4th Floor Beams** | | | | | | |
| B13 | 1.08 | 1.75 | 20.00 | 2 | 76 | 11375 |
| B14 | 1.08 | 1.75 | 20.00 | 2 | 76 | 11375 |
| B20 | 0.96 | 1.67 | 18.33 | 7 | 205 | 30741 |
| B21 | 0.96 | 1.67 | 25.83 | 7 | 289 | 43319 |
| B22 | 1.08 | 1.83 | 25.83 | 2 | 103 | 15390 |
| SB3 | 0.67 | 3.92 | 20.00 | 2 | 104 | 15667 |
| SB2 | 0.67 | 3.92 | 20.00 | 2 | 104 | 15667 |
| | | | | | | |
| total | | | | | 3606 | **540831** |

Buckner Building - Seismic Dead Weight

**D, F WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| **3rd Floor Walls** | 0.67 | 10.00 | 80.00 | 2 | 1067 | 160000 |
| | 0.58 | 10.00 | 18.33 | 2 | 214 | 32078 |
| | | | | | | |
| **3rd Floor Columns** | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 15 | 338 | 50625 |
| | | | | | | |
| **3rd Floor Slab** | 44.17 | 0.29 | 80.00 | 1 | 1031 | 154595 |
| | | | | | | |
| **3rd Floor Beams** | | | | | | |
| B13 | 1.08 | 1.75 | 20.00 | 2 | 76 | 11375 |
| B14 | 1.08 | 1.75 | 20.00 | 2 | 76 | 11375 |
| B20 | 0.96 | 1.67 | 18.33 | 7 | 205 | 30741 |
| B21 | 0.96 | 1.67 | 25.83 | 7 | 289 | 43319 |
| B22 | 1.08 | 1.83 | 25.83 | 2 | 103 | 15390 |
| SB3 | 0.67 | 3.92 | 20.00 | 2 | 104 | 15667 |
| SB2 | 0.67 | 3.92 | 20.00 | 2 | 104 | 15667 |
| | | | | | | |
| total | | | | | 3606 | **540831** |
| **2nd Floor Walls** | 0.58 | 10.00 | 18.33 | 2 | 214 | 32078 |
| | 0.67 | 10.00 | 80.00 | 2 | 1067 | 160000 |
| | | | | | | |
| **2nd Floor Columns** | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 15 | 338 | 50625 |
| | | | | | | |
| **2nd Floor Slab** | 44.17 | 0.29 | 80.00 | 1 | 1031 | 154595 |
| | | | | | | |
| **2nd Floor Beams** | | | | | | |
| B13 | 1.08 | 1.75 | 20.00 | 2 | 76 | 11375 |
| B14 | 1.08 | 1.75 | 20.00 | 2 | 76 | 11375 |
| B20 | 0.96 | 1.67 | 18.33 | 7 | 205 | 30741 |
| B21 | 0.96 | 1.67 | 25.83 | 7 | 289 | 43319 |
| B22 | 1.08 | 1.83 | 25.83 | 2 | 103 | 15390 |
| SB3 | 0.67 | 3.92 | 20.00 | 2 | 104 | 15667 |
| SB2 | 0.67 | 3.92 | 20.00 | 2 | 104 | 15667 |
| | | | | | | |
| total | | | | | 3606 | **540831** |

Buckner Building - Seismic Dead Weight

**D, F WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| **1st Floor Walls** | 0.58 | 10.00 | 18.33 | 2 | 214 | 32078 |
| | 0.67 | 10.00 | 80.00 | 2 | 1067 | 160000 |
| | 1.50 | 10.00 | 80.00 | 1 | 1200 | 180000 |
| **1st Floor Columns** | | | | | | |
| 18x18 | 1.50 | 10.00 | 1.50 | 15 | 338 | 50625 |
| | | | | | | |
| **1st Floor Slab** | 98.17 | 0.33 | 80.00 | 1 | 2618 | 392680 |
| | | | | | | |
| **1st Floor Beams** | | | | | | |
| SB1 | 0.67 | 3.33 | 20.00 | 2 | 89 | 13333 |
| SB2 | 0.67 | 3.33 | 20.00 | 2 | 89 | 13333 |
| B13 | 1.25 | 1.92 | 20.00 | 2 | 96 | 14375 |
| B14 | 1.25 | 1.92 | 20.00 | 2 | 96 | 14375 |
| B28 | 0.96 | 1.83 | 26.50 | 4 | 186 | 27935 |
| B29 | 0.96 | 1.83 | 25.83 | 2 | 91 | 13615 |
| B30 | 0.96 | 1.83 | 26.50 | 4 | 186 | 27935 |
| B31 | 0.96 | 1.83 | 26.50 | 4 | 186 | 27935 |
| B32 | 0.96 | 1.83 | 18.33 | 4 | 129 | 19323 |
| B33 | 0.96 | 1.83 | 25.83 | 4 | 182 | 27229 |
| B34 | 1.42 | 1.92 | 20.00 | 2 | 109 | 16292 |
| B35 | 1.42 | 1.92 | 20.00 | 2 | 109 | 16292 |
| B36 | 1.42 | 1.92 | 20.00 | 2 | 109 | 16292 |
| B37 | 1.42 | 1.92 | 20.00 | 2 | 109 | 16292 |
| B39 | 0.96 | 1.83 | 26.50 | 3 | 140 | 20952 |
| B40 | 0.96 | 1.83 | 26.50 | 3 | 140 | 20952 |
| B41 | 0.96 | 1.83 | 18.33 | 3 | 97 | 14492 |
| B42 | 0.96 | 1.83 | 25.83 | 3 | 136 | 20422 |
| | | | | | | |
| total | | | | | 7712 | **1156756** |

Buckner Building - Seismic Dead Weight

**D, F WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| **Ground Floor Walls** | 0.67 | 12.00 | 80.00 | 2 | 1280 | 192000 |
| | 0.50 | 12.00 | 80.00 | 1 | 480 | 72000 |
| | 0.58 | 12.00 | 18.33 | 2 | 257 | 38493 |
| **Ground Floor Columns** | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 10 | 270 | 40500 |
| 20x20 | 1.67 | 12.00 | 1.67 | 15 | 500 | 75000 |
| | | | | | | |
| **Ground Floor Slab** | 98.17 | 0.33 | 80.00 | 1 | 2618 | 392680 |
| | | | | | | |
| **Ground Floor Beams** | | | | | | |
| B19 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| B22 | 1.42 | 1.75 | 26.50 | 1 | 66 | 9855 |
| B23 | 1.42 | 1.75 | 26.50 | 1 | 66 | 9855 |
| B24 | 0.96 | 1.75 | 25.83 | 1 | 43 | 6498 |
| B25 | 1.08 | 1.75 | 26.50 | 2 | 100 | 15072 |
| B26 | 1.25 | 1.75 | 25.83 | 2 | 113 | 16951 |
| B27 | 1.08 | 1.75 | 26.50 | 1 | 50 | 7536 |
| B28 | 1.08 | 1.75 | 26.50 | 1 | 50 | 7536 |
| B29 | 1.25 | 1.75 | 18.33 | 1 | 40 | 6015 |
| B30 | 1.25 | 1.75 | 25.83 | 1 | 57 | 8475 |
| B31 | 1.08 | 1.75 | 26.50 | 2 | 100 | 15072 |
| B32 | 1.08 | 1.75 | 26.50 | 2 | 100 | 15072 |
| B33 | 1.08 | 1.75 | 26.50 | 2 | 100 | 15072 |
| B34 | 1.25 | 1.75 | 18.33 | 2 | 80 | 12029 |
| B35 | 1.25 | 1.75 | 25.83 | 2 | 113 | 16951 |
| B36 | 1.08 | 1.75 | 26.50 | 4 | 201 | 30144 |
| B37 | 1.25 | 1.75 | 25.83 | 2 | 113 | 16951 |
| B38 | 0.96 | 1.75 | 26.50 | 1 | 44 | 6666 |
| B39 | 0.96 | 1.75 | 26.50 | 1 | 44 | 6666 |
| B40 | 0.96 | 1.75 | 18.33 | 1 | 31 | 4611 |
| B41 | 0.96 | 1.75 | 25.83 | 1 | 43 | 6498 |
| B43 | 1.25 | 1.71 | 20.00 | 1 | 43 | 6406 |
| B44 | 1.25 | 1.83 | 20.00 | 1 | 46 | 6875 |
| B45 | 1.25 | 1.83 | 20.00 | 1 | 46 | 6875 |
| B46 | 1.25 | 1.83 | 20.00 | 1 | 46 | 6875 |
| B47 | 1.25 | 1.83 | 20.00 | 1 | 46 | 6875 |
| B48 | 1.25 | 1.96 | 20.00 | 1 | 49 | 7344 |
| B49 | 1.25 | 1.96 | 20.00 | 1 | 49 | 7344 |
| B50 | 1.25 | 1.96 | 20.00 | 1 | 49 | 7344 |
| B51 | 1.42 | 1.83 | 20.00 | 1 | 52 | 7792 |
| B52 | 1.42 | 1.96 | 20.00 | 1 | 55 | 8323 |
| B53 | 1.42 | 1.96 | 20.00 | 1 | 55 | 8323 |
| B54 | 1.42 | 1.96 | 20.00 | 1 | 55 | 8323 |
| | | | | | | |
| total | | | | | 2179 | **1137506** |

Buckner Building - Seismic Dead Weight

**D, F WINGS**

| | Width (ft) | Length (ft) | Pf (psf) | | | Weight (lbs) |
|---|---|---|---|---|---|---|
| **Basement Floor Walls** | 0.67 | 12.00 | 80.00 | 2 | 1280 | 192000 |
| | 0.50 | 12.00 | 70.00 | 1 | 420 | 63000 |
| | | | | | | |
| **Basement Floor Columns** | | | | | | |
| 18x18 | 1.50 | 12.00 | 1.50 | 10 | 270 | 40500 |
| 20x20 | 1.67 | 12.00 | 1.67 | 12 | 400 | 60000 |
| 22x22 | 1.83 | 12.00 | 1.83 | 3 | 121 | 18150 |
| | | | | | | |
| **Basement Floor Slab** | 98.17 | 0.33 | 80.00 | 1 | 2618 | 392680 |
| | | | | | | |
| total | | | | | 5109 | **766330** |

# ASCE41-13 - 16.10LS - Life Safety Checklist for Building Types C2: CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS

**Building Unit "A":**

| | | |
|---|---|---|
| W | 11717 | Building Seismic Weight |
| $C*S_a$ | 0.759 | |
| V | 8893 | Total Building Shear |
| k | 1 | |
| $M_s$ | 4 | Table 4-9 |

| Level | W (kips) | h (ft) | $W_x*h_x$ (k*ft) | $F_x$ (kips) | $V_j$ (kips) | $A_{wEast\text{-}West}$ (in$^2$) | $v_{jEast\text{-}West}^{avg}$ (ksi) | Quick Check (C, NC) | $A_{wNorth\text{-}South}$ (in$^2$) | $v_{jNorth\text{-}South}^{avg}$ (ksi) | Quick Check (C, NC) | Areq'd (in$^2$) | Lreq'd (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Basement | 680 | 0 | 0 | 0 | 8893 | | | | | | | 22233.36 | 231.5975 |
| Ground | 2691 | 12 | 32296 | 729 | 8893 | 18128 | 0.123 | NC | 10581 | 0.210 | NC | 22233.36 | 231.5975 |
| 1st | 1814 | 24 | 43545 | 983 | 8165 | 13696 | 0.149 | NC | 11546 | 0.177 | NC | 20411.6 | 212.6208 |
| 2nd | 1792 | 34 | 60925 | 1375 | 7182 | 15600 | 0.115 | NC | 11298 | 0.159 | NC | 17955.28 | 187.0342 |
| 3rd | 1530 | 44 | 67320 | 1519 | 5807 | 15888 | 0.091 | C | 19490 | 0.074 | C | 14518.53 | 151.2347 |
| 4th | 1535 | 54 | 82888 | 1870 | 4288 | 15888 | 0.067 | C | 19490 | 0.055 | C | 10721.07 | 111.6778 |
| Roof | 1675 | 64 | 107171 | 2418 | 2418 | 15888 | 0.038 | C | 19490 | 0.031 | C | 6045.436 | 62.9733 |
| ∑ | 11717 | | 394145 | | | | | | | | | | |

**Building Unit "B":**

| | | |
|---|---|---|
| W | 3594 | Building Seismic Weight |
| $C*S_a$ | 0.759 | |
| V | 2728 | Total Building Shear |
| k | 1 | |
| $M_s$ | 4 | Table 4-9 |

| Level | W (kips) | h (ft) | $W_x*h_x$ (k*ft) | $F_x$ (kips) | $V_j$ (kips) | $A_{wEast\text{-}West}$ (in$^2$) | $v_{jEast\text{-}West}^{avg}$ (ksi) | Quick Check (C, NC) | $A_{wNorth\text{-}South}$ (in$^2$) | $v_{jNorth\text{-}South}^{avg}$ (ksi) | Quick Check (C, NC) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Basement | 680 | 0 | 0 | 0 | 2728 | | | | | | |
| Ground | 1165 | 12 | 13975 | 581 | 2728 | 21888 | 0 | C | 17856 | 0.038 | C |
| 1st | 789 | 24 | 18934 | 788 | 2146 | 13120 | 0 | C | 11928 | 0.045 | C |
| Roof | 960 | 34 | 32645 | 1358 | 1358 | 12128 | 0 | C | 11440 | 0.030 | C |
| ∑ | 3594 | | 65554 | | | | | | | | |

## ASCE41-13 - 16.10LS - Life Safety Checklist for Building Types C2: CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS

**Building Unit "C, E, G":**

| | | |
|---|---|---|
| W | 7139 | Building Seismic Weight |
| $C*S_a$ | 0.759 | |
| V | 5419 | Total Building Shear |
| k | 1 | |
| $M_s$ | 4 | Table 4-9 |

| Level | W (kips) | h (ft) | $W_x*h_x$ (k*ft) | $F_x$ (kips) | $V_j$ (kips) | $A_{wEast\text{-}West}$ ($in^2$) | $v_{jEast\text{-}West}^{avg}$ (ksi) | Quick Check (C, NC) | $A_{wNorth\text{-}South}$ ($in^2$) | $v_{jNorth\text{-}South}^{avg}$ (ksi) | Quick Check (C, NC) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Basement | 676 | 0 | 0 | 0 | 5419 | | | | | | |
| Ground | 898 | 12 | 10778 | 226 | 5419 | 10147 | 0.133 | NC | 12656 | 0.107 | NC |
| 1st | 1260 | 24 | 30245 | 634 | 5192 | 9696 | 0.134 | NC | 9122 | 0.142 | NC |
| 2nd | 970 | 34 | 32979 | 692 | 4558 | 12192 | 0.093 | C | 8436 | 0.135 | NC |
| 3rd | 970 | 44 | 42679 | 895 | 3866 | 12192 | 0.079 | C | 8436 | 0.115 | NC |
| 4th | 970 | 54 | 52379 | 1099 | 2971 | 12192 | 0.061 | C | 8436 | 0.088 | C |
| Roof | 1395 | 64 | 89257 | 1872 | 1872 | 12192 | 0.038 | C | 8436 | 0.055 | C |
| ∑ | 7139 | | 258317 | | | | | | | | |

**Building Unit "D, F":**

| | | |
|---|---|---|
| W | 5170 | Building Seismic Weight |
| $C*S_a$ | 0.759 | |
| V | 3924 | Total Building Shear |
| k | 1 | |
| $M_s$ | 4 | Table 4-9 |

| Level | W (kips) | h (ft) | $W_x*h_x$ (k*ft) | $F_x$ (kips) | $V_j$ (kips) | $A_{wEast\text{-}West}$ ($in^2$) | $v_{jEast\text{-}West}^{avg}$ (ksi) | Quick Check (C, NC) | $A_{wNorth\text{-}South}$ ($in^2$) | $v_{jNorth\text{-}South}^{avg}$ (ksi) | Quick Check (C, NC) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Basement | 766 | 0 | 0 | 0 | 3924 | | | | | | |
| Ground | 1138 | 12 | 13650 | 372 | 3924 | 13692 | 0.072 | C | 1 | 980.985 | NC |
| 1st | 1157 | 24 | 27762 | 757 | 3552 | 3360 | 0.264 | NC | 1 | 887.967 | NC |
| 2nd | 541 | 34 | 18388 | 501 | 2795 | 3360 | 0.208 | NC | 3079 | 0.227 | NC |
| 3rd | 541 | 44 | 23797 | 649 | 2294 | 3360 | 0.171 | NC | 3079 | 0.186 | NC |
| 4th | 541 | 54 | 29205 | 796 | 1645 | 3360 | 0.122 | NC | 3079 | 0.134 | NC |
| Roof | 487 | 64 | 31155 | 849 | 849 | 3360 | 0.063 | C | 3079 | 0.069 | C |
| ∑ | 5170 | | 143957 | | | | | | | | |

All 7" & 8" walls are reinforced with 5/8" $\phi$ bar @ 15" o.c. each way per sheet 63.

| **Thickness** | **Unit Length** | **Area** | **$A_S$ Horiz** | **$A_S$ Vert** | **Horiz Reinf.** | **Criteria** | **Quick Check** | **Vert Reinf** | **Criteria** | **Quick Check** |
|---|---|---|---|---|---|---|---|---|---|---|
| **(in)** | **0.759** | **($in^2$)** | **($in^2$)** | **($in^2$)** | **Ratio** | **(min)** | **(C, NC)** | **Ratio** | **(min)** | **(C, NC)** |
| 7 | 15 | 105 | 0.31 | 0.31 | 0.0030 | 0.0020 | C | 0.0030 | 0.0012 | C |
| 8 | 15 | 120 | 0.31 | 0.31 | 0.0026 | 0.0020 | C | 0.0026 | 0.0012 | C |

Project: Buckner Building Location: Whittier, AK

Completed by: Peter Hewko Date:

## 16.17 NONSTRUCTURAL CHECKLIST

### Life Safety Systems

C NC N/A U LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)

C NC N/A U LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)

C NC N/A U LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4)

C NC N/A U LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)

### Hazardous Materials

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)

C NC N/A U LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)

### Partitions

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)

**Ceilings**

C NC N/A U LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 $ft^2$ of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 $ft^2$ of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 $ft^2$, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 $ft^2$ have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 $ft^2$ are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 $ft^2$ and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)

**Light Fixtures**

C NC N/A U LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)

C NC N/A U LS-not required; PR-H. PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft and, if rigidly supported, are free to move with the structure to which they are attached without damaging adjoining components. (Commentary: A.7.3.3. Tier 2: Sec. 13.7.9)

C NC N/A U LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)

**Cladding and Glazing**

C NC N/A U LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 $lb/ft^2$ are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. CLADDING ISOLATION: For steel or concrete moment frame buildings, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02. (Commentary: Sec. A.7.4.3. Tier 2: Section 13.6.1)

C NC N/A U LS-MH; PR-MH. MULTI-STORY PANELS: For multi-story panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicty, 0.02. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 $ft^2$ in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)

**Masonry Veneer**

C NC N/A U LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 $ft^2$, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. STUD TRACKS: For veneer with metal stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)

C NC N/A U LS-not required; PR-MH. OPENINGS: For veneer with metal stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)

**Parapets, Cornices, Ornamentation, and Appendages**

C NC N/A U LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)

C NC N/A U LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)

C NC N/A U LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)

C NC N/A U LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)

## Masonry Chimneys

C NC N/A U LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)

C NC N/A U LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)

## Stairs

C NC N/A U LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)

C NC N/A U LS-LMH; PR-LMH. STAIR DETAILS: In moment frame structures, the connection between the stairs and the structure does not rely on shallow anchors in concrete. Alternatively, the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.5.3.1 without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: 13.6.8)

## Contents and Furnishings

C NC N/A U LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)

C NC N/A U LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)

C NC N/A U LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)

C NC N/A U LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)

C NC N/A U LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)

C NC N/A U LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)

## Mechanical and Electrical Equipment

C NC N/A U LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)

C NC N/A U LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)

C NC N/A U LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)

C NC N/A U LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)

C NC N/A U LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)

**Piping**

C NC N/A U LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)

**Ducts**

C NC N/A U LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 $ft^2$ in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)

**Elevators**

C NC N/A U LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)

C NC N/A U LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)

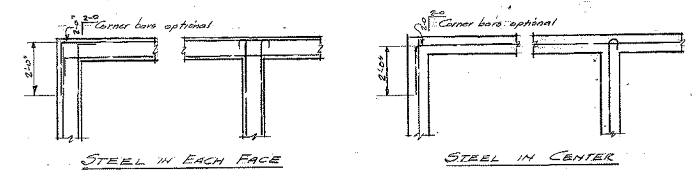
## GROUND FLOOR BEAM SCHEDULE FOR UNITS D, E, F & G

| CONCRETE MARK | W | D | STR. BARS No | STR. BARS SIZE | STR. BARS TYPE | BENT BARS No | BENT BARS SIZE | BENT BARS TYPE | STIRRUPS No | STIRRUPS SIZE | STIRRUPS SPACING EA. END | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 17 | 21 / 22½ | | | | 2 / 2 | 1⅛□ / 1¼□ | K / J | 7 / 11 | 3/8φ | 4,3@8,3@9 ext. / 2,4@5,3@8,3@9 int. | |
| B2 | 17 | 21 / 22½ | 2 | 1□ | A | | | | | 3/8φ□ | 6,@12" thruout | 2-1φ top - extend to ¼ pt of B1 and hook @ wall |
| B3 | 9½ | 14 | 1 | 1□ | B | 1 | 7/8φ | D | 8 | 3/8φ | 3,3@6 | |
| B4 | 18 | 16 / 20½ | 3 | 1¼□ | B | 2 | 1¼□ | D | 16 / 5 | 3/8φ | 1,3@2,4@3,3@4,3@6 Line C / 4,4@8 Line D | |
| B5 | 13 | 17 / 21½ | 1 | 1φ | C | 1 / 1 | 1□ / 1⅛□ | J / E | 3 / 18 | 3/8φ | 4,2@8 ext / 2,8@3,9@4 int | |
| B6 | 13 | 17 | 1 | 7/8φ | C | 1 | 1φ | F | 6 | 3/8φ | 3,2@7 | |
| B7 | 13 | 17 | 1 | 1φ | C | 1 | 1φ | E | 6 | 3/8φ | 3,2@7 | |
| B8 | 9½ | 12 | 1 | 7/8φ | B | 1 | 3/4φ | D | 6 | 3/8φ | 2,2@5 | |
| B9 | 9½ | 12 | 1 | 3/4φ | B | 1 | 3/4φ | D | 6 | 3/8φ | 2,2@5 | |
| B10 | 8 | 12 | 1 / 1 | 3/4φ / 3/4φ | C top 10 / B | | | | | 3/8φ□ | 6,@12" thruout | |
| B11 | 13 | 17 | 2 | 1¼□ | B | 2 | 1¼□ | D | 10 / 18 | 3/8φ | 3,9@7 ext / 2,6@3,11@4 int | |
| B12 | 11½ | 12 / 14 | 1 | 1φ | C | 1 | 1φ | E | 6 | 3/8φ | 3,2@5 | |
| B13 | 11½ | 14 / 18½ | 1 | 1φ | C | 1 | 1φ | E | 3 / 7 | 3/8φ | 3,2@7 ext / 3,6@5 int | |
| B14 | 17 | 21 | 2 | 1¼□ | C | 2 | 1¼□ | E | 8 / 14 | 3/8φ | 3,3@6,4@8 ext / 2,6@4,3@6,4@8 int | |
| B15 | 17 | 21 | 2 | 1φ | A | | | | | 3/8φ□ | 6,@12" thruout | 2-1□ top - extend to ¼ pt of B14 and hook @ wall. |
| B16 | 9½ | 14 | 1 | 1□ | B | 1 | 7/8φ | D | 8 | 3/8φ | 3,3@6 | |
| B17 | 11½ | 21 | 1 / 1 | 3/4φ / 1φ | N / C | 1 / 1 | 1φ / 1φ | J / E | 4 / 8 | 3/8φ | 4,3@8 ext / 4,7@8 int | |
| B18 | 11½ | 21 | 1 | 1φ | C | 2 | 1φ | E | 8 | 3/8φ | 4,3@8 | |
| B19 | 11½ | 21 | 2 | 1□ | C | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛□ cont. in top. Extend to ¼ pts of B18 & B20 only. |
| B20 | 11½ | 21 | 2 | 1φ | C | 2 | 1φ | E | | 3/8φ | 4,4@8 ext / 4,7@8 int | |
| B21 | 11½ | 21 | 4 | 1¼□ | B | 2 | 1□ | D | 18 | 3/8φ | 3,2@6,6@8 | Deck 2-1□ bars @ midspan. |
| B22 | 17 | 21 | 2 / 2 | 1¼□ / 1⅛□ | L @ B5 / C | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛□ cont. in top. |
| B23 | 17 | 18 / 21 | 2 / 2 | 1□ / 1⅛□ | L @ C5 / C | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛□ cont in top. |
| B24 | 11½ | 21 | 2 / 2 | 1⅛□ / 1¼□ | L @ D5 / C | | | | | 3/8φ□ | 4,2@8, Rem. @12" ext / 4,4@8 int | 2-1⅛□ cont. in top. Deck 1-1⅛□ 4" bar. |
| B25 | 13 | 21 | 2 / 2 | 1⅛□ / 1□ | L / C | 1 | 1¼□ | E | 9 / 11 | 3/8φ | 3,2@6,6@8 ext. / 2,2@5,2@6,6@8 int. | |
| B26 | 15 | 19½ / 21 | 2 | 1⅛□ | B | 2 | 1⅛□ | D | 12 | 3/8φ | 4,5@8 | |
| B27 | 13 | 21 | 1 / 2 | 7/8φ / 1□ | L @ B6 / C | 1 | 1¼□ | J | | 3/8φ□ | 3,2@6,6@8, Rem@12 ext / 2,2@5,2@6,6@8 int | 2-1⅛□ cont. in top. |
| B28 | 13 | 21 | 1 / 2 | 7/8φ / 1□ | L @ C6 / H | 1 | 1¼□ | F | | 3/8φ□ | 3,2@6,6@8, Rem. @12 | 2-1⅛□ cont. in top. |
| B29 | 15 | 21 | 2 | 1φ | H | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛□ cont. in top. |
| B30 | 15 | 21 | 1 / 2 | 1⅛□ / 1⅛□ | L @ D6 / C | 1 | 1¼□ | J | | 3/8φ□ | 4,4@8 Rem @12 ext / 4,6@8 int | 2-1⅛□ cont. in top. |
| B31 | 13 | 21 | 1 | 1□ | C | 1 / 1 | 1□ / 1⅛□ | J / E | | 3/8φ | 4,5@8 ext / 4,8@8 int | |
| B32 | 13 | 21 | 1 / 2 | 1□ / 1□ | L @ B / C | 1 | 1□ | J | | 3/8φ□ | 4,5@8 Rem@12 ext / 3,3@6,6@8 int | 2-1⅛□ cont in top |
| B33 | 13 | 21 | 1 / 2 | 1□ / 1□ | L @ C / H | 1 | 1□ | F | | 3/8φ□ | 4,5@8 Rem@12 ext / int | 2-1⅛□ cont. in top |
| B34 | 15 | 21 | 2 | 1φ | H | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛□ cont. in top |
| B35 | 15 | 21 | 1 / 2 | 7/8φ / 1⅛□ | L @ D / C | 1 | 1¼□ | J | | 3/8φ□ | 4,4@8, Rem@12 ext / 4,6@8 int | 2-1⅛□ cont. in top |
| B36 | 13 | 21 | 1 / 1 | 1φ / 1□ | C / M | 1 / 1 | 1□ / 1⅛□ | J / E | 6 / 10 | 3/8φ | 4,5@8 ext / 3,3@6,6@8 int | |
| B37 | 15 | 21 | 2 | 1⅛□ | B | 2 | 1⅛□ | D | 10 | 3/8φ | 4,4@8 | |
| B38 | 11½ | 21 | 1 | 1⅛□ | C | 1 | 1□ | J | | 3/8φ□ | 4,2@8 Rem @12" ext / 4,3@8 int | 2-1⅛□ cont. in top |
| B39 | 11½ | 21 | 1 | 1□ | H | 1 | 1φ | F | | 3/8φ□ | 4,2@8 Rem @12" | 2-1⅛□ cont. in top |
| B40 | 11½ | 21 | 2 | 7/8φ | H | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛□ cont. in top |
| B41 | 11½ | 21 | 1 | 1⅛□ | C | 1 | 1□ | J | | 3/8φ | 4,2@8 Rem @12" ext / 4,3@8 int | |
| B42 | 6 | 14 | 1 | 1□ | C | | | | 10 | 3/8φ□ | 2,4@5 | 1-3/4φ → top |
| B43 | 15 | 20½ | 1 | 1□ | C | 1 / 1 | 1¼□ / 1⅛□ | J / E | 6 / 11 | 3/8φ | 4,5@8 ext / 4,10@8 int | |
| B44 | 15 | 20½ / 22 | 1 | 1□ | H | 2 | 1□ | F | 6 / 4 | 3/8φ | 4,5@8 Line 6 / 4,3@8 Line 7 | |
| B45 | 15 | 22 | 1 | 1φ | H | 2 | 1□ | F | 6 | 3/8φ | 5,2@9 | |
| B46 | 15 | 22 | 1 | 1□ | C | 1 / 1 | 1□ / 1¼□ | J / E | 3 / 8 | 3/8φ | 5,2@9 ext / 5,7@9 int | |
| B47 | 15 | 22 | 1 / 1 | 1φ / 1⅛□ | L @ C6 / C | 1 / 1 | 1φ / 1¼□ | J / E | 5 / 10 | 3/8φ | 5,4@9 ext / 5,9@9 int | |
| B48 | 15 | 22 / 23½ | 1 | 1□ | H | 2 | 1□ | F | 6 | 3/8φ | 5,2@9 | |
| B49 | 15 | 23½ | 1 | 1□ | [illegible] | | | | | 3/8φ | 5,3@10 | |
| B50 | 15 | 23½ | 2 | 1¼□ | L @ C6 | 1 / 1 | 1¼□ | E | 8 | 3/8φ | 5,1@9 ext / 5,11@9 int | 2-1¼ M @ Col. B6. Deck 1-1⅛ L bar @ B6 |
| B51 | 17 | 22 | 1 | 1□ | L @ B7 | 1 | 1¼□ | F | | 3/8φ□ | 5,@10" thruout | |
| B52 | 17 | 23½ | 2 | 1□ | H | | | | | 3/8φ□ | 5,@10" thruout | |
| B53 | 17 | 23½ | 2 | 1⅛□ | [illegible] | 1 | 1□ | F | | 3/8φ | 5,10@10 ext / 4,10@8 int | |
| B54 | 17 | 23½ | [illegible] | [illegible] | [illegible] | 1 | 1⅛□ | F | | 3/8φ | 5,6@9 Rem @12 ext / 4,2@8,6@9 int | 2-1⅛ cont. in top. |
| B55 | 11½ | 22 | 2 | 1□ | H | 1 | 1□ | J | | 3/8φ□ | 5,2@8,6@9 | 2-1⅛ cont. in top. |
| B56 | 11½ | 22 | [illegible] | 1□ | C | 1 | 1□ | F | | 3/8φ□ | 5,2@9, Rem @12 | 2-1⅛ cont. in top |
| B57 | 11½ | 22 | 2 | 7/8φ | H | | | | | 3/8φ□ | 5,2@9, Rem @12 | 2-1⅛ cont. in top |
| B58 | 11½ | 22 | 2 | 1□ | C | | | | | 3/8φ□ | 6,@12" thruout | 2-1⅛ cont. in top |
| B59 | 15 | 22 | [illegible] | 1□ | [illegible] | 1 | 1□ | F | 11 | 3/8φ | 2,3@6,3@6,4@9 ext / 2,3@4,3@6,4@9 int | |
| B60 | 15 | 22 | [illegible] | [illegible] | [illegible] | 1 | 1□ | F | 22 | 3/8φ | 2,3@4,3@6,4@9 | |
| B61 | 15 | 22 | 1 | 1φ | N @ B11 | | | | 5 | 3/8φ | 5,2@9 ext / 5,2@9 int | |
| B62 | 9½ | 14 | 1 | 1□ | B | 1 | 7/8φ | D | 6 | 3/8φ | 3,2@6 | |
| B63 | 9½ | 14 | 1 | 3/4φ | B | 1 | 3/4φ | D | 6 | 3/8φ | 3,2@6 | |
| B64 | 11½ | 14 | 1 | 1□ | B | 1 | 1□ | D | 6 | 3/8φ | 3,2@6 | |
| B65 | 15 | 22 | 2 | 1φ | C | | | | | 3/8φ□ | 6,@12" thruout | 2-1φ top. Extend to splice and hook with bars from B67 |
| B66 | 15 | 22 | 2 | 1φ | H | 2 | 1¼□ | E | | 3/8φ | 2,3@4,3@6,4@9, Rem @12 | |
| B67 | 15 | 22 | 2 | 1φ | H | | | | 24 | 3/8φ | 2,3@4,7@9 | 2-1□ top. Extend to ¼ pts. of B66 & B68 |
| B68 | 15 | 22 | 2 | 1¼□ | C | 2 | 1¼□ | E | 9 / 16 | 3/8φ | 2,4@6,2@8,4@9 ext / 2,4@8,7@9 int | |
| B69 | 13 | 20½ / 22 | 1 | 1¼□ | C | 1 | 1⅛□ | J / E | 4 / 16 | 3/8φ | 4,3@9 ext / 4,2@8 int | |
| B70 | 13 | 20½ | 2 | 1φ | H | | | | 16 | 3/8φ | 4,2@9,13@12 | 2-1□ top. Extend to ¼ pts. of B69 & B71 |
| B71 | 13 | 22 | 1 | 1□ | C | [illegible] | 1□ / 1⅛□ | J / E | 10 | 3/8φ | 5,2@9 | |
| B72 | 13 | 22 | 3 | 1⅛□ | B | | | | | 3/8φ | 5,2@9 Rem @12 ext / 5,2@9 int | 4-1⅛ top. Hook / @ Col. B14. Extend 2 to ¼ pt of B73 |
| B73 | 13 | 20½ / 22 | 3 | 1□ | [illegible] | | | | 8 | 3/8φ | 4,3@8 | |
| B74 | 13 | 22 | 1 | 1□ | [illegible] | 1 | 1φ | E | 12 | 3/8φ | 5,2@9 | |
| B75 | 15 | 19½ / 21 | 2 | 1¼□ | B | 2 | 1□ | D | 12 | 3/8φ | 4,5@8 | |
| B76 | 17 | 22 | 1 | 1⅛□ | L @ B15 | 1 | 1¼□ | K | 6 | 3/8φ | 5,2@9 | 1-¾φ M bar @ B15 |
| B77 | 18 | 20 | 3 | 1□ | B | | | | | 3/8φ | 4,10@8, Rem @12 Line 16 / 4,12@8 Line 14 | 2-1¼ & 3-1⅛ → top |
| B78 | 15 | 21 | 3 | 1□ | A | | | | | 3/8φ | 4,6@8, Rem @12 int | 2-1¼ top. Extend to ¼ pt. of B79 |
| B79 | 15 | 19½ / 21 | 3 | 1¼□ | [illegible] | | | E | 12 | 3/8φ | 4,5@8 | |
| B80 | 11½ | 21 | 1 | [illegible] | N @ B13 | 1 | 1□ | E | 8 | 3/8φ | 4,3@9 | |
| B81 | 11½ | 21 | 1 | 1φ | H | 2 | 7/8φ | E | 8 | 3/8φ | 4,2@9 | |
| B82 | 11½ | 21 | 1 | 7/8φ | C | | | | | 3/8φ□ | 6,@12" thruout | 2-1¼ top. Extend to ¼ pt. of B81 |
| B83 | 15 | 20½ | 1 | 1□ | [illegible] | 1 | 1¼□ | F | 11 | 3/8φ | 4,5@8 ext / 4,10@8 int | |
| B84 | 15 | 22 | 1 | 1φ | H | 2 | 1□ | F | 6 | 3/8φ | 5,2@9 | |
| B85 | 15 | 20½ | 1 | 1⅛□ | L @ C15 | 1 | 1φ | E | 12 | 3/8φ | 4,6@8 ext / 4,11@8 int | |
| B86 | 15 | 22 | 1 | 1φ | H | 2 | 1□ | F | 6 | 3/8φ | 5,2@9 | |
| B87 | 15 | 22 | 1 | 1φ | H | 2 | 1□ | F | 6 | 3/8φ | 5,2@9 | |
| B88 | 15 | 22 | 1 | 1φ | C | 1 | 1⅛□ | F | 8 | 3/8φ | 5,2@9 ext / 5,7@9 int | 1-1□ M bar @ B16 |
| B89 | 17 | 22 | 2 | 1⅛□ | L @ B16 / C | 1 | 1⅛□ | F | | 3/8φ□ | 5,@9" thruout | 1-1⅛□ M bar @ B17 |
| B90 | 17 | 22 | 1 | 1□ | L @ B16 | 1 | 1□ | K | 15 | 3/8φ | 5,2@9 ext / 5,4@8,8@8 int | |

## TYPICAL WALL REINFORCEMENT & DETAILS

NOTES: Provide at least 2-5/8φ in bottom of all self-supporting walls.
Stub walls to footings or beams with ½φ x 2'-6" @18" o.c.
Stub walls to slabs with ½φ 1'6" 2'6" @18" o.c.
Lap wall bars 30 Dia. at splices; minimum lap to be 1'-6"
Provide 1" concrete cover over main wall bars, except where exposed to earth, provide 2" cover. Unless noted or shown otherwise, all walls shall be reinforced as shown in Schedule below.

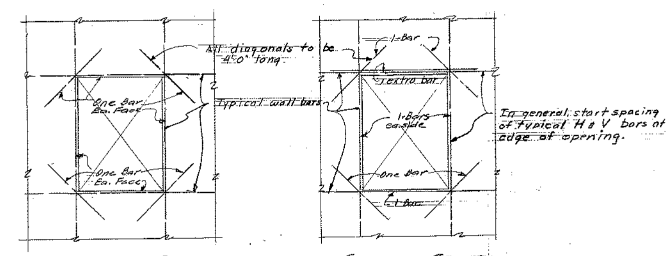
| WALL REINF. SCHEDULE | | |
|---|---|---|
| CONCRETE | HORIZ. & VERT. STEEL | REMARKS |
| 4" | ½φ @18" | In center of wall |
| 6" | 5/8φ @18" | " |
| 7 & 8 | 3/4φ @15" | " |
| 10" | 3/4φ @12" | " |
| Over 10" | 3/4φ @18" | In ea. face |




STEEL IN EACH FACE

STEEL IN CENTER

PLAN OF HORIZONTAL WALL BARS AT CORNERS AND INTERSECTIONS WHERE THERE ARE NO COLUMNS

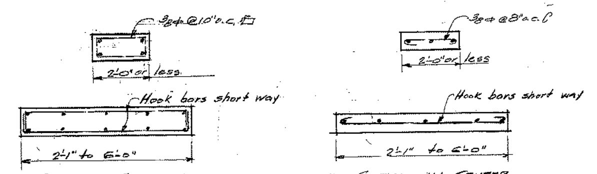



STEEL IN EACH FACE

STEEL IN CENTER

Unless noted or shown otherwise, typical wall bars are to be used around openings. Bars to extend a min. of 30 dia. beyond opening. If 30 dia. not available, extend as far as possible and hook. If opening is more than 6'-0" in horiz. direction use 3/4φ bars over top of opening.

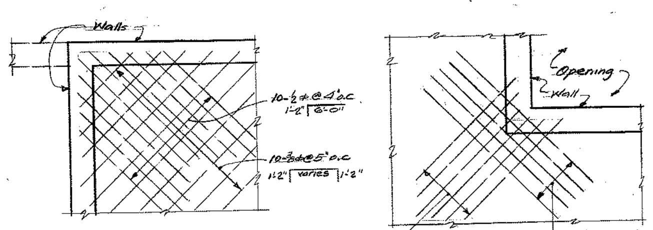
REINFORCING AROUND WALL OPENINGS



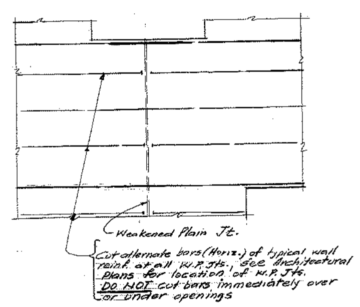

STEEL IN EA. FACE

STEEL IN CENTER

BARS IN SMALL WALL SECTIONS BETWEEN OPENINGS



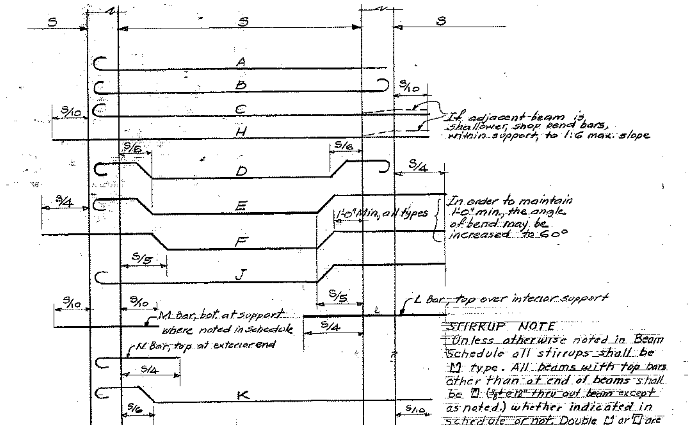

TYPICAL DETAIL SHOWING ADDITIONAL SLAB REINFORCING AT CORNERS OF BUILDING OR SHAFTS FRAMED WITH WALLS




TYPICAL DETAIL OF WALL REINFORCING AT WEAKENED PLAIN JOINTS

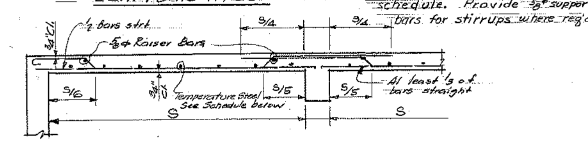
## TYPICAL BEAM & SLAB DETAILS

NOTES: Main beam bars to have 1½" concrete cover. Main slab bars ¾" concrete cover. In beams having bars top and bottom, lap bottom bars 2'-0" at supports and lap top bars 2'-0" between supports, unless noted otherwise. Where beams and girders intersect at columns top steel in deepest member to be decked 1".

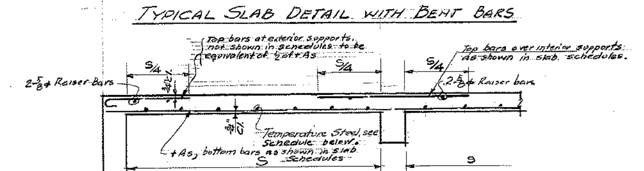



STIRRUP NOTE
Unless otherwise noted in Beam Schedule all stirrups shall be ⊔ type. All beams with top bars other than at end of beams shall be ⊓ (¼φ @12" thru out beam except as noted) whether indicated in schedule or not. Double ⊔ or ⊓ are shown thus ⊔⊔ or ⊓⊓ in schedule. Provide 3/8φ support bars for stirrups where req'd.

BEAM BAR TYPES



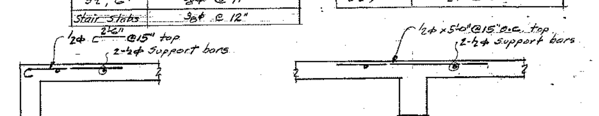

TYPICAL SLAB DETAIL WITH BENT BARS



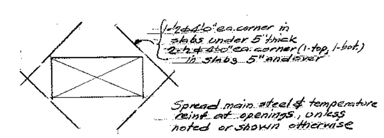

TYPICAL SLAB DETAIL - ALL BARS STRAIGHT

| FLOOR SLAB TEMPERATURE REINF. | |
|---|---|
| CONCRETE | STEEL |
| 3½", 4" | ¼φ @17" |
| 4½" | ¼φ @16" |
| 5", 5½" | ¼φ @13" |
| 5½", 6" | ¼φ @11" |
| Stair Slabs | 3/8φ @12" |

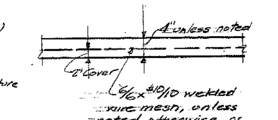
| ROOF SLAB TEMPERATURE REINF. | |
|---|---|
| CONCRETE | STEEL |
| 4" | ¼φ @15" |
| 4½", 5" | ¼φ @11" |
| 5½" | 3/8φ @10" |
| 5½", 6" | ½φ @16" |



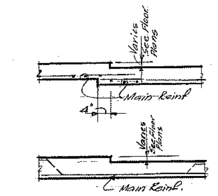

TOP SLAB BARS WHERE MAIN SLAB REINFORCING IS PARALLEL TO SUPPORT



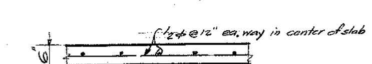

TYPICAL DETAIL OF REINFORCING AROUND SLAB OPENINGS



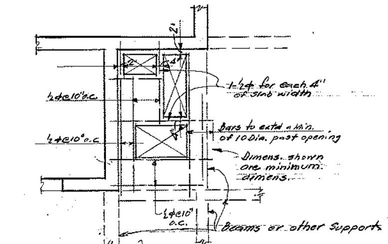

TYPICAL SLAB ON GROUND




TYPICAL DETAIL OF SLABS AT DEPRESSED AREAS



TYPICAL SLAB ON GROUND AT DRIVEWAYS




See Structural Plans for Slab thicknesses.

TYPICAL DETAIL OF REINFORCING AROUND MECHANICAL OPENINGS IN SLABS

## GENERAL NOTES

CONCRETE:
All concrete, except as noted, shall be f'c = 2500 psi, fc = 1125 psi, with a min of 5½ sacks of cement per cu. yd of concrete and a max of 6¾ gal. of mixing water per sack of cement. Concrete as noted f'c = 3000 psi, fc = 1550 psi, to have a min. of 6 sacks of cement per cu. yd of concrete and a max. of 6 gals. of mixing water per sack of cement. Concrete noted f'c = 2000 psi to have a min of 5 sacks per cu. yd and a max. of 7½ gals. of mixing water per sack of cement.
See specifications for aggregate proportions and grading.

REINFORCING STEEL:
fs = 20,000 psi in direct tension and bending. fs = 16000 psi in compression. Reinforcing steel shall be deformed bars per Fed. Spec. QQ-B-71a, Grades 2, 3, 4, or 5.
Reinforcing steel shall be tied securely in place with #16 double-annealed iron wire. Reinforcing steel in beams & slabs shall be supported on well cured concrete blocks or metal chairs.
Reinforcing steel shall be detailed in accordance with the A.C.I. "Manual of Standard Practice for Detailing Concrete Structures."

STEEL JOISTS
All steel joists to conform to specifications of the Steel Joist Institute

MISCELLANEOUS
Refer to Architectural Plans for all wall openings and architectural treatment not shown and all dimensions not shown
Refer to Mechanical Plans for size and location of all openings for piping, ducts, etc.

## DESIGN DATA

DESIGN LOADS

I. LIVING QUARTERS
A. Squad Rooms, NCO. & BOQ
L.L. = 30#/□'
Fin. = 6"
36#/□' + Slab
B. Corridors
L.L. = 40#/□'
Fin. = 6"
46#/□' + Slab
C. All other rooms
L.L. = 40#/□'
Fin. = 6"
46#/□' + Slab
With tile fl. 18"
64#/□' + Slab

II. HOSPITAL AREAS
A. Wards & Rooms
L.L. = 40#/□'
Fin. 6"
46#/□' + Slab
With tile fl. 18
64#/□' + Slab
B. Operating & similar Rooms
L.L. = 60#/□
Fin. = 6#/□'
66#/□' + Slab
With tile fl. = 18"
84#/□' + Slab
C. Corridors
L.L. = 60#/□'
Fin. 6"
66#/□' + Slab
D. Public Spaces
L.L. = 100#/□'
Fin. = 6"
106#/□' + Slab

III. OFFICES
L.L. = 50#/□'
Fin. = 6"
56#/□' + Slab

IV DAY ROOMS & MESS HALLS
L.L. = 100#/□'
Fin. = 6"
106#/□' + Slab

V KITCHENS
L.L. = 40#/□'
Fin. = 6#/□'
Tile fl. = 18#/□
64#/□' + Slab

VI ALL OTHER PUBLIC PLACES & STAIRS
L.L. = 100#/□'
Fin. = 6"
106#/□' + Slab

VII ROOFS
L.L. = 100#/□'
Rfg. & Ins. 10"
110#/□'
Fill for snow 48#/□'
melting areas 158#/□' + Slab

VIII MISC. AREAS
Light Storage - 125, 150#/□'
Heavy Storage & Stores - 250#/□'
Mechanical Rooms = 75#/□'

IX PARTITION LOADS.
Actual partition loads only in Living Quarters. All other areas designed for partition loads in accordance with "Engineering Manual Part IV Chap I" Corps of Engineers.

WIND LOAD: $W = 20\#/□'$
EARTHQUAKE: $F = W\left(\frac{0.60}{N+4.5}\right)$ where W = dead load only, N = no. of stories above the story being considered. Horizontal forces are distributed by the floor system to the vertical walls in proportion to their capacity to resist lateral deformation. Flexural, overturning, & torsional stress are transmitted by the walls to the footings. The seismic design is in accordance with the "Uniform Building Code, 1949." The static design is in accordance with the "A.C.I. Building Code 1948" and the "Engineering Manual, Chap I, Part IV," Corps of Engineers.
DESIGN SOIL BEARING VALUE = 10,000#/□'
Exterior footings and footings with greater percentage of dead load are designed for proportionately lower soil pressure based on ratios of dead load and live load.

## CONCRETE NOTE

f'c = 2000#/□" concrete shall be used only in interior slabs on ground.
f'c = 2500#/□" concrete shall be used thru-out the building and for all misc. construction outside the building which are exposed to the weather, except as otherwise noted.
f'c = 3000#/□" concrete shall be used for all column footings

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|
| | | | |

JOHN W. MALONEY
ARCHITECT-ENGINEER
754 Central Building Seattle, Washington

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY: T.E.K., V.A.P.
TRACED BY: T.E.K., V.A.P.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Selivanoff, CHIEF Struct. BRANCH

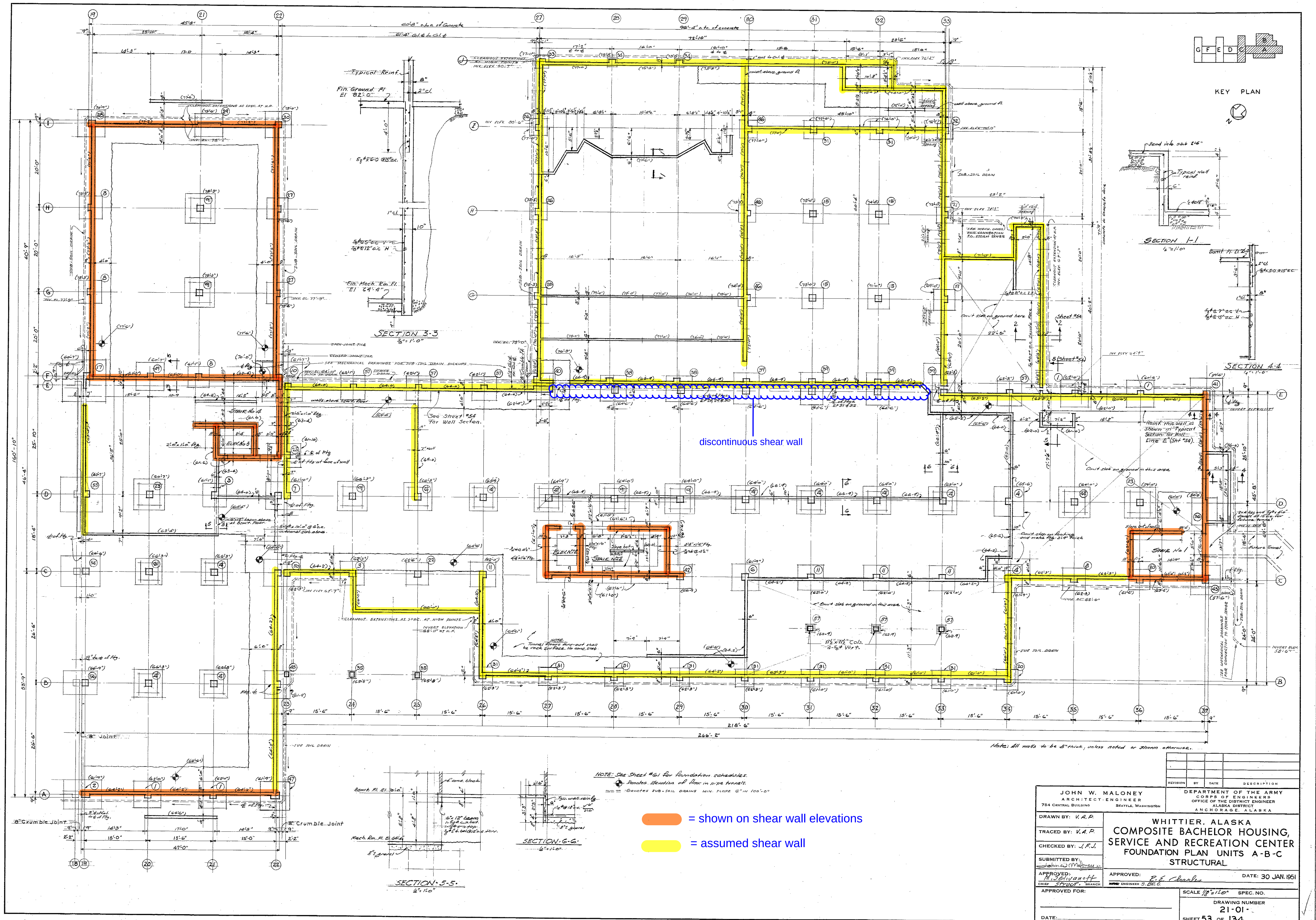
WHITTIER, ALASKA
COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
GROUND FLOOR BEAMS UNITS D·E·F·G
STRUCTURAL GENERAL NOTES

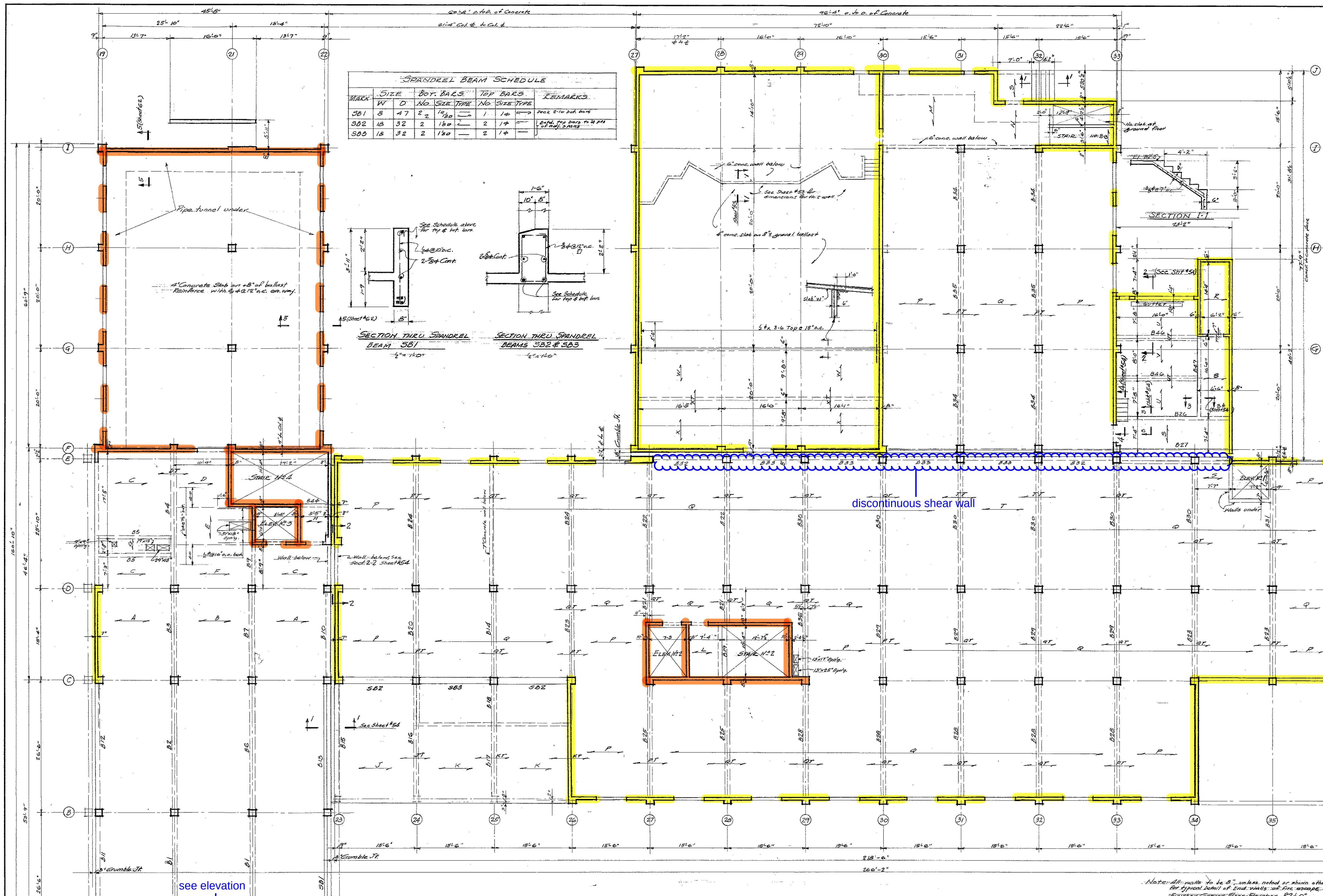
APPROVED: R. E. Charles, ENGINEER S.Be.O.
DATE: 30 JAN. 1951

APPROVED FOR:
DATE:

SCALE: SPEC. NO.
DRAWING NUMBER 21-01-
SHEET 63 OF 134

FILE 20-20
DA179
51-18


discontinuous shear wall
= shown on shear wall elevations
= assumed shear wall
KEY PLAN
SECTION 1-1
SECTION 3-3
SECTION 4-4
SECTION 5-5
SECTION 6-6
Typical Reinf.
Omit slab on ground in this area
NOTE: See Sheet #61 for foundation schedules.
Note: All walls to be 8" thick, unless noted or shown otherwise.
JOHN W. MALONEY
ARCHITECT-ENGINEER
754 CENTRAL BUILDING
SEATTLE, WASHINGTON
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA
WHITTIER, ALASKA
COMPOSITE BACHELOR HOUSING,
SERVICE AND RECREATION CENTER
FOUNDATION PLAN UNITS A-B-C
STRUCTURAL
DRAWN BY: V. A. P.
TRACED BY: V. A. P.
CHECKED BY: J. F. J.
SUBMITTED BY:
APPROVED:
APPROVED FOR:
DATE: 30 JAN. 1951
SCALE 1/8"=1'-0"
SPEC. NO.
DRAWING NUMBER
21-01-
SHEET 53 OF 134




| SPANDREL BEAM SCHEDULE | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| MARK | SIZE W | SIZE D | BOT. BARS No | BOT. BARS SIZE | BOT. BARS TYPE | TOP BARS No | TOP BARS SIZE | TOP BARS TYPE | REMARKS |
| SB1 | 8 | 47 | 2 2 | 1□ 1⅛□ | | 1 | 1φ | | Deck 2-1□ bot. bars |
| SB2 | 18 | 32 | 2 | 1⅛□ | | 2 | 1φ | | Extd. top bars to ¼ pts of adj. spans |
| SB3 | 18 | 32 | 2 | 1⅛□ | | 2 | 1φ | | |

KEY PLAN

| SLAB SCHEDULE | | | | | |
|---|---|---|---|---|---|
| CONCRETE MARK | CONCRETE DEPTH | BOT. BARS SIZE & SPACING | TOP BARS MARK | TOP BARS SIZE & SPACING | REMARKS |
| A | 5¼ | ½φ @ 6" | AT | ½φ @ 18 | Bend up ½ |
| B | 5¼ | ½φ @ 7" | | | Bend up ⅔ |
| C | 6 | ½φ @ 6½ | | | Bend up ½ |
| D | 6 | ½φ @ 8½ | DT | ½φ @ 13" | " |
| E | 6 | ½φ @ 7½ | | | All bars strt. |
| F | 6 | ⅝φ @ 8" | FT | ½φ @ 18" | Bend up ⅔ |
| G | 3½ | ⅜φ @ 10" | | | All bars strt. |
| H | 4 | ⅜φ @ 7" | | | " |
| J | 6" | ⅝φ @ 7" | JT | ½φ @ 16 | Bend up ½ |
| K | 6" | ⅝φ @ 9" | KT | ½φ @ 18" | Bend up ⅔ |
| L | 4 | ⅜φ @ 7 | | | All bars strt. |
| M | 6" | ¾φ @ 7½" | | | Bend up ½ |
| N | 6 | ½φ @ 9½ | | | " |
| P | 5½ | ½φ @ 5" | PT | ½φ @ 20 | " |
| Q | 5½ | ½φ @ 6½ | QT | ½φ @ 14 | " |
| R | 4" | ½φ @ 9½ | | | All bars strt. |
| S | 4" | ⅜φ @ 7" | | | " |
| T | 5½" | ⅝φ @ 7½ | TT | ½φ @ 10" | Bend up ½ |
| U | 6 | ½φ @ 6" | UT | ½φ @ 5" | All bars str. Top steel 1" cl. |
| V | 6 | ½φ @ 6" | | | " |
| W | 4 | ½φ @ 10" | | | All bars str. |
| X | 4 | ½φ @ 6½" | XT | ½φ @ 6" | " |

Note: All walls to be 8", unless noted or shown otherwise. For typical Detail of End walls of Fire escape shafts see Sheet #62. Finished Ground Floor Elevation 82'-0"

NOTE: All Beams NOT sheduled on this sheet to be as shown in Beam Schedule for 1ST FLOOR, Sheet #56

| BEAM SCHEDULE | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CONCRETE MARK | W | D | STRT. BARS No | SIZE | TYPE | BENT BARS No | SIZE | TYPE | STIRRUPS No | SIZE | SPACING EA. END | REMARKS |
| B1 | 13 | 22 | 1 1 | 1□ 1φ | C | 2 | 1φ | E | 6 7 | ⅜φ U | 4,5@9 at extr. end 4,6@9 at intr. end | 1-1¼□ L bar at Cols. B20 & B21 1-⅝φ N bar |
| B2 | 13 | 22 | 2 | 1φ | H | 2 | 1□ | F | 14 | ⅜φ U | 4,6@9 | |
| B3 | 13 | 22 | 2 | 1φ | H | | | | | ⅜φ ▯ | 3,2@6 ea. end then @ 12" thru | 2-1⅛□ top, extd. to ¼ pts. of B2 & B4 1-1⅛□ L bar at Col. D20 |
| B4 | 13 | 22 | 2 | 1¼□ | C | 2 | 1¼□ | E | 25 16 | ½φ U | 1½,12@3,6@4,2@6,4@10 @ B3 2,3@4,4@6,3@8,5@10 | Deck 1-"E" bar over col. D20 1-¾φ N bar |
| B5 | 7½ | 12 | 1 | ⅞φ | B | 1 | ¾φ | D | 6 | ⅜φ C | 3,2@6" | |
| B6 | 13 | 22 | 1 1 | ⅞φ 1φ | H | 2 | 1φ | F | 12 | ⅜φ U | 4,5@9 | 1-⅞φ L bar @ Col. C21 |
| B7 | 13 | 22 | 2 | 1φ | C | | | | | ⅜φ ▯ | 3,2@6 ea. end, bal. @ 12" thru | 2-1φ top, hook into Col. D21 and extd. to ¼ pt. of B6 |
| | | | | | | | | | | | | |
| B9 | 9½ | 14 | 1 1 | 1□ 1φ | B | | | | | ⅜φ ▯ | 3,4@6 ea. end then @ 12" thru | 2-¾φ top |
| B10 | 11½ | 20 | 1 1 | 1φ ⅞φ | C | | | | | ⅜φ ▯ | 4,9 ea. end, then @ 12" thru | 2-⅞φ top, extd. to ¼ pt. of B13 |
| B11 | 13 | 22 | 2 | 1⅛□ | C | | | | | ⅜φ ▯ | 4,2@9 then @ 12" thru @ extr. end 4,3@9 @ intr. end | 2-1φ top extd. to ¼ pt. of B12 1-1□ L bar at Col. B19 |
| B12 | 13 | 22 | 2 | 1¼□ | C | | | | | ⅜φ ▯ | 4,7@9 then @ 12" thru | 2-1φ top, extd. to ¼ pt. of B11 1-½φ N bar |
| B13 | 11½ | 20 | 1 1 | 1⅛□ 1¼□ | C | | | | | ⅜φ ▯ | 3,2@6,6@9 @ B13 then @ 12" thru 4,5@9 @ Col. B22 | 2-1φ top, extd. to ¼ pt. of B10 1-1⅛□ L bar at Col. C22 |
| B14 | 15 | 20 | 2 | 1⅛□ | C | 2 | 1□ | E | 10 | ⅜φ U | 3,6,3@9 | Extd. E bars over B18 to ¼ pt. of B17 1-½φ N bar |
| B15 | 13 | 20 | 1 1 | 1¼□ 1⅛□ | B | 2 | 1⅛□ | D | 12 | ⅜φ ▯ | 4,5@9 ea. end then @ 12" thru | Deck 1-D bar 2-⅝φ × 20'-0" top, See Sect. 1-1 |
| B16 | 13 | 20 | 4 | 1□ | C | | | | | ⅜φ ▯ | 3,2@6,2@8 ea. end then @ 12" thru; ⅜φ ▯ over wall | 2-⅞φ top, extd. to ¼ pt. of B23 Extd. "C" bars to Col. C24; Add 2-⅝φ N bars |
| B17 | 13 | 20 | 2 | 1□ | C | 2 | 1□ | E | 10 | ⅜φ U | 3,2@6,2@8 | Extd. "E" over B18 to ¼ pt. of B14 1-½φ N bar |
| B18 | 13 | 20 | 2 | ⅞φ | H | | | | | ⅜φ ▯ | 4"@9" thru | |
| B19 | 9½ | 12 | 1 | 1φ | B | 1 | 1φ | D | 4 | ⅜φ U | 2,5" | |
| B20 | 15 | 20 | 1 1 | 1□ 1⅛□ | H | | | | | | 3,3@9 @ Col. C24 then @ 12" thru 3,2@6,4@9 @ Col. D24 | 2-1φ 32'-0" top, extd. to ¼ pt. of B24 1-¾φ L bar @ Col. C24 |
| B21 | 15 | 20 | 3 2 | 1¼□ 1φ | top C | Extd. to ¼ pt. of B22 | | | | ⅜φ ▯ | 6, bal. @ 12" thru | Add 2-1¼□ × 6'-6" bot. at Cols. D27 D28 & D36 |
| B22 | 15 | 20 | 2 | 1⅛□ | B | 2 | 1⅛□ | D | 11 9 | ⅜φ U | 3,3@6,7@9 @ B21 4,8@9 @ Extr. end | Extd. D bars thru B21 and hook into wall Deck 1-D bar top @ Col.; Add 1-½φ N bar |
| B26 | 8 | 58 | 2 2 | 1□ 1⅛□ | top B | | | | | ½φ C | @ 10" thruout. | |
| B27 | 9½ | 20 | 2 | 1φ | B | 1 | 1⅛□ | D | 6 | ⅜φ U | 4,2@9 | |
| B36 | 13 | 20 | 1φ ⅞φ | | H | | | | | ⅜φ ▯ | 4,9" ea. end, then @ 12" thru | 2-1φ top, full lgth. of bm. extd. to ¼ pts. of adjacent spans |
| B46 | 11½ | 16 | 2 | 1⅛□ | B | 1 | 1□ | D | 12 | ⅜φ U | 3,5@7 | |
| B47 | 8 | 58 | 2 2 | 1□ 1□ | top B | | | | | ½φ C | @ 10" thruout | |

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|
| | | | |

JOHN W. MALONEY
ARCHITECT-ENGINEER
754 CENTRAL BUILDING SEATTLE, WASHINGTON

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY: V.A.P.
TRACED BY: V.A.P.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. [illegible] CHIEF Struct. BRANCH

# WHITTIER, ALASKA
# COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
## GROUND FLOOR PLAN UNITS A-B-C
## STRUCTURAL

APPROVED: C. E. Charles ENGINEER S. BR. O.
DATE: 30 JAN. 1951
APPROVED FOR:
DATE:
SCALE ⅛"=1'-0" SPEC. NO.
DRAWING NUMBER 21-01
SHEET 55 OF 134

FILE 90-20
STANDARD
DA 179

## SPANDREL BEAM SCHEDULE

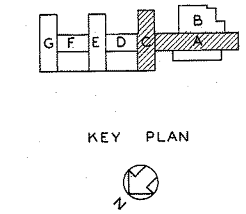
| MARK | SIZE W | SIZE D | BOT. BARS No | BOT. BARS SIZE | BOT. BARS TYPE | TOP BARS No | TOP BARS SIZE | TOP BARS TYPE | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| SB1 | 8 | 47 | 2 | 1 1/8□ | ⊂ | 1 | 1□ | ⊂ | 1-1φ N bar<br>1-1φ L bar |
| SB2 | 8 | 47 | 1 1 | 1□ 1φ | ⊂ | 1 | 1φ | ⊂ | 1-1□ N bar |
| SB3 | 8 | 47 | 1 1 | 1□ 1φ | — | 1 | 1φ | — | Add 1-1□ L bar, deck |

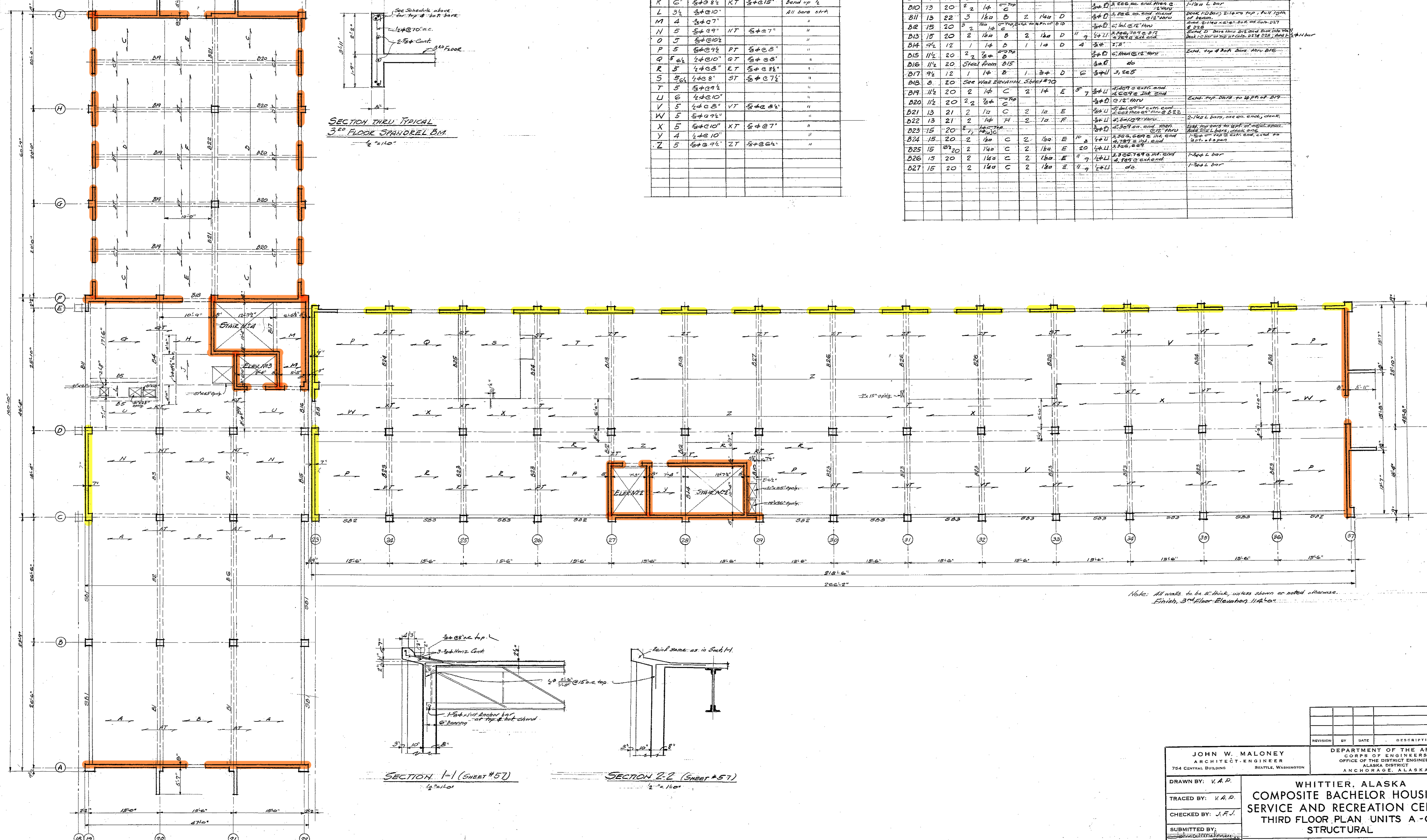
## SLAB SCHEDULE

| CONCRETE MARK | CONCRETE DEPTH | BOT BARS SIZE & SPACING | TOP BARS MARK | TOP BARS SIZE & SPACING | REMARKS |
|---|---|---|---|---|---|
| A | 4 1/4" | 1/2φ@7 1/2 | AT | 1/2φ@5 1/2 | All bars strt. |
| B | 4 1/4 | 1/2φ@7 1/2 | | | " |
| C | 3 1/2 | 3/8φ@10" | CT | 3/8φ@7 1/2 | " |
| D | 3 1/2 | 3/8φ@10" | DT | 3/8φ@8 1/2 | " |
| E | 3 1/2 | 3/8φ@6 1/2 | ET | 1/2φ@9" | " |
| F | 3 1/2 | 3/8φ@8" | FT | 1/2φ@10" | " |
| G | 5" | 1/2φ@6" | GT | 5/8φ@8" | " |
| H | 6" | 1/2φ@11" | | | " |
| J | 6" | 1/2φ@9 1/2" | | | " |
| K | 6" | 5/8φ@8 1/2 | KT | 3/8φ@15" | Bend up 1/2 |
| L | 3 1/2 | 3/8φ@10" | | | All bars strt. |
| M | 4 | 3/8φ@7" | | | " |
| N | 5 | 5/8φ@9" | NT | 5/8φ@7" | " |
| O | 5 | 5/8φ@10 1/2 | | | " |
| P | 5 | 5/8φ@9 1/2 | PT | 5/8φ@8" | " |
| Q | 5 6 1/2 | 1/2φ@10" | QT | 5/8φ@8" | " |
| R | 5 | 1/2φ@8" | RT | 5/8φ@8 1/2" | " |
| S | 5 6 1/2 | 1/2φ@8" | ST | 5/8φ@7 1/2" | " |
| T | 5 | 5/8φ@9 1/2 | | | " |
| U | 6 | 1/2φ@10" | | | " |
| V | 5 | 1/2φ@8" | VT | 5/8φ@8 1/2" | " |
| W | 5 | 5/8φ@9 1/2" | | | " |
| X | 5 | 5/8φ@10" | XT | 5/8φ@7" | " |
| Y | 4 | 1/2φ@10" | | | " |
| Z | 5 | 5/8φ@9 1/2" | ZT | 5/8φ@6 1/2" | " |

## BEAM SCHEDULE

| CONCRETE MARK | CONCRETE W | CONCRETE D | STRT. BARS No | STRT. BARS SIZE | STRT. BARS TYPE | BENT BARS No. | BENT BARS SIZE | BENT BARS TYPE | STIRRUPS No | STIRRUPS SIZE | STIRRUPS SPACING EA. END | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 13 | 22 | 2 | 1φ | C | 2 | 1φ | E | 5 7 | 3/8φ U | 4; 4@10 @ Ext. End<br>4; 2@9, 4@10 @ int. end | 1-1 1/8□ L bar<br>1-1/2φ N bar |
| B2 | 13 | 22 | 2 | 7/8φ | H | 2 | 1φ | F | 10 | 3/8φ | 3, 4@10" | |
| B3 | 13 | 22 | 2 | 1□ | H | | | | | 3/8φ ◻ | 5; 3@10 ea. end then @12" thru | 2-1φ top extd. to 1/4 pts. of B2 & B4 |
| B4 | 13 | 22 | 2 | 1 1/8□ | C | 2 | 1 1/8□ | E | 22 12 | 1/2φ U | 2; 12@4; 2@6, 3@8, 4@10 at B3<br>3; 2@6, 3@8, 6@10 @ Ext. End | 2-1 1/8□ L bars<br>1-1/2φ N bar |
| B5 | 7 1/2 | 12 | 1 | 7/8φ | B | 1 | 3/4φ | D | 6 | 3/8φ C | 3, 2@6 | |
| B6 | 13 | 22 | 2 | 1φ | C | 2 | 1φ | F | 7 5 | 3/8φ U | 4; 2@9, 4@10 @ B1<br>5; 4@10 @ B7 | 2-1□ L bars |
| B7 | 13 | 16 | 2 | 1□ | C | 2 | 1□ | F | | 3/8φ ◻ | 4; 3@8 ea. end then @ 12" thru | 2-7/8φ ⊂ Top, hook into Col. D21 and extd. to 1/4 pt. of B6 |
| B8 | 7 | 20 | 1 1 | 1 1/4□ | — Top<br>— Bot | | | | | 1/2φ C | @6" thru | Extd. T. & B. bars 40 dia. into wall past opening. |
| B9 | 9 | 14 | 2 | 1φ | B | | | | | 3/8φ ◻ | 2, 4@4 ea. end then @ 12" thru | 1-5/8φ ⊂ & 1-5/8φ ⊃ top; extd one top bar (⊂) to 1/4 pt. of B7 |
| B10 | 13 | 20 | 2 2 | 1φ | ⊂ Top<br>C | | | | | 3/8φ ◻ | 3, 2@6 ea. end then @ 12" thru | 1-1 1/8□ L bar |
| B11 | 13 | 22 | 3 | 1 1/8□ | B | 2 | 1 1/4□ | D | | 3/8φ ◻ | 3, 8@6 ea. end then @12" thru | Deck 1-D Bar; 2-1φ ⊃ top, full lgth of beam. |
| B12 | 15 | 20 | 3 2 | 1 1/4□ 1φ | ⊂ Top, Extd. to 1/4 Pt. of B13<br>C | | | | | 3/8φ ◻ | 6; bal. @12" thru | Add 2-1 1/4□ x 6'-6" bot. at Cols. D27 & D28 |
| B13 | 15 | 20 | 2 | 1 1/8□ | B | 2 | 1 1/8□ | D | 11 9 | 1/2φ U | 3, 9@6, 7@9 @ B12<br>4, 8@9 @ ext. end. | Extd. D Bars thru B12 and Hook into Wall<br>Deck 1-D bar at top at Cols. D27 & D28; Add 1-1/2φ N bar |
| B14 | 9 1/2 | 12 | 1 | 1φ | B | 1 | 1φ | D | 4 | 3/8φ | 2, 5" | |
| B15 | 11 1/2 | 20 | 2 2 | 7/8φ | ⊂⊃ Top<br>B | | | | | 3/8φ ◻ | 6; then @12" thru | Extd. top & Bot. Bars thru B16 |
| B16 | 11 1/2 | 20 | Steel | from | B15 | | | | | 3/8φ ◻ | do | |
| B17 | 9 1/2 | 12 | 1 | 1φ | B | 1 | 7/8φ | D | 6 | 3/8φ U | 3, 2@5 | |
| B18 | 8 | 20 | See Wall Elevation Sheet #70 | | | | | | | | | |
| B19 | 11 1/2 | 20 | 2 | 1φ | C | 2 | 1φ | E | 5 7 | 3/8φ U | 4; 4@9 @ extr. end<br>4, 6@9 @ Int. end | |
| B20 | 11 1/2 | 20 | 2 2 | 7/8φ | ⊃ Top<br>C | | | | | 3/8φ ◻ | @12" thru | Extd. top bars to 1/4 pt. of B19 |
| B21 | 13 | 21 | 2 | 1□ | C | 2 | 1□ | E | | 3/8φ U | 4; bal. @9" at extr. end<br>4; 6@8 then @9" thru @ B22 | |
| B22 | 13 | 21 | 2 | 1φ | H | 2 | 1□ | F | | 3/8φ U | 4; bal. @9" thru | 2-1 1/4□ L bars, one ea. end, deck |
| B23 | 15 | 20 | 2 1 | 1φ ⊂ Top<br>1φ 1□ } C | | | | | | 3/8φ ◻ | 4; 3@9 ea. end then @12" thru | Extd. top bars to 1/4 pt. of adjct. span<br>Add 2-1□ L bars, deck one |
| B24 | 15 | 20 | 2 | 1 1/8□ | C | 2 | 1 1/8□ | E | 10 8 | 1/2φ U | 3, 3@6, 6@9 @ int. end<br>4, 7@9 @ int. end | 1-5/8φ ⊂ top @ extr. end; extd to 1/4 pt. of span |
| B25 | 15 | 18 1/2 20 | 2 | 1 1/4□ | C | 2 | 1 1/8□ | E | 20 | 1/2φ U | 3, 3@6, 6@9 | |
| B26 | 15 | 20 | 2 | 1 1/4□ | C | 2 | 1 1/8□ | E | 11 9 | 1/2φ U | 3, 3@6, 7@9 @ int. end<br>4, 8@9 @ ext. end | 1-3/4φ L bar |
| B27 | 15 | 20 | 2 | 1 1/4□ | C | 2 | 1 1/8□ | E | 11 9 | 1/2φ U | do | 1-3/4φ L bar |




SECTION THRU TYPICAL 3RD FLOOR SPANDREL BM.

1/2" = 1'-0"



Note: All walls to be 8" thick, unless shown or noted otherwise.
Finish. 3rd Floor Elevation 114'-0"



SECTION 1-1 (SHEET #57)

1/2" = 1'-0"

SECTION 2-2 (SHEET #57)

1/2" = 1'-0"

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|

JOHN W. MALONEY
ARCHITECT-ENGINEER
754 CENTRAL BUILDING SEATTLE, WASHINGTON

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY: V. A. P.
TRACED BY: V. A. P.
CHECKED BY: J. F. J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Schwartz, CHIEF Struct. BRANCH

# WHITTIER, ALASKA
# COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
## THIRD FLOOR PLAN UNITS A-C STRUCTURAL

APPROVED: R. E. Charles
ENGINEER S.BR.O.
DATE: 30 JAN. 1951

APPROVED FOR:

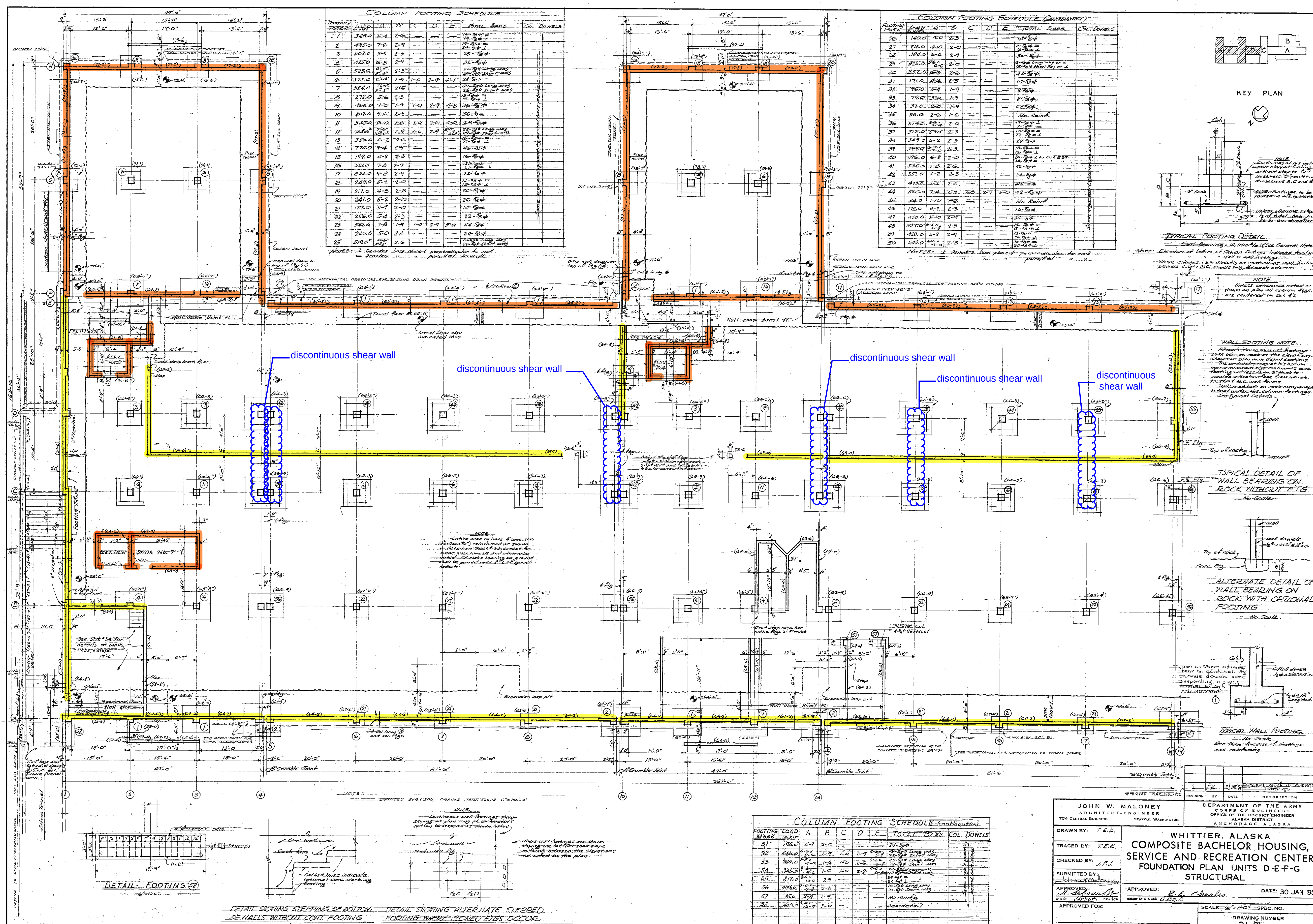
DATE:

SCALE 1/8" = 1'-0" SPEC. NO.

DRAWING NUMBER 21-01-
SHEET 58 OF 134

FILE 20-20

DA179
51-18




## COLUMN FOOTING SCHEDULE

| FOOTING MARK | LOAD IN KIPS | A | B | C | D | E | TOTAL BARS | COL. DOWELS |
|---|---|---|---|---|---|---|---|---|
| 1 | 369.0 | 6-4 | 2-6 | — | — | — | 16-5/8φ = 19-5/8φ ⊥ | Same size and quantity as col. bars above |
| 2 | 495.0 | 7-6 | 2-9 | — | — | — | 19-5/8φ = 24-5/8φ ⊥ | |
| 3 | 308.0 | 5-8 | 2-3 | — | — | — | 28-5/8φ | |
| 4 | 425.0 | 6-8 | 2-9 | — | — | — | 32-5/8φ | |
| 5 | 525.0 | 6'-4" × 8'-4" | 2'-3" | — | — | — | 21-5/8φ Long way 24-5/8φ Short way | |
| 6 | 374.0 | 6'-4" | 1-9 | 1-0 | 2-9 | 4'-4" | 28-5/8φ | |
| 7 | 584.0 | 7'-0" × 8'-8" | 2'-6" | — | — | — | 21-5/8φ Long way 26-5/8φ Short way | |
| 8 | 278.0 | 5-6 | 2-3 | — | — | — | 13-5/8φ = 15-5/8φ ⊥ | |
| 9 | 466.0 | 7-0 | 1-9 | 1-0 | 2-9 | 4-8 | 36-5/8φ | |
| 10 | 807.0 | 9-6 | 2-9 | — | — | — | 56-3/4φ | |
| 11 | 325.0 | 6-0 | 1-6 | 1-0 | 2-6 | 4-0 | 28-5/8φ | |
| 12 | 708.0[K] | 7'-6" × 10'-0" | 1-9 | 1-0 | 2-9 | 5'-0" × 6'-8" | 22-5/8φ Long way 29-5/8φ Short way | |
| 13 | 350.0 | 6-2 | 2-6 | — | — | — | 15-5/8φ = 17-5/8φ ⊥ | |
| 14 | 770.0 | 9-4 | 2-9 | — | — | — | 46-3/4φ | |
| 15 | 199.0 | 4-8 | 2-3 | — | — | — | 16-5/8φ | |
| 16 | 521.0 | 7-8 | 2-9 | — | — | — | 21-5/8φ = 25-5/8φ ⊥ | |
| 17 | 833.0 | 9-8 | 2-9 | — | — | — | 52-3/4φ | |
| 18 | 249.0 | 5-2 | 2-0 | — | — | — | 13-5/8φ = 15-5/8φ ⊥ | |
| 19 | 217.0 | 4-8 | 2-6 | — | — | — | 20-5/8φ | |
| 20 | 241.0 | 5-2 | 2-0 | — | — | — | 26-5/8φ | |
| 21 | 129.0 | 3-9 | 2-0 | — | — | — | 14-5/8φ | |
| 22 | 256.0 | 5-4 | 2-3 | — | — | — | 22-5/8φ | |
| 23 | 541.0 | 7-8 | 1-9 | 1-0 | 2-9 | 5-0 | 44-5/8φ | |
| 24 | 230.0 | 5-0 | 2-3 | — | — | — | 20-5/8φ | |
| 25 | 519.0[K] | 6'-6" × 8'-4" | 2-6 | — | — | — | 17-5/8φ Long way 22-5/8φ Short way | |

NOTES: ⊥ Denotes bars placed perpendicular to wall
= Denotes " " parallel to wall

## COLUMN FOOTING SCHEDULE (CONTINUATION)

| FOOTING MARK | LOAD IN KIPS | A | B | C | D | E | TOTAL BARS | COL. DOWELS |
|---|---|---|---|---|---|---|---|---|
| 26 | 140.0 | 4-0 | 2-3 | — | — | — | 14-5/8φ | Same size and quantity as col. bars above |
| 27 | 216.0 | 4-10 | 2-0 | — | — | — | 11-5/8φ = 13-5/8φ ⊥ | |
| 28 | 384.0 | 6-6 | 2-9 | — | — | — | 30-5/8φ | |
| 29 | 325.0 | 5-6 × 6-6 | 2-0 | — | — | — | 6-5/8φ Long Way or = 18-5/8φ Short Way or ⊥ | |
| 30 | 352.0 | 6-3 | 2-6 | — | — | — | 32-5/8φ | |
| 31 | 171.0 | 4-4 | 2-3 | — | — | — | 14-5/8φ | |
| 32 | 96.0 | 3-4 | 1-9 | — | — | — | 8-5/8φ | |
| 33 | 79.0 | 3-0 | 1-9 | — | — | — | 8-5/8φ | |
| 34 | 37.0 | 2-0 | 1-9 | — | — | — | 6-5/8φ | |
| 35 | 56.0 | 2-6 | 1-6 | — | — | — | No Reinf. | |
| 36 | 374.0 | 6-6 × 6-4 | 2-0 | — | — | — | 17-3/4φ ⊥ 7-5/8φ = | |
| 37 | 312.0 | 5-10 | 2-3 | — | — | — | 14-5/8φ = 17-5/8φ ⊥ | |
| 38 | 349.0 | 6-2 | 2-3 | — | — | — | 28-5/8φ | |
| 39 | 399.0 | 6-0 × 7-4 | 2-3 | — | — | — | 19-5/8φ = 16-5/8φ ⊥ | |
| 40 | 396.0 | 6-8 | 2-0 | — | — | — | 20-5/8φ ⊥ to Col. E27 14-5/8φ = " " " | |
| 41 | 536.0 | 7-8 | 2-6 | | | | 50-5/8φ | |
| 42 | 353.0 | 6-2 | 2-3 | — | — | — | 34-5/8φ | |
| 43 | 473.0 | 7-2 | 2-6 | — | — | — | 44-5/8φ | |
| 44 | 500.0 | 7-4 | 1-9 | 1-0 | 2-9 | 5-0 | 42-5/8φ | |
| 45 | 34.0 | 1-10 | 1-6 | — | — | — | No Reinf. | |
| 46 | 172.0 | 4-2 | 2-3 | — | — | — | 16-5/8φ | |
| 47 | 430.0 | 6-10 | 2-9 | — | — | — | 34-5/8φ | |
| 48 | 337.0 | 6-2 × 6-8 | 2-3 | — | — | — | 15-5/8φ = 18-5/8φ ⊥ | |
| 49 | 428.0 | 6-8 | 2-9 | — | — | — | 16-5/8φ = 19-5/8φ ⊥ | |
| 50 | 543.0 | 6-6 × 9-0 | 2-3 | — | — | — | 30-5/8φ = 20-5/8φ ⊥ | |

NOTES: ⊥ Denotes bars placed perpendicular to wall
= " " " parallel " "

## COLUMN FOOTING SCHEDULE (continuation)

| FOOTING MARK | LOAD IN KIPS | A | B | C | D | E | TOTAL BARS | COL. DOWELS |
|---|---|---|---|---|---|---|---|---|
| 51 | 196.0[K] | 4-8 | 2-0 | — | — | — | 24-5/8φ | Same size & quantity as col. bars above |
| 52 | 546.0 | 6-6 × 8-6 | 1-9 | 1-0 | 2-9 | 4-6 × 5-8 | 16-5/8φ Long way 22-5/8φ Short way | |
| 53 | 740.0 | 7-8 × 10-0 | 1-6 | 1-0 | 2-6 | 5-2 × 6-8 | 25-5/8φ Long way 32-5/8φ Short way | |
| 54 | 346.0 | 7-4 × 9-4 | 1-6 | 1-0 | 2-6 | 5-0 × 6-4 | 22-5/8φ Long way 28-5/8φ Short way | |
| 55 | 817.0 | 8-6 × 10-0 | 2-9 | — | — | — | 29-5/8φ = 24-5/8φ ⊥ | |
| 56 | 424.0 | 6-0 × 7-8 | 2-3 | — | — | — | 16-5/8φ Long way 20-5/8φ Short way | |
| 57 | 45.0 | 2-4 | 1-9 | — | — | — | No reinf'g | |
| 58 | 405.0 | 3-4 × 12-9 | 3-0 | — | — | — | See detail | |

APPROVED MAY 26 1952

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|
| 1 | L.H. | 10 JUNE 52 | Revised reinf. in column footings |

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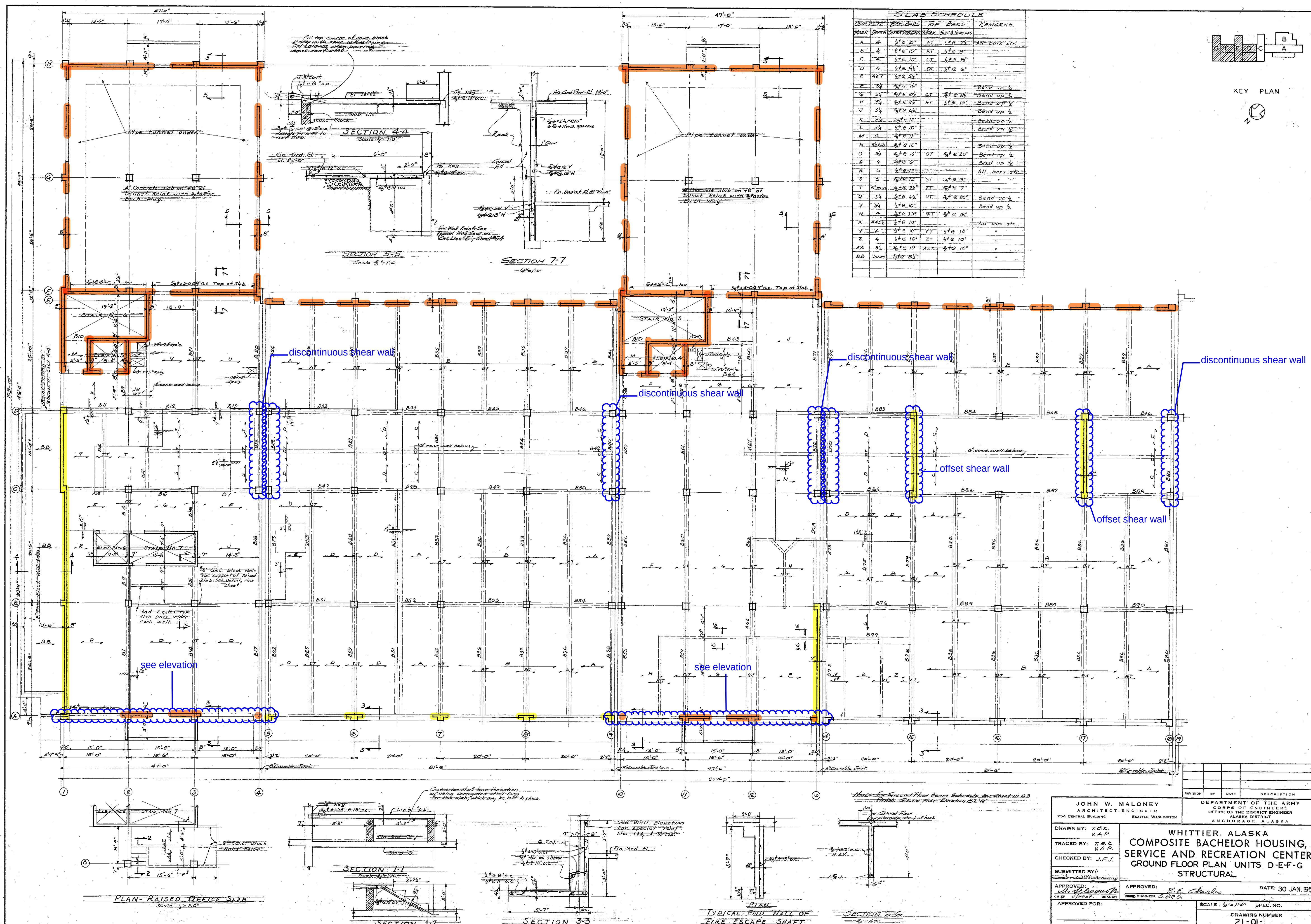
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY: T.E.K.
TRACED BY: T.E.K.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Selivaunt, CHIEF Struct. BRANCH

# WHITTIER, ALASKA
# COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
## FOUNDATION PLAN UNITS D-E-F-G
## STRUCTURAL

APPROVED: R. E. Charles, ENGINEER S.Br.O.
DATE: 30 JAN. 1951
APPROVED FOR:
DATE:
SCALE 1/8"=1'-0" SPEC. NO.
DRAWING NUMBER 21-01-
SHEET 61 OF 134

FILE 20-20




## SLAB SCHEDULE

| CONCRETE MARK | CONCRETE DEPTH | BOT. BARS SIZE & SPACING | TOP BARS MARK | TOP BARS SIZE & SPACING | REMARKS |
|---|---|---|---|---|---|
| A | 4 | ½φ @ 10" | AT | ½φ @ 7½ | All bars str. |
| B | 4 | ½φ @ 10" | BT | ½φ @ 8" | " |
| C | 4 | ½φ @ 10" | CT | ½φ @ 8" | " |
| D | 4 | ½φ @ 9½" | DT | ½φ @ 6" | " |
| E | 4 & 7 | ½φ @ 5½" | | | " |
| F | 5¼ | ⅝φ @ 9½" | | | Bend up ½ |
| G | 5¼ | ⅝φ @ 10½ | GT | ⅝φ @ 31½" | Bend up ⅔ |
| H | 5¼ | ⅝φ @ 9½" | HT | ½φ @ 15" | Bend up ½ |
| J | 5¼ | ⅝φ @ 6½" | | | Bend up ½ |
| K | 5¼ | ⅝φ @ 12" | | | Bend up ½ |
| L | 5¼ | ½φ @ 10" | | | Bend up ½ |
| M | 4 | ⅜φ @ 7" | | | |
| N | 5¼ & 6¾ | ⅝φ @ 10" | | | Bend up ½ |
| O | 5¼ | ⅝φ @ 10" | OT | ⅝φ @ 20" | Bend up ½ |
| P | 6 | ⅝φ @ 6" | | | Bend up ½ |
| R | 6 | ½φ @ 12" | | | All bars str. |
| S | 5 | ⅝φ @ 12" | ST | ⅝φ @ 9" | " |
| T | 5" min | ⅝φ @ 9½" | TT | ⅝φ @ 7" | " |
| U | 3¼ | ⅝φ @ 6½" | UT | ⅝φ @ 20" | Bend up ½ |
| V | 5¼ | ½φ @ 10" | | | Bend up ½ |
| W | 4 | ⅜φ @ 10" | WT | ⅜φ @ 18" | |
| X | 4 & 5½ | ½φ @ 10" | | | All bars str. |
| Y | 4 | ½φ @ 10" | YT | ½φ @ 10" | " |
| Z | 4 | ½φ @ 10" | ZT | ½φ @ 10" | " |
| AA | 3½ | ⅜φ @ 10" | AAT | ⅜φ @ 10" | " |
| BB | Varies | ⅝φ @ 8½" | | | " |

Notes: For Ground Floor Beam Schedule, see Sheet No. 63. Finish Grand Floor Elevation 82'0"

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|

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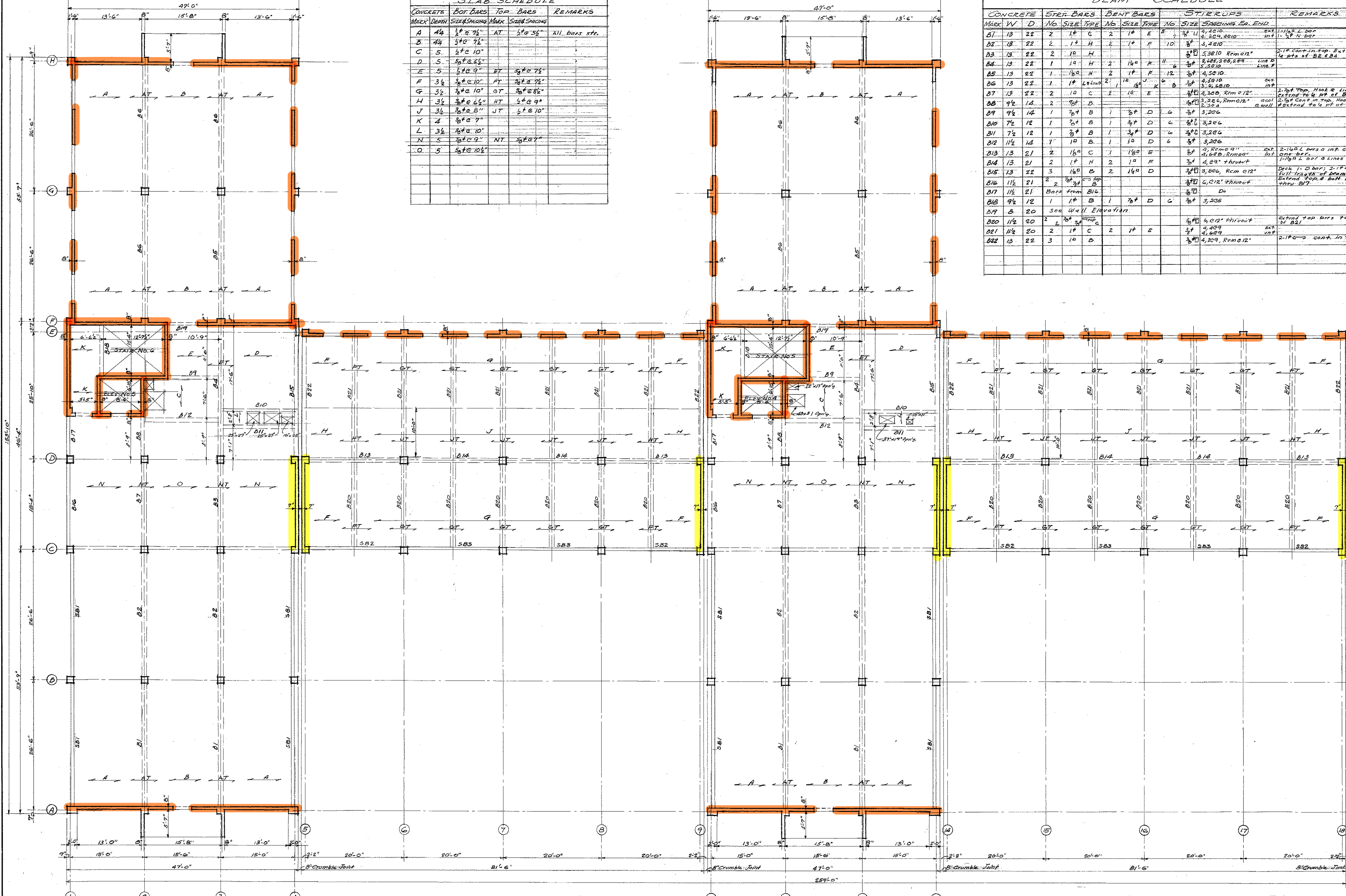
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY: T.E.K. V.A.P.
TRACED BY: T.E.K. V.A.P.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Selivanoff, CHIEF Struct. BRANCH

# WHITTIER, ALASKA
# COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
## GROUND FLOOR PLAN UNITS D-E-F-G
## STRUCTURAL

APPROVED: R. E. Charles, ENGINEER S. Br. O.
DATE: 30 JAN. 1951
APPROVED FOR:
DATE:
SCALE: ⅛" = 1'-0" SPEC. NO.
DRAWING NUMBER 21-01-
SHEET 62 OF 134

FILE 20-20
DA179



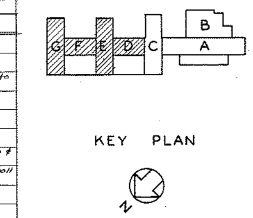
## SLAB SCHEDULE

| CONCRETE MARK | CONCRETE DEPTH | BOT. BARS SIZE & SPACING | TOP BARS MARK | TOP BARS SIZE & SPACING | REMARKS |
|---|---|---|---|---|---|
| A | 4 1/4 | 1/2φ @ 7 1/2" | AT | 1/2φ @ 5 1/2" | All bars str. |
| B | 4 1/4 | 1/2φ @ 7 1/2" | | | " |
| C | 5 | 1/2φ @ 10" | | | " |
| D | 5 | 5/8φ @ 6 1/2" | | | " |
| E | 5 | 1/2φ @ 9" | ET | 5/8φ @ 7 1/2" | " |
| F | 3 1/2 | 3/8φ @ 10" | FT | 3/8φ @ 7 1/2" | " |
| G | 3 1/2 | 3/8φ @ 10" | GT | 3/8φ @ 8 1/2" | " |
| H | 3 1/2 | 3/8φ @ 6 1/2" | HT | 1/2φ @ 9" | " |
| J | 3 1/2 | 3/8φ @ 8" | JT | 1/2φ @ 10" | " |
| K | 4 | 3/8φ @ 7" | | | " |
| L | 3 1/2 | 3/8φ @ 10" | | | " |
| N | 5 | 5/8φ @ 9" | NT | 5/8φ @ 7" | " |
| O | 5 | 5/8φ @ 10 1/2" | | | " |

## BEAM SCHEDULE

| CONCRETE MARK | W | D | STR. BARS NO | SIZE | TYPE | BENT BARS NO | SIZE | TYPE | STIRRUPS NO | SIZE | SPACING EA. END | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 13 | 22 | 2 | 1φ | C | 2 | 1φ | E | 5 / 7 | 3/8φ ⊔ | 4, 4@10 ext / 4, 2@9, 4@10 int | 1-1 1/8□ L bar / 1-1/2φ N bar |
| B2 | 13 | 22 | 2 | 1φ | H | 2 | 1φ | F | 10 | 3/8φ | 5, 4@10 | |
| B3 | 13 | 22 | 2 | 1□ | H | | | | | 3/8φ□ | 5, 3@10 Rem @12" | 2-1φ cont. in top. Extend to 1/4 pts of B2 & B4 |
| B4 | 13 | 22 | 1 | 1□ | H | 2 | 1 1/4□ | F | 11 / 6 | 3/8φ | 2, 6@5, 2@8, 2@9 Line D / 5, 5@10 Line F | |
| B5 | 13 | 22 | 1 | 1 1/8□ | H | 2 | 1φ | F | 12 | 3/8φ | 4, 5@10 | |
| B6 | 13 | 22 | 1 | 1φ | L @ Line B | 2 / 1 | 1□ / 1 1/8□ | J / K | 6 / 8 | 3/8φ | 4, 5@10 ext / 3, 6, 6@10 int | |
| B7 | 13 | 22 | 2 | 1□ | C | 2 | 1□ | E | | 3/8φ□ | 4, 3@8, Rem @12" | 2-7/8φ Top, Hook @ Line D & extend to 1/4 pt of B2. |
| B8 | 9 1/2 | 14 | 2 | 7/8φ | B | | | | | 3/8φ□ | 3, 2@6, Rem @12" @ col. / 2, 5@4 @ wall | 2-5/8φ Cont. in top. Hook @ wall & extend to 1/4 pt of B7 |
| B9 | 9 1/2 | 14 | 1 | 7/8φ | B | 1 | 7/8φ | D | 6 | 3/8φ | 3, 2@6 | |
| B10 | 7 1/2 | 12 | 1 | 7/8φ | B | 1 | 3/4φ | D | 6 | 3/8φ□ | 3, 2@6 | |
| B11 | 7 1/2 | 12 | 1 | 7/8φ | B | 1 | 3/4φ | D | 6 | 3/8φ□ | 3, 2@6 | |
| B12 | 11 1/2 | 14 | 1 | 1□ | B | 1 | 1□ | D | 6 | 3/8φ | 3, 2@6 | |
| B13 | 13 | 21 | 2 | 1 1/8□ | C | 1 | 1 1/8□ | E | | 3/8φ | 4, Rem @9" ext / 4, 6@8, Rem @9" int | 2-1 1/4□ L bars @ int. col. Deck one bar. |
| B14 | 13 | 21 | 2 | 1φ | H | 2 | 1□ | F | | 3/8φ | 4, @9" thruout | 1-1 1/8□ L bar @ Lines 7 & 16 |
| B15 | 13 | 22 | 3 | 1 1/8□ | B | 2 | 1 1/4□ | D | | 3/8φ□ | 3, 8@6, Rem @12" | Deck 1-D bar; 2-1φ top full length of beam. |
| B16 | 11 1/2 | 21 | 2 / 2 | 7/8φ / 7/8φ | top / B | | | | | 3/8φ□ | 6, @12" thruout | Extend top & bott bars thru B17 |
| B17 | 11 1/2 | 21 | Bars | from | B16 | | | | | 3/8φ□ | Do | |
| B18 | 9 1/2 | 12 | 1 | 1φ | B | 1 | 7/8φ | D | 6 | 3/8φ | 3, 2@5 | |
| B19 | 8 | 20 | See | Wall | Elevation | | | | | | | |
| B20 | 11 1/2 | 20 | 2 / 2 | 7/8φ / 7/8φ | top / C | | | | | 3/8φ□ | 6, @12" thruout | Extend top bars to 1/4 pt of B21 |
| B21 | 11 1/2 | 20 | 2 | 1φ | C | 2 | 1φ | E | | 3/8φ | 4, 4@9 ext / 4, 6@9 int | |
| B22 | 13 | 22 | 3 | 1□ | B | | | | | 3/8φ□ | 4, 2@9, Rem @12" | 2-1φ cont. in top. |

KEY PLAN




## SPANDREL BEAM SCHEDULE

| MARK | SIZE W | SIZE D | BOT BARS NO | BOT BARS SIZE | BOT BARS TYPE | TOP BARS NO | TOP BARS SIZE | TOP BARS TYPE | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| SB1 | 8 | 47 | 2 | 1 1/8□ | | 1 | 1□ | | Add 1-1φ N bar @ ext. & 1-1φ L bar @ int. col. |
| SB2 | 8 | 47 | 1 / 1 | 1□ / 1φ | | 1 | 1□ | | Add 1-1φ N bar @ ext. col. |
| SB3 | 8 | 47 | 1 / 1 | 1□ / 1φ | | 1 | 1□ | | Add 1-1φ L bar @ all int. cols. |

For detail of Spandrel Beam See Sht. #57

NOTES: All walls to be 8" thick unless shown or noted otherwise.
Finished 2nd Floor Elevation 104'-0"

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|

JOHN W. MALONEY
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754 CENTRAL BUILDING SEATTLE, WASHINGTON

DEPARTMENT OF THE ARMY
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OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

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TRACED BY: T.E.K.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Selevan CHIEF [illegible] BRANCH

WHITTIER, ALASKA
COMPOSITE BACHELOR HOUSING,
SERVICE AND RECREATION CENTER
SECOND FLOOR PLAN UNITS D-E-F-G
STRUCTURAL

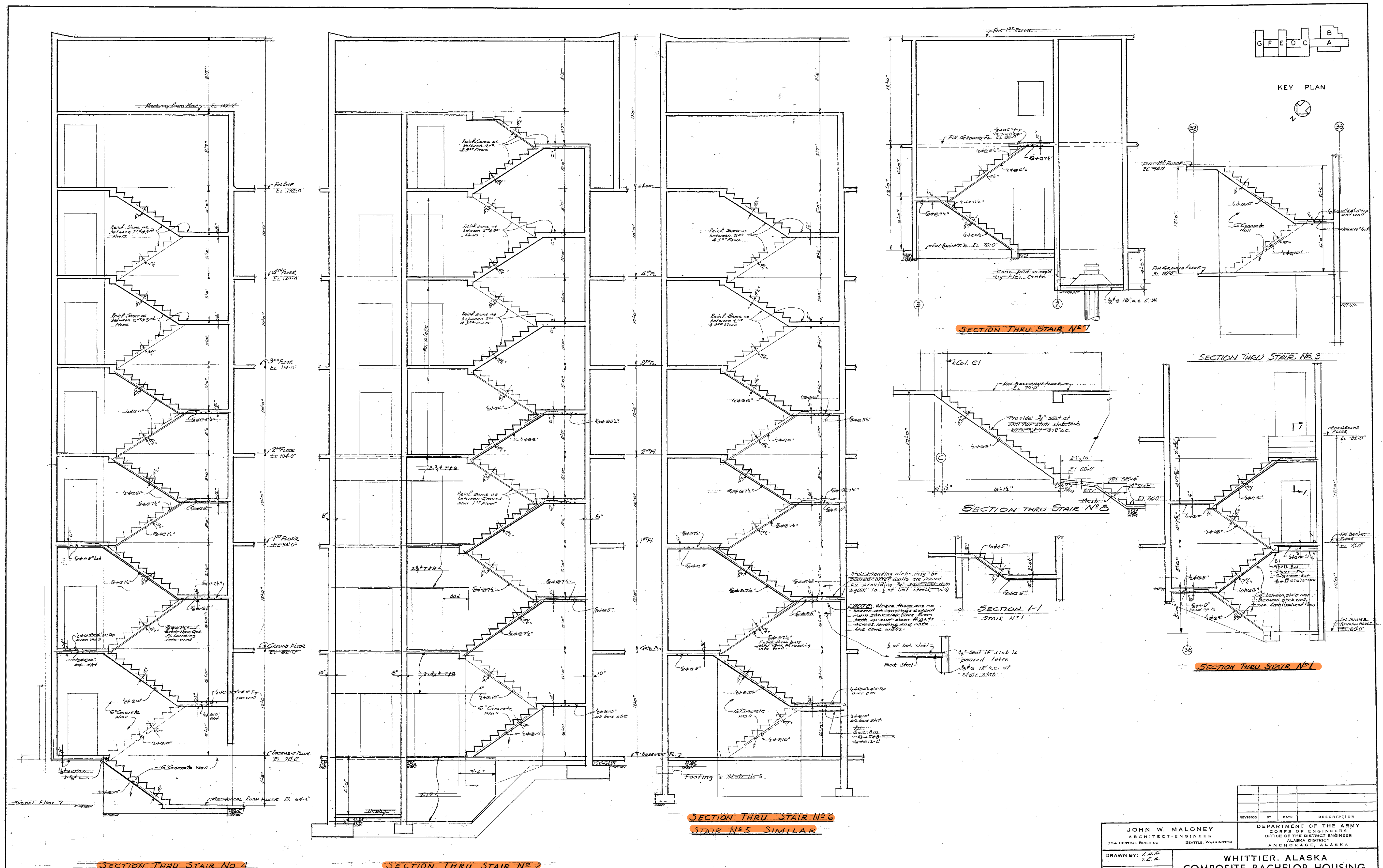
APPROVED: R. E. Charles ENGINEER S. BR. O.
DATE: 30 JAN 1951

APPROVED FOR:
DATE:

SCALE 1/8"=1'-0" SPEC. NO.
DRAWING NUMBER 21-01-
SHEET 65 OF 134

FILE 70-20

DA 79 51-18




STAIR NOTES: For Beams and slabs at floor levels, not shown on this drawing, see Structural floor plans.
See Architectural Plans for size and number of treads and risers, for openings, recesses and all dimensions not shown on this drawing
Dowel all stair slabs to walls with 3/8"φ @ 12" o.c. perpendicular to main slab steel. Provide 3/8"φ @ 12" o.c temperature bars in stair slabs.

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|

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TRACED BY: V.A.P. T.E.K.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Seliguam... CHIEF Struct. BRANCH

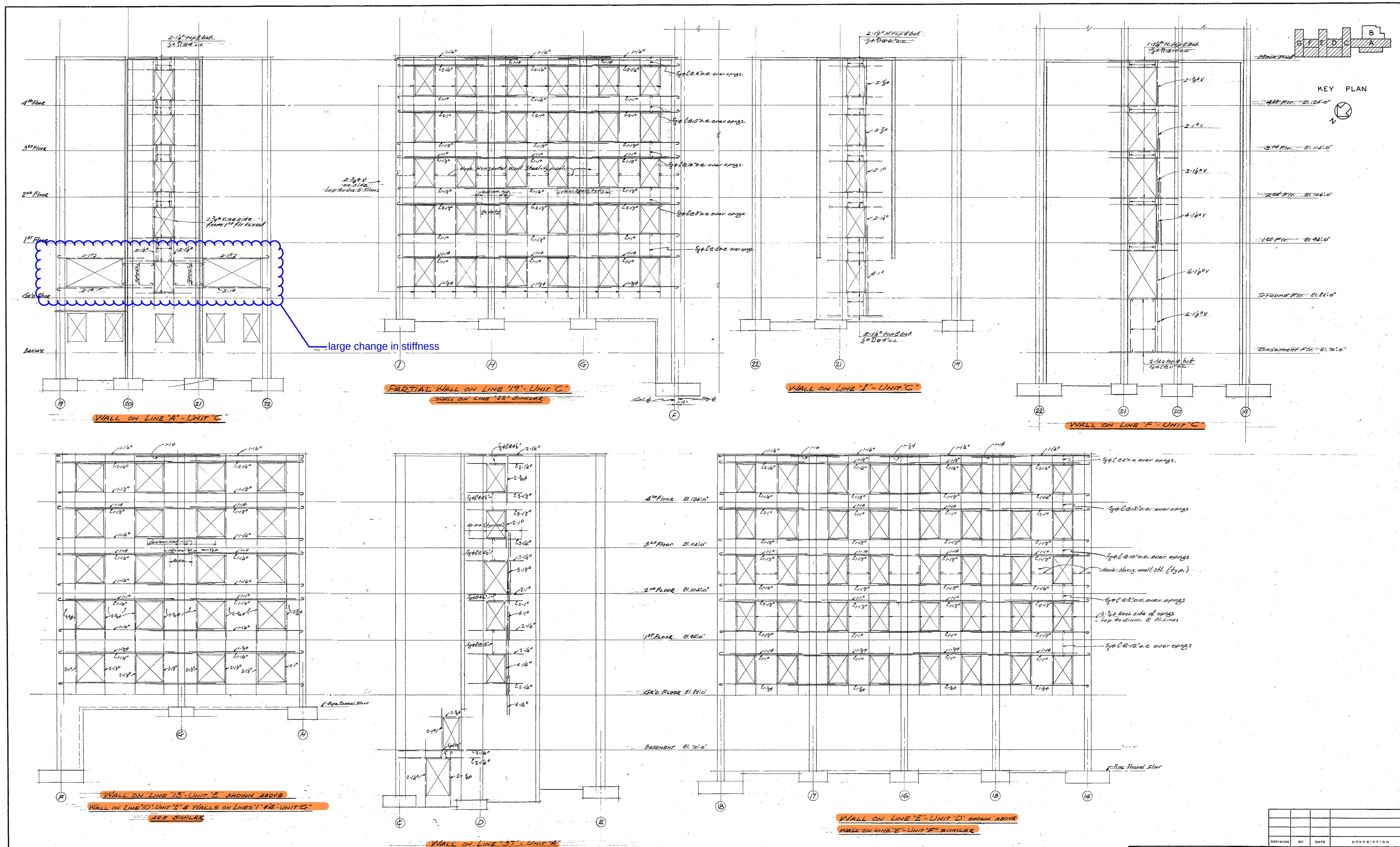
# WHITTIER, ALASKA
# COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
## STAIR SECTIONS
## STRUCTURAL

APPROVED: R. E. Charles ENGINEER S.BR.O.
DATE: 30 JAN 1951

APPROVED FOR:
DATE:

SCALE 1/4" = 1'-0" SPEC. NO.
DRAWING NUMBER 21-01-
SHEET 69 OF 134

FILE 20-20
DA129 51-18




NOTE.

All bars indicated on these elevations are in addition to typical wall reinforcing. See General Notes for typical wall reinforcing.

All diagonal bars at openings in these elevation shall be 1-3/4"ϕ x 4'-0" long.

See Foundation Plans for bottom elevations of walls and footings.

| REVISION | BY | DATE | DESCRIPTION |
|---|---|---|---|
| | | | |

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DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY: W.S.C. J.B.S.
TRACED BY: W.S.C. J.B.S.
CHECKED BY: J.F.J.
SUBMITTED BY: John W. Maloney
APPROVED: M. Selivanow, CHIEF Struct. BRANCH

# WHITTIER, ALASKA
# COMPOSITE BACHELOR HOUSING, SERVICE AND RECREATION CENTER
## LATERAL BRACING WALL ELEVATIONS
## STRUCTURAL

APPROVED: R. E. Charles, ENGINEER S.B.E.O.
DATE: 30 JAN. 1951

APPROVED FOR:

DATE:

SCALE 1/8" = 1'-0" SPEC. NO.

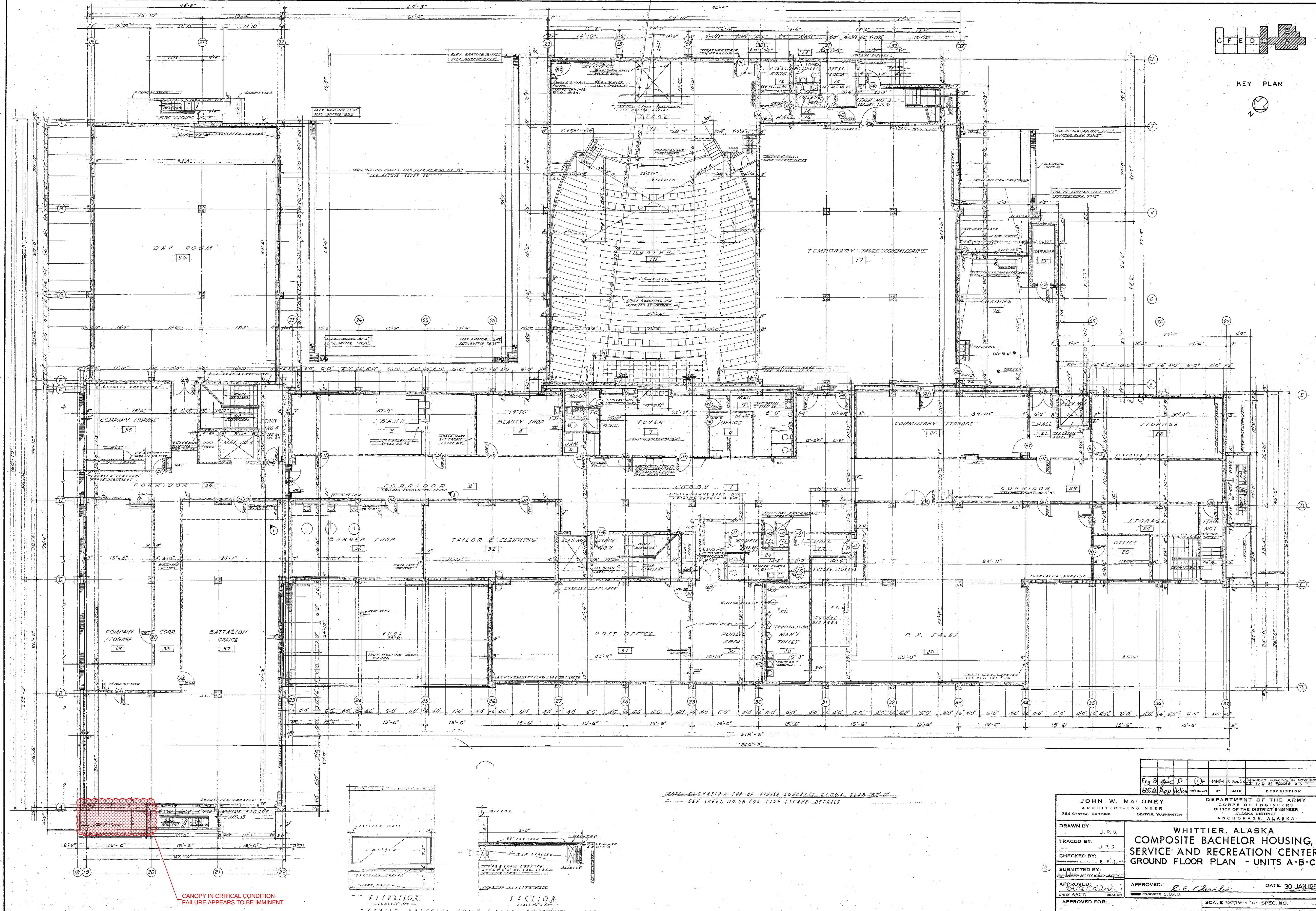
DRAWING NUMBER 21-01-
SHEET 70 OF 134 DA179

FILE 20-20

51-18

# Appendix E

GROUND FLOOR PLAN




| Eng-8 | | P | 1 | MMH | 21 Aug 52 | CHANGED FURRING IN CORRIDOR 2 AND IN ROOM 37. |
|---|---|---|---|---|---|---|
| RCA | App | Action | REVISION | BY | DATE | DESCRIPTION |

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DEPARTMENT OF THE ARMY
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OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
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DRAWN BY: J. P. D.
TRACED BY: J. P. D.
CHECKED BY: E. F. S.
SUBMITTED BY: John W. Maloney
APPROVED: H. T. Kelso
CHIEF ARCT. BRANCH

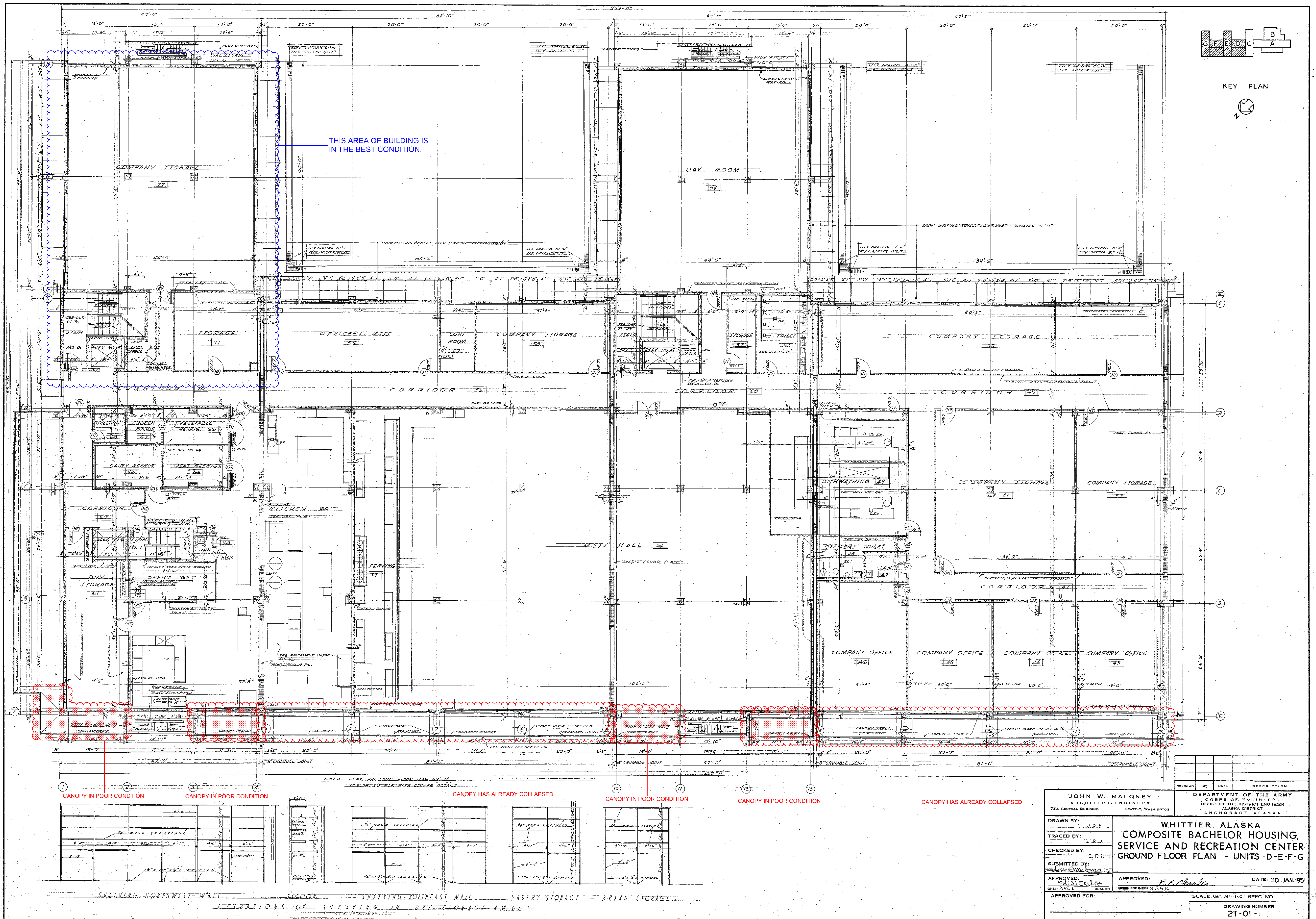
WHITTIER, ALASKA
COMPOSITE BACHELOR HOUSING,
SERVICE AND RECREATION CENTER
GROUND FLOOR PLAN - UNITS A-B-C

APPROVED: R. E. Charles
ENGINEER S. BR. O.
DATE: 30 JAN. 1951

APPROVED FOR:
DATE:

SCALE: 1/8", 1/2" = 1'-0" SPEC. NO.
DRAWING NUMBER
21-01-
SHEET 4 OF 134
DA179

FILE 20-20 STANDARD
51-18


THIS AREA OF BUILDING IS IN THE BEST CONDITION.
CANOPY IN POOR CONDTION
CANOPY IN POOR CONDITION
CANOPY HAS ALREADY COLLAPSED
CANOPY IN POOR CONDITION
CANOPY IN POOR CONDITION
CANOPY HAS ALREADY COLLAPSED
KEY PLAN
COMPANY STORAGE 72
DAY ROOM 51
OFFICERS' MESS 56
COAT ROOM 57
COMPANY STORAGE 55
STORAGE 71
COMPANY STORAGE 35
CORRIDOR 53
CORRIDOR 50
CORRIDOR 40
KITCHEN 60
MESS HALL 54
SERVING 59
DISHWASHING 49
COMPANY STORAGE 41
COMPANY STORAGE 39
CORRIDOR 42
COMPANY OFFICE 46
COMPANY OFFICE 45
COMPANY OFFICE 44
COMPANY OFFICE 43
FROZEN FOODS 67
VEGETABLE REFRIG. 66
DAIRY REFRIG. 64
MEAT REFRIG. 65
DRY STORAGE 61
OFFICE 62
FIRE ESCAPE NO. 7
FIRE ESCAPE NO. 5
SHELVING NORTHWEST WALL
SECTION
SHELVING NORTHEAST WALL
PASTRY STORAGE
BREAD STORAGE
ELEVATIONS OF SHELVING IN DRY STORAGE RM. 61
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DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA
WHITTIER, ALASKA
COMPOSITE BACHELOR HOUSING,
SERVICE AND RECREATION CENTER
GROUND FLOOR PLAN - UNITS D-E-F-G
DRAWN BY: J.P.D.
TRACED BY: J.P.D.
CHECKED BY: E.F.S.
DATE: 30 JAN. 1951
DRAWING NUMBER
21-01-
SHEET 11 OF 134
DA179
FILE 20-20