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Prepared for the U.S. Department of Energy
Assistant Secretary for Environmental Management

Project Hanford Management Contractor for the
U.S. Department of Energy under Contract DE-AC06-96RL13200
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P.O. Box 1000
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J.T. Baxter
Fluor Federal Services, Inc.

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Abstract: Results are presented of an initial quantitative structural assessment on the capability of the 221-U Building to withstand the structural loads associated with the conceptual waste disposal alternatives postulated by the Canyon Disposition Initiative (CDI). A master structural strategy is described that uses the waste emplacement process to establish short and long-term structural stability. The assessment establishes bounding structural loading conditions. Structural calculations are at a rough-order-of-magnitude (ROM) level, for conceptual feasibility scoping purposes only.

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1.0 INTRODUCTION AND SUMMARY

The purpose of this Conceptual Structural Study (CSS) is to supplement the Canyon Disposition Initiative (CDI) with an initial quantitative structural assessment on the capability of the 221-U Building (Figure 1-1) to withstand the structural loads associated with CDI Conceptual Alternative 4. Structural calculations used in this study (Appendices A and B) are at a rough-order-of-magnitude (ROM) level, for conceptual feasibility scoping purposes only.

The CDI Feasibility Study (FS) (DOE/RL, 97) describes several conceptual disposition alternatives for the 221-U canyon facility. Alternative No. 4 (*Entombment with internal/external waste disposal*), postulates emplacing low-level wastes inside and around the outside of the 221-U Building, and then covering the waste-filled facility with an engineered environmental barrier cap (Figure 1-2). Of all the CDI conceptual alternatives addressed in the FS, either CDI Alternative 3 (*Entombment with internal waste disposal*), or CDI Alternative 4 can be used to develop the CSS. Alternative 4 is the alternative of choice for the CSS because it is the only alternative that includes waste placement on both the inside and the outside of the 221-U facility. Alternative No. 4 allows postulation of waste loading scenarios that can bound the structural conditions of all the CDI entombment alternatives. As such, the structural viability of Alternative No.4 also encompasses the structural viability of Alternative No. *3 (Entomb With Internal Waste Disposal*), and Alternative No.6 (*Close in Place-Collapsed Structure*).

The two major types of waste materials being considered for disposal in the 221-U Building are:

a) Compacted bulk soil waste analogous to that currently being disposed in the Environmental Restoration Disposal Facility (ERDF) (Bond and Casbon, 1999), and

b) Cube-containers analogous to Immobilized Low-Activity Waste (ILAW) containers and fill between the containers to eliminate voids (Burbank and Hohl, 1999).

A third type of waste material in the form of compacted equipment/pieces, is also identified in the CDI FS, but is not considered major fill for this CSS.

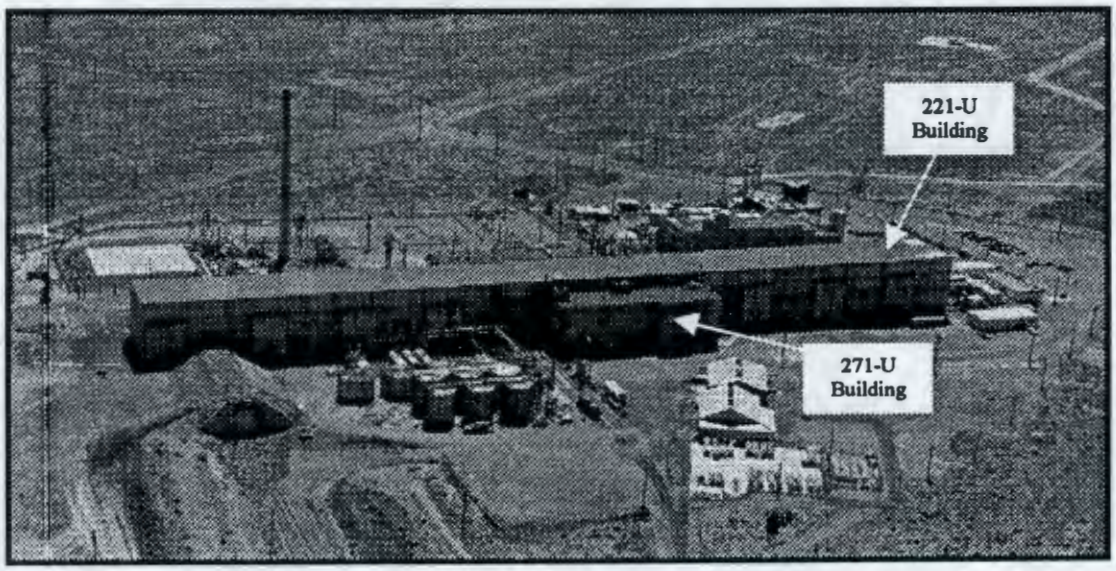
In this CSS, these waste materials are generally referred to as "fill" because they make up the major volume of the materials that will fill the 221-U facility. The structural loads caused by each of the two forms of fill materials establish the lower and upper limits (a and b above, respectively) that bound the CSS. These bounds are used to evaluate the existing 221-U Building structural systems for adequacy to withstand such loads, and to conceptualize fill emplacement sequences.

1.1 Master Structural Strategy

Preliminary structural calculations (Appendices A and B) indicate that CDI Alternative 4 is structurally feasible to achieve short and long term structural adequacy. This can be accomplished by adopting a master structural strategy that uses the waste emplacement process to establish short and long-term structural stability. Short-term is generally considered as the

time during which the site is being constructed, filled, closed and finally covered. Long-term is the subsequent 500-year time period stipulated by NRC regulations for NRC Class C waste sites.

Figure 1-1 221-U Facility and Cross Section Sketch



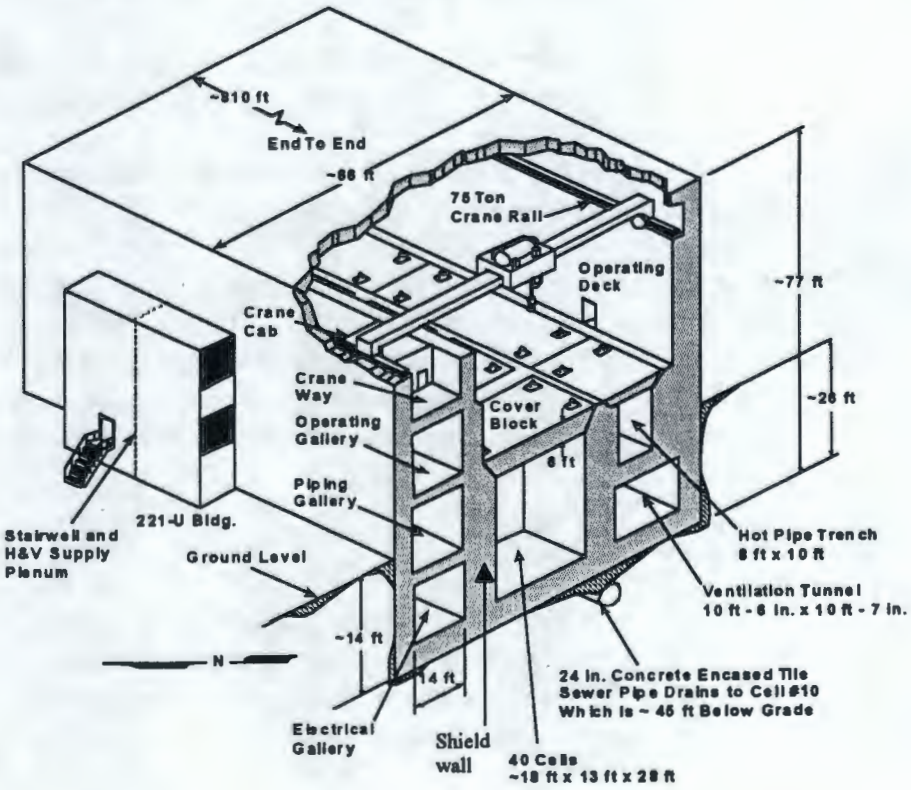
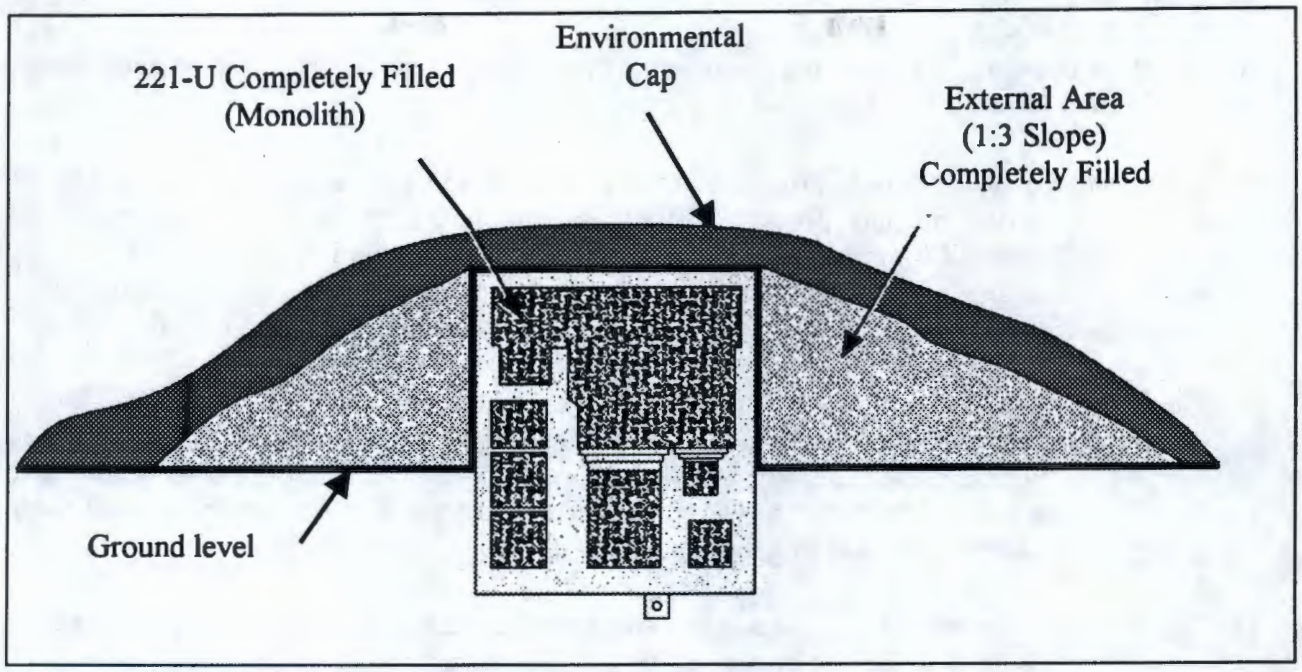


Figure 1-2 Entombment with Internal/External Waste (CDI Alternative No. 4)



The master strategy is comprised of the following four strategic structural objectives:

Strategic Objective No. 1: Long-term structural stability can be accomplished by using the fill process to gradually convert Building 221-U from a building "shell" into a structurally stable "monolith" (Figure 1-2). The monolith is defined as a structurally stable, completely waste-filled facility having no significant internal void spaces. It is engineered to behave as a structural unit that carries and transfers the major structural loads through the fill materials and the building superstructure, to the existing 221-U foundation and down to the soils below the building.

Strategic Objective No. 2: Short-term structural stability can be achieved during the fill process by sequencing the overall fill process and balancing the fill loads on opposite sides of the building walls so that the structural elements of the building are not loaded beyond their existing static and dynamic structural capacities. This also will minimize the amount of supplementary structural reinforcement required for walls, floors, and the roof. In general, floors and the roof will have to be supported by fill materials from below, before they are subjected to additional fill loads on their upper surfaces.

Strategic Objective No. 3: The endwalls will most likely require supplementary structural reinforcement

Strategic Objective No. 4: If the CDI design effort moves beyond the conceptual feasibility stage, detailed structural analyses will be required to establish specific structural design requirements for the fill process, end-wall reinforcements, rail tunnel access, and for the structurally stable monolith. Structural calculations used in this CSS (Appendices A and B) are at a rough-order-of-magnitude (ROM) level, for conceptual feasibility scoping purposes only.

1.2 Conclusions

As a result of this CSS, the following building structure-related conclusions can be made about the 221-U entombment alternative:

- The waste emplacement (fill) process must be sequenced and engineered. For example, it will be necessary to preclude potential differential settlement between building sections. Therefore the overall fill process can be conducted by layering the fill, both inside and outside the building so that the entire facility is filled circumferentially upwards and evenly, rather than filling one end of the facility first and then proceeding from end-to-end.

- The filling process must provide structural support for subsequent fills where needed. For example, structural support for the Piping and Operating Gallery floors can be provided by completely filling each of the lower galleries with well-compacted fill/grout before starting to fill the gallery immediately above. The fill, rather than the gallery floor structures, will carry gravity loads down to the building foundation.

- The filling process must be conducted and engineered to balance the fill loads on opposite sides of each wall. Exterior walls, end walls, and the internal shield wall of the Operating Gallery can endure envisioned seismic lateral forces <u>only if</u> the filling process is conducted to balance the fill loads on opposite sides of each wall. Reinforcing walls to withstand such forces is viable, but is deemed impractical in cost and complexity.

- For internal soil fills, compaction requirements will have to be established and met. Fill combinations of soils and cube-containers are acceptable. Where overhead clearances inside Building 221-U limit compaction of bulk fill, the filling process will be completed using flowable grout. In all cases, balancing fill loading forces on opposite sides of walls will be required. Balanced fill means that the differential elevations between fill on opposite sides of a wall are maintained within some design value (preliminary estimates indicate ~10 ft. maximum differential elevation). Balancing fill limits the horizontal thrust on a wall from fill or soil loads and earthquake loads.

- For internal grout fills, lift heights and areal extents will have to be controlled to limit thermal expansion effects during placement and curing. In all cases, balancing of fill loading forces on opposite sides of walls will be required. Grouting for CDI fills should take advantage of experience developed at the Savannah River Site Saltstone Project (Wilhite et.al., 1988). Detailed thermal analyses are recommended for future design phases.

- As the waste placement on the operating deck initially approaches the rail tunnel, it will be necessary to fill the rail tunnel to provide structural stability to that portion of the building. At that time, alternate access ways into the building will be needed for waste placement on, and above the operating deck.

- The roof is a structural factor for natural phenomena only during the short-term (Prepare and Operate) phases of the CDI project. After the building is filled, the compacted fill/grout

material will carry roof gravity loads down to the operating deck and then the foundations and the external fill will carry lateral natural phenomena loads (e.g., earthquake loads).

- It is recommended that the end-walls be reinforced to withstand natural phenomena loads for the preparation and operations phases of the project. However, this reinforcement is not required to be designed to withstand internal fill loads or seismically-induced fill loads. A balanced fill process will be used to insure structural adequacy for the latter.

- Building 221-U can be shown to comply with current structural design requirements for a Category 3 hazard classification as specified in DOE Orders for all project phases. This may require use of the provisions of DOE Order 420.1 that allow relaxation of natural phenomena structural loading requirements (if necessary) for existing facilities that have limited future mission duration and hazards

- The arrangement of the reinforcing steel in Building 221-U is not compliant with current building codes because codes have changed over the ~50 years since the building was originally designed. However, special code provisions for mass concrete can be invoked that allow accepting the existing reinforcing steel arrangement provided the building is analyzed and shown to have sufficient strength to satisfy current design loading requirements. Scoping calculations indicate that a controlled fill sequence coupled with supplemental structural reinforcement of selected walls, can limit loads during all project phases to assure adequate structural margins (factors of safety).

- The finally-filled 221-U Building, when covered with an engineered surface barrier and including the underlying supporting soils, can behave as a structurally stable monolith that can meet the Nuclear Regulatory Commission (NRC) 500-year long-term structural stability requirement for a land disposal facility.

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2.0 ASSUMPTIONS AND CRITERIA FOR STRUCTURAL SCENARIO DEVELOPMENT

- Primary fill materials: Two forms of waste materials are considered as primary fill materials for placement inside the 221-U Building and around the building perimeter. The materials are:

  a) Compacted bulk soil waste analogous to that currently being disposed in the ERDF is referred to as the base case material for the scenario development, and

  b) Cube-containers analogous to ILAW waste containers are referred to as the alternate case material for the scenario development.

- Void fill materials: Flowable grout and compacted bulk soils are assumed acceptable as void-filler materials. Also, individual equipment items without internal void spaces are assumed acceptable to be interspersed among the void fill material inside the 221-U building only.

- ILAW containers are 1.4 m (4.6 ft) cubical containers with a maximum weight of 10,000 kg (22,040 lbf). They can be stacked, up to 10 m (32.8 ft) high, and are capable of supporting a maximum total load of 100,000 kg (220,400 lbf). Maximum heat loads (range from 0.3 watts to 1.3 watts per container) do not significantly impact the structural scenarios.

- CSS scenario development includes an assumption that structural modifications to Building 221-U are to be minimized.

- Hazards classification: To develop the structural scenarios, an assumption must first be made on the level of hazard associated with the CDI entombment alternatives. There are three hazard categories established in DOE-STD-1027-92 (DOE, 1997):

  Category 1: The hazard analysis shows the potential for significant off-site consequences.

  Category 2: The hazard analysis shows the potential for significant on-site consequences.

  Category 3: The hazard analysis shows the potential for only significant localized consequences.

  These hazard categories are used to specify the required level of conservatism in the facility structural design to resist natural phenomena events (e.g., earthquake and wind). For this CSS, it is assumed that the activities to be conducted during the three CDI project phases (defined in the CDI FS as *Prepare, Operate and Close*) can be classified as Hazard Category 3. (DOE, 1997).

- DOE Order 420.1 specifies natural phenomena design loadings and applicable design criteria based on the facility hazard classification. Given a Category 3 hazard classification, the structural scenarios are subject to an "occupancy classification" from the Uniform Building Code (UBC) (ICBO, 1997) which equates the CDI project with an "Essential" facility. This is consistent with current Project Hanford Procedure HNF-PRO-97, Rev. 0, *Engineering Design and Evaluation* (Conrads, 1997). This UBC occupancy correlates to Performance Category 2 (PC-2) as discussed in HNF-PRO-97. For the scenarios studies, seismic design loadings are consistent with UBC Zone 2B, "Essential Facilities," and wind loads are consistent with UBC wind loads for "Essential Facilities."

- The Code of Federal Regulations 10 CFR 61 (U.S. NRC) identifies a set of primary performance objectives for Land Disposal of Radioactive Waste in Subpart C, "Performance Objectives", Paragraph 61.44, "Stability of the disposal site after closure" states:

  "The disposal facility must be sited, designed, used, operated, and closed to achieve long-term stability of the disposal site and to eliminate to the extent practicable the need for on-going active maintenance of the disposal site following closure so that only surveillance, monitoring, or minor custodial care are required."

  The functional requirement stemming from this performance objective is that short and long-term settlement be limited to provide a "stable" site. A stable site preserves and protects the functions of other parts of the disposal system, such as covers, from natural forces and geologic processes which might degrade their functions. Short-term is generally considered as the time during which the site is being constructed, filled, closed and finally covered. Long-term is the subsequent 500-year time period stipulated by NRC regulations for NRC Class C waste sites.

- The Code of Federal Regulations 10 CFR 61 (U.S. NRC) Paragraph 61.52 (a) (2), "Land Disposal Facility Operation and Site Closure" states:

  "Wastes designated as Class C pursuant to § 61.55, must be disposed of so that the top of the waste is a minimum of 5 meters below the top surface of the cover or must be disposed of with intruder barriers that are designed to protect against an inadvertent intrusion for at least 500 years."

3.0 STRUCTURAL SCENARIOS

Conceptual structural conditions are described in this section for scenarios postulated for filling the 221-U facility with the fill materials discussed in Section 2.0. In general, the postulated scenarios address structural conditions such as: building modifications; operational loading limitations of the existing structures during waste emplacement; and structural constraints imposed on the methods of waste emplacement to assure adequate structural control over both short and long-term settlements.

3.1 Incorporation of CDI Functional Analysis

Functional analysis was used in the CDI Feasibility Study (FS) to define the three top-level major CDI project functions *(DOE/RL, 1991)*. These are synonymous with the major CDI project phases. They are:

1.0 Prepare the Complex
2.0 Operate the Complex
3.0 Close the Complex

The same functional categories are used in this CSS to provide a framework for organizing the discussion and developing the loading scenarios and their inherent structural analyses. The functional definitions for CDI Alternative 4 that are applicable to the development of the structural scenarios were excerpted from the CDI FS and are presented in Table 3-1.

Table 3-1. Top-level Functions and Definitions Applicable to Conceptual Structural Studies for CDI Alternative 4.

Function Title	Definition
1.0 Prepare Existing Complex	Prepare the existing 221-U Facility Site into an operationally-ready waste disposal complex
1.3 Modify External Area	Changes made to the 221-U Facility site to enable initiation of waste operations
1.4 Modify Facility	Changes made to the entombment complex to enable initiation of waste operations
2.0 Operate the Complex	The capability to accept and dispose of waste in the entombment complex
2.3 Dispose Waste	Waste handling operations to situate the waste within the entombment complex
3.0 Close the Complex	Entombment of the waste-filled complex, including post-closure surveillance and maintenance
3.1 Establish Closure Systems	Establish disposition systems to enable capping and closure of the entombment complex

The CDI entombment conceptual alternatives rely on the existing 221-U Building structure, with some modifications, to provide adequate structural load capacities. Adequate structural capacity must be furnished for gravity loads and for natural phenomena lateral loads (e.g., wind, earthquake) that will be encountered during the three major project phases (*1.0 Prepare the Complex ; 2.0 Operate the Complex; and 3.0 Close the Complex)*. For structural evaluation purposes, the project phases can be grouped into two time periods: short and long-term. The short-term period is defined as the time that the facility must remain stable to permit modification of the facility and fill it with waste (i.e., during CDI phases 1.0 *Prepare the Complex* , and 2.0, *Operate the Complex*, respectively). The long-term period is defined as the time that the facility must remain structurally stable during installation of the environmental barrier cap and during the monitoring for long-term stability (i.e., during CDI phase 3.0, *Close the Complex*). Design life for land disposal facilities that handle Nuclear Regulatory Commission (NRC) Class C waste is stipulated to be 500 years minimum.

3.2 Structural Aspects Associated with CDI Function 1.3 (*Modify External Area*)

The site, buildings and structures that lay within the boundary of the proposed external fill around Building 221-U can be prepared for waste emplacement. For the structural scenario scope, this includes the following aspects:

- Proper demolition, removal or preparation of existing external buildings and structures are not anticipated to impact the current structural capacity of 221-U.

- The bearing strength of the granular foundation soils under the main building footing depend in part on the vertical confining gravity loads from the soil overburden around the 221-U building. Because the current elevation of the ground surface varies at different locations around building, the perimeter ground can initially be filled, compacted, and brought to a minimum elevation that assures adequate stability of the building foundations during subsequent waste emplacement operations. This includes filling at both ends of the building Sections 1 and 20.

3.3 Structural Aspects Associated with CDI Function 1.4 (*Modify Facility*)

It is projected that the following building structure-related activities can be conducted without degrading the current structural capacity of the 221-U Building:

- Access ways can be made available for waste emplacement activities in Building 221-U. For example, access into the operating deck can be accomplished via the rail tunnel. The rail tunnel can be used for access to the operating deck before waste emplacement is started on the operating deck itself. At this point in the fill sequence, the rail tunnel will have to be filled with waste to provide an adequate foundation for waste emplacement on top the deck. This will require that access ways to the operating deck be shifted to one or both of the endwalls. Access into the galleries can be accomplished through the existing roll-up door in the south end of the Piping Gallery, and by removal of the unreinforced end walls of the

Electrical and Operating Galleries. For the ventilation tunnel, an access way suitable for construction activities can be made through the end wall.

- The front and rear stair towers of Building 221-U, and Building 271-U will be removed.

- Openings in the exterior walls of Building 221-U can be plugged or sealed prior to starting waste disposal inside the building. There are about 100 openings through the outside wall of Building 221-U into the galleries and another 10 openings from the outside wall of the building into the process area above the cell cover blocks. These openings vary in size from 2 ft by 2 ft electrical penetrations to roll-up garage doors 10 ft wide in the end wall of Section 20. Plugs for these openings can be made compatible with all phases of the project and can be designed to have sufficient strength to resist all loads encountered in the three project phases.

- The endwalls at building Sections 1 and 20 of the 221-U are unreinforced concrete. A structural analyses of the endwalls (Scott and Moody, 1996), identifies that the shear keys, which hold the endwalls in place, will crack and perhaps fail during the design earthquake. Based on limitations described in that analysis, it is recommended that the endwalls be reinforced. Such reinforcement is not intended, nor required to carry proposed fill lateral seismic loads inside the building to grade, because the balanced fill process described in Section 3.4 minimizes the need for the walls to carry such fill loads. Reinforcement design is beyond the scope of this study; however, two reinforcement approaches are briefly introduced below for example purposes:

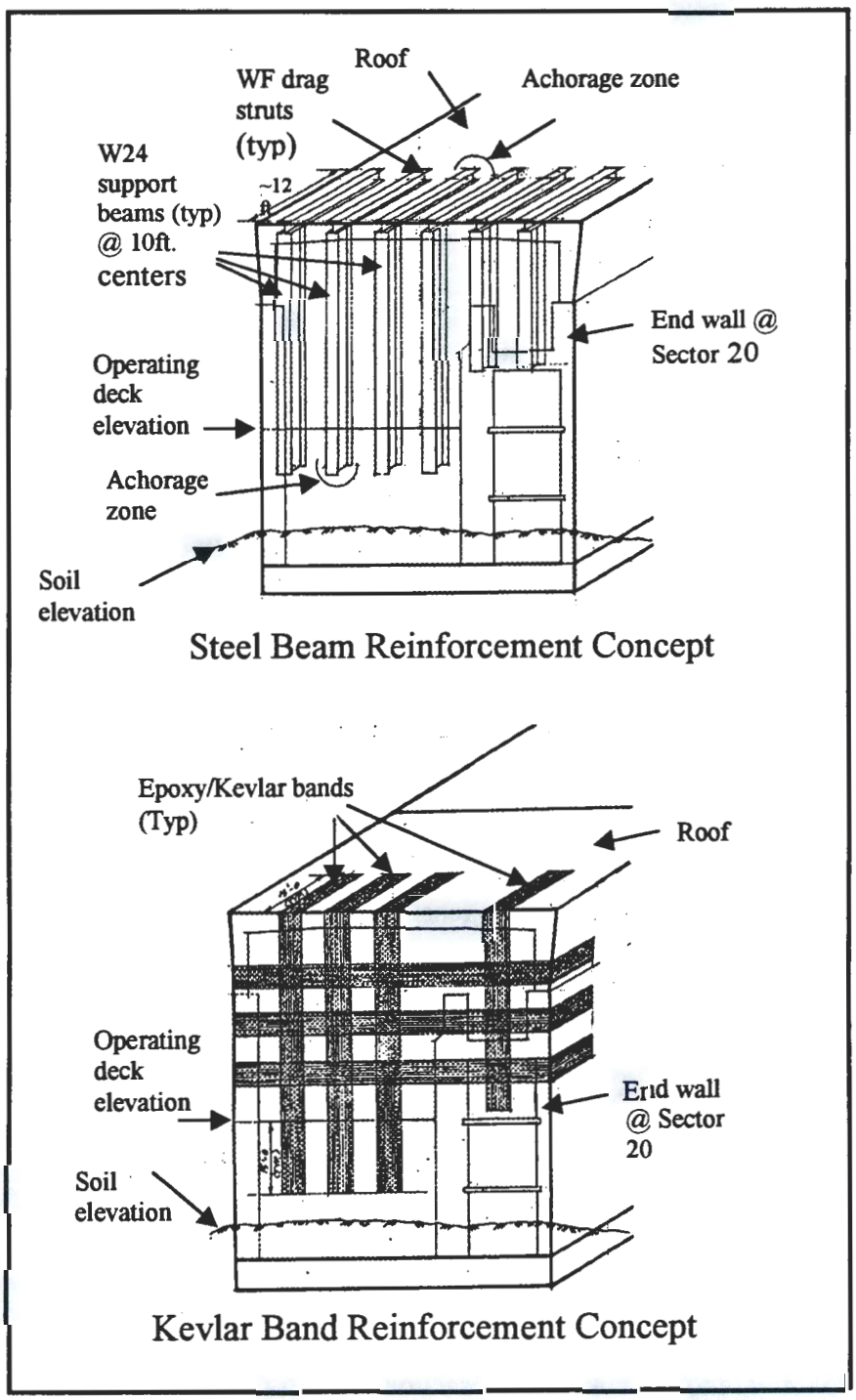
    - ◊ The first approach is reinforcing with a set of steel frames spanning from the end wall below the operating deck vertically to the roof as sketched in Figure 3-1. The frames can be anchored into the mass concrete around the cells and the roof slab at some distance removed from the edge of the roof. They can be designed to carry the entire seismic lateral load of the unreinforced endwall to grade.

    - ◊ A second approach for reinforcement is using Kevlar or Carbon Fiber blankets saturated with epoxy to produce tension ties between the building exterior walls and between the roof and the mass concrete around the cells (Figure 3-1). This option would rely on the epoxy composite bands to produce tensile restraint on the building elements surrounding the unreinforced concrete endwalls. Tensile restraint forces will provide kinematic restraint for the blocks preventing their sliding or tipping out of position. Kevlar and Carbon Fiber blankets are commonly used in combination with epoxy injection to repair and strengthen concrete structures in areas that have experienced damaging earthquakes.

- Visual observations of Building 221-U during walkdowns in 1999 (Baxter, 2000) identified some fracturing of concrete around the expansion joints between several of the 221-U building sections which are numbered 1 through 20. Fracturing typically occurs at locations where vertical and horizontal expansion joints intersect. It appears to have been produced by differential vertical motion between the sections during facility operations. Waste emplacement operations are likely to generate additional differential settlement between

building sections as they are filled. Differential fill depths between adjacent segments can be limited to prevent additional fracturing at these expansion joint intersections by layering the fill so that the entire facility is filled upwards and relatively evenly, rather than filling one end of the facility first and then proceeding from end-to-end.

Figure 3-1. Conceptual Examples of End Wall Reinforcement Approaches



3.4 Structural Aspects Associated with CDI Function 2.3 (*Dispose Waste*)

Constraints on the fill processes discussed in this subsection will maintain the structural elements of Building 221-U within their existing static and dynamic structural limits. Short and long-term structural stability can be achieved by the engineered placement of the fill and waste materials. These fills and waste materials can be placed so they will carry the loads from the emplaced wastes and the proposed surface barrier through the existing foundations to the soils below the building. This structural strategy transforms Building 221-U from a building "structure" into a stable structural "monolith." To assure short-term stability, the fill operations can be conducted in a sequence to balance the fill loads on opposite sides of the building walls during the filling process. With the exception of the endwalls, this strategy eliminates the need for supplemental reinforcement of other walls and floors.

For cement based grout fills it will be necessary to limit the lateral extent of fills (lifts) to control thermal expansion forces that could damage the existing building structural systems. A separate detailed analysis for heat transfer and structural effects of grout lifts is recommended.

3.4.1 Constraints on Fill Placement

The cross-section of Building 221-U shows the interior spaces that will have to be filled to assure long-term stability of the building (Figures 1-1 and 3-2). The two major waste materials that are being considered for disposal in the 221-U Building are listed below.

a) Base Case: Compacted bulk soil waste analogous to that currently being disposed in the Environmental Restoration Disposal Facility (ERDF), and

b) Alternate Case: Cube-containers analogous to Immobilized Low-Activity Waste (ILAW) containers and fill between the containers to eliminate voids.

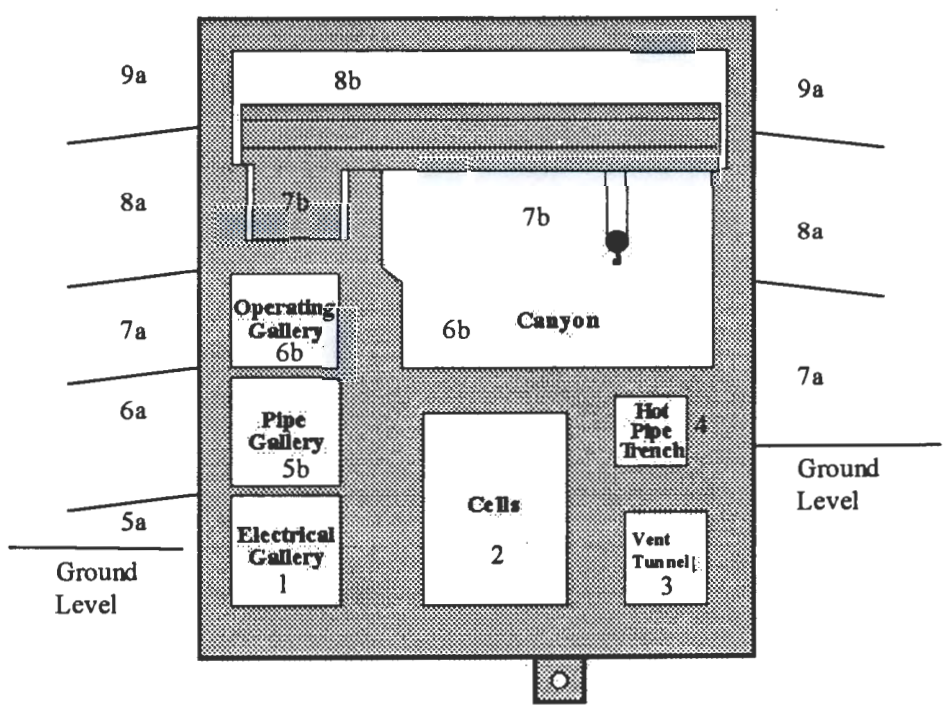
Compacted loose equipment/material pieces is a third type of waste fill material that is also identified in the CDI FS. Such equipment/material is not considered major fill for this CSS because the structural loads that could be caused by loose equipment are not expected to exceed the bounding loads analyzed for ILAW packages. Therefore, there would be no impact to this structural report if loose equipment were to be considered as major fill. From a structural perspective, the only limitations that would be placed on loose equipment is that the equipment be void-free, stabilized in compacted bulk soil or grout, and not exceed the structural capacity bounds established in this report.

A specific waste emplacement sequence is recommended for all fill scenarios because of structural limitations of the existing building.. The recommended sequence is as follows (See Figure 3-2):

- Electrical gallery
- Cells
- Ventilation tunnel
- Hot Pipe trench

- Piping gallery
- Rail Tunnel
- Operating gallery
- Operating Deck to top of Crane Way shield wall (Multiple lifts)
- Crane Way and space above to underside of roof (Multiple lifts)

Figure 3-2. Conceptual Fill Sequence for 221-U



Note: Building is to be filled layer-by-layer, circumferentially.

Waste emplacement scenarios for each of the above areas are discussed in the following subsections for both the base case that uses compacted bulk earthen material and the alternate case that uses cube containers.

Fill materials proposed for placement in the interior spaces of Building 221-U range in weight as follows:

- Well-compacted Hanford Site soils in the range of 110 to 120 lb/ft^3
- Structural grouts ~ 150 lb/ft^3
- ILAW-type waste packages ~ 227 lb/ft^3.

- Miscellaneous equipment (average weight is assumed to be less than the ILAW weight). For the purposes of this scenario study, emplacement of miscellaneous equipment is not considered to create bounding structural conditions. It is assumed that miscellaneous equipment will be size- reduced, collapsed, or cut into relatively small pieces to eliminate void spaces that may negatively affect the long term settlement of the facility during the operational and post-closure periods. Such equipment can be incorporated into granular soil or grout fills. Larger, bulky equipment such as tanks will have to be either size reduced, or filled with a structural grout material. In cases where large tanks and runs of small pipe and tubing are to be left in place; grout fill will have to be used to eliminate voids.

The base case assumes that granular earthen material will be placed in these interior spaces of the building and compacted to a specific relative density. Maximum dry densities for the soils in the areas near Building 221-U range from 110 lb/ft^3 to 116 lb/ft^3 (Shannon & Wilson, 1995). The compaction design for the ERDF facility is 90 percent of modified proctor density (ASTM 1557) (Casbon, 1995). Relative densities for fill placed inside Building 221-U will be selected to be equal to, or higher than that for fill placed outside around the building to preclude differential settlement where the building might settle more than the exterior fill. For comparison, a compaction requirement of 95 percent of modified proctor density is commonly specified for "structural fill" to be placed immediately below foundations for heavy structures. Thus, the expected compaction requirements for fill inside Building 221-U would be in the range of 90 to 95 percent of modified proctor density.

Several types of equipment are available for compacting bulk soil such as rubber tire rollers, vibratory rollers, and vibrating base-plate compactors. Because of the limited headroom available in the gallery spaces, low-boy equipment similar to that used in mining operations would be applicable to the fill operations.

3.4.2 Thermal Considerations for Grout Fill

Flowable grout is proposed for filling interstices between waste containers and for final fills in enclosed spaces where the head room is limited. These grout fills can be controlled in both the height of each lift (pour) and the lateral extent of each pour, to control heat build-up from the heat of hydration of the cementitious material during the curing period. Limits on the lateral extent of the lifts will be necessary to limit thermal expansion forces that could damage the existing building structural systems. A separate detailed analysis for heat transfer and structural effects is recommended. If the CDI project progresses to the next phase, detailed design analyses should be conducted for flowable grout fills to determine maximum lift heights and the maximum horizontal dimensions of monolithic pours at Building 221-U.

For this scenario study, basic information from the design and operation at the Savannah River "Saltstone " Project was considered adequate for conceptualizing limits on grout pour lift heights and lateral extents (Wilhite, 1988). The Saltstone project has operational experience with hazardous waste grouts poured into large concrete vaults. Current web-site information indicates that the vaults are comprised of 6 and 12 cells. Each cell measures 100 feet long, 100 feet wide

and 25 feet tall. The vaults are designed to allow access for pouring a flowable grout with a tremie tube using a concrete pump system. Relatively small lifts, on the order of one foot, are poured sequentially, one after the other, in each of the cells. After a period of time, the next lift is poured in the first cell. The time period is selected to allow sufficient time for the heat of hydration during the curing period to dissipate, and prevents cumulative build-up of temperature and associated thermal expansion that could damage the monolith concrete structure.

3.5 Structural Aspects Associated with CDI Function 3.1 (*Establish Closure Systems)*

After Building 221-U is filled with waste materials and the project proceeds to closure systems; the building, building contents, and surrounding fills will furnish a stable base over which the Environmental Cap is constructed as discussed in the Master Structural Strategy, Section 1.0. Settlements are expected to occur immediately upon the initial waste loadings, with little additional long-term settlement after closure of the site as discussed in Appendix A, Section A1.6. This long-term stability will help retain the integrity of the environmental cap.

4.0 INTERIOR AND EXTERIOR FILL SCENARIOS

This section provides detailed discussions of the waste emplacement scenarios for each of the interior spaces in Building 221-U and for the exterior area around the outside walls of the building. The discussions address both the base case fill material (bulk soil waste), and the alternate case (cube-containers). Note that the waste emplacement sequence discussed in Section 3.4.1 will govern fill emplacement rather than the presentation sequence in this section.

4.1 Conceptual Scenario for the Galleries (Electrical, Piping, and Operating Galleries)

The galleries are located one-above-the-other behind the shield wall as shown in Figures 1-1 and 3-2. The floor of the Electrical gallery is the top of the main foundation for Building 221-U and therefore has sufficient capacity for loads considered for this scenario study (the original foundation plate bearing test capacity exceeded 18 tons per square foot). Floors of the piping and electrical galleries are 12 inch thick concrete slabs with limited reinforcing steel. The load limits for the Piping and Operating Galleries floors were originally 350 lb/ft^2 and 400 lb/ft^2 respectively. Minimum fill loads that the floors of the piping and electrical galleries will have to resist, as noted in appendix A, Table A-1, are 1500 lb/ft^2 (minimum clear height in the three galleries is ~12.0 ft). These loads will fail the floor structures if the floors are left unsupported while emplacing waste. However, the necessary floor support can be provided by completely filling each lower gallery before starting to fill the one immediately above. In this manner, the fill in the lower galleries will support superimposed gravity loads from the upper floors during the period when emplacing waste, and during the following closure operations and the post-closure periods.

The base case fill material for these galleries is compacted bulk soil waste. This material is to be emplaced in lifts with adequate compaction of each lift. Because of the limited ceiling height in the galleries, it is likely that the last fill in each gallery will be with flowable grout if the physical clearances preclude efficient compaction of soil materials. Final lifts of grout will be used to fill void spaces up to the ceiling of each gallery.

The alternate case fill material for the galleries is cube containers analogous to ILAW packages. Emplacement loads for these containers range up to a weight of 227 lb/ft^3. ILAW package emplacements could vary from an arrangement 2 packages wide by 2 packages high to an alternative arrangement of 3 packages wide by 2 packages high in all three galleries. In both cases, the interstices between the containers will require filling to provide structural stability. This fill will probably have to be flowable grout because of limited clearances around the packages. Grout lifts will have to be limited in height and lateral extent as discussed in section 3.4.2. Existing vertical floor penetrations for electrical conductors and control circuits can allow placement of grout in the Electrical and Piping Galleries from the gallery immediately above. Final grouting of the Operating Gallery may require construction of new access ways for grout placement from the Crane Way, or from outside the building exterior wall.

The existing building gallery walls are capable of resisting lateral loads from placement and compaction of bulk soil wastes or grout fills without additional reinforcement provided that the balanced fill sequence discussed in Section 3.4 is followed. Similarly, flowable grout fill similar

to that being used in the Saltstone project at Savannah River Site, can be poured in small lifts so that thermal expansion loads will not challenge the existing wall lateral structural capacities. Detailed estimates of these wall capacities are recommended for development during later design phases for the CDI.

4.2 Conceptual Scenario for the Cells

The base case fill material for the cells is compacted bulk soil waste. This material is to be emplaced in lifts with adequate compaction of each lift.

For cube-container waste, containers can be stacked four high in the cells. This assumes that the cell internals are removed (i.e. empty cells are made available). It may be necessary to construct a load carrying intermediate floor above the cover blocks to limit loads on the containers in the cells. Such alternate methods are beyond the scope of this CSS study.

The overall fill process for the cells will follow general fill sequencing and constraints similar to those discussed in Sections 3.4 and 4.1.

4.3 Conceptual Scenario for the Ventilation Tunnel

For most of the length of the 221-U Building, the Ventilation Tunnel is 10 ft – 6 in. x 10 ft –6 in. in dimension. The base case fill material for the ventilation tunnel is compacted bulk soil waste. This material is to be emplaced in lifts with adequate compaction of each lift.

The alternate material for waste emplacements is cube containers. Container emplacement could be as dense as 2 packages wide by 1 package high in the Ventilation Tunnel.

The overall fill process for the Ventilation Tunnel will follow general fill sequencing and constraints similar to those discussed in Sections 3.4 and 4.1.

4.4 Conceptual Scenario for the Hot Pipe Trench

The Hot Pipe Trench is an 8 feet wide by 10 feet. high trench (See Figure 1.1) which length runs from the south end of the building ending at Cell 5. Current concepts include disposal of significant amounts of miscellaneous size-reduced equipment and materials in the Hot Pipe Trench. For this CSS, it is assumed that this will be the only type of material to be emplaced in the Hot Pipe Trench. This kind of emplacement will require that the fill be a flowable type grout to preclude formation of voids.

The overall fill process for the Hot Pipe Trench will follow general fill sequencing and constraints similar to those discussed in Sections 3.4 and 4.1.

4.5 Conceptual Scenario for the Rail Tunnel

The rail tunnel will provide the initial access way for moving materials into the building and on to the operating deck. As the waste placement on the operating deck initially approaches the rail

tunnel, it will be necessary to fill the rail tunnel to provide structural stability to that portion of the building. As a consequence, of the need to fill the rail tunnel, alternate access ways into the building will be needed for waste placement on, and above the operating deck. The recommended option is to cut access ways through the endwalls.

The base case fill material for the rail tunnel is compacted bulk soil waste. This material is to be emplaced in lifts with adequate compaction of each lift. For cube-container waste, containers can be stacked in an array three wide by four high in the rail tunnel. Because of limited clearance, the fill for the interstices between the containers will probably be flowable grout.

The overall fill process for the rail tunnel will follow general fill sequencing and constraints similar to those discussed in Sections 3.4 and 4.1

4.6 Conceptual Scenario for the Space Above the Operating Deck

Vertical space to be filled above the Operating Deck extends upward 39.2 ft high to the underside of the roof. The existing service cranes in Building 221-U can be used to place fill until the fill reaches an elevation a few feet below the crane hook. If access ways are needed to place fill above this elevation, access ways can be constructed. The simplest locations for new access ways with minimum impact on the existing building structural systems would be access ways cut in the unreinforced endwalls. In general, it is undesirable to cut any of the existing reinforcing steel in the Building 221-U structures.

For the base case fill material (compacted bulk soil waste), the soil will be placed in a series of lifts because of the large height of the space to be filled, and to accomplish balancing the loads on opposite sides of the walls. Also, when the headroom becomes too low for effective compaction, the last fill placement will be grout pours. A detailed analysis will have to be conducted to evaluate acceptable locations for cutting access ways into the building roof to place the final grout lifts.

The alternate material for waste emplacements is cube containers. Space above the operating deck can accommodate a maximum stack of six to seven levels of containers inside the process area, above the cover blocks. This option includes a one foot thick layer of fill above each layer of waste packages. This arrangement is consistent with the current conceptual design range for ILAW package placements (Burbank, 1999). The ILAW waste packages have sufficient structural capacity to carry the loads from a surface barrier above the building roof, the building roof, and a seven package high column of waste packages and fill to the canyon deck structure. Small size-reduced contaminated equipment could be placed in each soil lift provided that the size is no more than about one-half of the lift depth, and the equipment has been collapsed to eliminate internal void spaces. Larger intact equipment could also be placed in the soil fill as long as void spaces are eliminated. Internal spaces in the equipment would have to be solid grouted.

The overall fill process for the space above the operating deck will follow general fill sequencing and constraints similar to those discussed in Sections 3.4 and 4.1.

4.7 Conceptual Scenario for the Space In-and-Above the Crane Way

Vertical space to be filled in-and-above the Crane Way extends upward ~23.7 ft high to the underside of the roof.

For the base case fill material (compacted bulk soil waste), the soil will be placed in a series of lifts because of the large height of the space to be filled, and to accomplish balancing the loads on opposite sides of the walls. When the headroom becomes too low for effective compaction, the last fill placement will be grout pours. A detailed analysis will have to be conducted during the next CDI project phase to evaluate acceptable locations for cutting access ways into the building roof to place the final grout lifts.

The alternate material for waste emplacements is cube containers. Vertical space to be filled above the crane way floor deck would allow for a maximum stack of four levels of containers. This option includes a one foot thick layer of fill above each layer of waste containers, and fill in the interstices between the containers.

The overall fill process for the space in-and- above the crane way will follow general fill sequencing and constraints similar to those discussed in Sections 3.4 and 4.1.

4.8 Conceptual Scenario for the External Fill

Both base case and alternate fill materials can be used in external fills outside the building envelope. The Master Structural Strategy discussed in Section 1.0 will have to be implemented to protect the building integrity during waste emplacement. In general, external fill placement will have to be coordinated with waste emplacement inside the building to balance loads across the existing walls. Details of the external waste emplacement are beyond the scope of this study.

4.9 Other Fill Scenarios Considered but not Developed

Several other scenarios were considered while developing the above scenarios, but were eliminated from further consideration earlier in the process of preparing this report. These scenarios focused on waste emplacement inside the building before waste emplacement outside the building. Some of the structural scenarios considered were:

- Soil fill with tension ties across the building.
- Self-supporting reinforced earth fill inside the building.
- Self-supporting cube containers with reinforcing steel in the grout matrix around the containers.

Each of these candidate scenarios was rejected for reasons ranging from basic structural inadequacy to excessive structural complexity. For example: Self-supporting reinforced earth fill was rejected because of the fundamental incompatibility between the deformation characteristics (modulus of deformation) of this material and the reinforced concrete building structure. Another option considered was fills with soils with spaced tension ties (about 10 feet

on center vertically and horizontally spanning the entire width of the building from outside wall to outside wall). This option was rejected because the existing walls cannot carry horizontal seismic loads from the proposed fills to the foundations in a structural flexural mode. They lack the structural flexural capacity. Calculations were not developed for these scenarios.

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DRAWINGS:

Drawing No. 69334, Rev. 81, Building 221 T-U-B Sections 1 & 2 Concrete Plans and Details.

Drawing No. W69566, Rev. 73, "Bldg 221 T-U-B Std. Sects. Concrete Sections and Details."

Drawing No. 70080, Rev. 8, Building 221 T-U-B Std. Sects. 10, 11, 12, & 13 Concrete Elev of Wall Adjoining Bldg # 271.

APPENDIX A -- LOADS CONSIDERED FOR CONCEPTUAL SCENARIOS

INTRODUCTION

The following fundamental engineering parameters were examined while developing the conceptual scenarios to assure that the Building 221-U scenarios are achievable for CDI Alternative 4:

- Gravity loads with fill
- Foundation bearing
- Gallery floor capacities
- Gallery wall capacities
- Process Area exterior wall capacity
- Short and long-term settlements

Scoping assessments were performed for each of the above parameters to guide the scenario development. Structural calculations for this study are at a rough-order-of-magnitude (ROM) level, and are furnished in Appendix B for conceptual feasibility scoping purposes only.

A1.1 Gravity Loads with Fill

Gravity loads attributable to a given area are basically the product of the unit weight of the fill material times the height of the fill. Simple one-dimensional models can be used to provide upper-bound estimates for the anticipated floor loads due to filling the building. Floor loadings are calculated as the clear height of the space times the unit weight of the proposed fill materials. These evaluations are furnished in Table A-1.

A1.2 Foundation Bearing Capacity

The original foundation pressure limits for design of Building 221-U are shown on drawing W69334, Rev. 81. An allowable foundation load of 8000 lb/ft^2 is listed in a note on the drawing. This number was based on results from New York City Plate Bearing Tests conducted at the sites where the major Hanford process structures were built. The acceptance criteria for these tests was a maximum settlement of 10/32 of an inch (DuPont, 1945, Vol. III, p.638) for an allowable bearing load of 8000 lb/ft^2. The tests were described (Ibid., p. 816) as follows:

> "Wooden test tables, having bearing plates twelve inches square and platforms six feet square, were set at a minimum of four to five feet below ground level in undisturbed soil. Additional test tables were set up in the deep excavations of buildings such as 221 and 241. Weight was added at regular twenty-four hour intervals in 2,000 pound increments over a period of ten days. From the settlement readings made daily, settlement diagrams were plotted for each test to ascertain the actual soil-bearing capacity."

Table A-1. Floor Loads due to Fill

Building Space	Ceiling Height h, (ft)	Unit Weights/Floor Loads w, (lbf/ft^3) FL, (lbf/ft^2)		
		Base Case	Grout	Grout + ILAW
		FL = h x w	FL = h x w	FL = h x w
Electrical Gallery	13.5	120	150	*
		1620	2025	2730*
Piping Gallery	13.0	120	150	*
		1560	1950	2660*
Operating Gallery	12.5	120	150	*
		1500	1875	2580*
Ventilation Tunnel	10.5	120	150	*
		1260	1575	1930*
Hot Pipe Trench	6.0	120	150	N/A
		720	900	
Cells	22.0	120	150	N/A
		2640	3300	
Operating Deck	39.2	120	150	*
		4700	5880	8360*
Crane Way	23.7	120	150	*
		2840	3560	4970*

* See fill load calculations in Appendix B.

Results from one of the two plate bearing tests made at the Building 221-U construction site are included in the Appendix B calculations. This information is from an old soil report (Udine, 1956). Two borehole drill logs from holes located near the plate bearing test site are also included. From information in these records, the plate bearing test was conducted at an elevation of around 689 feet. Core Drill hole No. 2 was proximate to the plate bearing test with a ground elevation of 703 feet. Therefore the plate bearing test location was about 14 feet below local grade. Soils at this location are described as "Coarse gravel and sand – some cobbles" in the drill log. After testing for a period of 240 hours, the plate bearing test was terminated at a load of 18,000 pounds with a total settlement of 8/32 inch. The test did not fail the soil under the plate. Plate bearing test data show that the soil at Building 221-U exceeds the minimum design strength specified for the building in the original design of 8000 lb/ft^2, but doesn't indicate what the failure load may be.

A geotechnical site investigation was conducted for a proposed new tank farm, W-236A, with two double-shell tanks across the street from Building 221-U in 1995 (Shannon & Wilson, 1995). Results of this investigation can be used to estimate the probable soil bearing capacity for the soils below the foundations of Building 221-U. This report recommended a design value of 37 degrees for the internal angle of friction for the undisturbed local soils in Table 8-1 which is included in Appendix B. A standard estimate for bearing capacity of footings on soils is known as the Buisman-Terzaghi equation as discussed by Vesic in Winterkorn and Fang (1975):

$$q_0 = cN_c + qN_q + 1/2\gamma BN_\gamma \tag{A.1}$$

where q_o is the average ultimate bearing stress over the footing contact area A, q is the overburden or surcharge pressure at the foundation base and γ is the unit weight of the soil. Three bearing capacity factors N_c, N_q, and N_γ represent the effects due to soil cohesion, surface loading, and soil unit weight. These factors are all a function of the angle of internal friction ϕ for the soil. B is the foundation width. Since we are dealing primarily with deep deposits of sand and gravel on the Hanford Site, the term relating to cohesion drops out of the expressions. Maximum allowable soil bearing stress q_a is commonly calculated using a minimum factor of safety (F.S.) of 3.0.

$$q_a = q_o / 3.0 \tag{A.2}$$

The fill scenario that produces the highest bearing stress with the lowest F.S. on bearing capacity is the case where the galleries, ventilation tunnel, and process area above the cover blocks is filled with ILAW waste packages and grout. This scenario includes placement of reinforcing steel in the waste package-grout matrix so that simultaneous external fill is not required to balance loads across the existing building walls. For this scenario under gravity loading only, the average soil bearing load is 8.1 tons per square foot and the F.S. against soil bearing failure is 18.2. Simultaneous external fill would increase the ultimate bearing capacity through the increase in the term qN_q.

Soil bearing capacity for this structural scenario was also evaluated for a set of equivalent static lateral seismic loadings ranging from 0.2W, where W is the weight of the building and fill, to 0.3W and 0.4W, acting horizontally through the center of gravity of the filled building. These values span most of the range of seismic design requirements for the current Uniform Building Code. Load eccentricity was addressed using correction factors for footing effective width and inclination factors as discussed by Vesic (Ibid., pp. 129-131). The overall F.S. against punching shear at the toe of the foundation ranged from 8.1 for gravity loads combined with a static lateral load of 0.2W, to 3.1 for gravity loads combined with a static lateral load of 0.4W.

Soil bearing capacity below the existing Building 221-U foundations is adequate for the structural scenarios associated with Alternative 4 for the FS. Large factors of safety for soil bearing capacity under gravity loads are available for the case where the building is filled with a self-supporting system of ILAW waste packages, grout, and reinforcing steel. A simplified seismic lateral load analysis shows that adequate factors of safety are available to preclude punching shear failure in the soils at the toe of the building foundation under UBC level seismic loadings. The minimum F.S. is 3.1 for gravity loads combined with a static lateral load of 0.4W. Scenarios where simultaneous fill occurs inside and outside Building 221-U will all have higher factors of safety on soil bearing capacity than the scenario discussed herein.

A1.3 Gallery Floor Structural Capacities

Ceiling heights for the Building 221-U interior spaces are taken from Drawing No. W69566, Rev. 73, "Bldg 221 T-U-B Std. Sects. Concrete Sections and Details." Inspection of anticipated

floor loads listed in Table A-1 for areas such as the Building 221-U galleries, the Crane Way, and the Process Area above the cover blocks shows that the floor loads exceed original design capacities by factors of 4 to 5 or more. For example, assuming the fill is ILAW and grout, this ratio for the Piping Gallery is 2660 $lb/ft^2 \div 350$ $lb/ft^2 = 7.6$, and for the Operating Gallery the ratio computes as 2580 $lb/ft^2 \div 400$ $lb/ft^2 = 6.5$. These ratios far exceed the original design margins for the concrete floor structures of Building 221-U and the design margins used in contemporary building codes.

Structural design margins for floor dead loads are typically in the range of 1.4 to 1.5 and design margins for floor live loads are typically 1.7 to 1.9. Design margins depend on the design medium such as steel, concrete, or wood products, and vary depending on the implementing design code. The large overload ratios on the floor slabs point out the need for careful installation of engineered fills in the building spaces to support the final loads for FS Alternative 4 after closure.

Floor slabs in the piping and operating galleries will fail in shear when the fill is placed. These floor slabs also serve as tension ties to carry lateral loads imposed on the galleries outside building wall back into the shield wall. The floor failures will disrupt these tension ties which will reduce the capacity of the gallery outside wall to carry lateral loads. Lateral loads to be considered are those caused by the fill process itself, and lateral loads imposed on the outside wall by wind and earthquake during the waste emplacement operational period as discussed in the next section.

A1.4 Gallery Wall Lateral Structural Capacity

Waste emplacement inside the Operating, Piping, and Electrical Galleries introduces two lateral loads for which the gallery exterior walls were not designed. Lateral soil loads due to waste soil emplacement and compaction and lateral seismic loads imposed on the external wall from self-weight and the weight of the fill.

The first load is the lateral load produced by placing and compacting the base case bulk soil waste. Shannon & Wilson provided lateral earth pressure estimates in Chapter 11of that soil report, which have been used to estimate this lateral load. The at-rest earth pressure coefficient, K_o, was taken as 0.36 and additional load due to compaction close to the wall was included. The resulting lateral load on a one foot wide strip of wall spanning vertically between the gallery floors varied linearly from 410 lb/ft at the ceiling to 708 lb/ft at the floor.

The second component of the lateral load is the inertial load of the exterior wall self-weight and the fill. For the scoping study, it was assumed that the fill weight was that of the cube-containers (227 lb/ft^3) to maximize the load, and the tributary width contributing to the load is the 14 ft – 0 in. width of the gallery. The gallery sidewall is modeled as the wall of a storage bin. Assuming a 0.30 g lateral acceleration, the lateral load on the one foot wide strip is 1088 lb/ft.

As noted in Section A1.3, the fill loads will fail the floor slabs in the Piping and Operating galleries sometime during the fill process. This failure precludes their use as tension drag struts delivering load from the exterior wall to the Shield Wall. Therefore, the exterior fill on the

outside of the building will have to be brought up to the elevation of the floor slab on which the fill is to be placed *before* placing the fill. The exterior fill provides the lateral support for the wall after the floor slab fails.

Simple scoping calculations were done to evaluate how much differential elevation can be allowed between fills placed inside the galleries and the exterior fill placed around the outside of Building 221-U. For the base case fill, bulk soil waste, the piping and operating galleries can be filled for their full height before having to bring the exterior fill up to the next floor level. The allowable difference in fill elevation across the gallery walls is about 14 feet. For the alternate fill, cube-containers and flowable grout, the fill weighs about twice as much as the compacted soil base case. The maximum allowable difference in fill elevation across the gallery walls is about 7 feet when placing fill consisting of cube-containers and grout. The limiting structural elements that control these differential elevations are the floors of the Operations and Piping Galleries.

A1.5 Operating Deck Outside Wall Lateral Structural Capacity

Waste emplacement above the cover blocks on the Operating Deck introduces two lateral loads for which the Operating Deck exterior walls were not designed. Lateral soil loads due to waste soil emplacement and compaction and lateral seismic loads imposed on the external wall from self-weight and the weight of the fill.

The first load is the lateral load produced by placing and compacting the base case bulk soil waste. Shannon & Wilson provided lateral earth pressure estimates in Chapter 11 of their soil report which have been used to estimate this lateral load. The at-rest earth pressure coefficient, K_o, was taken as 0.36 and additional load due to compaction close to the wall was included. The resulting lateral load on a one foot wide strip of wall which is filled to an elevation 14 feet above the outside fill varied linearly from 410 lb/ft at the top of the fill to 708 lb/ft at the bottom of the current lift of fill.

The second component of the lateral load is the inertial load of the exterior wall self-weight and the fill. For the Operating Deck exterior wall it was assumed that the fill weight was that of the bulk waste soil (115 lb/ft^3). An additional uniform dynamic incremental load of ten times the height of the fill, 140 lb/ft^2, was used to represent the inertial load from the fill in accordance with Shannon & Wilson recommendations. The lateral component for the wall self-weight was taken as 0.30 g times the wall weight uniformly distributed over the 14 foot height of the fill.

These loads impose a cantilever bending moment of about 1400 kips-in (including load factor) on the exterior wall during each lift. When the external fill is brought up to match the elevation of the internal fill, the loads are again balanced across the wall. Given the height of the building roof, it will take 3 lifts to fill the space. The maximum moment during the first lift occurs at the level of the deck and cover blocks. During the second lift, the maximum moment occurs at the base of the second lift which is 14 feet elevation on the wall above the deck. The added load is around 25 percent of the reserve bending strength at each location which is described in Chapter 7 of the Winkel building seismic analysis. These loads are well within the Operations Deck exterior wall moment capacities.

The evaluation of the external gallery walls in Section A1.4 resulted in a recommendation that unbalanced fill heights on the inside of the gallery walls be limited to no more than 14 feet of bulk waste soil, and 7 feet of cube-containers with flowable grout in the interstices between the containers. A similar recommendation for limitations on the unbalanced height of fill is also appropriate for fill on the Operating Deck.

There are residual uncertainties involved in imposing unbalanced fill loads on this exterior wall. The vertical and lateral load distribution from the roof downward into the building walls depends on the vertical and lateral stiffness of the support walls. Lateral deflections of the support walls due to unbalanced fill loads will impact the percentage of the lateral seismic loads taken by each wall at the elevation of the crane rail. It is recommended that this interaction be quantified in detail in future structural studies.

A1.6 Short and Long-term Settlements

A1.6.1 Settlement Without External Fill

A one-dimensional settlement analysis was made for the structural scenario where the canyon is filled internally before it is filled around the exterior. The method used was developed by Schmertmann (1970) with subsequent modifications reported in Schmertmann et al. (1978) as discussed in Fang (1985, Chapter 5). This is an empirical procedure which is expressed as:

$$s_i = C_1 C_2 \Delta q \sum_{i=1}^{n} \left(\frac{I_{zp}}{E} \right)_i \Delta z_i \qquad \text{(A.3)}$$

where: Δq = net load intensity at the foundation depth
I_{zp} = strain influence factor from the distribution for plane strain.
E = appropriate Young's modulus at the middle of the ith layer of thickness Δz_i
C_1, C_2 = correction factors

Schmertmann's procedure assumes that strain contributions to settlement occur in a soil volume that has a depth of four times the foundation width below the building foundation. One of the two correction factors was used in the analysis. Factor C_1 is a modification that increases the maximum strain influence factor based on the ratio of the in-situ overburden stress at the depth of the foundation to the net foundation pressure. The second factor C_2 is intended to address time dependent settlement increases for foundations on cohesionless soils. This factor was not included because it is generally agreed that settlement on sands and gravels with little organic content occurs immediately on loading. This is consistent with recommendations in the Shannon and Wilson soil report for the proposed W-236 facility in the 200 West Area.

Values for the in-situ Young's Modulus as a function of depth were taken from Table 8-1 in the soil report which is titled "Summary of Static Soil Properties." Building 221-U is located immediately northwest of the intersection of Beloit Avenue and 16th Street in the 200 West Area. The site location for the proposedW-236 facility was described as "east of Beloit Avenue and

north of 13th Street on the southeast side of the 200 West Area complex." This location is diagonally across the intersection from Building 221-U; therefore, the soil investigation has considerable applicability to the Building 221-U site.

A maximum settlement of 2.0 inches is predicted for Building 221-U when filled including loads from the surface barrier. Settlement will be immediate, as soon as the fill is placed, and there should be little additional long-term settlement. The figure of 2.0 inches is larger than the 1/2 inch gaps that were designed into the expansion joints in Building 221-U. Therefore, fill of the building will have to be done in layers that extend the entire length of the building to preclude fracturing between sections at the horizontal portions of the expansion joints. These areas are located in the roof and the main Shield Wall above the operating deck.

A1.6.2 Settlement With External Fill

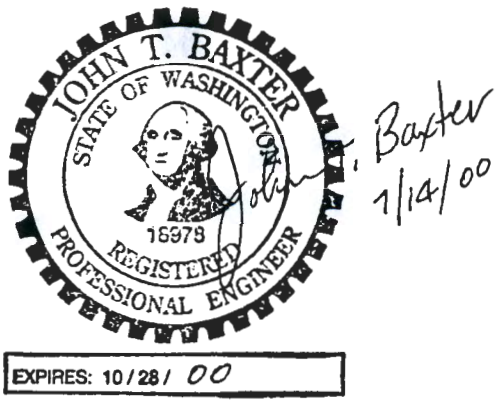
Settlement of the entire entombment complex as shown in Figure 1-2, can be bounded for feasibility purposes without additional calculations using the following logic. Two factors are considered: a) instantaneous elastic deformations in the foundation soils below the filled building and the surrounding fill, plus b) potential inelastic post-closure consolidation in the fill placed around the building. The elastic settlement below the building and surrounding fill is estimated based on the author's engineering judgement to be two times the 2.0 inch value discussed in the previous paragraph. Additional post-closure consolidation in the fill placed around the outside of building can be bounded by the consolidation estimate for the ERDF fill. This estimate is reported as 2.3 inches maximum (Casbon, 1995, pp. T97 – T108). Therefore, a conservative bounding estimate for the total settlement can be summed up to be in the range of 4 to 6 inches adjacent to the Building 221-U walls.

References:

For References see main document.

APPENDIX B -- CALCULATION PACKAGE CDI-C-001

Content: Index and 21 pages.



Civil/Structural

HNF-6325, Rev. 0
KW

Fluor Federal Services	CALCULATION IDENTIFICATION AND INDEX	Calc. No.: CDI-C-001 T.O./Job No.: 65100241-A0-13 Date: 5/18/00 Sht i of i

Status and description of the attached Calculation Sheets.

Discipline: STRUCTURAL

Project No. & Title: Canyon Disposition Initiative

Calculations: Selected evaluations of basic design parameters to develop fill scenarios for Building 221-U.

These calculations apply to:

Drawing No. _____ Rev. No. _____

Drawing No. _____ Rev. No. _____

Other (Study, CDR) HNF-6325, Rev. 0, 221-U Conceptual Structural Study (CSS) for the Canyon Disposition Initiative (CDI) Rev. No. 0

The status of these calculations is:

☒ Final Calculations

☐ Void Calculations (reason voided): _____

Were calculations incorporated into the final drawings? ☐ Yes ☒ No

Were calculations verified by independent "check" calculations? ☐ Yes ☒ No

Original and Revised Calculation Approvals

	Rev. 0 Signature/Date	Rev. 1 Signature/Date	Rev. 2 Signature/Date
Originator	J.T. Baxter 5/18/00		
Checked By	George J. Zyp 5/19/00		
Approved By	J.T. Baxter 5/19/00		
Checked Against Approved Vendor Data			

INDEX

Calculation Sheet Page No.	Description
1	Introduction, Objective, Results and References
2	Backfill loads
3-5	Foundation bearing for gravity and seismic lateral loads
6-10	Foundation settlement analysis
10-17	Evaluate exterior gallery walls for backfill and lateral seismic loads
18-19	Variation in lateral seismic acceleration over building height
19-21	Evaluate Operating Deck exterior wall for backfill and lateral seismic loads

FLUOR DANIEL NORTHWEST, INC.

Calc. No. CDI-C-001
Revision 0
Page No. 1 of 21

DESIGN ANALYSIS

Client Canyon Disposition Initiative (CDI)	WO/Job No. 65100241 A0 13		
Subject Scoping Calcs, Structural Scenarios	Date 5/18/00	By J.T. Baxter	
	Checked	By	
Location Bldg 221-U	Revised	By	

INTRODUCTION: The purpose for these calculations is to support a Canyon Disposition Initiative (CDI) report on structural backfill scenarios for options three and four for the CDI Feasibility Study (FS).

OBJECTIVE: These calculations are intended to identify the constraints that the existing building structural systems will impose on the backfill process. One of the goals of the study is to identify the minimum set of structural modifications that will allow the building to be used for the proposed ~~mission~~ F.S. Alternative 4a
JB

Key areas to be reviewed in developing the scenarios include the range of anticipated backfill loadings for each internal floor area, the foundation bearing capacity and associated factors of safety for gravity and the combination of gravity and seismic lateral loadings. A rough estimate is made for anticipated settlement due to the heaviest backfill load proposed with delayed placement of external backfill around building 221-U. Building exterior walls are evaluated for their ability to resist unbalanced backfill loads including effects of earthquake ground motion.

RESULTS: See HNF-6325, Rev. 0, *221-U Conceptual Structural Study (CSS) for the Canyon Disposition Initiative (CDI).*

REFERENCES:

1. Drawing W-69566, Building 221 T-U-B Std. Sections Concrete Sections and Details.
2. Shannon & Wilson, 1995, "Geotechnical Investigation KEH W-236A, Multi-Function Waste Tank Facility; 200 West Area, Hanford Site Richland, Washington," Volume 1, Project H-1070-50, Shannon & Wilson, Inc. Seattle Washington for ICF Kaiser Hanford Company, Richland, Washington, June, 1995 (Document No. W236A-TI-MT5).
3. Winterkorn & Fang, 1975, *Foundation Engineering Handbook*, 1st Edition, Van Nostrand Rheinhold, New York, N.Y.
4. Fang, H. Y., 1985, *Foundation Engineering Handbook*, 2nd Edition, Van Nostrand Rheinhold, New York, N.Y.
5. Schmertmann, J. H., (1970), Static Cone to compute static settlement over sand, *Journal of the Soil Mechanics and Foundation Division, ASCE*, **96**, No. SM3, pp. 1011-1043.
6. Schmertmann, J. H., (1978), Improved strain influence factor diagrams, *Journal of the Geotechnical Engineering Division, ASCE*, **104**, No. GT8, pp. 1131-1135.
7. Baxter, J. T., 1999, *221-U Facility Concrete and Reinforcing Steel Evaluations, Canyon Disposition Initiative (CDI)*, Fluor Daniel Northwest, Inc., Richland, Washington.
8. Udine, G., 1956, *HAPO Soil Information*, HW-50239, Architectural & Civil Design, Design Engineering Operation, Construction Engineering Operation, Hanford Atomic Products Operation, General Electric, Camp Hanford, Washington.

FLUOR DANIEL NORTHWEST, INC.

DESIGN ANALYSIS

Calc. No. CDI-C-001
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Client Canyon Disposition Initiative (CDI) WO/Job No. 65100241 AO 13
Subject Scoping Calcs, Structural Scenarios
Date 2/22/00 By J.T. Baxter
Checked 5/08/00 By George T. Ryu
Revised By
Location BLDG 221-U

Backfill Loads: 1-D model, assumes unit weight smeared in layers

W, base case 120 lbf/ft³, grout 150 lbf/ft³, ILAW 227 lbf/ft³
ILAW = 1.4m (4.6 ft) cube, 10,000 kg weight
$W = 2.205 \times 10{,}000 / 4.6^3 \text{ ft}^3 = 227 \text{ lbf/ft}^3$

Electrical Gallery

Grout + ILAW $h_g = 13.5 - 2 \times 4.6' = 4.3'$ $h_{ILAW} = 2 \times 4.6' = 9.2'$

$FL = 4.3 \times 150 + 9.2 \times 227 = 2730 \text{ lbf/ft}^2$

Piping Gallery

Grout + ILAW $h_g = 13.0 - 2 \times 4.6 = 3.8'$ $h_{ILAW} = 2 \times 4.6 = 9.2'$

$FL = 3.8 \times 150 + 9.2 \times 227 = 2660 \text{ lbf/ft}^2$

Operating Gallery

Grout + ILAW $h_g = 12.0 - 2 \times 4.6 = 2.8'$ $h_{ILAW} = 2 \times 4.6' = 9.2'$

$FL = 2.8 \times 150 + 9.2 \times 227 = 2508 \text{ lbf/ft}^2$

Ventilation Tunnel

Grout + ILAW $h_g = 10.5' - 4.6' = 5.9'$ $h_{ILAW} = 4.6'$

$FL = 5.9 \times 150 + 4.6 \times 227 = 1930 \text{ lbf/ft}^2$

Operating Deck - 7 package high

Grout + ILAW $h_g = 39.2' - 7 \times 4.6' = 7.0'$ $h_{ILAW} = 7 \times 4.6' = 32.2'$

$FL = 7.0 \times 150 + 32.2 \times 227 = 8360 \text{ lb/ft}^2$

Crane Way - 4 packages high

Grout + ILAW $h_g = 23.7' - 4 \times 4.6' = 5.30'$ $h_{ILAW} = 4 \times 4.6' = 18.4'$

$FL = 5.30 \times 150 + 18.4 \times 227 = 4970 \text{ lb/ft}^2$

Process Cells - 4 packages high

Grout & ILAW $(28'-0 - 6'-0) - 4 \times 4.6 = 3.60'$ $h_{ILAW} = 4 \times 4.6 = 18.4'$

$FL = 3.6 \times 150 + 18.4 \times 227 = 4716 \text{ lb/ft}^2$

Client CDI	WO/Job No.		
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Location BLDG 221-U	Revised	By	

Foundation Bearing Check

Use soils data from:

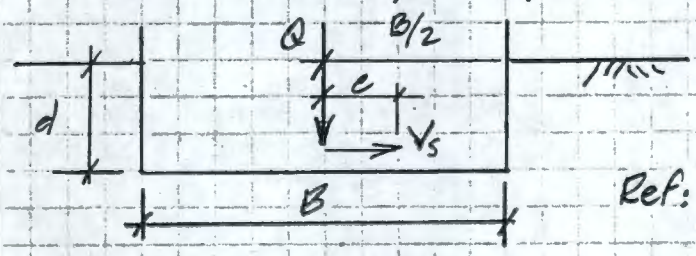
Shannon & Wilson, 1995, "Geotechnical Investigation KEH W-236A, Multi-Function Waste Tank Facility; 200 West Area, Hanford Site Richland, Washington," Volume 1, Project H-1070-50, Shannon & Wilson, Inc. Seattle, Washington for ICFKaiser Hanford Company, Richland, Washington, June, 1995 (Document No. W236A-T1-MT5)

Table 8-1 included in this calc. package

Assume 221-U is a strip footing 66'-2" wide by 800 ft long

B = 66.17 ft.

Minimum embedment occurs at Section 20 and is ~ 12 ft. of soil above the lower edge of the 6 ft. mat foundation. See Figure 2-6, HNF 3915, Rev. 0, "221-U Facility Concrete & Reinforcing Steel Evaluations, Canyon Disposition Initiative" d = 12 ft fill



From Table 8-1

$\phi = 37°$ (design value)

Ref: Foundation Engineering Handbook, 1st Ed. Winterkorn & Fang, 1975 p. 127

Net ultimate soil pressure

$\phi_{design} = 37°$

$$q_d = \frac{B}{2}\gamma N_\gamma + \gamma d (N_q - 1)$$

$N_q = 42.9$ $N_\gamma = 66.2$

$\gamma = 110$ pcf

$$= \frac{66.17'}{2} \times 110\text{ pcf} \times 66.2$$

$$+ 110\text{ pcf} \times 12\text{ ft} \times (42.9 - 1)$$

$$= 2.41 \times 10^5\ \text{lb/ft}^2 + 5.53 \times 10^4\ \text{lb/ft}^2$$

$$= 2.96 \times 10^5\ \text{lb/ft}^2$$

Allowable Soil Bearing $= q_d/3.0 = 2.96 \times 10^5\ \text{lb/ft}^2/3 = 98{,}700\ \text{lb/ft}^2$

Client CDI		WO/Job No. 65100241 A0 13	
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		Checked 5/08/00	By George J. Lyn
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Max. soil load occurs below process cells containing ILAW & Grout

$w_{conc.} = 150\ lb/ft^3 \times (6' + 6' + 4')$ = 2400 lb/ft²
(6' mat, 6' cell cover blocks, 4' roof)

$w_{ILAW+grout} = 4716\ lb/ft^2 + 8360\ lb/ft^2 = 13{,}076\ lb/ft^2$
(4716 lb/ft² cells; 8360 lb/ft² operating deck)

$w_{surface\ barrier} = 15\ ft \times 150\ lb/ft^3 = 2250\ lb/ft^2$
(assume concrete)

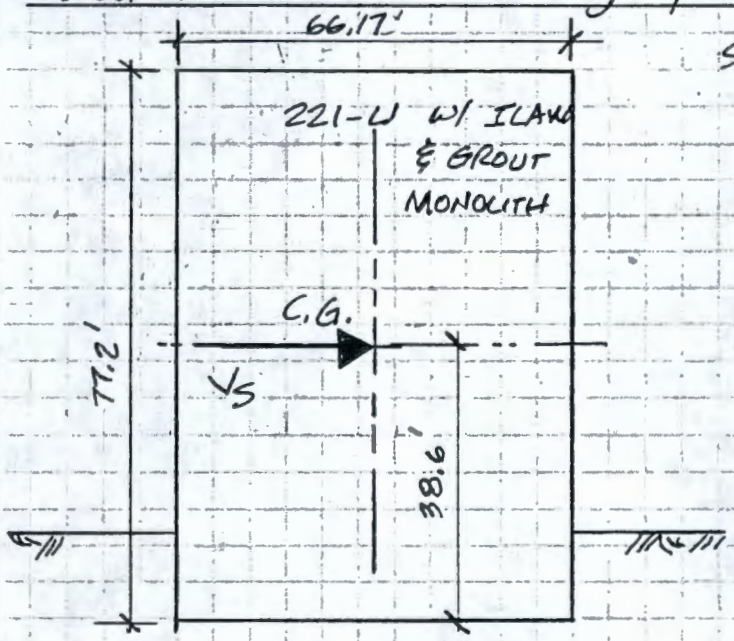
$w_{Max.} = 2400 + 13{,}076 + 2250 = 17{,}726\ lb/ft^2 \ll$ bearing allow. $= 98{,}700\ lb/ft^2$

$q = 17{,}726\ lb/ft^2 - 110\ pcf \times 12\ ft = 16{,}406\ lb/ft^2 = 8.20\ ton/ft^2$

Check Overall F.S.

$F.S. = (98{,}700/16{,}406) \times 3 = 18.0$

Effect of lateral loads on bearing capacity [Winterkorn & Fang, pg 129]



Static seismic loads

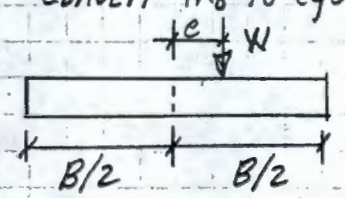
$V_S = 0.2W,\ 0.3W,\ 0.4W$

$M_o^{0.2} = 0.2W \times 38.6' = 7.72\ W\ Ft$

$M_o^{0.3} = 0.3W \times 38.6' = 11.6\ W\ Ft$

$M_o^{0.4} = 0.4W \times 38.6' = 15.4\ W\ Ft$

Convert M_o to equiv. eccentricity



$M_o = e \times W \quad e = M_o/W$

$e^{0.2} = 7.72\ Ft \quad e^{0.3} = 11.6\ Ft \quad e^{0.4} = 15.4\ Ft$

DESIGN ANALYSIS

Client CDI	WO/Job No. 65100241, AO 13		
Subject Scoping Calcs, structural Scenarios	Date 2/24/00	By J. T. Baxter	
	Checked 5/8/00	By Gregg J. Lyon	
Location BLDG 221-U	Revised	By	

Adjustments for eccentricity

$B' = B - 2e$ for $e \leq B/4 = 66.17'/4 = 16.5'$

$B'_{0.2} = 66.17' - 2(7.72') = 50.7'$
$B'_{0.3} = 66.17' - 2(11.6') = 43.0'$
$B'_{0.4} = 66.17' - 2(15.4') = 35.4'$

$$\zeta_{qi} = \left[1 - \frac{P}{Q + \cancel{B'L'c\cot\phi}^{\,0}}\right]^{m} \qquad m_B = \frac{2 + B/L}{1 + B/L} = \frac{2 + 66.2/800}{1 + 66.2/800} = 1.92$$

$$\zeta_{\gamma i} = \left[1 - \frac{P}{Q + \cancel{B'L'c\cot\phi}^{\,0}}\right]^{m+1} \qquad c = 0 \text{ for both expressions}$$

$$\zeta_{qi} = \left[1 - \frac{V_s}{W}\right]^{1.92}$$

$$\zeta_{\gamma i} = \left[1 - \frac{V_s}{W}\right]^{2.92}$$

V_s	ζ_{qi}	$\zeta_{\gamma i}$
0.2 W	0.65	0.52
0.3W	0.50	0.35
0.4W	0.375	0.23

$$q_d = \frac{B'}{2}\gamma N_\gamma \zeta_{\gamma i} + \gamma d (N_q - 1)\zeta_{qi}$$

$$q_d = 2.41\times10^5\ \text{lb/ft}^2 \times \frac{B'}{B} \times \zeta_{\gamma i} + 5.53\times10^4\ \text{lb/ft}^2 \times \zeta_{qi}$$

$$q_d^{0.2W} = \quad '' \quad \times \frac{50.7}{66.17} \times 0.52 + \quad '' \quad \times 0.65 = 132{,}000\ \text{lb/ft}^2$$

$$q_d^{0.3W} = \quad '' \quad \times \frac{43.0}{66.17} \times 0.35 + \quad '' \quad \times 0.50 = 82{,}500\ \text{lb/ft}^2$$

$$q_d^{0.4W} = \quad \times \frac{35.4}{66.17} \times 0.23 + \quad '' \quad \times 0.375 = 50{,}400\ \text{lb/ft}^2$$

$$q_d^{0.2W}/3 = 44{,}000\ \text{lb/ft}^2$$

$$q_d^{0.3W}/3 = 27{,}500\ \text{lb/ft}^2$$

$$q_d^{0.4W}/3 = 16{,}800\ \text{lb/ft}^2$$

Client CDI		WO/Job No. 65100241, A0 13	
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Location BLDG. 221-U		Revised	By

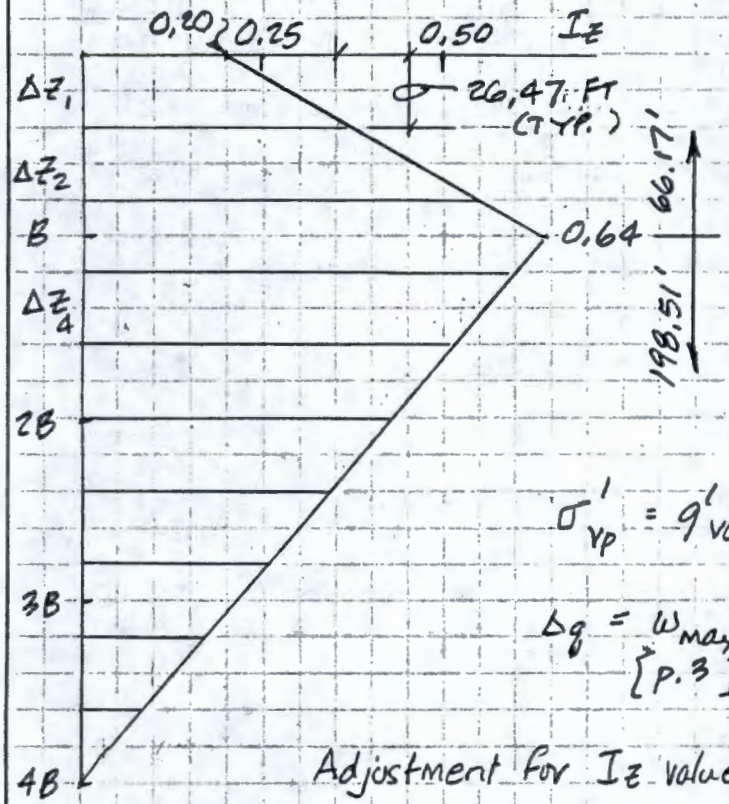
Ref: Foundation Engineering Handbook, 2nd Ed., [Fang, H.Y., 1985] Chapter 5, pp 179 through 184

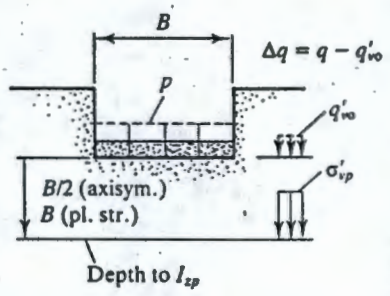
Estimate Settlement due to Canyon Backfill — Schmertmann method including 1978 modifications

$$S_i = C_1 C_2 \Delta q \sum_{i=1}^{10} \left(\frac{I_z}{E}\right)_i \Delta z_i$$

Foundation length $L = 800^{FT}$, width $B = 66.17^{FT}$, $L/B = 800/66.17 = 12.1$
∴ plane strain solution

Divide soil column below canyon into 10 zones for a depth of $4B = 4 \times 66.17^{FT} = 264.68^{FT}$
Determine I_{z_i} influence coefficients at mid-heights of zones.





$$\sigma'_{vp} = q'_{vo} + B\gamma = (12' \times 110\,pcf + 66.17' \times 110\,pcf)/2000$$
$$= 4.30\ tons/ft^2$$

$$\Delta q = \underset{[p.3]}{w_{max}} - q'_{vo} = (17{,}726\ lb/ft^2 - 12' \times 110\,pcf)/2000$$
$$= 8.20\ tons/ft^2$$

Adjustment for I_z values $I_{zp} = 0.5 + 0.1\sqrt{8.20/4.30}$
$= \underline{\underline{0.64}}$

In-Situ Modulus

Soil boring data and Plate Bearing Test data is available from the original siting for Building 221-U in 1943 [Udine, 1956]. Core drill hole No. 2 [pg. 6, this calc] was noted to be close to Plate Bearing Test No. 7 [pg. 7, this calc]. Ground elevation at drill hole No. 2 was 703.4'.

DESIGN ANALYSIS

Client	CDI	WO/Job No.	65100241 AO 13
Subject	Scoping Calcs, Structural Scenarios	Date	2/28/00 By J.T. Baxter
		Checked	3/08/00 By George T. Lyn
Location	BLDG 221-U	Revised	By

Ref: [Udine, 1956] p. 20

200 WEST AREA

BUILDING NO. 221 U

Core Drill Hole No. 1 N 38160 W 73500 Ground Elevation 700.7
Begin 6-30-43 Finished 7-3-43

Depth in Feet From	To	Description	Date
0'	5'	Loose yellow sand	6-30-43
5	20	Medium and coarse sand	6-30-43
5	20	Gravel - some cobbles - sand	6-30-43
20	30	Medium and coarse gravel -some cobbles-sand	7-1-43
30	41	Sand-partially cemented	7-1-43
41	58	Sand-medium and coarse graael-some cobbles	7-1-43
58	60	Cemented sand and gravel	7-1-43
60	90	Cemented sand and gravel	7-2-43

Completed 7-2-43 Final depth 90 Feet

Core Drill Hole No. 2 N38410 W 73285 Ground Elevation 703.4
Begin 6-30-43 Finished 7-3-43

Depth in Feet From	To	Description	Date
0	5	Loose yellow sand	6-30-43
5	25	Coarse gravel and sand-some cobbles	6-30-43
25	55	Coarse gravel and sand-some cobbles	7-1-43
55	70	Coarse gravel and sand-some cobbles	7-2-43
70	85	Cemented sand-small gravel-mud streaks	7-2-43
85	90	Cemented sand-small gravel-mud streaks	7-3-43

Completed 7-3-43, Final Depth 90 Feet

Note: Core Drill Hole No. 2 is near Test No. 7 location

The elevation of the test plate for plate test No. 7 was reported as 689.1 FT. The plate test was 703.4 FT - 689.1 FT = 14 FT below grade. USE 15 FT

Table 8-1 from [Shannon & Wilson, 1995] is included on p. 8 of this calc. Reported values for the Elastic Modulus, E, are reported in tons per ft.^{2} Assume data in Table 8-1 applies to 221-U site. Bottom of footing

Client CDI
WO/Job No. 65100241 A0 13
Subject Scoping Calcs, Structural Scenarios
Date 2/28/00 By J.T. Baxter
Checked 5/08/00 By Gerge J. Zyn
Location BLDG 221-U
Revised By

Ref: [Udine, 1956] p. 25

200 West Area

BUILDING NO. 221 U

Test No. 7 38307 W 73351 - Elevation of Tested Surface - 689.1
Bearing on Fine dry gravel and coarse black sand :
Date set 8-16-43
Wt. of Table - 400 Lbs. Initial Load 2000 Lbs.
Settlement immediately 1/32"

Load Lbs.	Time Hrs.	Settlement Before Load	After Load	Date
4000	24	2/32"	2/32"	8-17-43
6000	48	4/32"	4/32"	8-17-43
8000	72	5/32"	5/32"+	8-19-43
10000	96	7/32"	7/32"+	8-20-43
12000	120	8/32"	8/32"	8-21-43
	144	No Loading		8-22-43
14000	168	8/32"	8/32"	8-23-43
16000	192	8/32"	8/32"	8-24-43
18000	216	8/32"	8/32"+	8-25-43
	240	8/32"+	Terminated	8-26-43

of 221-U = -15 ft. Based on Table 8-1 data from pg. 6 Core Drill Hole #2

$E_{\Delta z_1} = (5' \times 720\ tsf + 21.5' \times 1440\ tsf)/26.5' = 1304\ tsf$

$E_{\Delta z_2} = [(30' - 21.5') \times 1440\ tsf + (26.5' - 8.5') \times 5760\ tsf]/26.5'$
$= 4375\ tsf$

$E_{\Delta z_3} - E_{\Delta z_{10}} = 5760\ tsf$

$C_1 = 1 - 0.5\left(\frac{\sigma'_{vo}}{\Delta q}\right) \geq 0.5 = 1 - 0.5\left(\frac{0.66\ tsf}{8.20\ tsf}\right) = 0.96$

C_2 - neglect $= 1.0$

$S_i = C_1 C_2 \Delta q \times \sum_{i=1}^{10}\left(\frac{I_z}{E}\right)_i \cdot \Delta z_i$
$= 0.96 \times \Delta q \times \sum\left(\frac{I_z}{E}\right)_i \Delta z_i$

$0.96 \times 8.20\ \frac{tons}{ft^2} = 7.87\ \frac{tons}{ft^2}$

TABLE 8-1

SUMMARY OF STATIC SOIL PROPERTIES

Material	Dry Density (pcf)	Moist Density (pcf)	Moisture Content (%)	Friction Angle (degree)	Elastic[3] Modulus (E) (tsf)	Bulk[3] Modulus (B) (ksi)	Horizontal Modulus of Subgrade Reaction (n_h) (pci)	Coefficient of Subgrade Reaction[3] (K_{s1}) (pci)	Vertical Modulus of Subgrade Reaction[5] (pci)
Stratum 2 (Sand with Gravel) El. 685 to 670	110	113	3	37	720	5	30	250	25[6.2] 100[6.3]
Stratum 3 (Sand) Surface Structures[1]	110	113 (design)	3	37 (design)					
El. 670 to 640		115 (mean)		41 (mean)	1,440	10	40	500	100[6.3]
El. 640 to 135 (rock)		±3 (range)		±4 (range)	5,760	40	50	2,000	
Stratum 3 (Sand) Tank Structures[2]	110	113 (design)	3	37 (design)					
El. 631 to 581		115 (mean)		41 (mean)	1,440	10	40	500	100[6.4]
El. 581 to 135 (rock)		±3 (range)		±4 (range)	5,760	40	50	2,000	
Backfill Depth below surface	116	130	12.5[4]	40					
0 - 20 ft.					720	5	30	250	25[6.2]
20 - 50 ft.					1,440	10	40	500	
>50					5,760	40	50	2,000	

Notes:
1 Parameters apply to soils beneath surface structures.
2 Parameters apply to soils beneath tank foundation.
3 Moduli have been adjusted to be compatible with strain levels anticipated with structural loadings.
4 Moisture content anticipated during field compaction of backfill.
5 Range of data (±) based on test data from this report and review of other reports in the 200 Areas.
6 Design moduli of subgrade reaction have been adjusted for anticipated foundation size and location.
6.1 Piping within 20 feet of ground surface.
6.2 Surface structures and floor slabs.
6.3 Exhaust stack mat.
6.4 Tank mat foundation.
7 Modulus values may be considered approximate with ± standard deviation values corresponding to ±25 percent.

6-6-95/TABLE.8-1/H1070-lkd/dgw

H-1070-50

Client CDI
WO/Job No. 65100241 A0 13
Subject Scoping Calcs, Structural Scenarios
Date 2/28/00 By P.T. Baxter
Checked 5/08/00 By George T. Zyw
Location BLDG 221-U
Revised By

ELEV. -15 FT

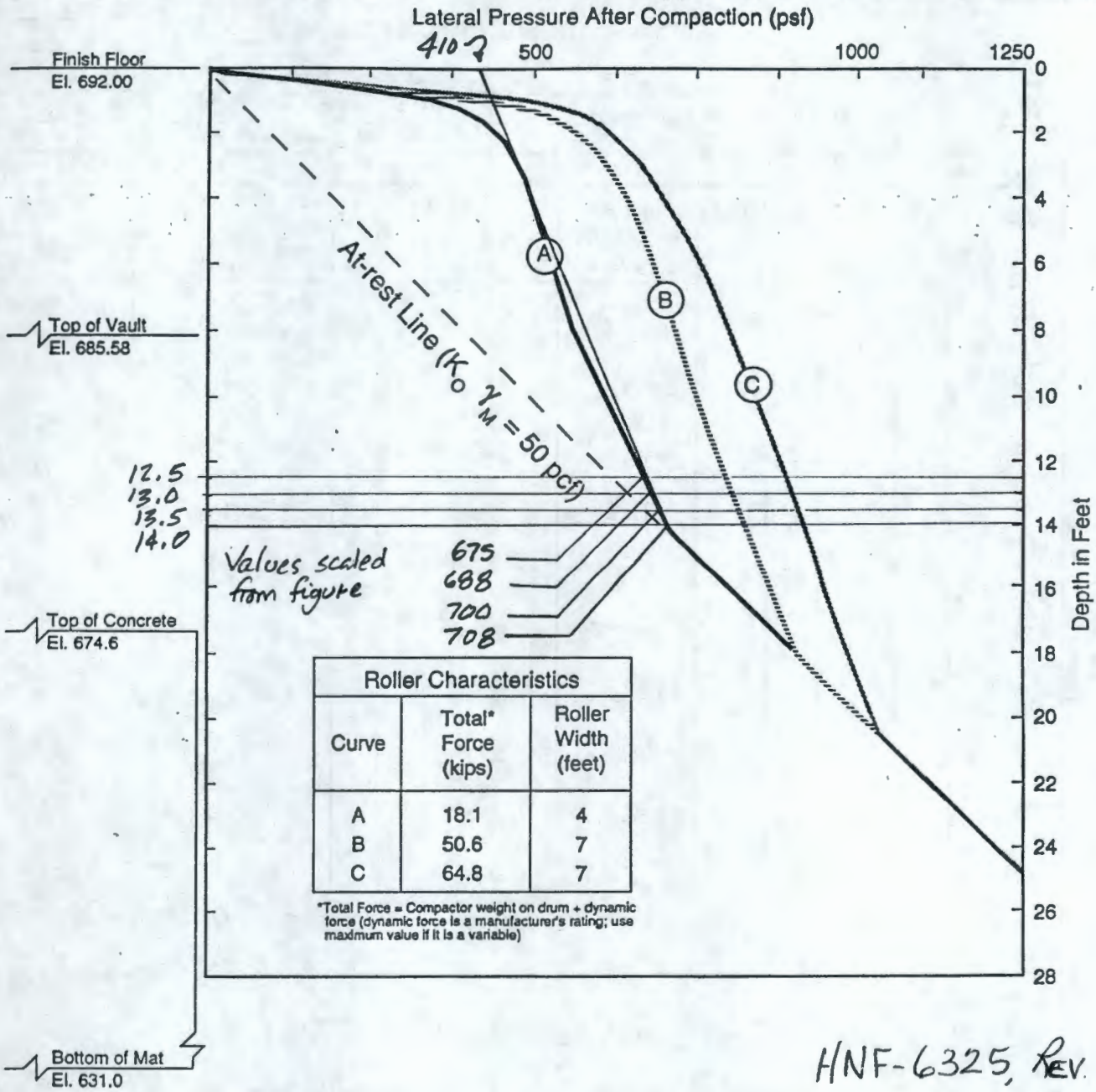
Layer	Δz_i	Average E (tsf)	Distance to center of interval	I_z	q (tsf)	$I_z \Delta z_i / E_i$	TOTAL
1	26.47 FT	1304	13.24'	0.288	8.20	.00585	$S_i = 7.87 \sum_1^{10} \frac{I_z}{E} \Delta z_i$
2		4375	39.71'	0.464		.00281	
3		5760	66.18'	0.640		.00294	$S_i = 7.87 \times 0.0212$
4			92.65	0.555		.00255	$= 0.167' \times 12''/'$
5			119.12	0.469		.00216	$= \underline{2.00 \text{ in.}}$
6			145.59	0.384		.00177	
7			172.06	0.299		.00137	
8			198.53	0.213		.00098	
9			225.00	0.128		.00059	
10			251.47	0.043		.00020	
						$\Sigma = 0.0212$	

///

Evaluate Backfill Loads for Gallery Outside Walls

Table 8-1 [Shannon & Wilson] γ_{soil} varies from 110 pcf to 116 use 115

Soil will be placed and compacted after interfering equipment is removed. Backfill lateral loads will be those for a retaining wall with a passive K_o. Shannon & Wilson provided curves for backfill pressures on subsurface tanks in Figure 11-8. These loads are used for evaluating backfill loads on the outside walls of Building 221-U. Analyze the exterior gallery wall as a 3 span beam with fixity at the ends. Span dimensions taken to ℄ of Operating & Piping floor slabs. [W69566] Assume fill is placed in lifts and compacted all the way to the overhead slab



Roller Characteristics		
Curve	Total* Force (kips)	Roller Width (feet)
A	18.1	4
B	50.6	7
C	64.8	7

*Total Force = Compactor weight on drum + dynamic force (dynamic force is a manufacturer's rating; use maximum value if it is a variable)

HNF-6325, Rev. 0
KN

NOTES

1. Above curves are based on charts developed by Duncan, et. al. (1991 and 1993).
2. Assumptions considered in analysis:
   Distance between compactor and tank ≥ 1 foot
   Backfill lift thickness = 12 inches
3. Backfill Sand Parameters:
   ϕ = Friction Angle = 40°
   γ_M = Moist Unit Weight = 130 pcf
   K_o = At-rest Earth Pressure Coefficient = 0.36
4. Extend "At-rest" line to estimate pressure at depths below range shown above.
5. Add a uniform dynamic incremental pressure of 10H (psf) to the above values to represent effects of earthquake loading (H is the wall embedment depth in feet) for 0.35g.

B13

Checked by R.W. Smith

KEH W-236A MWTF2W
Hanford Site
Richland, Washington

LATERAL EARTH PRESSURES STORAGE TANK ONE-FOOT-THICK BACKFILL LIFT

March 1994 H-1070-50

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 11-8

FLUOR DANIEL NORTHWEST, INC.

DESIGN ANALYSIS

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	Checked 5/19/00	By George T. Zyx	
Location BLDG 221-U	Revised	By	

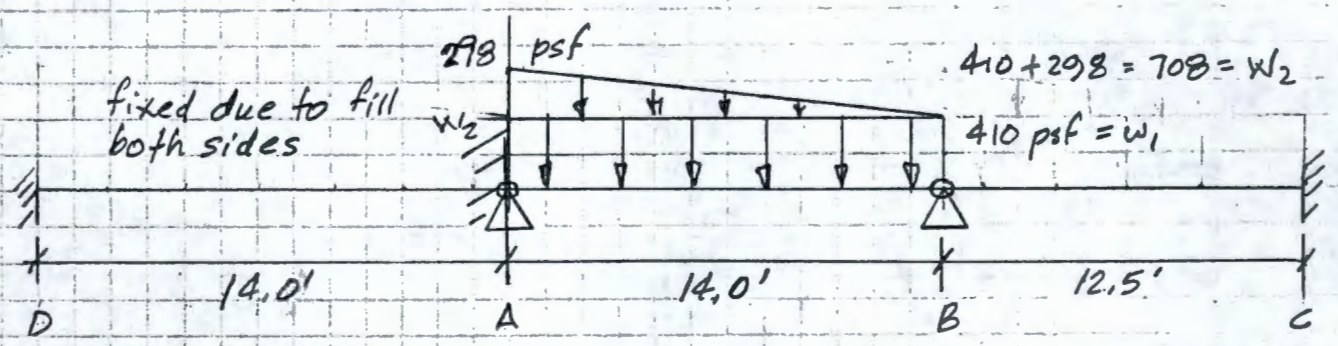
Use curve (A) for smaller equipment as idealized on pg 11.

$h^{electrical}_{gallery} = 14.0^{Ft}$ $h^{piping}_{gallery} = 14.0^{ft.}$ $h^{operating}_{gallery} = 12.5^{ft.}$

Uniform Load N/A Backfilled both sides 410 plf

Triangular Load N/A 708 − 410 = 293 plf

Maximum moment @ A occurs with span BC unloaded



Fixed End Moments [Rogers, 1965, p 40]

$M^F_{AB} = \frac{w_1 L^2}{12} + \frac{(w_2 - w_1) L^2}{20}$ $W_1 = 410$ plf $W_2 = 708$ plf $L = 14.0'$

$M^F_{BA} = \frac{w_1 L^2}{12} + \frac{(W_2 - w_1) L^2}{30}$ $M_{AB} = 9{,}617$ lb-ft $= 9.62^{K-Ft}$ $M_{BA} = 8{,}644^{lb-ft} = 8.64^{k-Ft}$

$R^F_A = Q \times n$ $W_1/W_2 = 0.58$, $n = 0.57$

$= 7{,}826 \times 0.57$
$= 4{,}460$

$R^F_B = Q - R_A = 7{,}826 - 4460 = 3{,}366$ lb

[Rogers, pg. 98]
Simple beam moment $Q = \frac{W_1 + W_2}{2} \times L = \left(\frac{410+708}{2}\right) \times 14.0$ Ft $= 7{,}826$ lb

$W_1/W_2 = 410/708 = 0.58$ $m = 7.94$ $M^F = \frac{Q \times L}{m} = \frac{7{,}826 \times 14^{Ft}}{7.94} = 13{,}798^{lb-ft.}$

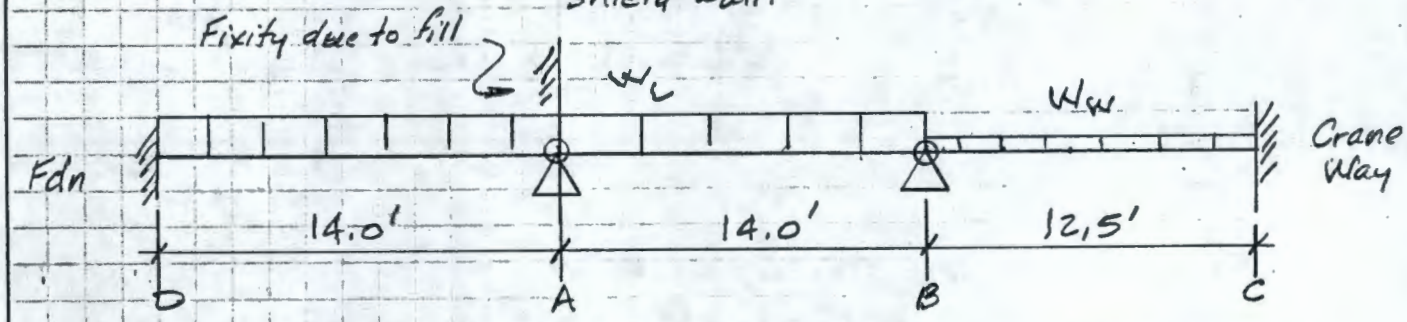
Client	CDI	WO/Job No.	65100241 A0 13
Subject	Scoping Calcs, Structural Scenarios	Date	5/18-00 By J.T. Baxter
		Checked	5/19/00 By George P. Eyer
Location	BLDG 221-U	Revised	By

Review Gallery Sidewall Ability to Carry Lateral Loads for Earthquake with Waste Fill:

Assumptions: Fill weight: Low 115 pcf, high 227 pcf

Equiv. Static Load: 0.2g, 0.3g, 0.4g

Fixity: Wall is fixed-end at top of foundation and bottom of crane way slab. Continuous 3 span over floor slabs. Floor slabs function as drag struts to carry lateral load into shield wall.



Assume the electrical gallery is already backfilled, and the external fill has been brought up to the elevation of the Piping Gallery floor, Point A. This fixes the wall at Pt. A

Range of fill weight: Galleries are 14' wide. Assuming rigid body acceleration

$$W_{fill}^{Lower} = \underset{fill}{14' \times 115\,pcf} + \underset{conc.\ wall}{3' \times 150\,pcf} = 2060\ lb/ft$$

$$W_{fill}^{upper} = \underset{ILAW\ boxes}{14' \times 227\,pcf} + \underset{conc.\ wall}{3' \times 150\,pcf} = 3628\ lb/ft$$

$$W_{conc} = 3 \times 150\,pcf = 450\ lb/ft$$

W_L = seismic coefficient × W_{FILL} use 0.2W, 0.3W, 0.4W; analyze wall for 0.3W case

W_W = " " × W_{conc}

W_W	W_{conc}	450 lb/ft	×0.20 = 90 lb/ft	×0.30 = 135 lb/ft	×0.40 = 180 lb/ft
W_L	W_{fill}^{Lower}	2060 lb/ft	×0.20 = 412 lb/ft	×0.3 = 618 lb/ft	×0.4 = 824 lb/ft
	W_{fill}^{upper}	3628 lb/ft	×0.20 = 726 lb/ft	×0.30 = 1088 lb/ft	×0.40 = 1451 lb/ft

FLUOR DANIEL NORTHWEST, INC.

DESIGN ANALYSIS

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Subject	Scoping Calcs, Structural Scenarios	Date 5/18/00	By J.T. Baxter
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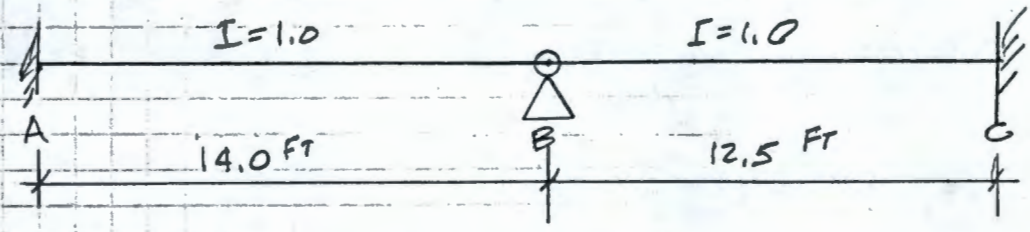


$K_{BA} = \frac{4EI}{L} = \frac{4}{14.0} = 0.286$

$K_{BC} = \frac{4EI}{L} = \frac{4}{12.5} = 0.320$

$DF^{BA} = \frac{.286}{.286+.320} = .472$

$DF^{BC} = 1 - .472 = .528$

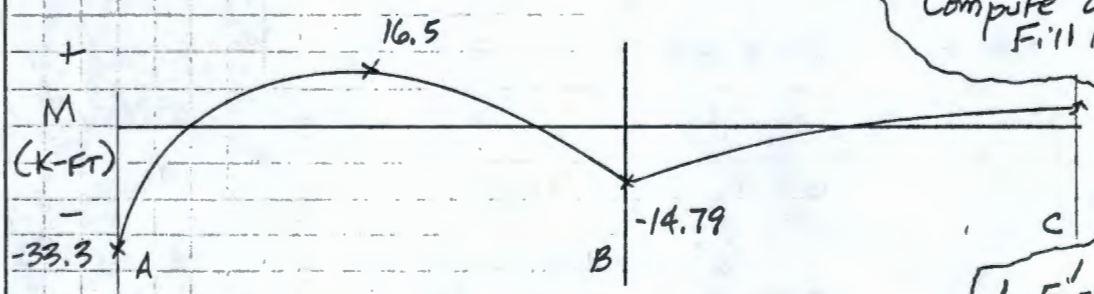
	AB	BA	BC	CB
K	–	0.286	0.320	–
DF		0.472	0.528	
COF		0.5	0.5	
Fill	9.62	-8.64		
W_L	17.8	-17.8	1.76	-1.76 W_W
	+5.83	+11.65	+13.0	+6.52
	33.25	-14.79	14.76	4.76

Simple beam moment AB $M^{W_L} = wL^2/8 = 1.09\ ^{K}/_{FT} \times (14.0\ FT)^2/8 = 26.7\ ^{K\text{-}FT}$

Fixed End Moments: Use for moment distribution above.

Seismic ILAW, 0.30g $M^{W_L}_{AB} = M_{BA} = \frac{wL^2}{12} = 1.09\ ^{K}/_{FT} \times (14.0\ FT)^2/12 = 17.8\ ^{K\text{-}FT}$

Seismic W_W, 0.30g $M^{W_W}_{BC} = M_{CB} = \frac{wL^2}{12} = 0.135\ ^{K}/_{FT} \times (12.5\ FT)^2/12 = 1.76\ ^{K\text{-}FT}$



$M^+ \cong 26.7 + 13.8 - \frac{(33.3 + 14.79)}{2} = 16.5\ ^{K\text{-}FT}$

Maximum moment occurs at A, $-33.3\ ^{K\text{-}FT}$

Compute composite load factor
Fill load is a soil load with LF = 1.7.
Lateral seismic load of fill is earthquake load with L.F. = 1.4

$L.F.' = \frac{9.62^{K\text{-}FT} \times 1.7 + 17.8^{K\text{-}FT} \times 1.4}{9.62 + 17.8}$

L.F.' = 1.50 use composite L.F. for capacity check.

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Location BLDG 221-U	Revised	By	

Check tension capacity of Oper. Gallery floor to support exterior wall.

$T = \Sigma R_B = R_B^F + R_B^{W_L} + R_B^{W_W}$

$= 3{,}366 \text{ lb} + \left(\frac{14.0^{FT} \times 1.09^{K/FT}}{2}\right) + \left(\frac{12.5^{FT} \times 0.135^{K/FT}}{2}\right)$

$T = 3{,}366 + 7{,}630 + 844 = 11{,}840$ lb/Ft of wall.

Dwg. No. W69566, Sect. H-H
W69333, DET. G, DETAIL SHOWING REINF. OF 2ND FLOOR SLAB

Bars No. 605 provide tension capacity

(5) 3/4" ϕ x 8'-0 MK 605 between slots 5 places + 2 ends = 7 x

no. bars = 7 x 5 = 35 bars $A_s = 35 \times 0.44 \text{ in}^2/\text{bar} = 15.4 \text{ in}^2$

$T_n = 15.4 \text{ in}^2 \times 40 \text{ ksi} = 616^K / 40 \text{ FT} = 15.4^K/\text{FT OF WALL}. > 11.8^K$

$T_u = \underset{L.F.}{1.5} \times T = 1.5 \times 11.84^{K}/\text{FT OF WALL} = 17.8^{K}/\text{FT WALL} \leq \phi T_n \leq .9 \times 15.4^K$

$17.8^{K}/\text{FT WALL} \leq 13.9^{K}/\text{FT WALL}$ N.G

Tension capacity $= 13.9/17.8 \times 100 = 78\%$ of that required.

Change fill assumption to soil rather than ILAW, compare Fixed End Moments at A

ILAW: Fill 9.62 + W_L 17.8 = 27.4

SOIL FILL: Fill 9.62; W_L $17.8 \times \frac{618 \text{ lb/Ft}}{1088 \text{ lb/ft}} = 10.1$ = 19.72

$\frac{19.72}{27.4} = 0.72$ 72 percent is within floor tensile capacity. Limit soil unbalanced fill lift height to 14 ft in galleries and limit ILAW w/ grout unbalanced lift height to 7 ft. May require placing L.L. load limits on floor above gallery being filled.

B17

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Gallery Wall Capacity: Reference BHI prior calculation: (attached, next page)

Project Title: 221-U Structural Sampling, CDI
Discipline: Civil Job. No. 22192
Calc. No. 0200W-CA-C0037

From p. 9 of 13 The P-M diagram shows that ϕM_n should be $\leq 62^{k\text{-}ft}$ at Point M. For the load case $U = 0.9D \pm 1.4E$, the D.L. in the wall has not changed $M_u = \pm 1.5 \times 33.3^{k\text{-}ft} = \pm 50.0^{k\text{-}ft} \leq 62^{k\text{-}ft}$
L.F.

∴ Wall has adequate capacity.

The exterior wall of the Piping and Operating Gallery has adequate strength to carry unbalanced fill loads with a lift height of 14.0 FT. However, the Operating Gallery floor lacks adequate tension capacity to deliver the

PD-6002-142 (12/95)

DESIGN CALCULATIO

(1) Drawing _____ (2) Doc. No. _____

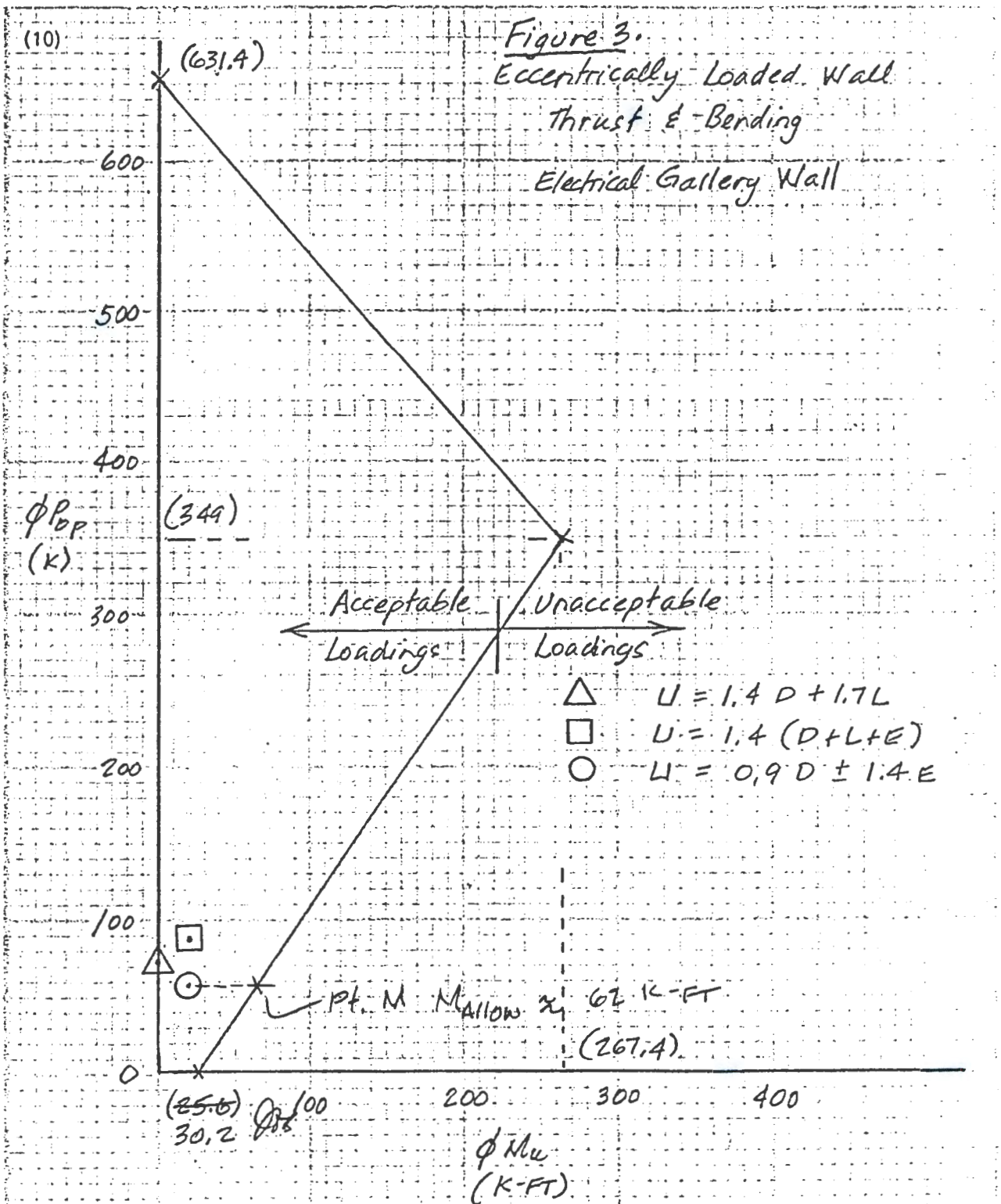
(4) Building 221-U (5) Rev. _____ CDI Scoping Calcs.

(7) Subject Structural sampling impact on gallery wall structural capacities.

(8) Originator J.T. Baxter J.T. Baxter Date 6/19/98 5/18/00

(9) Checker [illegible] Date 7/23/98

(10)



ILAW reaction loads to the Shield Wall. For conceptual design limit fill lift heights in the galleries to a maximum of 7 Ft. height for ILAW and 14 Ft. For Soil.

Evaluate Variation in Seismic Acceleration over Building Height:

Reference: Winkel, 1990, B-Plant Canyon Structure Seismic Evaluation, WHC-SD-WM-SA-005, Rev. 0, Westinghouse Hanford Co., Richland, WA.

The reference report includes a 2-D non-linear time history seismic analysis. Input ground motion was a 0.20g ZPA time history that was processed through a FLUSH soil-structure interaction model to obtain foundation level T-H accelerations. Lateral acceleration varies over the height of the building from the foundation to the roof. The 0.2g acceleration is a reasonable approximation for local acceleration at the foundation. Evaluate the maximum acceleration at the Crane Rail level below the roof. Page D15 shows an element plot of the ANSYS model. Elements 22 and 76 represent the non-linear shear keys at the spring line for the roof bent. The roof bent for the standard sections of Building 221-U produces a lateral thrust at the elev. of the Crane Rail caused by D.L. frame action. Seismic

Reference Pages	North Wall ELEM. 22	South Wall ELEM. 76
Table C-1, p. D6	DL P = 22.9 K	P = 23.3 K
Fig. 8-19 p. 67 shear @ t = 0 sec.	V = 8.3 K	V = 8.3 K
p. D51 Peak Shear and Normal Forces 0.20g DBE	-17,150 lb @ t = 1.850 sec.	1039 @ t = 1.160
	777.8 @ 1.620 sec.	15,590 @ t = 1.640

shears are determined by taking the difference between peak + and -

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seismic shears and the static D.L. value.

North Wall $-17,150\ lb - (-8.3^k) = -8.85^k \quad t = 1.850$
$778\ lb - (-8.3^k) = 9.08^k \quad t = 1.620$

South Wall $8300 - -1039 = 7.26^k \quad t = 1.160$
$8.3^k - 15.59^k = -7.29^k \quad t = 1.640$

$\Sigma V_E = 9.08^k + 7.29^k = 16.37^k$ Combining the largest components

The maximum vertical response is not time-related to the maximum horizontal response from inspection of info on pg. D51.

Calculate Equiv. Peak $\vec{a}_H$ at Crane Rail

$\vec{a}_H = 16.37^k/(22.9^k + 23.3^k) = 0.35\ g$

Thus, fill materials will see lateral accel. that ranges from 0.20g near the foundations to 0.35g near the roof

Check Lateral Capacity of Exterior Wall of Operating Deck (South Wall)

Evaluation of the exterior wall of the Operating Deck, try 14 ft lift of bulk waste soil. Use Shannon & Wilson for backfill lateral pressure and compaction load. Use Shannon & Wilson Fig. 11-8, note 5. (Calc. Sht 10) with a dynamic increment of $10H = 10 \times 14' = 140$ psf seismic uniform load over 14' lift height. Wall self-weight $= 5' \times 150\ lb/ft^3 = 750\ lb/ft^2$

Lateral seismic $= 0.30\ g \times 750\ lb/ft^2 = 225\ lb/ft^2$

The South wall is asymmetrically reinforced, from [Winkel] p. 51 the steel reinforcement is:

S. Wall	As/ft. (in^2) at Deck	As/ft (in^2) +14^{Ft} elev.
- Outside	0.311	0.311
- Inside	4.167	2.667

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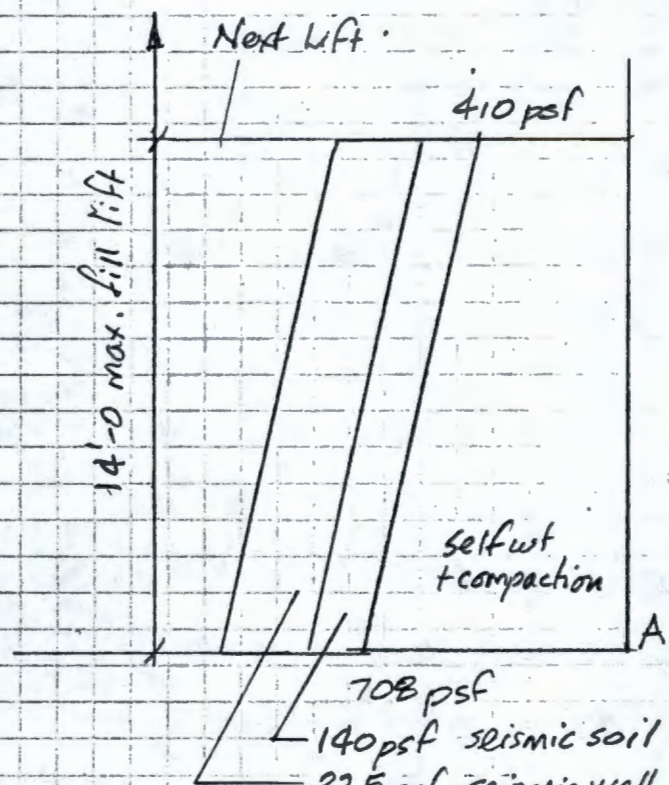
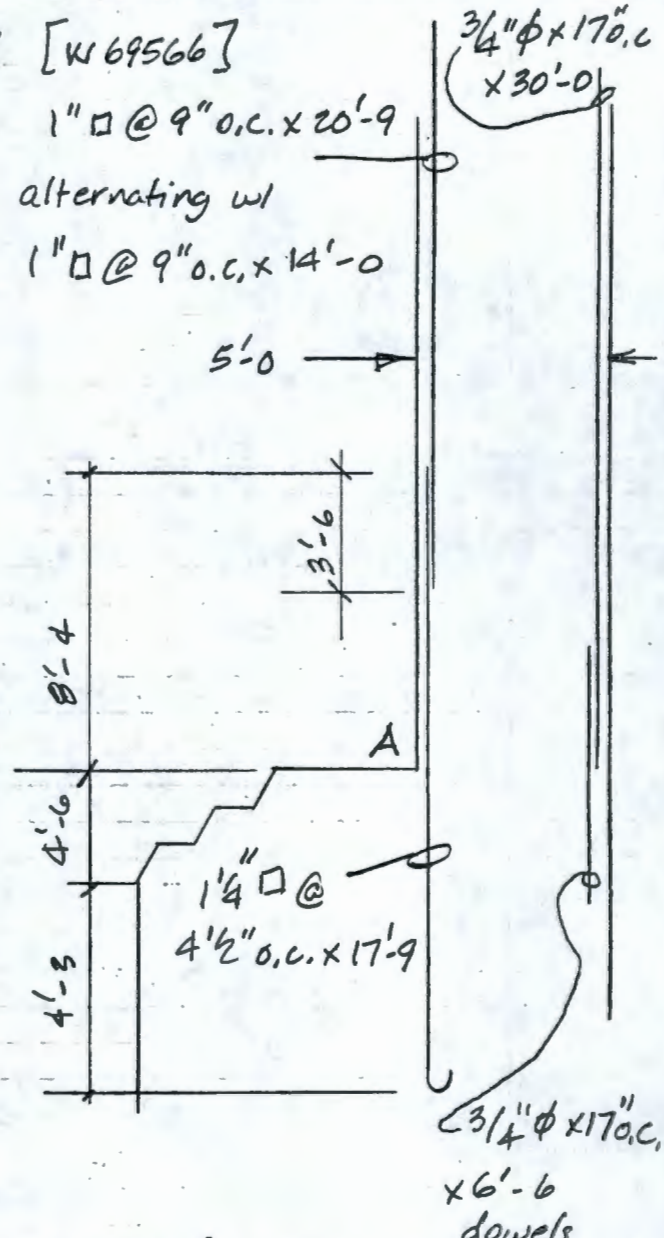
Cantilever Moment on 5'-0" thick Wall [W 69566]





$$M_A = \frac{wL^2}{2} + \frac{wL}{3}$$

$$= (410+140+225) \times \frac{(14^{Ft})^2}{2} + (708-410) \times \frac{14^{Ft}}{3}$$

$$= 75{,}950^{lb\text{-}ft} + 1{,}391^{lb\text{-}ft} = 77{,}340^{lb\text{-}ft} \times 12'' = 928{,}000^{lb\text{-}in}$$

$$= 928^{K\text{-}in}$$

$M_u = L.F. \times M_A = 1.50 \times 928^{K\text{-}in} = 1{,}392^{K\text{-}in}$ Compare to reserve capacity of Wall at the Operating Deck elev. & +14'-0. Use Moment-Rotation Diagrams from Winkler p. 37 and p. 39 for source.

FLUOR DANIEL NORTHWEST, INC.

DESIGN ANALYSIS

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		Checked	5/19/00 By Jerry J. Zyr
Location	BLDG 221-U	Revised	By

Capacity at +14' elev. Reference Winkel Fig. 7-7

$M_{DL} = -626$ K-IN $M_{DL} + 0.2g$ DBE $= 482$

$+0.2g$ DBE $= 482 - (-626) = 1{,}108$ K-IN

$M_{DL} - 0.2g$ DBE $= -626 - (1{,}108$ K-IN$) = -1{,}734$ K-IN

$M_u = -6900$ K-IN (INTERPOLATED FROM FIGURE)

Margin on yield $= -6900 - (-1734) = 5166$ K-IN

$\frac{\text{Cantilever } M_u}{\text{Margin on yield}} = \frac{1392 \text{ K-IN}}{5166 \text{ K-IN}} = 0.27$ OK

Capacity at Deck Elev. Reference Winkel Fig. 7-9

$M_{DL} = -1793$ K-IN $M_{DL} + 0.2g$ DBE $= +1357$ K-IN

$0.2g$ DBE $= 1357 - (-1793) = 3150$ K-IN

$M_{DL} - 0.2g$ DBE $= -1793 - (3150) = -4943$ K-IN

$M_u = -10600$ K-IN (Interpolated from Figure)

Margin on yield $= -10600 - (-4943) = 5657$ K-IN

$\frac{\text{Cantilever } M_u}{\text{Margin on yield}} = \frac{1392 \text{ K-IN}}{5657 \text{ K-IN}} = 0.25$ OK

South Wall can withstand 14 FT high lifts of compacted soil. Structural demand is about 25% of the walls reserve strength. Wall deflections at the top have not been checked, a future structural study (detailed) should evaluate to interaction between the roof bent and the canyon main lower structures for the backfill loads. Use same lift height limit for ILAW waste as used for galleries, 7 ft. maximum

BD-8002-142 (12/96)