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Page 1 of 2	Proj. ECN

2. ECN Category (mark one)	3. Originator's Name, Organization, MSIN, and Telephone No.		4. Date
Supplemental [] Direct Revision [x] Change ECN [] Temporary [] Standby [] Supersedure [] Cancel/Void []	R. A. Giller, Structural Assessments, H5-55, 376-2662 BD440		9-23-94
	5. Project Title/No./Work Order No. 2736Z Analysis/E23205	6. Bldg./Sys./Fac. No. 2736Z	7. Approval Designator QS
	8. Document Numbers Changed by this ECN (includes sheet no. and rev.) WHC-SD-CP-SA-023 Rev. 0	9. Related ECN No(s). NA	10. Related PO No. NA

11a. Modification Work	11b. Work Package No.	11c. Modification Work Complete	11d. Restored to Original Condition (Temp. or Standby ECN only)
[] Yes (fill out Blk. 11b) [X] No (NA Blks. 11b, 11c, 11d)		Cog. Engineer Signature & Date	Cog. Engineer Signature & Date

12. Description of Change

Add Appendix B, C, & D to Table of Contents

Revise Facility Use Category, Safety Class wording, page 1 of the report.

Add reference to equipment analysis report, page 7 of the report.

Add reference to appendix B on pages 2 and 10 of the report.

Revise wall calculations in Appendix A, pages A4-1, A4-7, A4-9, A4-11.

Add finite elements analysis calculations, Appendix B

Add column base shear discussion and position statements, Appendix C & D.

13a. Justification (mark one)			
	Criteria Change []	Design Improvement []	Environmental []
As-Found [x]	Facilitate Const. []	Const. Error/Omission []	Design Error/Omission []

13b. Justification Details

Additional calculations were performed in response to review comments.

Design verification method used was independant review by E.O. Weiner

14. Distribution (include name, MSIN, and no. of copies)

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15. Design Verification Required	16. Cost Impact				17. Schedule Impact (days)	
	ENGINEERING		CONSTRUCTION			
[X] Yes	Additional	[] $	Additional	[] $	Improvement	[]
[] No	Savings	[] $	Savings	[] $	Delay	[]

18. Change Impact Review: Indicate the related documents (other than the engineering documents identified on Side 1) that will be affected by the change described in Block 12. Enter the affected document number in Block 19.

SDD/DD	[]	Seismic/Stress Analysis	[X]	Tank Calibration Manual	[]
Functional Design Criteria	[]	Stress/Design Report	[]	Health Physics Procedure	[]
Operating Specification	[]	Interface Control Drawing	[]	Spares Multiple Unit Listing	[]
Criticality Specification	[]	Calibration Procedure	[]	Test Procedures/Specification	[]
Conceptual Design Report	[]	Installation Procedure	[]	Component Index	[]
Equipment Spec.	[]	Maintenance Procedure	[]	ASME Coded Item	[]
Const. Spec.	[]	Engineering Procedure	[]	Human Factor Consideration	[]
Procurement Spec.	[]	Operating Instruction	[]	Computer Software	[]
Vendor Information	[]	Operating Procedure	[]	Electric Circuit Schedule	[]
OM Manual	[]	Operational Safety Requirement	[]	ICRS Procedure	[]
FSAR/SAR	[]	IEFD Drawing	[]	Process Control Manual/Plan	[]
Safety Equipment List	[]	Cell Arrangement Drawing	[]	Process Flow Chart	[]
Radiation Work Permit	[]	Essential Material Specification	[]	Purchase Requisition	[]
Environmental Impact Statement	[]	Fac. Proc. Samp. Schedule	[]	Tickler File	[]
Environmental Report	[]	Inspection Plan	[]		[]
Environmental Permit	[]	Inventory Adjustment Request	[]		[]

19. Other Affected Documents: (NOTE: Documents listed below will not be revised by this ECN.) Signatures below indicate that the signing organization has been notified of other affected documents listed below.

Document Number/Revision	Document Number/Revision	Document Number Revision

20. Approvals

Signature	Date	Signature	Date
OPERATIONS AND ENGINEERING		ARCHITECT-ENGINEER	
Cog. Eng. R.A. Giller R.A. Giller	10-5-94	PE	
Cog. Mgr. T.J. Conrads T.J. Conrads	10-5-94	QA	
QA A.K. SHARMA A.K. Sharma	10/10/94	Safety	
Safety M.D. Jaka (M.D. JAKA)	10/18/94	Design	
Environ.		Environ.	
Other		Other	
Independant Reviewer E.D. Wein	10-10-94		
		DEPARTMENT OF ENERGY	
		Signature or a Control Number that tracks the Approval Signature	
		ADDITIONAL	

RELEASE AUTHORIZATION

Document Number:	WHC-SD-CP-SA-023, Rev.1
Document Title:	Structural Evaluation of the 2736Z Building for Seismic Loads
Release Date:	October 26, 1994

This document was reviewed following the procedures described in WHC-CM-3-4 and is:

APPROVED FOR PUBLIC RELEASE

WHC Information Release Administration Specialist:

Angela Laughery For — M.N. Boston   10/26/94

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APPROVED FOR PUBLIC RELEASE
A. Laughery FOR M. Boston 10-26-94

7. Abstract

The 2736Z building structure is evaluated for high-hazard loads. The 2736Z building is analyzed herein for normal and seismic loads and is found to successfully meet the guidelines of UCRL-15910 along with the related codes requirements.

9. Impact Level SQ

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RECORD OF REVISION	(1) Document Number WHC-SD-CP-SA-023	Page 1

(2) Title
Structural Evaluation of the 2736Z Building for Seismic Loads

CHANGE CONTROL RECORD

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1 RS	ECN 615157 Add Appendix B, C & D to Table of Contents Revise Facility Use Category, Safety Class wording, page 1 of report Add reference to equipment analysis report, page 7 of the report Add reference to appendix B on pages 2 and 10 of the report Revise wall calculations in Appendix A, pages A4-1, A4-7, A4-9, A4-11 Add finite elements analysis calculations, Appendix B Add column base shear discussion and position statements, Appendix C & D	R.A. Zillar	T.J. Conrads 10/27/94

STRUCTURAL EVALUATION
OF THE 2736Z BUILDING
FOR SEISMIC LOADS

Prepared by: R.A. Giller Date: 9-22-94
R. A. Giller, Senior Engineer
Structural Margin Assessments

Checked by: E.O. Weiner Date: 9-29-94
E. O. Weiner, Fellow Engineer
Structural Margin Assessments

Approved by: G.R. Wagenblast for TJC Date: 9-29-94
T. J. Conrads, Manager
Structural Margin Assessments

WESTINGHOUSE HANFORD COMPANY
Hanford Operations and Engineering Contractor
for the U.S. Department of Energy
Richland, Washington

TABLE OF CONTENTS

1.0 INTRODUCTION

Existing structural design documents for the 2736Z Building state that the structure is adequate to resist specified earthquake loads. However, the analyses and calculations to support these statements are not retrievable. The purpose of this document is to provide the structural analysis documentation supporting the structural qualification of the 2736Z Facility to current site design criteria.

The 2736Z Building is described by Z Plant facility management as a Safety Class 3 structure that houses Safety Class 1 equipment. The "Class 3 over 1" function requires that the building be evaluated to Safety Class 1 seismic loads.

Design criteria originate from DOE Order 6430.1A, which directs the use of current design criteria, specifically, the guidelines for natural phenomena hazards evaluation found in UCRL-15910 (June 1990) and site-specific design criteria. The site-specific design criteria for the Hanford Site as approved by DOE-RL are found in SDC-4.1, Rev. 12 (DOE-RL 1993).

The structural evaluation plan, was to utilize simple conservative methods and proceed to more rigorous methods as needed. The natural phenomena seismic, wind, and ashfall loads appropriate for Safety Class 1 structures were used. For the seismic analysis simple equivalent static analysis methods were employed first, and then dynamic analysis methods could be used as needed.

2.0 SUMMARY

The 2736Z Facility consists of three main structures: 2736Z Building, 2736ZA Building, and 2736ZB Building. All of these structures are made of reinforced concrete roof slabs, walls, floors and foundations. These buildings are supported on spread footings close to the surface of the ground but below frost level.

The current design and evaluation criteria for facilities at the Hanford Site appear in UCRL-15910 and SDC-4.1, Rev. 12, and the documents referenced therein. The design code for 2736Z Building was ACI-318-63; the code used in the current evaluation is ACI-349-90.

The 2736Z Building was analyzed for both normal operating and natural phenomena loads. The design of different components of the structure are controlled by different loads, load combinations, and loading directions. The required strength of the roof slab, the in-plane bending of the wall panels and the size of four of the columns is controlled by normal dead and live loads. The size of all other components of the structure is controlled by earthquake loads.

As a result of this evaluation, all structural members of the 2736Z Building show positive margins of safety with respect to the material design strengths. Margin-of-safety is defined as

$$M.S. = \frac{Allowable\ load}{Actual\ load} - 1$$

which must be greater than 0.0. The controlling margin of safety for each type of structural member is as follows:

- Roof: Dead and live load (vertical bending) M.S. = 0.009
- Walls: Dead and live load (vertical bending) M.S. = 0.630
- Columns: Earthquake (lateral base shear) M.S. = 0.170
- Foundation Wall: Earthquake (lateral shear friction) M.S. = 0.620
- Footing: Dead, live, and earthquake (vertical bending) M.S. = 0.037
- Soil bearing: Dead, live, and earthquake (vertical load) M.S. = 0.017

Most reinforcing details in the structural members meet the current ACI-349-90 criteria. The one exception is the depth to which the precast wall panel reinforcement is embedded into the surrounding column and beam members. The existing embedment length is 15 cm (6-in.), which meets the ACI-318-63 design code criteria. The 15 cm (6-in.) length also meets the bond stress criteria of ACI-349-90. Fifteen centimeters (6-in.) does not meet ACI-349-90 requirements for an overall minimum length of 30.5 centimeters (12-in.).

The requirement that the minimum embedment length be greater than the bond stress length criteria was added to the code to enhance the system ductility. Because the bond stress portion of the required embedment length is met, although the ductility portion is not, the wall member is acceptable for non-plastic stress levels.

Calculations to qualify the 2736Z structure for the safety class 3/1 function are included in Appendix A. More rigorous calculations were performed in response to review comments and are included in Appendix B, which was added in Revision 1 of this report. These additional calculations do not change the results, but demonstrate even more structural capacity.

3.0 STRUCTURE DESCRIPTION

The 2736Z Building was designed as a single-story reinforced concrete structure. The walls, columns, roof, floor, and foundation were all constructed of reinforced concrete. A later construction project added concrete thickness to the three outside walls.

The 2736Z Building was built originally to construction specification HWS-8937, project number HAP-642, dated July 10, 1970. Later in the 1982/1983 time period, project number B-246A added more wall concrete. Drawing numbers H-2-26648 to H-2-26654 show the original structural design and drawings H-2-90318 and H-2-90319 show the additions by project B-246a.

The overall dimensions of the building are 20 m (65-ft 6-in.) long by 17.5 m (57-ft 2-in.) wide by 3.7 m (12-ft 2-in.) high. The roof slab is 16.5 cm (6½-in.) thick with roof beams alternating with wall top beams that run in both directions. The wall and column footings are a minimum of 84 cm (2-ft 9-in.) below grade with the east wall 1.5 m (5-ft) below grade.

The foundation elements in the 2736Z Building consist of the wall footings, column footings, foundation wall, and column pedestals. The footings are 30.5 cm (1-ft) thick by 61 cm (2-ft) wide for the walls and 1.2 m (4-ft) wide for the columns. The foundation wall was poured monolithically with the column pedestals and supported on the footing. The foundation wall is 20.3 cm (8-in.) thick, as is the building wall, but has slightly less reinforcement. The column pedestals have the same dimensions and reinforcing as the columns and are 12.7 cm (5-in.) shorter than the foundation walls.

The floor slab is supported on soil backfill with expansion joint material between the slab edges and all foundation walls. Supported on the floor slab are storage racks, storage cubicles, and the stored material.

The building walls, columns, beams, and roof slab are supported by the column pedestals. The building walls are precast wall panels that are set in place with a 6 mm (¼-in.) gap between the wall panel and the underlying foundation wall. Holding the wall in place are cast-in-place columns and wall beams, which in turn are poured monolithically with the roof slab and roof beams. Once these other cast-in-place members have stabilized the wall, the gap is grouted shut.

The roof slab system is designed and constructed as a two-way slab unit. The slab spans are nearly equal in both directions and the reinforcing is identical. Poured monolithically with the slab are very shallow roof beams that alternate with the wall lines to support the roof slab. The roof slab also acts as a shear diaphragm to distribute lateral loads evenly to the precast shear walls.

4.0 ANALYSIS

4.1 CODES AND STANDARDS

Codes and standards at the Hanford Site originate from DOE Order 6430.1A, which directs the use of other design and evaluation documents. In addition to national codes and standards, the following DOE-specific guidelines and criteria documents were used:

- UCRL-15910, *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Loadings* (UCRL-15910)
- Site specific design criteria as approved by DOE-RL, SDC-4.1, Rev. 12 *Standard Architectural-Civil Design Criteria Design loads for Facilities* (DOE-RL 1993).

UCRL-15910 provides load combinations for use with extreme wind loadings and recommends the use of ACI-349 load combinations for seismic loadings. Other UCRL-15910 recommendations for use in seismic analyses are as follows:

- UBC 1988 and 1991, special design details for reinforced concrete structures
- Moderate seismic provisions of the UBC special design details when the peak ground acceleration is less than 0.24 *g*
- Elastic evaluation methods and F_u of unity when the UBC special details cannot be met
- 10% damping for post-yield inelastic stress conditions
- Square-root-sum-of-the-squares methodology (SRSS) or 100-40-40 methods for combining seismic load directions
- Use of ground motion equivalent to one-half of the appropriate return period for existing buildings that cannot meet the guidelines provided in UCRL-15910, Section 4.4.5
- Use of equivalent static analysis methods for simple structures.

SDC-4.1, Rev. 12, provides basic natural phenomena and other design loads for Hanford Site buildings. These loads include seismic, wind, ashfall, dead, live, roof, soil, and snow. SDC-4.1 directs the use of national codes and standards for design and evaluation methods. For example, ANSI A58.1 (1982) is specified for wind, snow, and roof load design, UCRL-15910 for some load combinations, and ACI and UBC for reinforced concrete design.

The design for ashfall loads is found only in SDC-4.1, which becomes both the design and evaluation document for this load. The magnitude of the load, the extreme nature of the load, and how it is to be combined with other loads all are defined in SDC-4.1.

Nuclear Regulatory Guide 1.61 was used to select the proper seismic damping value. Reg. Guide 1.61 recommends the use of 7% damping for the seismic design of reinforced concrete structures to remain within elastic limits. The 2736Z Building does not meet all of the currently required ACI reinforcing details; per the UCRL-15910 recommendation it must remain within elastic stress limits.

Tank Seismic Experts Panel (TSEP) Guidelines also recommend the use of 7% damping for response level 2-type loadings. Response level 2 loadings are high loadings, but are intended to remain within the material elastic limits.

ACI-349 and ACI-318 concrete building codes provide load combinations and design methods for use in the evaluation of reinforced concrete structures. These codes also provide reinforcing details required for structures subjected to seismic loads.

The UBC provides special concrete reinforcing details for use in the evaluation of concrete structures subjected to seismic loads. However, the 2736Z structures rely on shear wall to resist lateral loads and therefore are not required to meet the UBC special detailing. The UBC also provides recommendations on mass acceleration for seismic load determinations.

4.2 DESIGN ASSUMPTIONS

The 2736Z, 2736ZA, and 2736ZB structures are being analyzed simultaneously and contain similar assumptions which are listed below.

- The 2736Z facilities house Safety Class 1 materials and shall be evaluated as Safety Class 3/1.

- These structures shall be evaluated to current site-specific natural phenomena loads and criteria, as specified in UCRL-15910 and SDC-4.1.

- Seismic loads are expected to be the controlling lateral load. These structures shall be evaluated initially for seismic loads that use equivalent static methods as allowed by UCRL-15910 and progress to more exact and rigorous methods as required.

- Equivalent static evaluations multiply the mass of the structure by a static seismic acceleration. This acceleration equals the peak of the response spectra curve times a multiple mode factor. The mode factor recommended by ASCE 4-86 is 1.0 for simple regular structures and 1.5 for complex structures. The 2736Z structures are simple regular structures, so the 1.0 mode factor will be used.

- For the 2736Z structures, 10% seismic damping and F_u factors as recommended by UCRL-15910 shall be used if all reinforcing details meet the applicable ACI and UBC requirements. When the reinforcing details do not meet these requirements, then 7% damping and F_u equal to unity shall be used so that the structure stays within the elastic stress range.

- Review of the drawings show the 2736Z structures to be reinforced concrete with the following support systems:

  - Lateral loads resisted by the concrete shear walls and rigid roof diaphragm
  - Lateral loads proportionally distributed to the shear walls by the roof slab acting as a diaphragm
  - The roof slab acting as a two-way slab (2736Z and ZA) or a one-way slab (2736ZB) to resist vertical loads.

- Material strengths used in this evaluation are original design values depicted in the construction specifications or the drawings.

- Drawings and design specifications listed in the calculations are labeled as-built and are considered to be valid design information.

- For the calculation of seismic inertial loads, only the mass of the deadweight will be included; the mass of snow or roof live load will not be included per UBC Chapter 23 interpretation.

- For overturning effects, 100% of the deadweight mass will be used to calculate the overturning moment, but only 90% for the dead load resisting moment.

- Soil bearing pressures from the original design media are as follows:

  - 2736Z = 1,860 N/m^2 (4,500 lb/ft^2) foundation on undisturbed soil
  - 2736ZA = 2,066 N/m^2 (5,000 lb/ft^2) design drawing
  - 2736ZB = 2,066 N/m^2 (5,000 lb/ft^2) design specification.

  The 242S Evaporator soil report provides the basis for the validation of the stated soil bearing pressures.

- Because the 2736Z structures rely on shear walls to provide the lateral seismic loads, the UBC special seismic details for frames do not apply.

4.3 CONSTRUCTION MATERIALS AND IN SITU PROPERTIES

Construction of the 2736Z Building uses reinforced concrete for the precast and cast-in-place members. The construction specification, HWS-8937 (HWS 1970), references several ACI standards, codes, and practices for the design, detailing, and placement of concrete. This specification also lists several ASTM standards to be followed for material properties and material tests:

- ASTM A615-68 — Deformed Billet Steel Bars for Concrete Reinforcement
- ACI 306-66 — Recommended Practice for Cold Water Concreting
- ACI 315-65 — Manual of Standard Practice for Detailing Reinforced Concrete Structures
- ACI 318-63 — Building Code Requirements for Reinforced Concrete
- ACI 605-59 — Recommended Practice for Hot Weather Concreting.

The strengths of the concrete and reinforcing steel materials are given in the design drawings and the HWS specification. Concrete strength is listed on drawing H-2-26650 as 8.6 N/m^2 (3,000 lb/in^2) at 28 days. The reinforcement steel strength is given in HWS specification 6 as ASTM A615, grade 40. The grade 40 translates to a minimum yield strength 115 N/m^2 (40,000 lb/in^2).

The 2736 Facility was visited on December 21, 1993. This visit had several objectives: first, to verify the configuration of the structures as shown on the design drawings; second, to verify equipment loads on the structures; third, to inspect the structural condition of the concrete members.

The site visit resulted in the following observations.

- The structural configuration of the 2736Z Building is as shown on the drawings; see drawing list in Section 5.0.
- Wall- and ceiling-mounted equipment loads are very light, less than 16 Kg/m^3 (1 lb/ft^3). Floor-mounted storage racks and cubicles are much heavier, but are supported vertically by the floor and horizontally by the floor and wall as assumed in the analysis. See WHC-SD-CP-SA-026 for further evaluation of equipment loads on the structure.
- The condition of the structural members was very good. The concrete members had no visible cracking or spalling, aggregate voids were very small, and no patches or repair areas were found. The reinforcing obviously could not be seen, but no rust stains or spalled rebar cover was found either.

Based on these observations the conclusion is that equipment loads as assumed are reasonable. It is also reasonable to conclude that the in situ concrete and rebar properties meet those specified by the original material design properties since there were no signs of deterioration.

4.4 ANALYSIS METHOD

The UCRL-15910 recommends that high-hazard facilities (Safety Class 1) normally be analyzed with dynamic methods. UCRL-15910 Appendix A also states that equivalent static methods are an acceptable alternative for simple high-hazard structures. The 2736Z Building is a simple structure with a regular floor plan that fits the UCRL-15910 description; thus equivalent static methods were employed.

To analyze the 2736Z Building, static load hand-calculation methods were employed. The normal dead and live loads were combined with the natural phenomena seismic, wind, or ashfall loads to produce equivalent static total loads on the structure. Dead and live loads are normal static loads, ashfall is an extreme static load, and seismic and wind are extreme dynamic loads.

To convert dynamic seismic loads into static loads, the methods recommended in ASCE 4-86 were employed. Site-specific response spectra peak accelerations were multiplied by the mass of the structure to simulate dynamic earthquake effects. This method is acceptable only for simple regular structures such as the 2736Z Building.

Dynamic wind loads are calculated from the ANSI A58.1 standard (ANSI 1982). This standard provides a simple formula for calculating a static wind load given a dynamic wind speed value that is multiplied by gust response factors.

4.5 EQUIVALENT STATIC ANALYSIS JUSTIFICATION

The "Equivalent Static Analysis Method" is a solution to a single-degree-of-freedom model where the response spectrum represents an exact dynamic analysis. Structures that have a single degree of freedom or nearly so can be dynamically analyzed by this method. The justification and use of the equivalent static analysis method for simple regular structures is discussed in the following documents: UCRL 15910 Appendix A, ASCE 4-86 Section 3.2.5, and UBC 1991 Section 2333.

These documents agree that the equivalent static method should be used only on simple regular structures that have uniform mass and stiffness distributions. The ASCE document further defines simple and regular to mean cantilever or simple multi-degree-of-freedom structures. The UBC document provides quantifiable measures of what regular, uniform mass, and uniform stiffness mean.

UBC Tables 23-M and 23-N define what constitutes irregularities in dimensions, mass, and stiffness. The vertical irregularities of Table 23-M are for multi-story buildings only and thus are not applicable to the 2736Z facilities. The plan irregularities of Table 23-N are applicable and shall be addressed item by item as follows.

A. Torsional Irregularities must be limited to about 10% total eccentricity based on displacements at the ends of the buildings. The 2736Z facilities all have less than 5% eccentricity.

B. Re-entrant corners between floor levels must be limited to 15% of the total building dimension. The 2736Z facilities are all single-story structures.

C. Diaphragms in regular buildings must not have abrupt variations in stiffness, openings greater than 50% of the gross area, or greater than 50% stiffness variation between floor levels. The 2736Z facilities diaphragms meet these cirterian.

D. Regular building shall not have discontinuities in lateral load path including out-of-plane members. The 2736Z facilities use shear wall diaphragms for lateral load resistance, and all lateral load members are in-line with these diaphragms.

E. Regular buildings shall not have nonparallel lateral load systems. The 2736Z facilities all have parallel lateral load systems.

Based on the preceding discussions and reference documents, the conclusion is that the 2736Z facilities meet the description of simple, regular structures. Therefore, use of the equivalent static method of analysis is justified.

4.6 STRUCTURAL COMPONENT EVALUATION

Below is a list of the structural components analyzed in the course of this evaluation. The analysis items checked are listed for each component.

APPENDIX A

LOAD CALCULATION:

LATERAL LOAD RESISTANCE:

ROOF:
- Design loads and load combinations
- Two-way slab strength analysis
- Shear-diaphragm strength analysis
- Wall-beam strength analysis
- Reinforcing detail check

WALL PANELS:
- Design loads and load combinations
- Shear-diaphragm strength analysis
- Reinforcing detail check

COLUMNS:
- Design loads and load combinations
- Compressive strength analysis
- Shear strength analysis
- Combined direction analysis
- Reinforcing detail check

FOUNDATION WALLS:
- Design loads and load combinations
- Compressive strength analysis
- Shear strength analysis
- Reinforcing detail check

FOOTING:
- Design loads and load combinations
- Bending and shear strength analysis
- Overturning and sliding resistance

DEFLECTIONS:
- Shear wall deflection
- Out-of-plane deflection
- Soil-spring deflection

APPENDIX B

COLUMN-SHEAR WALL:
- Column-Foundation Detail & Capacity Check
- Shear-Wall Unit computer model by RA Giller
- Shear-Wall Unit computer model by EO Weiner

5.0 REFERENCES

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STRUCTURAL EVALUATION OF THE 2736Z BUILDING FOR SEISMIC LOADS

APPENDIX A

CALCULATIONS

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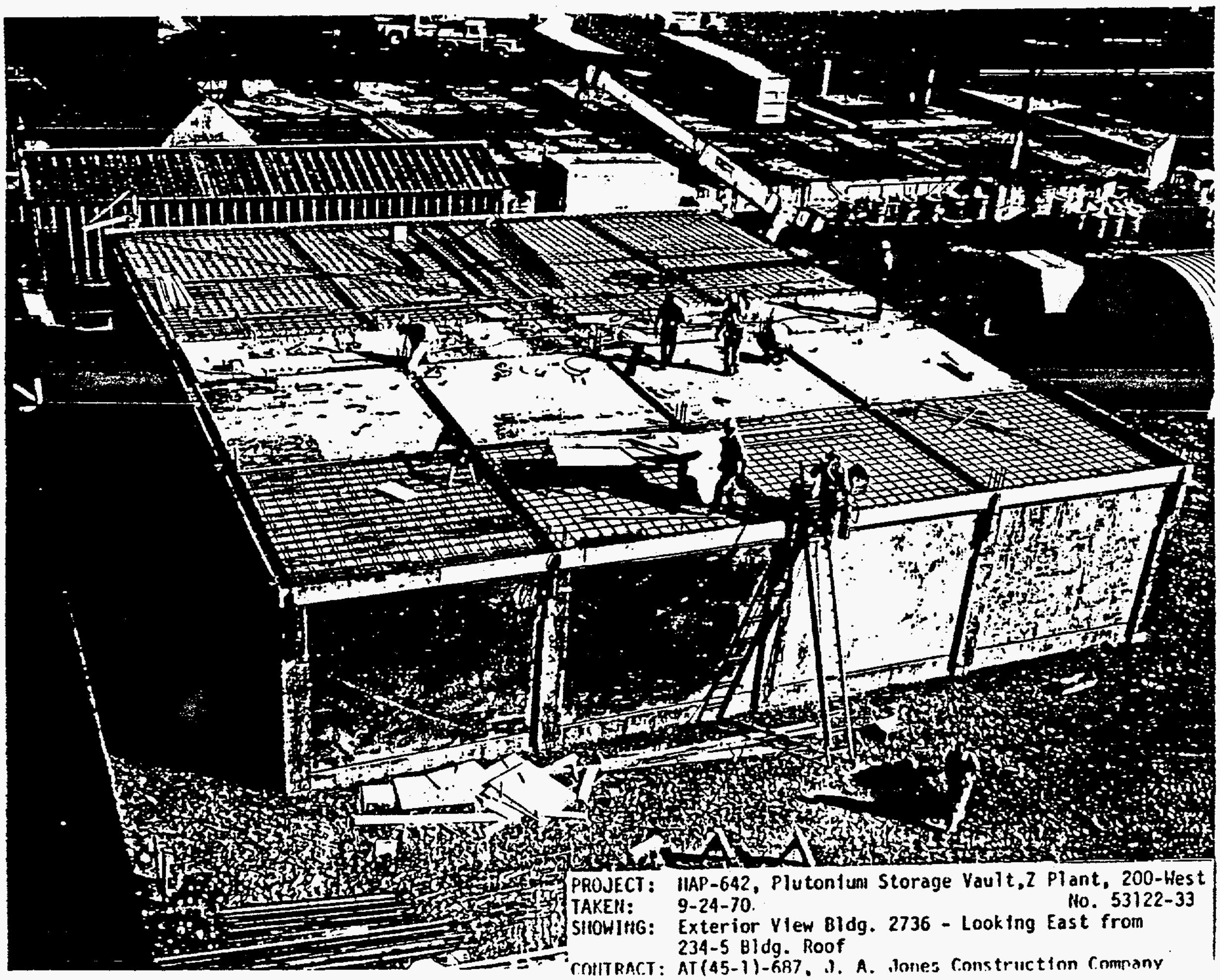
CONSTRUCTION PHOTO

2736Z BUILDING



PROJECT: HAP-642, Plutonium Storage Vault,Z Plant, 200-West
TAKEN: 9-24-70 No. 53122-33
SHOWING: Exterior View Bldg. 2736 - Looking East from 234-5 Bldg. Roof
CONTRACT: AT(45-1)-687, J. A. Jones Construction Company

DESIGN VERIFICATION METHOD

The need for design verification has been reviewed with the method selected as indicated below:

____X____ Independent Review

________ Alternate Calculations

________ Qualification Testing

________ Formal Design Review

T. J. Conrads [signature]
Cognizant/Project/Design Manager

SD # WHC-SD-CP-SA-023 R1

ECN # 615157

DWG(S) # N/A

CHECKLIST FOR INDEPENDENT REVIEW

Document: Structural Evaluation of the 2736Z Building For Seismic Loads

Author: R. A. Giller

Yes	No	N/A	
[✓]	[ ]	[ ]	Problem completely defined.
[✓]	[ ]	[ ]	Necessary assumptions explicitly stated and supported.
[ ]	[ ]	[✓]	Computer codes and data files documented.
[✓]	[ ]	[ ]	Data used in calculations explicitly stated in document.
[✓]	[ ]	[ ]	Data checked for consistency with original source information as applicable.
[✓]	[ ]	[ ]	Mathematical derivations checked including dimensional consistency of results.
[✓]	[ ]	[ ]	Models appropriate and used within range of validity or use outside range of established validity justified.
[✓]	[ ]	[ ]	Hand calculations checked for errors.
[ ]	[ ]	[✓]	Code run streams correct and consistent with analysis documentation.
[ ]	[ ]	[✓]	Code output consistent with input and with results reported in analysis documentation.
[✓]	[ ]	[ ]	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
[✓]	[ ]	[ ]	Safety margins consistent with good engineering practices.
[✓]	[ ]	[ ]	Conclusions consistent with analytical results and applicable limits.
[✓]	[ ]	[ ]	Results and conclusions address all points required in the problem statement.

MANDATORY Software QA Log Number None

E.O. Weiner
Reviewer

2-8-94
Date

09/22/93

DEAD LOADS

Roof:

Concrete roof 6.5" slab =	81.25 psf
Roofing, 3 ply felt and gravel =	5.5 psf
Load = (81.25 + 5.5) X 65.5' X 57.16' =	**325 kips**

Roof Beams:

Concrete beam 2.5" deep, 12" wide, 13.3' long

Load = 2.5" X 12" X 13.3' X 16 beams X 150 pcf / 144 si = **7 kips**

Walls beams:

Concrete beams

8" X 12.5" X 57'long X 2 beams X 150 pcf / 144 si =	12 kips
3.5" X 11" X 57'long X 2 beams X 150 pcf / 144 si =	5 kips
8" X 8" X 30'long X 2 beams X 150 pcf / 144 si =	4 kips
8" X 11" X 65.5'long X 2 beams X 150 pcf / 144 si =	12 kips
Total load for wall beams =	**33 kips**

Interior Walls:

Concrete wall 8" thick

8" X 10.5' X 13.16' X 12 walls X 150 pcf / 12 in. = **166 kips**

Exterior Walls:

Concrete wall 8" thick

8" X 10.5' X 13.16' X 16 walls X 150 pcf / 12 in. =	221 kips
8" X 10.5' X 7' X 2 walls X 150 pcf / 12 in. =	15 kips
Total load for Exterior walls =	**236 kips**

6" Add-on Walls:

Concrete wall 6" thick

0.5' X 11.25' X 181' X 150 pcf =	**153 kips**

Columns:

Columns 10.416' high

8" X 12" X 10.416' X 6 columns X 150 pcf / 144 si =	6 kips	
11" X 12" X 10.416' X 12 columns X 150 pcf / 144 si =	17 kips	
13" X 13" X 10.416' X 4 columns X 150 pcf / 144 si =	7 kips	
12" X 13" X 10.416' X 8 columns X 150 pcf / 144 si =	14 kips	
Total load for columns =		**44 kips**

Floor Slab:

Floor Slab 6" thick

0.5' X 7'wide X 55.4'long X 150 pcf =	29 kips	
0.5' X 27.67'wide X 27.3'long X 4 slabs X 150 pcf =	227 kips	
Total load for floor slab =		**256 kips**

Foundation Walls:

Foundation Wall 8" thick

0.67' X 1.9'high X 340'long X 150 pcf =	65 kips	
0.67' X 4.16'high X 84'long X 150 pcf =	35 kips	

Foundation Wall 6" thick

0.5' X 1.9'high X 123'long X 150 pcf =	18 kips	
0.5' X 4.16'high X 57'long X 150 pcf =	18 kips	
Total load for foundation walls =		**136 kips**

Column Pedestal:

Column Pedistal 1.6′ or 0.75′ high

14" X 14" X 1.6′high X 12 col. X 150 pcf / 144 si = 4 kips

11" X 14" X 1.6′high X 12 col. X 150 pcf / 144 si = 3 kips

8" X 12" X 0.75′high X 6 col. X 150 pcf / 144 si = 1 kip

Total load for Column Pedistals = 8 kips

Footings:

Column Footings 1′ thick

4′ X 11.67′ X 1′thick X 3 footings X 150 pcf = 21 kips

4′ X 4′ X 1′thick X 24 footings X 150 pcf = 58 kips

Wall Footing 1′ thick

2′ X 10′ X 1′thick X 30 footings X 150 pcf = 90 kips

Total load for footings = 169 kips

Dead Load Summary:

Roof =	325 kips
Roof Beams =	7 kips
Wall Beams =	33 kips
Interior Walls =	166 kips
Exterior Walls =	236 kips
6" Add-on Walls =	153 kips
Columns =	44 kips
Floor Slab =	256 kips
Foundation Walls =	136 kips
Column Pedistal =	8 kips
Footings =	169 kips
Total	1533 kips

LIVE LOADS (SDC-4.1 and ANSI-A58.1)

Roof:

Roof live load = 20 psf X 65.5' X 57.16' = 75 kips

Roof snow load = 20 psf X 65.5' X 57.16' = 75 kips

use either roof live load or snow load but not both

Total load = **75 kips**

WIND LOADS (SDC-4.1 and ANSI A58.1)

Straight wind pressure

$q_z = 0.00256\ K_z\ (IV)^2$

$K_z = 0.8$ for heights upto 15' and exposure category C

$I = 1.0$

$V = 90$ mph

$q_z = 16.6$ psf

Building wind pressure

$p = q_z\ (GhCp - GCpi)$

p (windward wall) = 16.6 psf (1.32 X 0.8 + 0.25) = + 21.7 psf

p (leeward wall) = 16.6 psf (1.32 X -.5 + 0.25) = - 6.8 psf

Total lateral pressure = 21.7 + 6.8 = 28.5 psf

Total lateral force = 28.5 psf X 12' X 65.5' = **22 kips**

p (Roof) = 16.6 psf (-0.7) = - 11.6 psf

Total Upward force = 11.6 psf X 65.5' X 57.16' = **43 kips**

SEISMIC LOADS (SDC-4.1 and UCRL-15910)

Vertical Seismic Loads:

2/3 X (7% damped, 0.2g spectra peak) X Dead Weight D

2/3 X (0.38g) W = 0.25D *

Horizontal Seismic Load:

(7% damped, 0.2g spectra peak) X Dead Weight D

(0.38g) W = 0.38D *

ASH LOADS (SDC-4.1)

Ash Load: (24 psf extreme load similar to wind and earthquake)

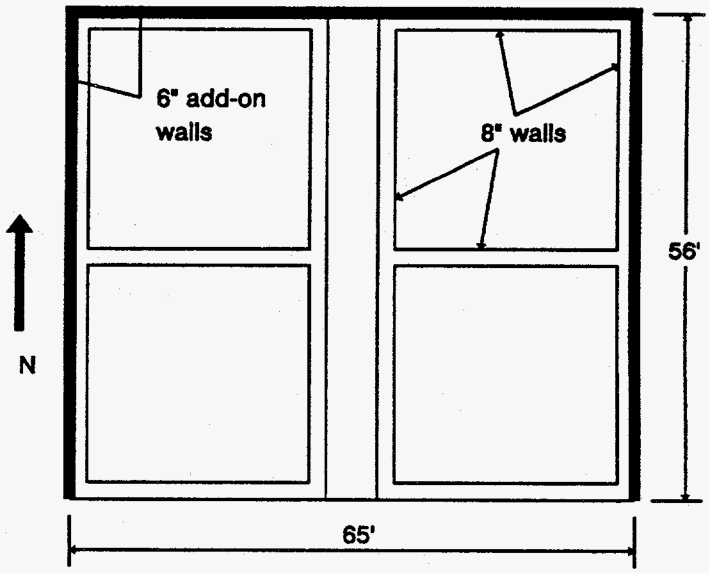
24 psf X 65.5' X 57.16' = 90 kips

* See SDC-4.1 ~~See~~ Table 3 for spectra peak and frequency values.

Lateral loads are resisted by wall shear, 4 - 56′ long walls in the NS direction, and 6 - 28′ long walls in the EW direction.

The roof slab acts in diaphragm shear as a rigid plate to distribute lateral forces evenly between the shear walls.

UBC requires that accidental torsion be accounted for in the design of the lateral force resisting system. this torsion is derived from an offset of the lateral seismic force. The amount of the offset is 5% of the building dimension in each direction.



5% of 56′ = 2.8′ 5% of 64′ = 3.3′

The EW direction has a designed offset from the 6" add-on walls. The designed offset is as follows;

total lateral load = 583^k

lateral load for the 6" wall (north side only)

$$0.5' \times 11.25' \times 67' \times 0.150\ \text{ksf} \times 0.38g = 21.5^k$$

$$\text{Offset} = \frac{(583 - 22)(56'/2) + 22(56')}{583} - \frac{56'}{2} = 1.1'$$

Total NS offset = 2.8′ + 1.1′ = 3.9′

Total EW offset = 3.3′

DESIGN CALCULATION

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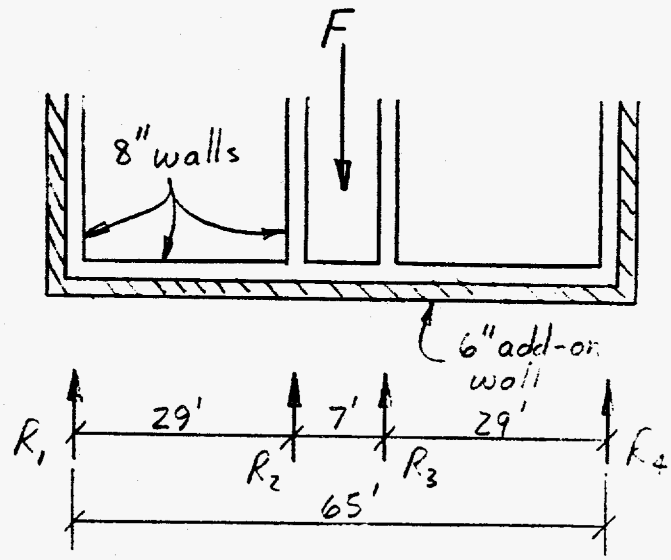
(1) Drawing ______ (2) Doc. No. ______ (3) Page A2-2 of ____
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator RA Giller Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10) Lateral Resistance N.S. Direction

Lateral Force, F

E.W. offset of N.S. Load = 3.3'

Moment M = 3.3(F)



Walls at R_2 & R_3 have 2 door openings each. Subtracting the door widths from the overall wall length yields an effective wall length = 56' − 4.3' − 4.3' = 47.4'

Based on effective wall lengths, relative resistances can be determined. $R_2 = R_3 = (47.4'/56')R_1 = .85 R_1$

The 6" add-on walls are considered to add mass but no stiffness or strength to the structure.

the case where the 6" wall does add stiffness and strength to the structure is discussed on page A2-7.

DESIGN CALCULATION

(1) Drawing ______ (2) Doc. No. ______ (3) Page A2-3 of ______

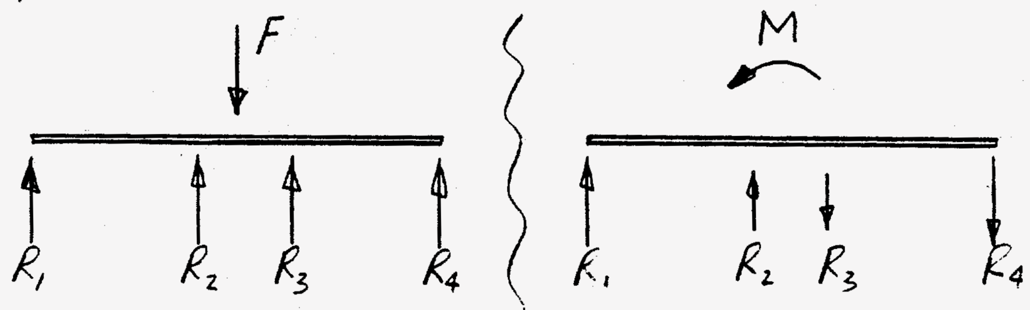
(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator R A Hille Date 2-8-94

(9) Checker Ed Weaver Date 2-8-94

(10)



$R_2 = R_3 = .85R_1 = .85R_4$

$1.85(R_1 + R_4) = F$

$3.7R_1 = F$

$R_1 = F/3.7$

$R_2 = .85F/3.7 = F/4.35$

$R_1 = R_4 \qquad R_2 = R_3$

$R_1 = \frac{65'}{7'}R_2 = 9.3R_2$

$M = R_1(65') + R_2(7')$

$M = 611R_2 = 65.7R_1$

Total Reactions

$$R_1 = \frac{F}{3.7} + \frac{M}{65.7} = \frac{F}{3.7} + \frac{3.3F}{65.7} = .32F$$

$$R_2 = \frac{F}{4.35} + \frac{M}{611} = \frac{F}{4.35} + \frac{3.3F}{611} = .235F$$

$$R_4 = \frac{F}{3.7} - \frac{3.3F}{65.7} = .22F$$

$$R_3 = \frac{F}{4.35} - \frac{3.3F}{611} = .225F$$

$$\left.\begin{matrix} R_1 \\ R_2 \\ R_4 \\ R_3 \end{matrix}\right\} = F$$

DESIGN CALCULATION

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(1) Drawing ______ (2) Doc. No. ______ (3) Page A2-4 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator R.A. Hille Date 2-8-94

(9) Checker Ed Weiner Date 2-8-94

(10) Lateral Resistance EW direction:

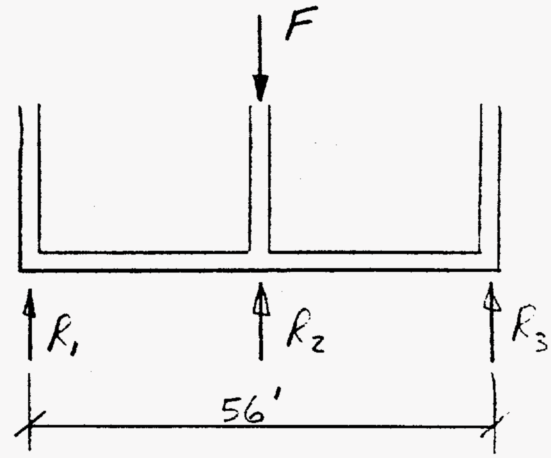
Lateral Load = F

N.S. offset of EW load

= 3.9'

Moment M_z

= 3.9 F



$$R_1 = \frac{F}{3} + \frac{M_z}{56'} = \frac{F}{3} + \frac{3.9F}{56} = .403\ F$$

$$R_2 = \frac{F}{3} = .333\ F$$

$$R_3 = \frac{F}{3} - \frac{M}{56} = .263\ F$$

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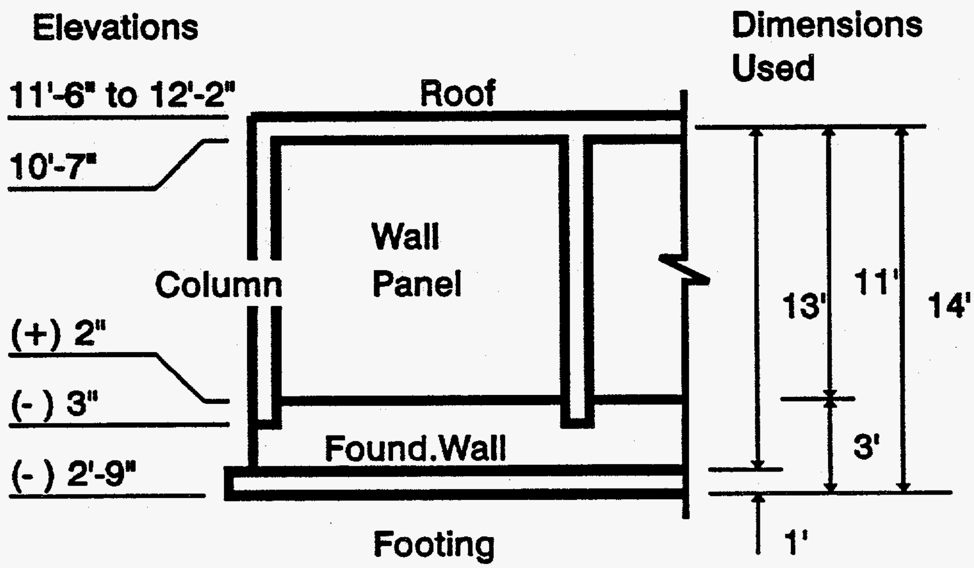
Vertical Distribution Of Lateral Loads:



Typical Wall Section

The lateral loads are distributed as follows;

- Roof, Roof Beams, Wall beams, + 1/2 [Columns and Walls] are placed at the roof level; 14' above the base and 11' above the foundation.
- 1/2 [Columns and Walls] + Foundation and Floor are placed at the Wall-Foundation interface; 3' above the base.

Actual lateral Loads:

@ Roof level
F = [(325 + 7 + 33) + 1/2(166 + 236 + 153 + 44)] 0.38 = 253 kips

@ Wall bottom level
F = [1/2(166 + 236 + 153 + 44)] 0.38 + 253 = 367 kips

@ Foundation wall top
F = (256 + 136 + 8) 0.38 + 367 = 519 kips

@ Base level
F = (169) 0.38 + 519 kips = 583 kips

@ Roof Level									
	R_1		R_2		R_3		R_4		
NS	81	+	59	+	57	+	56	=	253 kips
EW	102	+	84	+	67		-	=	253 kips
@ Wall Bottom									
	R_1		R_2		R_3		R_4		
NS	117	+	86	+	83	+	81	=	367 kips
EW	148	+	122	+	97		-	=	367 kips
@ Foundation Wall Top									
	R_1		R_2		R_3		R_4		
NS	166	+	122	+	117	+	114	=	519 kips
EW	210	+	173	+	136		-	=	519 kips
@ Base									
	R_1		R_2		R_3		R_4		
NS	187	+	137	+	131	+	128	=	583 kips
EW	235	+	194	+	154	+	-	=	583 kips

If the connection between the 6" add-on walls and the 8" precast walls is adequate to transfer all lateral loads, then the overall strength and stiffness will be affected.

Wall stiffness is based on wall area, thickness X length. For the case where the 6" wall has no stiffness the wall stiffness distribution is as follows;

outside wall = 8" X 56" X 12" = 5376 si

inside wall = 8" X 47.4' X 12" = 4550 si

Relative stiff. for outside wall = 5376 / (5376 + 4550) = 54%

Relative stiff. for inside wall = 46%

For the case where the 6" wall has stiffness the total stiffness distribution is as follows;

outside wall = 14" X 56' X 12" = 9408 si

inside wall = 8" X 47.4' X 12" = 4550 si

Rel. stif. outside wall = 9408R / (9408R + 4550R) = 67%

Rel. stif. inside wall = 33%

Stiffness increase;

67 / 54 = 1.24 = 24% increase.

As the stiffness increases so do the loads, a stiffness increase of 24% equals a load increase of 24%. However, the wall thickness increases from 8" to 14" or an increase of 14 / 8 = 1.75 (75% increase) which increases the wall strength by 75%.

In conclusion, the thicker stiffer wall increases the loads 24% and the wall strength 75% which is a less severe condition than neglecting the stiffness and strength contributions of the 6" add on wall.

A3.0 ROOF EVALUATION

The 2736Z building roof slab is a reinforced concrete slab, poured monolithic with the roof beams and columns and anchored by dowel action to the precast wall panels. Construction details indicate that the slab was designed as a Two-Way slab. The slab also serves as a lateral load diaphragm that ties together the building shear walls for lateral loading.

- **DESIGN LOADS AND LOAD COMBINATIONS**

  Design loads are specfied in SDC-4.1 ~~rev. 11~~ rev 12, UCRL-15910 and ANSI A58.1. Load combinations are listed in UCRL-15910 and ACI-349.

- **TWO-WAY SLAB BENDING AND SHEAR STRENGTH ANALYSIS**

  Two-way slab design for vertical loads follows the methods of ACI-349 chapter 13.

- **SHEAR DIAPHRAGM STRENGTH ANALYSIS**

  The roof slab diaphragm action for lateral loads follows ACI-349 chapter 11 shear strength methods.

- **WALL BEAM COMPRESSIVE STRENGTH ANALYSIS**

  Compression is developed in the slab edge wall beams as lateral loads are transfered from the roof slab to the wall panels. ACI-349 chapter 10 provides compressive strength evaluation methods.

- **REINFORCING DETAIL CHECK**

  Reinforcing details are found in several ACI-349 chapters. Minimum reinforcing is specified in chapter 7, rebar anchorage in chapter 12, flexure and shear strengths are found in chapters 10 and 11, and two-way slab details in chapter 13.

DESIGN LOADS

Vertical Loads:

Dead Loads

roof self weight =	81.25 psf	X 3744 sf =	304 kips
3 ply roofing weight =	5.5 psf	X 3744 sf =	21 kips

Live Load

roof live load =	20 psf	X 3744 sf =	75 kips
snow load =	20 psf	X 3744 sf =	75 kips

Ash Load

ash load =	24 psf	X 3744 sf =	90 kips

Wind Load

wind load =	- 12 psf	X 3744 sf =	- 45 kips

Earthquake Load

earthquake load = $\pm$ 0.25 X (81.25 + 5.5) = 22 psf

$\pm$ 22 psf X 3744 sf = $\pm$ 82 kips

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A3-3

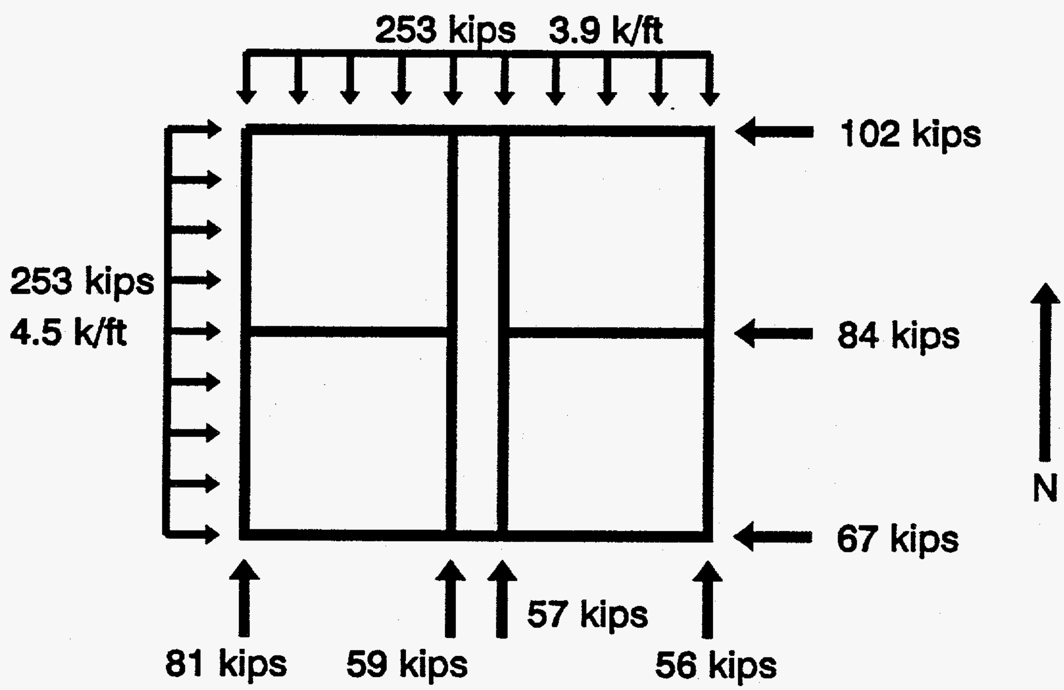
Horizontal Earthquake Loads:

From the "Lateral Load Resistance" section the total lateral load at the roof level is;

253 kips

The following diagram shows; Lateral loads at the roof level,

Shear wall reactions due to a lateral loads and due to accidental offset of the lateral loads.



Building Plan

LOAD COMBINATIONS (ACI-349 and UCRL:-15910)

Per ASCE 7-88 (ANSI-A58.1), roof live or snow loads may be used but not both.

Per SDC-4.1, ash, wind and earthquake loads are extreme loads and are not used at the same time.

Vertical Loads:

ACI-349 Section 9.2:

1.) 1.4D + 1.7(Lr or S)

1.4(86.75) + 1.7(20) = 156 psf

4.) D + (Lr or S) + E

86.75 + 20 + 22 = 129 psf

4.) 0.9D - E

0.9(86.75) - 22 = 56 psf

4.) D + (Lr or S) + A

86.75 + 20 + 24 = 131 psf

5.) 0.9D - W

0.9(86.75) - 12 = 66 psf

UCRL-15910 Section 5.2.3:

5.4(a) D + 1.3W

86.75 + 1.3(-12) = 71 psf

5.4(b) 1.1D + 0.5L + 1.2W

1.1(86.75) + 0.5(20) + 1.2(-12) = 91 psf

Horizontal Loads:

ACI-349, 4.) E = 253 kips — 253 kips

ACI-349, 5.) W = $\frac{(29psf)\ (12' \times 64')}{1000}$ = 22 kips

UCRL-15910, 5.4(a) 1.3W = $\frac{1.3\ (29psf)\ (12' \times 64')}{1000}$ = 29 kips

TWO-WAY SLAB BENDING & SHEAR: (Vertical Loads)

The roof slab is designed and constructed as a Two-Way slab system and shall be analyzed as such, according to ACI-349 Chapter 13 rules.

Use the "Direct Design Method" of chapter 13 :

- A minimum of 3 spans in each direction is required; 2736Z has 4 ~~spansin~~ spans in the NS direction and 5 in the EW direction.
- Ratio of longest to shortest span, not greater than 2.0; 2736Z has 13.33'/7' = 1.9 : O.K.
- ~~Successive~~ Succesive span ratios not greater than 1/3 of the longer; 2736Z has 7'/14' = 1/2 for one span, but since the shorter span has a weaker cross-section it is roughly equivalent to the 1/3 requirement.
- Columns may be offset by no more than 10%; 2736Z has 0% offsets.
- All loads are from gravity, and live loads do not exceed 3 times the dead load.

Total Factored Static Moment:

$$M_o = \frac{w_u l_2 l_n^2}{8}$$

w_u = factored load

l_2 = panel width

l_n = clear span, from the face of supports

$$M_o = \frac{156 psf (14.167)(13.33)^2}{8}$$

M_o = 49,088 ft-lbs = 589 in-kips

Distribution of Static Moment: (ACI Section 13.6.3)

Interior Span Negative Moment =	$0.65\ M_o$	ACI, 13.6.3.2
Interior Span Positive Moment =	$0.35\ M_o$	ACI, 13.6.3.2
End Span Interior Negative Moment =	$0.70\ M_o$	ACI, 13.6.3.3
End Span Exterior Negative Moment =	$0.30\ M_o$	ACI, 13.6.3.3
End Span Positive Moment =	$0.50\ M_o$	ACI, 13.6.3.3

Distribution of Pos. & Neg. Moments to Panel Strips: (ACI 13.6.4)

$l_2/l_1 =$ 14′ / 14.167′ = 0.99

$\alpha_{1\ \text{approx.}} =$ bh^3 beam / bh^3 slab = $17(9^3)\ /\ 151(6.5^3) =$ 0.30

$\alpha_1 l_2/l_1 =$ ~~0.20~~ 0.30

$\beta_t =$ 0.0, for masonary edge wall
2.5, for a monolithic concrete wall

$\beta_t =$ use 1.25 for a slab with an edge beam and a partially restrianing precast wall.

Interior Negative Moment ACI, 13.6.4.1

Column Strip = 75% of the negative moment

Middle Strip = 25% of the negative moment

Exterior Negative Moment ACI, 13.6.4.2

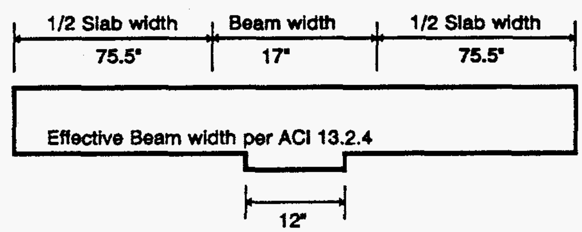
Column Strip = 87.5% of the negative moment

Middle Strip = 12.5% of the negative moment

Positive Moment ACI, 13.6.4.4

Column Strip = 64.0% of the positive moment

Middle Strip = 36.0% of the positive moment



Final Moment Distributions: When there are beams between supports.

Moment Type And Location	Moment in Panel	Column Strip	Middle Strip
Interior Span Negative Moment	$0.65M_o$	75%	25%
Interior Span Positive Moment	$0.35M_o$	64%	36%
End Span Interior Negative Moment	$0.70M_o$	75%	25%
End Span Exterior Negative Moment	$0.30M_o$	87.5%	12.5%
End Span Positive Moment	$0.50M_o$	64%	36%

Design Moments: *** (M_o = 589 in-kips)

Interior Span Negative Moment;	Column Strip =	$0.65\ (0.75)\ M_o$ =	287 in.-kips
	Middle Strip =	$0.65\ (0.25)\ M_o$ =	96 in.-kips
Interior Span Positive Moment;	Column Strip =	$0.35\ (0.64)\ M_o$ =	132 in.-kips
	Middle Strip =	$0.35\ (0.36)\ M_o$ =	74 in.-kips
End Span Interior Negative Moment;	Column Strip =	$0.70\ (0.75)\ M_o$ =	309 in.-kips
	Middle Strip =	$0.70\ (0.25)\ M_o$ =	103 in.-kips
End Span Exterior Negative Moment;	Column Strip =	$0.30\ (0.875)\ M_o$ =	155 in.-kips
	Middle Strip =	$0.30\ (0.125)\ M_o$ =	22 in.-kips
End Span Positive Moment;	Column Strip =	$0.50\ (0.64)\ M_o$ =	188 in.-kips
	Middle Strip =	$0.50\ (0.36)\ M_o$ =	106 in.-kips

*** See further adjustments, next page.

Column Strip Positive Moments:

Shall be further divided between the beam and the slab

For $\alpha_1 l_2/l_1 = 1.0$	Column strip beam resists 85% of strip moment
For $\alpha_1 l_2/l_1 = 0.0$	Column strip beam resists 0% of strip moment
For $\alpha_1 l_2/l_1 = 0.30$	Column strip beam resists 0.30(85%) = 25% of the strip moment. And the column strip slab resists 100 - 25 = 75%.

Interior Span, Column Strip, Positive Moment;	Beam =	132 X .25 =	33 in.-kips
	Slab =	132 X .75 =	99 in.-kips
End Span, Column Strip, Positive Moment;	Beam =	188 X .25 =	47 in.-kips
	Slab =	188 X .75 =	141 in.-kips

Chapter 13, Section 13.6.7

This section allows a further 10% readjustment in moments so long has the absolute sum of positive + negative moments ~~dose~~ does not fall below the minimum

$$\frac{w_u l_2 l_u^2}{8}$$

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Total Factored Shear Load:

$Vu = \dfrac{156 \text{ psf}}{1000} \times 13.33' \times 13.33' =$ 27.7 kips, per panel

13.9 kips, each end of member

Distribution of Shear Load:

Column strip, for Beam with $\alpha_1 = 1.0$: Fig. R13.6.8

$Vu = \dfrac{156 \text{ psf}}{1000} \times \dfrac{14.2'}{2} \times 13.33 =$ 14.8 kips, per panel

Column strip, for Beam with $\alpha_1 = 0.30$

Beam Vu = 0.30(14.8) = 4.4 kips, per panel

2.2 kips, each end

Slab Vu = 14.8 - 4.4 = 10.4 kips, per panel

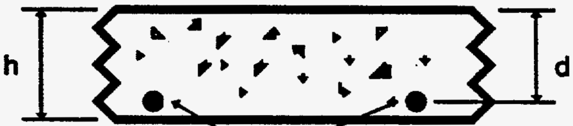
5.2 kips, each end

Middle strip:

Vu = 27.7 - 14.8 = 12.9 kips, per panel

6.5 kips, each end

Middle Strip, Positive Moment Capacity:



h = 6.5"

d = h - 1.5" cover - 1/2 bar ϕ

d = 4.8"

b = $\frac{14.2'}{2}$ = 7.1' = 85"

column strip width = 7.1'

middle strip width = 7.1'

#3 bars @ 10" centers;

(12"/10") 7.1' = 8.52 bars : 8 bars

As (8 - #3 bars) = 0.88 si

From Whitney stress block theory;

Asfy = 0.85f'c (ab) :

$a = \frac{Asfy}{0.85\ f'c\ (b)}$ a = 0.16"

Moment = ϕAsfy (d - a/2)

ϕMn = 0.9 (0.88 X 40) (4.8 - 0.16/2) = 150 in-kips

Slab Shear Capacity:

Shear = $\phi 2\sqrt{f'c}\ bd$

ϕVn = $\phi 2\sqrt{3000}$ (85")(4.8")

ϕVn = ϕ45 kips = 38 kips

Column Strip (Slab only), Positive Moment Capacity:

$\phi Mn = 150$ in.-kips $\left(\frac{7.1' - 1'}{7.1'}\right) =$ 129 in.-kips

Column Strip (Beam only), Positive Moment Capacity:

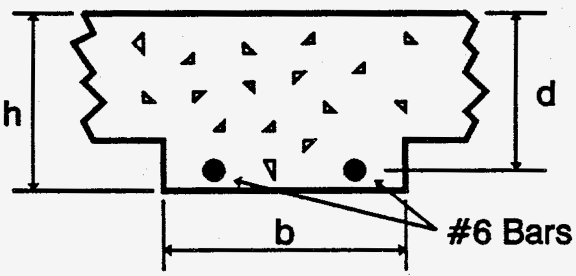
h = 9"

b = 12"

d = h - 1.5" cover - 1/2 bar ϕ

d = 7.1"

As (2 - #6 bars) = 0.88 si



From Whitney Stress Block Theory and Equilibrium;

$Asfy = 0.85f'c\,(ab)$

$a = \frac{Asfy}{0.85\ f'c\ (b)}$ a = 1.15"

Moment = Asfy (d - a/2)

$\phi Mn = .9Asfy\,(7.1" - \frac{1.15"}{2}) =$ 207 in.-kips

Beam Shear Capacity:

Shear = $\phi 2\sqrt{f'c}\ bd$

$\phi Vn = \phi 2\sqrt{3000}\ (12")(7.1")$

$\phi Vn = \phi 9.3$ kips = 8 kips

Column & Middle Strip, Interior Neg. Moment capacity:

h = 6.5"

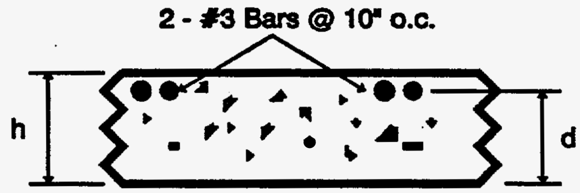
d = 4.8"

b = 85"

2 - #3 Bars @ 10" centers
(#3 Bars @ 5" centers)

(12"/5") 7.1′ = 17.04 bars

As (17 - #3 Bars) = 1.87 si



From Whitney Stress Block Theory;

$$Asfy = 0.85f'c\ (ab)$$

$$a = \frac{Asfy}{.85f'c\ (b)} \qquad a = 0.345"$$

$$\text{Moment} = .9Asfy\ (d - a/2)$$

$$\phi Mn = .9Asfy\ (4.8 - .345/2) = \qquad 312 \text{ in.-kips}$$

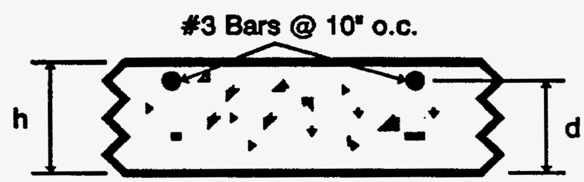
Column & Middle Strip, End Span Neg. Mom. Capacity:

h = 6.5"

d = 4.8"

b = 85"

As for 8 - #3 Bars = 0,88 si



From Whitney Stress Block Theory;

$$Asfy = 0.85f'c\ (ab)$$

$$a = \frac{Asfy}{0.85f'c\ (b)} \qquad a = 0.16"$$

$$\text{Moment} = 0.9Asfy\ (d - a/2)$$

$$\phi Mn = 0.9Asfy\ (4.8 - 0.16/2) = \qquad 150 \text{ in.kips}$$

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A3-13

Strength Check:

Moments : in.-Kips, Shears : Kips

Interior Span:

(-) Moment, Column Strip =	287 < 312	O.K.
Middle Strip =	96 < 312	O.K.
(+) Moment, Column Strip, Beam =	33 < 207	O.K.
Column Strip, Slab =	99 < 129	O.K.
Middle Strip =	74 < 150	O.K.

End Span, Interior Support:

(-) Moment, Column Strip =	309 < 312	O.K.
Middle Strip =	103 < 312	O.K.

End Span, Exterior Support:

(-) Moment, Column Strip =	155 > 150	*********
Middle Strip =	22 < 150	O.K.

End Span,

(+) Moment, Column Strip, Beam =	47 < 207	O.K.
Column Strip, Slab =	141 > 129	*********
Middle Strip =	106 < 150	O.K.

********* Using the 10% moment redistribution from 13.6.7:
155 in.-kips minus 10% = 139.5 < 150 and thus O.K.
141 in.-kips minus 10% = 126.9 < 129 and thus O.K.

Shear,

Column Strip, Beam Shear =	2.2 < 8.0	O.K.
Column Strip, Slab Shear =	5.2 < 38	O.K.
Middle Strip =	6.5 < 38	O.K.

DESIGN CALCULATION

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(9) Checker EO Werner Date 2-8-94

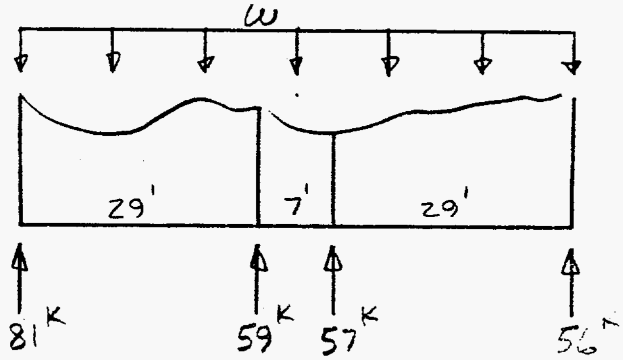
(10) Roof Diaphragm Action

Loads and wall reactions are from the "Lateral Load Resistance" section.

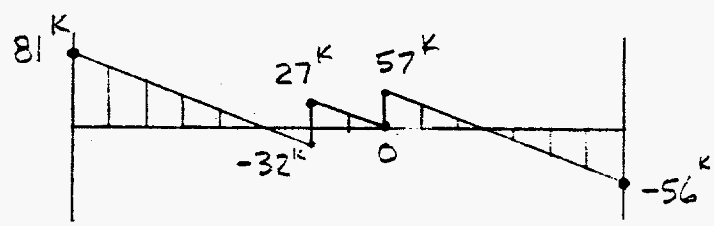
N.S. Loads

253^K

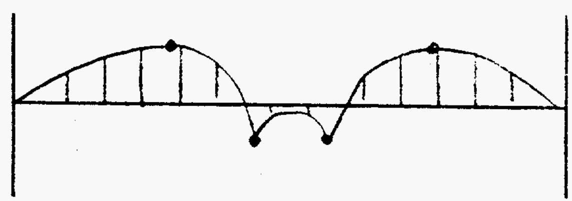
$$w = \frac{253}{65'} = 3.9\ ^K/_{ft}$$



Shear diagram



Moment diagram



Maximum moment @ any point will not exceed $\frac{w\ell^2}{8}$ ∴ so design for $\frac{w\ell^2}{8}$

$$\frac{w\ell^2}{8} = \frac{3.9(29)^2}{8} = 410^{\text{ 'K}}$$

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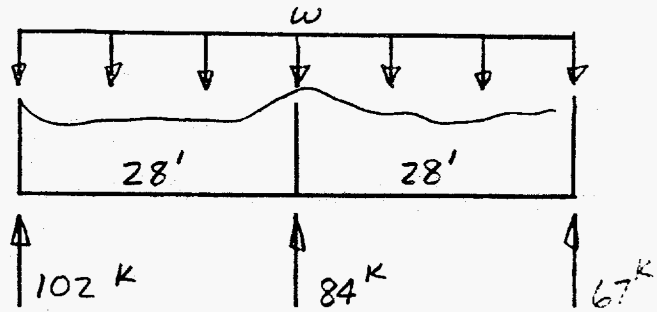
(9) Checker EO Werner Date 2-8-94

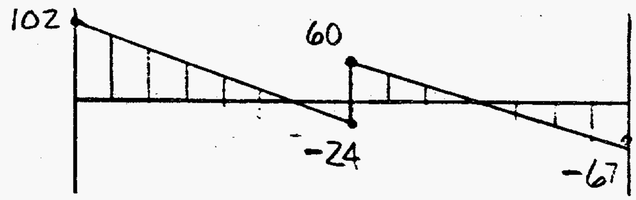
(10)

EW Loads

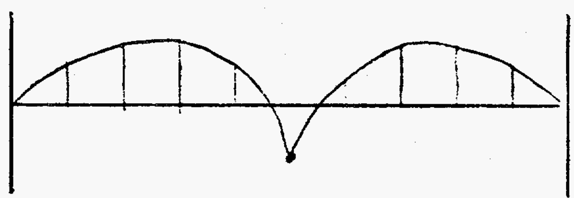
$w = 4.5^K$



Shear diagram:



Moment diagram:



Maximum moment at any point will not exceed $\frac{wl^2}{8}$

$$\frac{wl^2}{8} = \frac{4.5(28)^2}{8} = 441^{\prime \cdot K}$$

Design for larger of NS or EW Forces

102^K shear & $441^{\prime \cdot K}$ moment

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(9) Checker EO Weiner Date 2-8-94

(10)

Shear Capacity Check

$$V_c = 2\sqrt{f_c'}\,bd \quad \text{where } d = .8h$$

$$= 2\sqrt{3000}\,(6.5'')(.8 \times 56' \times 12'')$$

$$= 383^{K}$$

$$V_s = \frac{A_v F_y d}{s}$$

$$= \frac{(.11)(40\text{ ksi})(.8 \times 56' \times 12'')}{10''}$$

$$= 237^{K}$$

$$\phi V_n = .85(383 + 237) = 527^{K} > 102^{K} \quad \text{O.K.}$$

Moment Capacity Check

use wall beam reinf. as tension reinf. for the in-place roof slab bending.

$A_s = 1.24$ for 4 - #5 bars

$$a = \frac{A_s F_y}{.85 f_c' b} = \frac{1.24(40)}{.85(3)(6.5)} = 3.00$$

$$\phi M_n = .9(A_s F_y)(d - a/2) = .9(1.24 \times 40)(.8 \times 56 \times 12 - 3/2)$$

$$= 23{,}931^{\text{in}\cdot K} = 1994^{\text{ft}\cdot K} > 441^{\text{ft}\cdot K} \quad \text{O.K.}$$

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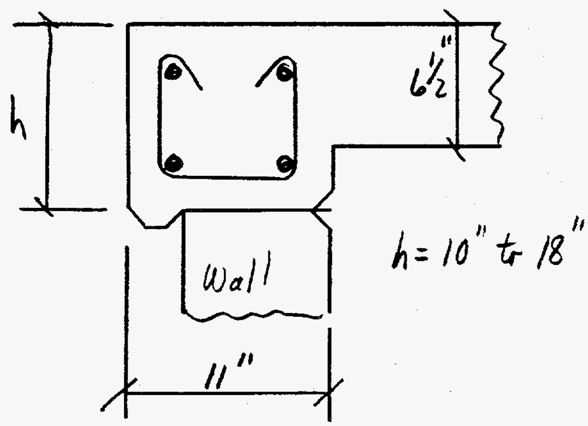
(8) Originator RA Gille Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) WALL BEAM COMPRESSIVE FORCE ANALYSIS

Exterior Wall Beam
Gross Area:

$11'' \times 10'' = 110$ si
to
$11'' \times 18'' = 198$ si

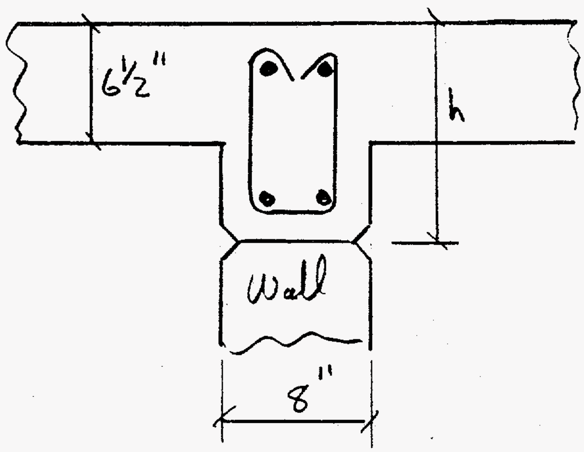


h = 10" to 18"

H-2-26652 Section 39, 40

Interior Wall Beam
Gross Area:

$8'' \times 10 = 80$ si
to
$8'' \times 18'' = 144$ si



h = 10" to 18"

H-2-26652 Section 41

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(10) WALL BEAM CONT.

From the roof shear diagrams, the maximum total shear load to be transfered to wall shear is 102^K along one outside wall.

This 102^K shear load becomes compression in the wall beam before being transfered into the wall shear.

The 102^K compression load is transfered by shear friction and wall reinforcement to the wall, and the maximum compression load in the roof beam is the total load divided by the wall length and multiplied by the rebar spacing.

$$\frac{102^K}{(57' \times 12'')} 5'' \text{ spacing} = 0.75^K$$

Check Capacity: (4-#5 bars $A_s = 1.24 si$)

$$\phi P_n = .8\phi\left[.85 f_c'(A_g - A_s) + f_y A_s\right]$$
$$= .8(.7)\left[.85(3)(80si - 1.24) + 40(1.24)\right]$$
$$= 140^K > 0.75^K$$

O.K.

DESIGN CALCULATION

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(9) Checker EO Weiner Date 2-8-94

(10) Check Shear Friction

ACI-349 Section 11.7

$V_n = A_v f_y \mu$ A_v, steel reinf. for wall panels and columns

30-#3 bars × 4 wall panels

$A_v = .11(30)4 = \underline{13.2\ si}$

4-#5 bars × 5 columns

$A_v = .31(4)(5) = \underline{6.2}$

$A_v = 13.2 + 6.2 = \underline{19.4\ si}$

$f_y = 40$ ksi

$\mu = 0.6$

$\phi V_n = .85(19.4)40(0.6)$

$= 395.76^K > 102^K$ O.K.

DESIGN CALCULATION

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(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Margins of Safety

$$MS = \frac{\text{Allowable}}{\text{Actual}} - 1$$

Two Way Slab Bending $= \frac{129}{127.8} - 1 = 0.009$

Two Way Slab Shear $= \frac{9.3}{3.3} - 1 = 1.82$

In Plane Diaphragm Shear $= \frac{527}{102} - 1 = 4.17$

In Plane Diaphragm Bending $= \frac{1994}{441} - 1 = 3.52$

Wall Beam Compression $= \frac{140}{0.75} - 1 = 186$

Shear Friction $= \frac{396}{102} - 1 = 2.88$

Reinforcing Detail Check

ACI-349

Chapter 7

ρ_{min} = .0018 in each direction

ρ = As/bd = .11/(10" X 4.8") = .0023 > .0018 O.K.

maximum slab spacing; 3h = 3 X 6.5" = 19.5" > 10" O.K.

18" > 10" O.K.

Chapter 13 In-plane, two-way bending (See figure 13.4.8)

maximum slab spacing; 2h = 2 X 6.5" = 13" > 10" O.K.

Column strip straight top bars
hooked wall embedment required, hooked provided
50% slab development length (l_d) at 0.3l required
50% slab l_d at 0.2l required
100% slab l_d at 0.35l provided O.K.

Column strip straight bottom bars
6" straight wall embedment required, 7" provided
50% slab l_d at l - 3" required
50% slab l_d at 0.875l required
100% slab l_d at 1.0l provided O.K.

Middle strip
same reinforcing provided and less reinf. required O.K.

Chapter 11 In-plane diaphragm action

For deep flexural members 11.8

A_v > .0015 $b_w s$

.0015 (6.5" X 5") = 0.04875 si

0.11 si > 0.04875 si O.K.

A_{vh} > .0025 $b_w s_2$

.0025 (6.5" X 5") = 0.08125 si

0.11 si > 0.08125 si O.K.

A4.0 WALL PANEL EVALUATION

Orig RA Hiller 7-12-94 Checker E.O. Weiner 7/12/94

The 2736Z building wall panels are precast reinforced concrete panels. These panels have rebar extending outward 6" on the top edge and two sides edges which are then cast into the surrounding beams and columns. The bottom edge is left free during the beam and column construction with a 1/4" gap which is grouted-in after the rest of the building has been poured.

The precast wall panels act as shear wall units once the beam and column cast-in-place concrete has hardened. These shear wall units transfer the entire lateral load to the building foundation.

- **DESIGN LOADS AND LOAD COMBINATIONS**

  Design loads are specfied in SDC-4.1 ~~rev.11~~ rev.12, UCRL-15910 and ANSI A58.1. Load combinations are listed in UCRL-15910 and ACI-349.

- **SHEAR DIAPHRAGM STRENGTH ANALYSIS**

  The wall provides lateral building shear resistance through diaphragm action, which is a shear check using ACI-349 chapter 11.

- **OUT-OF-PLANE BENDING**

  The wall panels also experience out-of-plane bending which is checked with bending formulas obtained from "Roark's Formulas For Stress and Strain.

- **REINFORCING DETAIL CHECK**

  Reinforcing details are found in several ACI-349 chapters. Minimum reinforcing is specified in chapter 7 and rebar anchorage in chapter 12. Shear strength, flexural strength and shear friction are found in chapters 10, 11 and 13.

DESIGN LOADS

Length of Walls:

56' X 4 + 64.33' X 2 + 28.33' X 2 = 409.32'

Vertical Loads:

Dead Loads:

roof =	325 kips		
beams =	40 kips		
8" walls =	402 kips	Total =	767 kips

Live Loads:

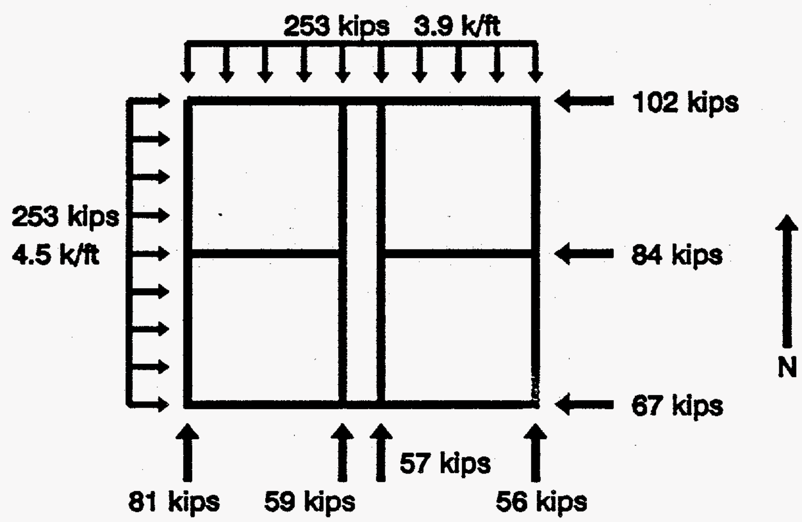
roof = 75 kips

Earthquake Loads:

roof =	325 kips	X .25 =	81 kips		
beams =	40 kips	X .25 =	10 kips		
8" walls =	402 kips	X .25 =	101 kips	Total =	192 kips

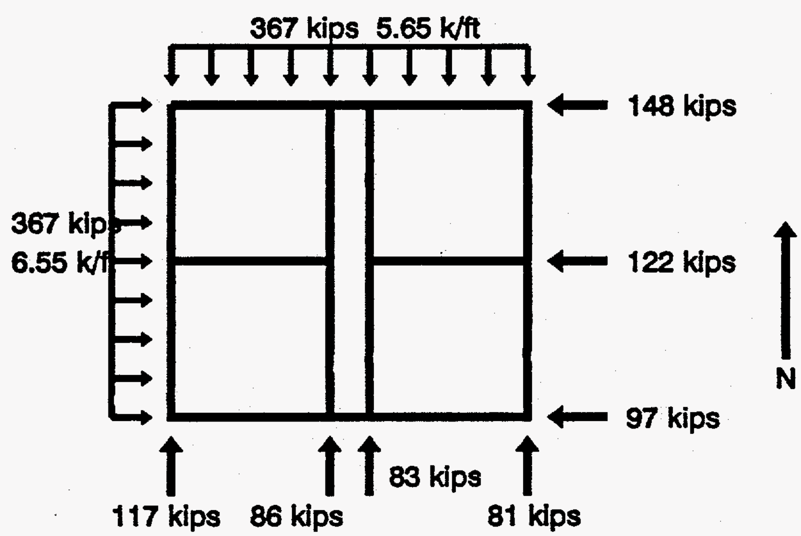
<u>Horizontal Loads</u>:

Earthquake Loads at the roof level



Building Plan

Earthquake Loads at the Wall Bottom (includes the above roof level loads)



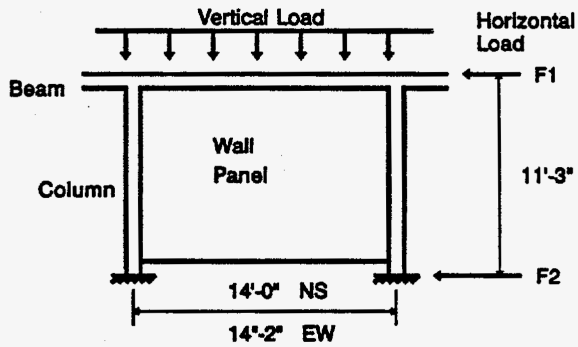
Building Plan

Load Combinations

28 wall panels not counting 7' in-fill walls

4 walls resist N.S. lateral loads, the two outside walls have 4 wall panels with no openings, the two inside walls have 2 door openings in the 4 wall panels so the walls are assumed to be only 85% effective in resisting lateral loads

3 walls, with 4 wall panels each, resist E.W. lateral loads



ACI-349 Section 9.2:

1.) 1.4D + 1.7L

$$u = \frac{1.4(767^k) + 1.7(75^k)}{28 \text{ wall panels}} = 43\ ^k/\text{wall panel}$$

$$F = 0$$

4.) D + L + E

$$U = \frac{767^k + 75^k + 192^k}{28 \text{ wall panels}} = 37\ ^k/\text{wall panel}$$

$$F_1 = \frac{81^k}{4 \text{ wall panels}} = 20\ ^k/\text{wall panel} \quad \text{NS direction}$$

$$F_1 = \frac{102^k}{4 \text{ wall panels}} = 26\ ^k/\text{wall panel} \quad \text{EW direction}$$

$$F_2 = \frac{117^k - 81^k}{4 \text{ wall panels}} = 9\ ^k/\text{wall panel} \quad \text{NS direction}$$

$$F_2 = \frac{148^k - 102^k}{4 \text{ wall panels}} = 12\ ^k/\text{wall panel} \quad \text{EW direction}$$

DESIGN CALCULATION

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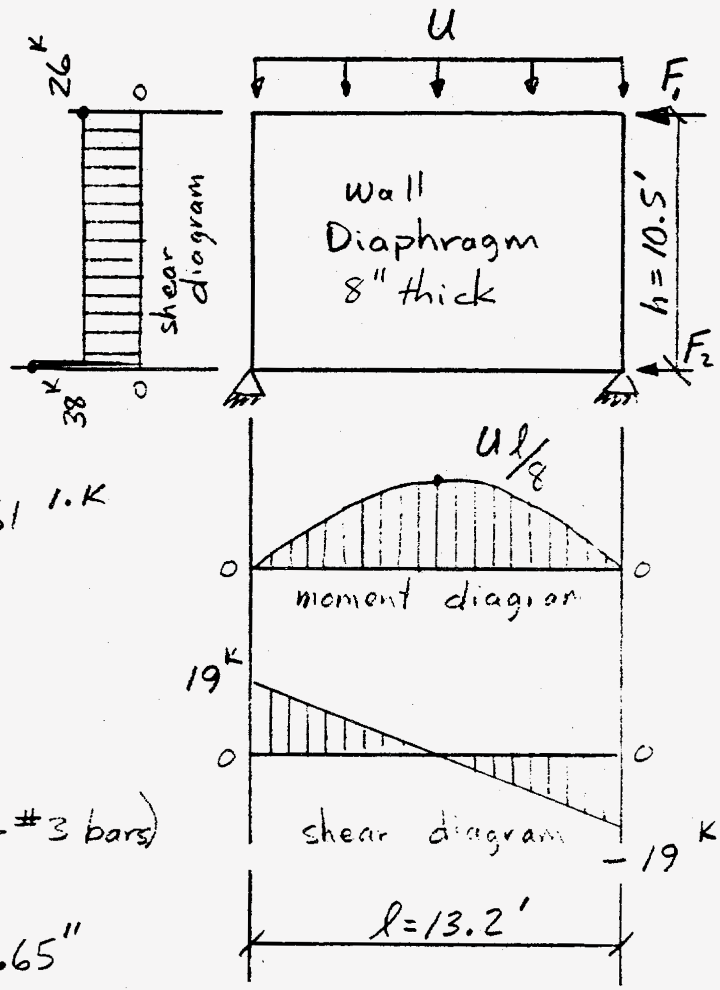
(10) SHEAR DIAPHRAGM ACTION ANALYSIS



$U = 37^K$ uniform load

$F_1 = 26^K$

$F_2 = 12^K$

$$\frac{u\ell}{8} = \frac{37\,(13.2)}{8} = 61^{\prime K}$$

Check Capacity:

Bending:

$$a = \frac{A_s F_y}{.85 f_c' b} \quad (3 - \#3 \text{ bars})$$

$$= \frac{.33\,(40)}{.85\,(3)(8)} = .65''$$

d = .8h = 8.4'

$$\phi M_n = A_s F_y \left(d - a/2\right)$$

$$= .33(40)\left(8.4 \times 12'' - {.65}/{2}\right)$$

$$= .9\left(1474^{''K}\right)\left(\frac{1}{12}\right) = 110.5^{\prime K} > 61^{\prime K} \quad \text{O.K.}$$

Note: Bending due to F_1 shows up as column compression due to overturning; see column analysis

DESIGN CALCULATION

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(8) Originator RA Hiller Date 2-8-94
(9) Checker EO Werner Date 2-8-94

(10) Check Capacity:

Use ACI-349 section 11.10 for shear in walls

Horizontal Shear:

$$10\sqrt{f_c'}\,hd = \frac{10\sqrt{3000}\,(8'')\,(0.8\times 13.2')}{1000}(12'')$$

$$= 555^K > 38^K$$

$$V_c = 2\sqrt{f_c'}\,hd = \frac{2\sqrt{3000}\;8''(0.8\times 13.2'\times 12'')}{1000}$$

$$= 111^K$$

$$V_s = \frac{A_v f_y d}{S_2} = \frac{.11\,(40)(0.8\times 13.2)(12'')}{5''}$$

$$= 112^K$$

$$\phi V_n = .85(111+112) = 190^K > 38^K \quad \text{O.K.}$$

Vertical Shear:

$$V_c = 2\sqrt{f_c'}\,hd = \frac{2\sqrt{3000}\;8''(0.8\times 10.5'\times 12'')}{1000} = 88^K$$

$$V_s = \frac{A_v f_y d}{S_2} = \frac{.11\,(40)(0.8\times 10.5)\,12''}{5''} = 89^K$$

$$\phi V_n = .85(88+89) = 150^K > 19^K$$

DESIGN CALCULATION

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(8) Originator R.A. Hille Date 7-12-94
(9) Checker E.O. Weiner Date 7-12-94

(10) Check Shear Friction

Development length l_d

ACI-318-63 design code:

allowable $\frac{6.7\sqrt{f_c'}}{D}$ 560 psi max

$D = .375''$ $\frac{6.7\sqrt{f_c'}}{.375} = 978$ psi

bar circumfrence $d = \pi D = 1.18''$

$(6''\text{ emb.})(1.18'')(560\text{ psi}) = 3965^{\#}$

$20{,}000$ psi allowable stress $\times .11$ si $= 2200^{\#} < 3965^{\#}$

OK.

ACI-349-90 code:

$l_d = .04 A_b f_y / \sqrt{f_c'} = .04(.11)(40)/\sqrt{3000} = 3.2'' < 6''$

OR

$= .0004 d_b f_y = .0004(3/8)(40) = 6.0'' = 6''$

OK.

Section 12.2.5

$l_{d_{min}} = 12'' \neq 6''$ see page A4-11 for Justification

DESIGN CALCULATION

(1) Drawing ____ (2) Doc. No. ____ (3) Page A4-8 of ____

(4) Building ____ (5) Rev. ____ (6) Job No. ____

(7) Subject ____

(8) Originator RA Hiller Date 2-8-94

(9) Checker Eo Weiner Date 2-8-94

(10) Horizontal Shear Friction Force:

$V_n = A_v f_y \mu$ $A_v = 31$ bars $\times .11 = 3.41$ si

$f_y = 40$ ksi

$\mu = 0.6$

$\phi V_n = (3.41(40)0.6).85$

$\phi V_n = 69^K > 38^K$ O.K.

Vertical Shear Friction Force:

$V_n = A_v f_y \mu$ $A_v = 25$ bars $\times .11 = 2.75$ si

$f_y = 40$ ksi $\mu = 0.6$

$\phi V_n = (2.75(40)0.6).85$

$\phi V_n = 56^K > 19^K$ O.K.

DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A4-9 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator R. A. Gille Date 7-12-94

(9) Checker EO Weiner Date 7-13-94

(10) OUT-OF-PLANE BENDING

Typical wall panels have 3 edges supported and the bottom edge free. From "ROARK" the stresses are as follows:

Page 461 2a.

$$Max\ \sigma = \frac{\beta q b^2}{t^2} \quad \beta = .54 \ @ \ \frac{a}{b} = \frac{10.5'}{13.2'} = 0.8$$

$$= \frac{.54 q (13.2')^2}{(8''/12'')^2} = 211.5 q$$

$$q = \frac{(6''+8''\text{walls})}{12''}(150 pcf)(.38g) = .0665 ksf$$

$$Max\ \sigma = 211.5(.0665) = 14.06 ksf$$

$$M_u = \sigma S_b = 14.06 \frac{(8/12)^2}{6}(1) = 1.04 ^{1.K}/ft$$

$$A_s f_y = .85 f_c' a b \quad \therefore \quad a = \frac{A_s f_y}{.85 f_c' b} = \frac{.264(40)}{.85(3)(12)} = .345''$$

$$M_n = A_s f_y (d - a/2) = .264(40)\left(4 - \frac{.345}{2}\right)$$

$$= 40.4 ^{\prime\prime\text{-}K}$$

$$\phi M_n = .9(40.4)\left(\frac{1}{12}\right) = 3.03 ^{1.K}/ft > 1.04 \quad O.K$$

Note: Equipment loads will effect the above calculations see equipment qualification analysis WHC-SD-CP-SA-026.

DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A4-10 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Margins of Safety

$$M.S. = \frac{\text{Allowable}}{\text{Accutual}} - 1$$

In Plane Wall Bending $= \frac{99.3}{61} - 1 = 0.63$

In Plane Vert. Wall Shear $= \frac{150}{19} - 1 = 6.89$

In Plane Horiz Wall Shear $= \frac{190}{38} - 1 = 4.00$

In Plane Shear Friction $= \frac{69}{38} - 1 = 0.82$

Development length Bond Stress $= \frac{6''}{3.2} - 1 = 0.875$

Out of Plane Bending $= \frac{3.03}{1.04} = 1.913$

Reinforcing Detail Check

ACI-349

Chapter 14; wall design refers to sections 7.12 & 11.10 for reinforcing.

Chapter 7.12 $\rho_{min.}$ = .0012 in each direction

Chapter 11.10, $\rho_{min.}$ = .0025 for walls in horizontal direction

$\rho_{min.}$ = .0025 for walls in vertical direction

ρ = As/(bs$_2$)

= (.11 si)/(8" X 5") = .00275 > .0025 O.K.

s_2 maximum = l_w/5 = 158"/5" = 31.6" > 5" O.K.

= 18" > 5" O.K.

Chapter 12,

The 6" embedment length of wall reinforcing into beams and columns does not meet current ACI-349-90, 12.2.5 code requirement of 12".

The rebar bond stress was calculated using both the original ACI-318-63 design code and the current ACI-349 building code and found to be acceptable;

ACI-318-63

Bonding strength for a #3 bar with 6" embed.	=	3,965 lbs.
Allowable bar tension for #3 bar	=	2,200 lbs.
Margin of safety	=	0.80 O.K.

ACI-349-90 12.2.2

Min. embed for bonding of a #3 bar	=	6.0"
Actual embed.	=	6"
Margin of Safety	=	0.0 O.K.

A5.0 COLUMN EVALUATION

Orig RA Miller 2-8-94 Checker EO Weiner 2-8-94 WHC-SD-CP-SA-023
Rev. ~~0~~ 1
Page A5-1

The 2736Z building columns are cast-in-place columns poured monolithic with the beams and foundation wall, and are used to secure the precast wall panels in place. Some of the columns are laterally supported by the wall panels and thus have no bending or slenderness considerations. The remaining columns are not laterally supported in one or both directions and thus do have bending moments to be considered.

- **DESIGN LOADS AND LOAD COMBINATIONS**

  Design loads are specfied in SDC-4.1 ~~rev.11~~ rev. 12., UCRL-15910 and ANSI A58.1. Load combinations are listed in UCRL-15910 and ACI-349.

- **COMPRESSIVE STRENGTH ANALYSIS**

  Check column compression using ACI-349 chapter 10 and the column design charts of ACI-340.2R. All columns in the 2736Z building are braced against side sway and they have three different types of lateral support. The three types are, full support both directions (18 cloumns), full support in one direction only (8 columns) and no lateral support (4 columns).

- **SHEAR STRENGTH ANALYSIS**

  All lateral building loads are transfered to the foundation through column base shear. Twenty columns resist the NS direction shear and 18 resist the EW direction shear. Some columns resist shear in both directions and will be checked for combined effects.

- **REINFORCING DETAIL CHECK**

  Reinforcing details are found in several ACI-349 chapters. Minimum reinforcing is specified in chapter 10 and rebar anchorage in chapter 12. Shear strength, flexural strength and shear friction are found in chapters 10 and 11.

DESIGN LOADS:

Vertical Loads:

Dead Loads

roof =	325 kips		
beams =	40 kips		
8" walls =	402 kips		
columns =	44 kips	Total =	811 kips

Live Loads

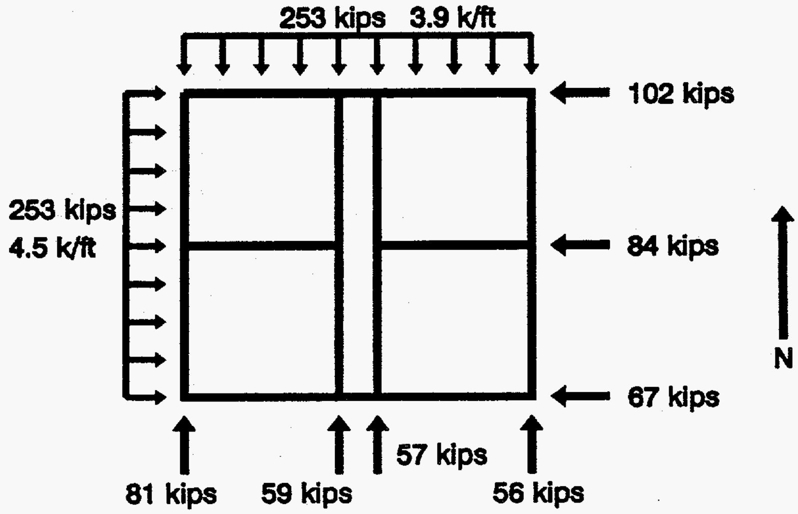
roof =	75 kips	Total =	75 kips

Earthquake Loads

roof =	325	X .25 =	81 kips		
beams =	40	X .25 =	10 kips		
8" walls =	402	X .25 =	101 kips		
columns =	44	X .25 =	11 kips	Total =	203 kips

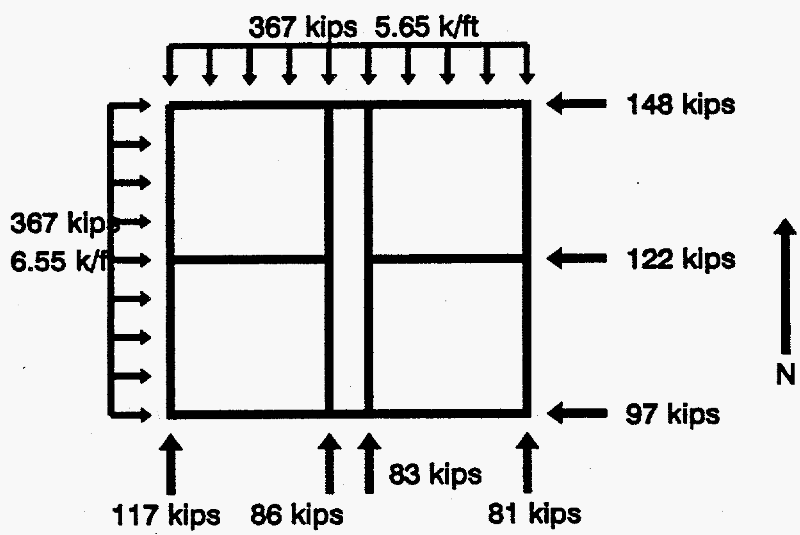
Horizontal Loads:

Earthquake Loads at the roof level



Building Plan

Earthquake Loads at the column base (includes the above roof level loads)

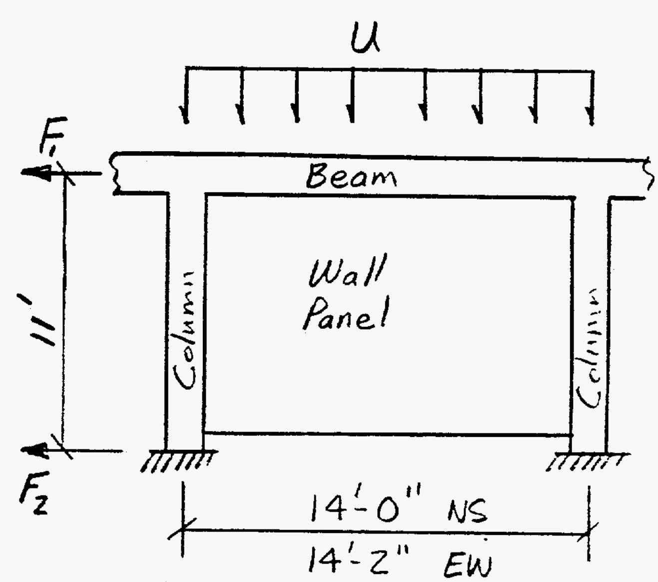


Building Plan

(1) Drawing ______ (2) Doc. No. ______ (3) Page AS-4 of ____
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator RA Hiller Date 2-8-94
(9) Checker LEO Weiner Date 2-8-94

(10) LOAD COMBINATIONS

ACI-349 Section 9.2 :



1.) 1.4D + 1.7L

$$U = \frac{1.4(811) + 1.7(75)}{30 \text{ Columns}} = 42.1^{K} \qquad F = 0$$

4.) D + L + E

$$U = \frac{811 + 75 + 203}{30 \text{ Columns}} = 36.3^{K}$$

$F_1 = 253^{K}$ ∴ 81^{K} highest loaded wall NS / 102^{K} highest loaded wall EW

$F_2 = 367^{K}$ ∴ 117^{K} highest loaded wall NS / 148^{K} highest loaded wall EW

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(7) Subject ______
(8) Originator RA Giller Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

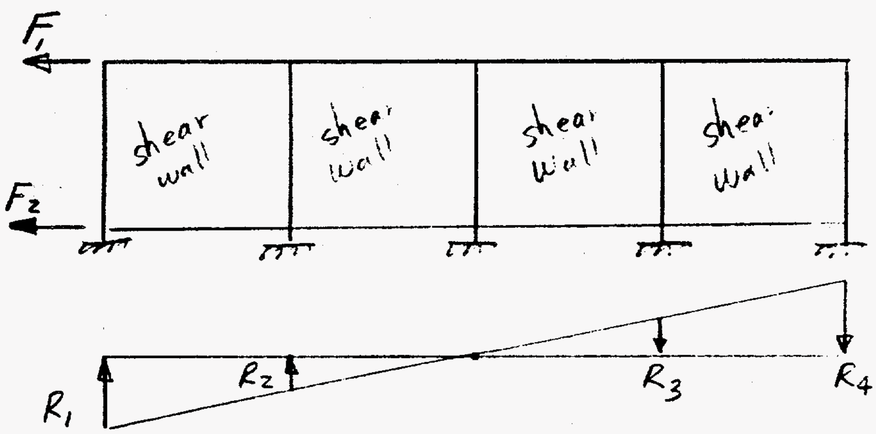
(10) COMPRESSION ANALYSIS

Compression due to vertical load:

$U = 36.3^K/\text{column}$

Compression due to horiz. loads:

N.S. direction: (4 wall lines)



$R_1 = R_4$
$R_2 = R_3$
$R_1 = 2R_2$

$$F_1(11.0') = R_1(56') + R_2(28')$$
$$= 2R_2(56') + R_2(28')$$
$$= 140\,R_2$$

$$R_2 = \frac{F_1(11.0)}{140} = \frac{(81^K)\,11.0}{140} = 6.4^K$$

$$R_1 = 2R_2 = 12.7^K$$

DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page AS-6 of ______

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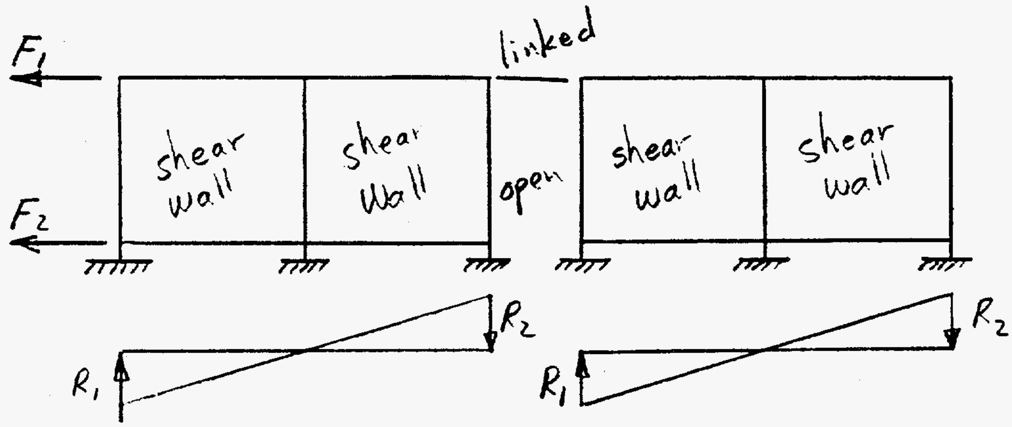
(7) Subject ______

(8) Originator RA Miller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Compression due to horiz. Loads

EW direction :



$R_1 = R_2$

$F_1 (11.0') = R_1 (28.4')$

$R_1 = \dfrac{F_1 (11.0)}{28.4}$ where $F_1 = \dfrac{102^k}{2} = 51^k$

$= \dfrac{51 (11)}{28.4} = 20^K$

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(7) Subject ______
(8) Originator RA Ziller Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10) Total Compression Load:

Use 100-40-40 method to combine earthquake direction effects

$$P_u = \frac{D + L + .4E_V}{30 \text{ columns}} + 1.0E_{EW} + 0.4E_{NS}$$

$$= \frac{811 + 75 + .4(203)}{30} + 20^K + .4(12.7^K)$$

$$= 57^K$$

ACI-349 Section 10.3

$$\phi P_n = .8\phi[.85f'_c(A_g - A_s) + f_y A_s]$$

$A_s = 1.24$

$A_g = 96$

$$= .8(.7)[.85 \times 3(96 - 1.24) + 40 \times 1.24]$$

$$= 163^K > 57^K \quad \text{O.K.}$$

(1) Drawing ______ (2) Doc. No. ______ (3) Page A5-8 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Check Combined Bending & Compression

4 columns are not laterally supported in either direction, and thus have biaxial bending. See drawing H-2-26650 Foundation Plan for column.

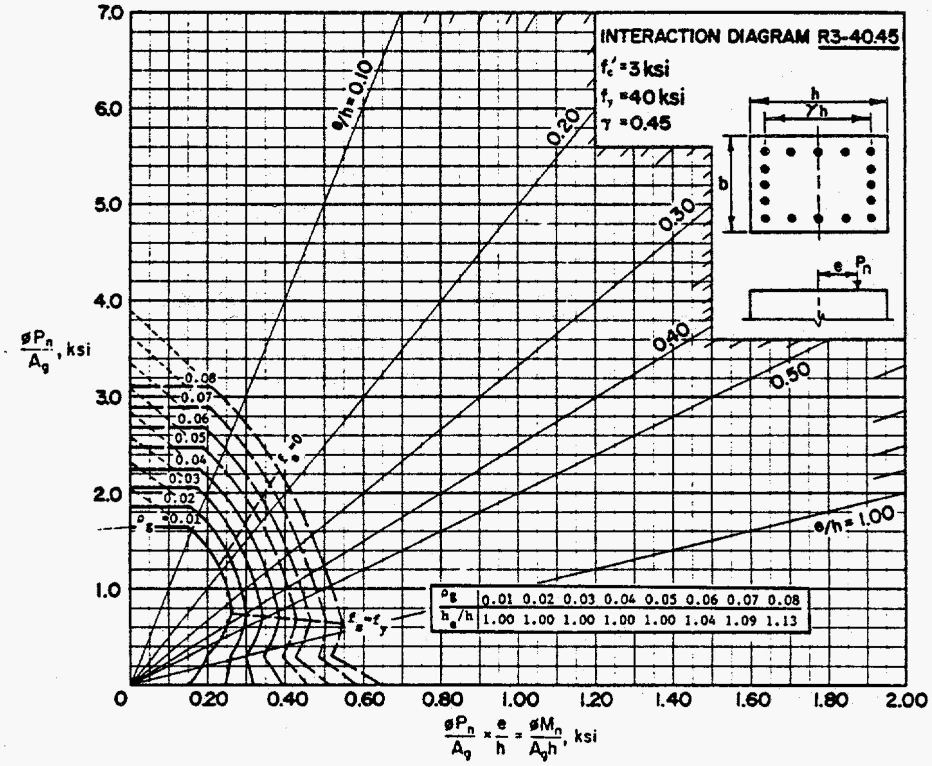
(H.8 - 16.5)
(H.8 - 14.7)
(L - 16.5)
(L - 14.7)

8 columns are laterally supported in one direction only. See drawing H-2-26650 Foundation Plan for column.

(H.1 - 16.5)
(H.1 - 14.7)
(H.8 - 17.3)
(H.8 - 14.1)
(L - 17.3)
(L - 14.1)
(M - 16.5)
(M - 14.7)

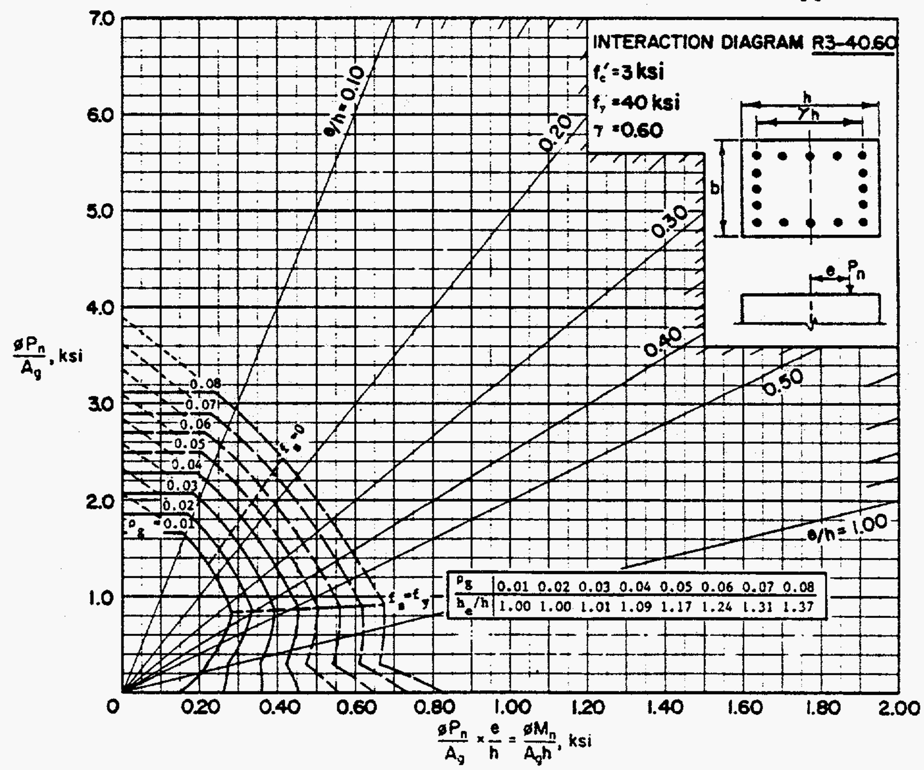
COLUMNS 7.1.1—Load-moment strength interaction diagram for R3-40.45 columns

References: ACI 318-83, Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182



COLUMNS 7.1.2—Load-moment strength interaction diagram for R3-40.60 columns

References: ACI 318-83, Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182



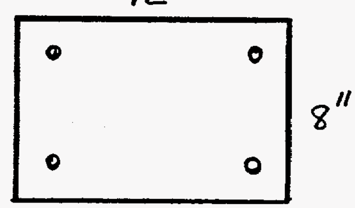
For use of these Design Aids, see Columns Examples 1–8 and 11–16.

DESIGN CALCULATION WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page AS-10 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator RA Giller Date 2-8-94
(9) Checker EO Wemer Date 2-8-94

(10) Biaxial Columns

Factored load $P_u = 42.1^K$



4- #5 bars

From roof analysis :

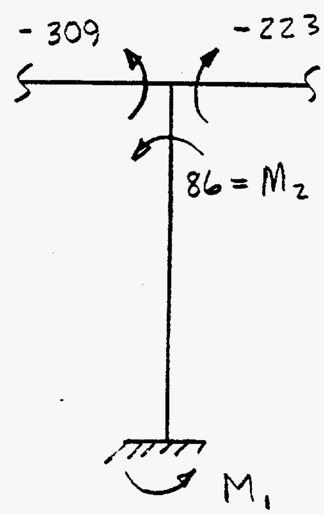
Interior span end moment
$= -287^{\prime\prime \cdot K}$

Exterior span end moment
$= -309^{\prime\prime \cdot K}$

Interior span (dead load only) end moment
$$= \left(\frac{1.4 \times 86.75 \text{psf}}{156 \text{psf}} \right) 287 = 223^{\prime\prime \cdot K}$$

Maximum unbalance end moment
$= 309 - 223 = 86^{\prime\prime \cdot K}$ each way

Base moment M_1
$= \frac{1}{2}$ top moment M_2
$= 43^{\prime\prime \cdot K}$
see Bloddget 8.1-18
Fig 5E



(1) Drawing ____ (2) Doc. No. ____ (3) Page AS-11 of ____

(4) Building ____ (5) Rev. ____ (6) Job No. ____

(7) Subject ____

(8) Originator RA Miller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10)

β_d Ratio of dead to total load

$$= \frac{86.75 \text{ psf}}{86.75 + 20} = .81$$

l_u Column length K = 1.0

$$= 10'\text{-}10'' = 130''$$

E_c Concrete modulus (ACI-349 8.5)

$$= 57000\sqrt{f_c'} = 3\,122\,000 \text{ psi}$$

I_g strong axis

$$= \frac{(12)^3\,8}{12} = 1152 \text{ in}^4$$

I_g weak axis

$$= \frac{(8)^3\,12}{12} = 512 \text{ in}^4$$

EI (from ACI-349 10.11)

$$= \frac{E_c I_g / 2.5}{1 + \beta_d}$$

= 794,816,353 strong axis = 353,251,712 weak axis

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(7) Subject ______
(8) Originator RA Keller Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10) Strong Axis Magnified Moment (ACI-349 10.11)

P_c buckling load

$$= \frac{\pi^2}{(k l_u)^2} EI = 464{,}173 = 464^K$$

$$C_m = .6 + .4\left(\frac{M_1}{M_2}\right) = .6 + .4\left(\frac{-43}{86}\right) = .4$$

$$\delta = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} = .44 \Longrightarrow 1.0 \quad \text{No Magnification}$$

$$\underline{\underline{M_c = M_2 = 86^{\text{''-K}}}}$$

Weak Axis Mag. Moment

$$P_c = \frac{\pi^2}{(K l_u)^2} EI = 206{,}299 = 206^K$$

$$C_m = .4$$

$$\delta = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} = .56 < 1.0 \quad \text{No Mag}$$

DESIGN CALCULATION

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(1) Drawing ______ (2) Doc. No. ______ (3) Page A5-13 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator R.A. Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

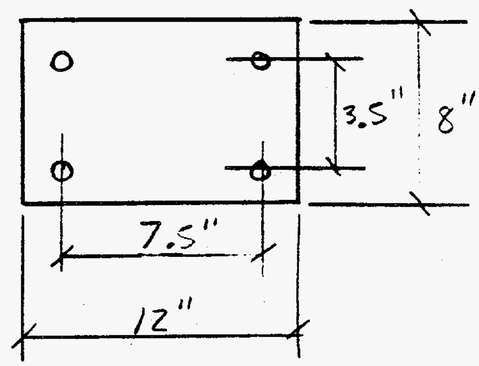
(10) Strength Check

Using the Interaction diagrams of ACI-340.2R-85 design hand book.

$f_c' = 3$ ksi $f_y = 40$ ksi

Strong Axis:

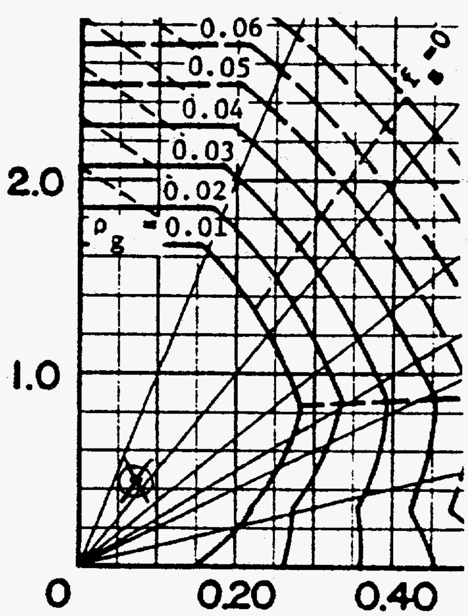
$\gamma = \frac{7.5''}{12''} = .625 \simeq .6$



Use Chart 7.1.2

$\rho_g = (.31)4/_{8 \cdot 12} = .0129$

$\frac{P_u}{A_g} = \frac{42.1}{8 \cdot 12} = .44$ ksi $\quad \frac{M_u}{A_g h} = \frac{86}{8 \cdot 12 \cdot 12} = .075$ ksi



Acceptable - well with-in the interaction $\rho_g = .01$ curve.

BD-6400-060.1 (12/87)

DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A5-14 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

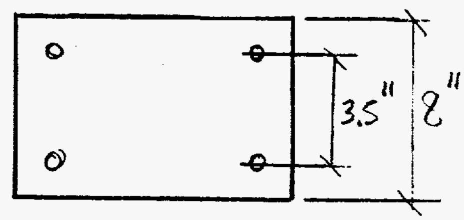
(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Weak Axis:

$$\gamma = \frac{3.5''}{8''} = .44 \simeq .45$$

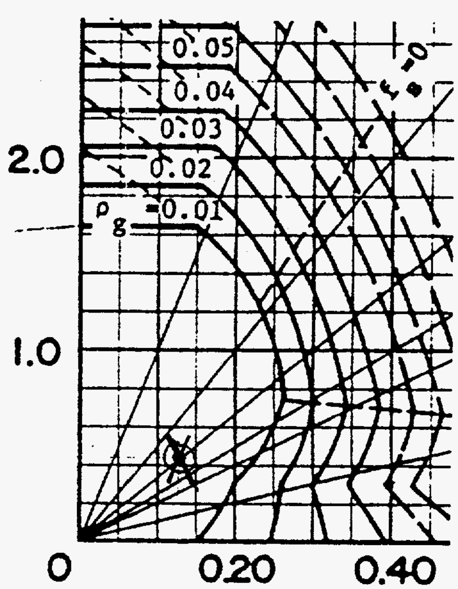


$h = 8''$ $A_g = 8'' \times 12''$

$\rho_g = .0129$

$$\frac{P_u}{A_g} = .44 \text{ ksi} \qquad \frac{M_u}{A_g h} = \frac{86}{8 \cdot 12 \cdot 8} = .112 \text{ ksi}$$

use chart 7.1.1



Acceptable: Well with-in the $\rho_g = .01$ curve ⊗

(1) Drawing ________ (2) Doc. No. ________ (3) Page A5-15 of ____

(4) Building ________ (5) Rev. ________ (6) Job No. ________

(7) Subject ________

(8) Originator RA Miller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Biaxial Bending

Use ACI-340.2R-85 design charts

$P_u = 57^K$ (from vertical & horz. loads)

$\phi P_n = 163^K \quad \phi P_o = \dfrac{\phi P_n}{.8} = 204^K$

$$\frac{\phi P_n'}{\phi P_o} = \frac{P_u}{\phi P_o} = \frac{57}{204} = .28$$

$\rho_g f_y / f_c' = .01(40)/3 = .13$

$\beta = .62$ from chart 11.1.1

$$\frac{\phi P_n}{A_g} = \frac{P_u}{A_g} = \frac{57}{8'' \times 12''} = .59 \text{ ksi}$$

$$\frac{\phi M_{nox}}{A_g h} = .243 \text{ ksi}$$ @ Table R3-40

$\gamma = .6$

Between points 5 & 6

$\rho_g = .01$

$\phi M_{nox} = .243(8'' \times 12'')8''$

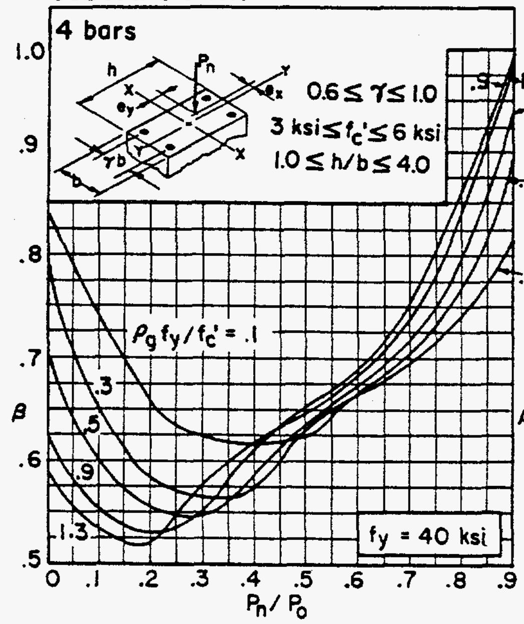
$= 187$ in.k simultaneous moments

187 in.k > 86 in.k O.K.

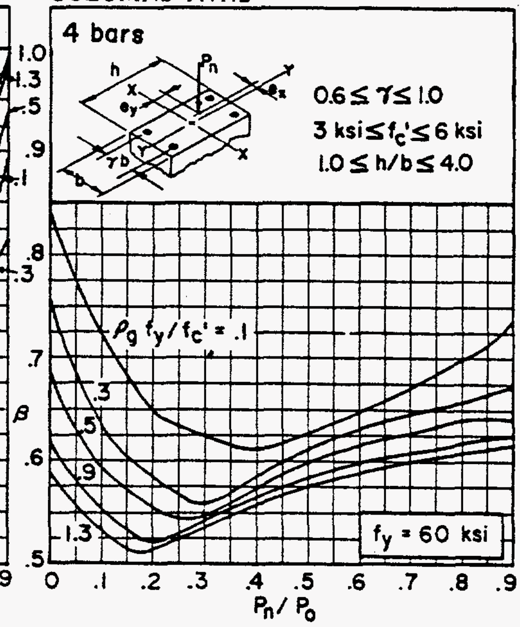
COLUMNS 11.1—Biaxial bending design constant* β—For rectangular columns with two bars on each of four faces

Reference: Parme, Alfred, L.: Nieves, Jose M.; and Gouwens, Albert, "Capacity of Reinforced Rectangular Columns Subject to Biaxial Bending," ACI JOURNAL, *Proceedings* V. 63, No. 9, Sept. 1966, pp. 911–923

COLUMNS 11.1.1



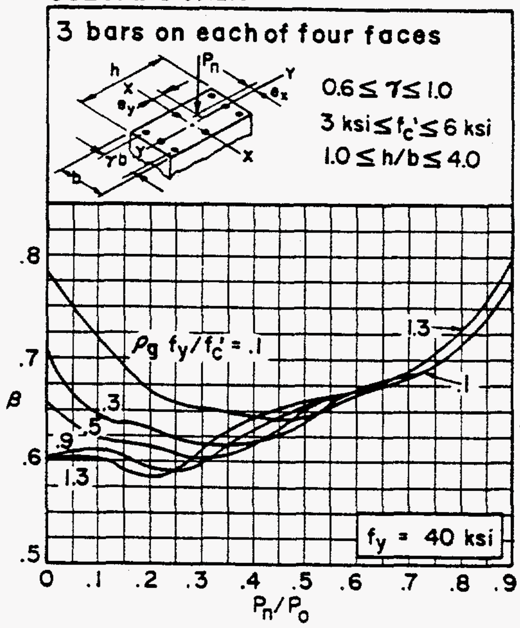
COLUMNS 11.1.2



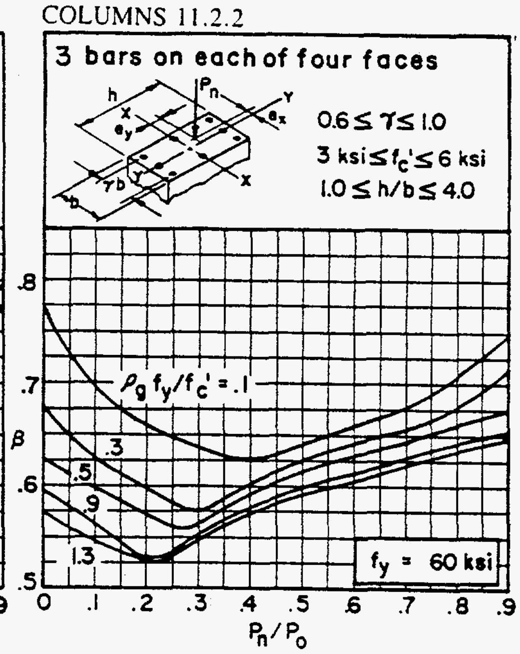
COLUMNS 11.2—Biaxial bending design constant* β—For rectangular columns with three bars on each of four faces

Reference: same as for COLUMNS 11.1

COLUMNS 11.2.1



COLUMNS 11.2.2

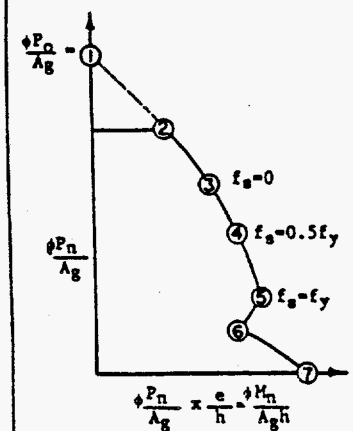


*β = constant portion of uniaxial factored moment strengths M_{nox} and M_{noy} which may be permitted to act simultaneously on the column cross section; value of β depends on P_n / P_o and properties of column material and cross section

For use of this Design Aid, see Columns Examples 13 and 16.

COLUMNS 8.1—Basic limits of factored axial load and factored moment for R3-40 columns (Design load and moment strengths)

References: ACI 318-83, Sections 9.3.2.2, 10.2, and 10.3; ACI Publication SP-7, pp. 152–182



Coordinates tabulated below provide more precise values for critical points on corresponding interaction diagrams (COLUMNS 7).

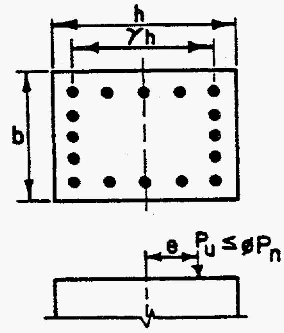
units: $\phi\frac{P_n}{A_g}$, ksi $\phi\frac{M_n}{A_g h}$, ksi

BASIC LIMITS TABLE R3-40

Rectangular column with bars on four faces

$f_c' = 3$ ksi

$f_y = 40$ ksi



ρ_g	② maximum P_u/A_g $\phi\frac{P_n}{A_g}$	② $\phi\frac{M_n}{A_g h}$	③ $f_s = 0$ $\phi\frac{P_n}{A_g}$	③ $\phi\frac{M_n}{A_g h}$	④ $f_s = 0.5f_y$ $\phi\frac{P_n}{A_g}$	④ $\phi\frac{M_n}{A_g h}$	⑤ $f_s = f_y$ $\phi\frac{P_n}{A_g}$	⑤ $\phi\frac{M_n}{A_g h}$	⑤ $\frac{e}{h}$	⑥ ϕ changes $\phi\frac{P_n}{A_g}$	⑥ $\phi\frac{M_n}{A_g h}$	⑦ $P_u = 0$ $\phi\frac{M_n}{A_g h}$
					R3-40.45 γ = 0.45							
.01	1.638	.150	1.246	.232	.963	.254	.742	.259	.349	.300	.197	.1512
.02	1.847	.158	1.391	.253	1.031	.286	.730	.300	.411	.300	.261	.2453
.03	2.057	.167	1.537	.274	1.100	.317	.718	.341	.475	.300	.300	.3223
.04	2.267	.176	1.683	.295	1.168	.348	.706	.382	.542	.300	.345	.3930
.05	2.477	.185	1.828	.316	1.236	.380	.694	.424	.610	.300	.397	.4567
.06	2.686	.195	1.974	.337	1.305	.411	.682	.465	.681	.300	.429	.5188
.07	2.896	.205	2.120	.358	1.373	.442	.670	.506	.755	.300	.485	.5803
.08	3.106	.214	2.265	.379	1.441	.473	.658	.547	.831	.300	.520	.6411
					R3-40.60 γ = 0.60							
.01	1.638	.155	1.367	.222	1.071	.264	.841	.279	.331	.300	.202	.1490
.02	1.847	.167	1.520	.250	1.155	.307	.851	.336	.394	.300	.271	.2618
.03	2.057	.180	1.673	.277	1.239	.349	.861	.392	.455	.300	.358	.3617
.04	2.267	.193	1.826	.305	1.323	.392	.871	.449	.515	.300	.424	.4576
.05	2.477	.206	1.979	.333	1.408	.435	.881	.506	.574	.300	.473	.5523
.06	2.686	.219	2.132	.360	1.492	.478	.891	.563	.631	.300	.541	.6444
.07	2.896	.233	2.285	.388	1.576	.521	.901	.619	.687	.300	.609	.7332
.08	3.106	.246	2.438	.416	1.660	.564	.911	.676	.742	.300	.663	.8193
					R3-40.75 γ = 0.75							
.01	1.638	.161	1.487	.204	1.171	.267	.936	.295	.315	.300	.211	.1525
.02	1.847	.179	1.646	.239	1.263	.320	.962	.367	.381	.300	.316	.2869
.03	2.057	.197	1.805	.273	1.354	.373	.989	.438	.443	.300	.404	.4157
.04	2.267	.215	1.964	.307	1.446	.426	1.015	.510	.503	.300	.483	.5339
.05	2.477	.233	2.123	.341	1.538	.479	1.041	.582	.559	.300	.548	.6473
.06	2.686	.252	2.283	.375	1.629	.533	1.068	.654	.612	.300	.632	.7590
.07	2.896	.270	2.442	.410	1.721	.586	1.094	.725	.663	.300	.717	.8695
.08	3.106	.288	2.601	.444	1.813	.639	1.121	.797	.711	.300	.786	.9727
					R3-40.90 γ = 0.90							
.01	1.638	.169	1.605	.179	1.270	.265	1.021	.306	.300	.300	.241	.1643
.02	1.847	.193	1.768	.220	1.368	.328	1.055	.392	.371	.300	.343	.3205
.03	2.057	.217	1.931	.261	1.466	.391	1.089	.477	.438	.300	.432	.4647
.04	2.267	.240	2.094	.301	1.564	.455	1.123	.563	.501	.300	.544	.6002
.05	2.477	.263	2.258	.342	1.662	.518	1.157	.648	.560	.300	.625	.7336
.06	2.686	.286	2.421	.383	1.760	.582	1.191	.734	.616	.300	.726	.8660
.07	2.896	.309	2.584	.424	1.858	.645	1.225	.820	.669	.300	.809	.9967
.08	3.106	.332	2.747	.464	1.956	.708	1.259	.905	.719	.300	.913	1.1253

For use of this Design Aid, see Columns Examples 11 and 13.

DESIGN CALCULATION

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(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

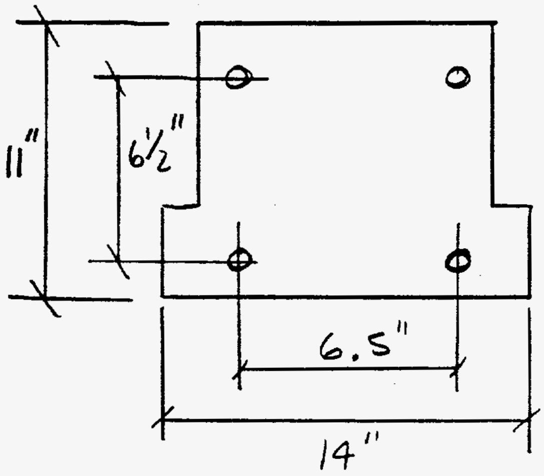
(10) Single axis Columns

Factored Load $P_u = 36.3^K$
average

From roof analysis:

End span end moment
$= -82^{''\cdot K}$

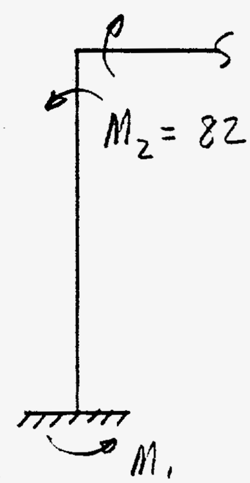
Maximum unbalanced end Moment
$= 82^{''\cdot K}$



effective size = 11" x 11"

Base Moment M_1
$= \frac{1}{2}$ top moment M_2
$= 41^{''\cdot K}$

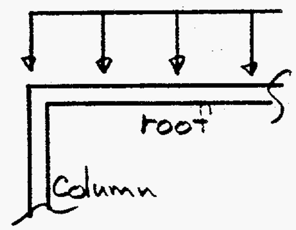
see Bloddget 8.1-18
Fig 5E



(1) Drawing ______ (2) Doc. No. ______ (3) Page A5-19 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator RA Hiller Date 2-8-94
(9) Checker EO Weaver Date 2-8-94

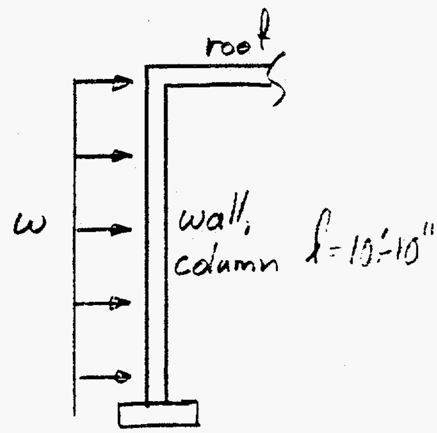
(10) Column Bending Moment

End Moment from factored D & L roof loads $= 82^{'' \cdot K}$



End Moment from factored D, L, & E roof loads $= \frac{129\,psf}{156\,psf} 82 = 68^{'' \cdot K}$

Moments from out-of-plane wall loads



$w = .0665$ ksf from wall analysis

$= .0665 \times 14.17'$ wide

$= 0.94\ ^{K}/_{ft}$

Since the column end have partial fixity, use the following moment approximations

End moments $= \frac{w\ell^2}{10} = \frac{.94(10.83)^2}{10} = 11^{' \cdot K} = 132^{'' \cdot K}$

Center mom. $= \frac{w\ell^2}{11} = \frac{.94(10.83)^2}{11} = 10^{' \cdot K} = 120^{'' \cdot K}$

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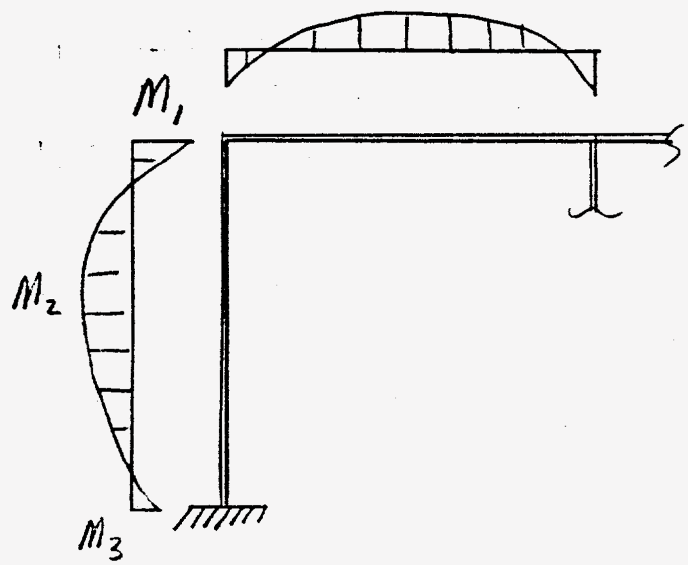
(4) Building ________ (5) Rev. ________ (6) Job No. ________
(7) Subject ________
(8) Originator RA Giller Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10)

Combine Roof & Wall Moments:

Moment Diagram



$$M_1 = 132^{'\cdot K} + 68^{'\cdot K} = 200^{'\cdot K}$$

$$M_2 = 120^{'\cdot K} + 0 = 120^{'\cdot K}$$

$$M_3 = 132^{'\cdot K} + \tfrac{1}{2}\, 68^{'\cdot K} = 166^{'\cdot K}$$

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(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator RA Miller Date 2-8-94
(9) Checker Ed Weiner Date 2-8-94

(10) $\beta_d = .81 \qquad \ell_u = 130'' \qquad K = 1.0$

$E_c = 3\,122\,000 \text{ psi}$

$I_g \quad \frac{11^3 \cdot 11}{12} = 1220$

$$EI = \frac{E_c I_g / 2.5}{1 + \beta_d}$$

$$= \frac{3122000(1220)/2.5}{1.81} = 841{,}732{,}597$$

<u>Magnified Moment</u> (ACI-349 10.11)

$$P_c = \frac{\pi^2}{(K \ell_u)^2} EI = \frac{\pi^2}{130^2} EI = 491572 = 492^K$$

$C_m = .4$

$$\delta = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} = .46 \Rightarrow 1.0$$

No Magnification

$M_c = 200''\cdot K$

DESIGN CALCULATION

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(7) Subject ________

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Strength Check

Using interaction diagrams from ACI-340.2R-85 design handbook

$f'_c = 3$ ksi $f_y = 40$ ksi

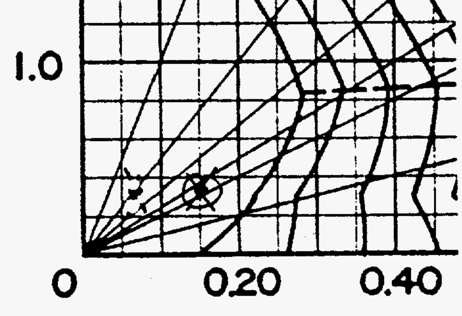
$$\gamma = \frac{6.5''}{11''} = .59 \simeq .6$$

$$\rho_g = (.31)4 / 11 \cdot 11 = .0102 \simeq .01$$

Use chart 7.1.2

$$\frac{P_u}{A_g} = \frac{36.3}{11 \cdot 11} = .30 \text{ ksi}$$

$$\frac{M_u}{A_g h} = \frac{200}{11 \cdot 11 \cdot 11} = .15 \text{ ksi}$$



Acceptable - Well with-in the interaction $\rho_g = .01$ curve ⊗

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(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Miller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) LATERAL SHEAR LOAD ANALYSIS

N.S. Shear @ Base F_z

$117^K \div 5$ columns $= 23.4^K$

E.W. Shear @ Base F_z

$148^K \div 6$ columns $= 24.7^K$

ACI-349 Section 11.3 & 11.5

$$V_c = 2\sqrt{f_c'}\,bd\left(1 + \frac{N_u}{2000 A_g}\right)$$

N_u average $= 36.3^K$/column

$A_g = 14'' \times 14'' = 196$

$bd = 11'' \times 14'' = 154$

$$= 2\sqrt{3000}(154)\left(1 + \frac{36300}{2000(196)}\right)$$

$$= 18.4^K$$

$$V_s = \frac{A_v f_y d}{s}$$

$d = 11''$

$s = 5''$

$A_v = \#3$ ties $= .22$ si

$$= \frac{.22(40)(11)}{5}$$

$$= 19.4^K$$

$$\phi V_n = .85(18.4 + 19.4) = 32.1^K > 23.4 \text{ \& } 24.7$$

O.K.

$\phi V_c = 2\sqrt{f_c'}\,bd = \phi 16.9^K \quad \therefore \phi V_n = .85(16.9 + 19.4) = 30.9^K$

(no comp. load)

DESIGN CALCULATION

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(1) Drawing ______ (2) Doc. No. ______ (3) Page A5-24 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

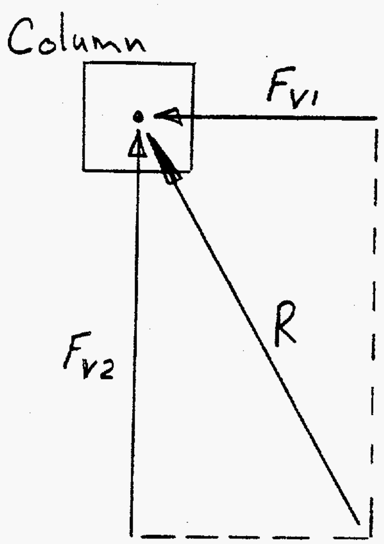
(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) COMBINED DIRECTIONS ANALYSIS

Use 100-40-40 method per UCRL-15910



$F_{V2} = 100\% \, (24.7) = 24.7^K$

$F_{V1} = 40\% \, (23.4) = 9.4^K$

$$R = \sqrt{F_{V1}^2 + F_{V2}^2} = 26.4^K < 32.1^K \quad \text{O.K.}$$

(1) Drawing ______ (2) Doc. No. ______ (3) Page A5-25 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator RA Hiller Date 2-5-94
(9) Checker EO Weiner Date 2-8-94

(10) Check Shear Friction

@ EL = 0'-2" (Base of Wall)

$V_n = A_v f_y \mu$ $\quad A_v = 4\text{-}\#5\text{ bars} = 1.24$ si

$f_y = 40$ ksi

$\mu = 1.4$ monolithic pour

$\phi V_n = .85(1.24)(40)(1.4) = 59^K > 26.4^K$ O.K.

@ EL = (-)0'-3" (Base of Column)

$V_n = A_v f_y \mu$ $\quad A_v$ column $= 1.24$

$\mu = 0.6$ non-monolithic

A_v foundation =

#4 @ 15" = 40 bars per wall

$\frac{.2 \times 40}{5 \text{ columns}} = 1.0$ si

$\mu = 1.4$ monolithic

$\phi V_n = .85(40)\left[1.24 \times 0.6 + 1.0 \times 1.4\right]$

$= 73^K > 26.4^K$ O.K.

(1) Drawing ______ (2) Doc. No. ______ (3) Page AS-26 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Margin of Safety

$$MS = \frac{\text{Allowable}}{\text{Actual}} - 1$$

Compression Load $\frac{163}{57} - 1 = 1.86$

Combined Bending & Axial loads (See Moment Interaction curve) $\simeq 1.0$

Biaxial Bending $\frac{187}{97} - 1 = 0.92$

Lateral Column Base Shear $\frac{32.1}{26.4} - 1 = 0.22$

Shear Friction @ Column Base $\frac{73}{26.4} - 1 = 1.77$

REINFORCING DETAIL CHECK

Longitudinal reinforcement:

4 - #5 bars, As = 1.24 si

ρ_g = As/Ag   Ag min. = 8" X 12" = 96 si
Ag max. = 14" X 14" = 196 si

ρ_g = 1.24/96 = .013
ρ_g = 1.24/196 = .006 **

Required reinf. (ACI-349 Section 10.9)

ρ_g min. = .01 > .006 **
ρ_g max. = .08 > .013 O.K.

Lateral reinforcement:

#3 bars @ 5" near base of column
#3 bars @ 10" in the remainder of the column

Required spacing (ACI-349 Section 7.10)

least dimension,	6 columns require 8" spacing < 10"	No Good
	24 columns require 11" or 14" > 10"	O.K.

Required reinf.

$$Av\ min. = 50bs/fy = \frac{50(14")10"}{40{,}000\ psi} = .175\ si < .22\ si \quad O.K.$$

Required lap splice at dowels:

Class C splice = 1.7 l_d,

l_d = 10" for a #5 bar with Fy = 40 ksi and f'c = 3 ksi

1.7 X 10" = 17" Drawing shows a minimum lap length of 4 ties spaced at 5" which equals 15" plus additional over lap of about 1" top and bottom which equals = 17" O.K.

** By reducing the effective column area down to 11" X 11", ρ_g becomes .013 which is acceptable. The reduction in compressive capacity is also reduced;

$\phi Pn = (11^2 / 14^2)$ X 140 kips = 86.4 kips > 78.3 kips O.K.

The base of the column has additional longitudinal reinf. in the form of dowels, and so do not require a smaller effective area to meet minimum reinforcing.

A6.0 FOUNDATION WALL EVALUATION

The 2736Z building foundation wall is a combination foundation wall column pedestal constructed of reinforced concrete poured monolithic with the footings and the columns. All loads are transfered to the foundation wall through the columns, there is no wall panel-foundation wall load transfer since the wall panels are set-in-place with a 1/4" gap between panel and foundation.

- **DESIGN LOADS AND LOAD COMBINATIONS**

  Design loads are specfied in SDC-4.1 ~~rev.11~~ rev.12, UCRL-15910 and ANSI A58.1. Load combinations are listed in UCRL-15910 and ACI-349.

- **COMPRESSIVE STRENGTH ANALYSIS**

  Check column compression using ACI-349 chapters 10 and 14. Chapter 10 is the basic compression strength requirements and chapter 14 is the strength requirements specific to the design of load bearing walls.

- **SHEAR STRENGTH ANALYSIS**

  All lateral building loads are transfered from the column base shear through the foundation wall to the footing. The shear stength requirements to be meet are in ACI-349 chapter 11.

- **REINFORCING DETAIL CHECK**

  Reinforcing details are found in several ACI-349 chapters. Minimum reinforcing is specified in chapter 10 and 14 and rebar anchorage in chapter 12. Shear strength, and shear friction is found in chapter 11.

DESIGN LOADS:

Vertical Loads:

Dead Loads

roof =	325 kips		
beams =	40 kips		
8" walls =	402 kips		
columns =	44 kips		
foundation wall =	136 kips	Total =	947 kips

Live Loads

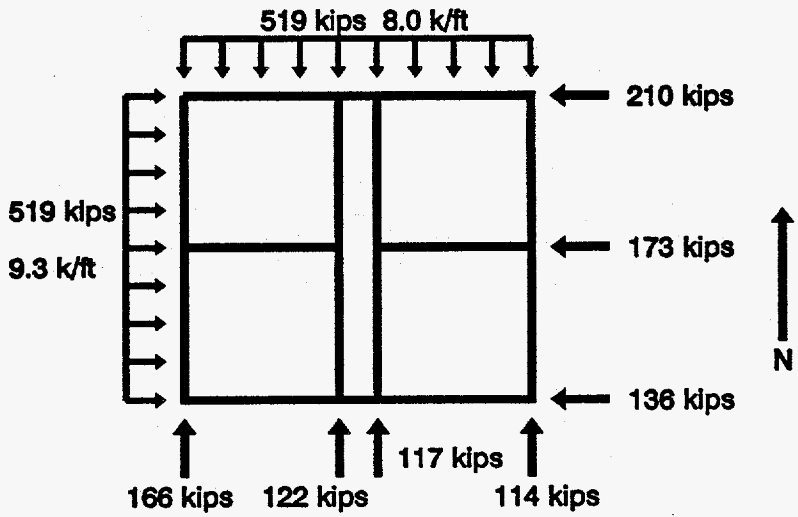
roof =	75 kips	Total =	75 kips

Earthquake Loads

roof =	325 X .25 =	81 kips		
beams =	40 X .25 =	10 kips		
8" walls =	402 X .25 =	101 kips		
columns =	44 X .25 =	11 kips		
fdn. wall =	136 X .25 =	34 kips	Total =	237 kips

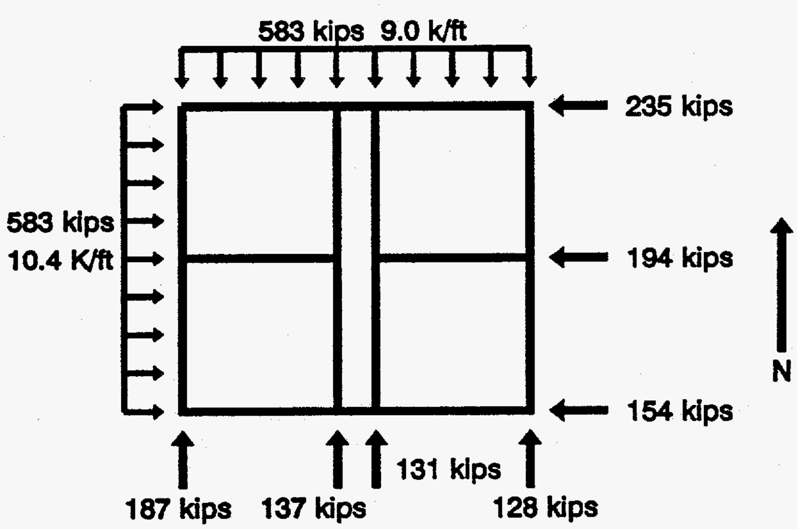
Horizontal Loads:

Earthqake Loads at the top of the foundation wall



Building Plan

Earthquake loads at the footing base (includes the above foundation loads)



Building Plan

DESIGN CALCULATION WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A6-4 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) LOAD COMBINATIONS

ACI-349 Section 9.2 :

1.) $1.4D + 1.7L$

$1.4(947) + 1.7(75) = 1453^K$

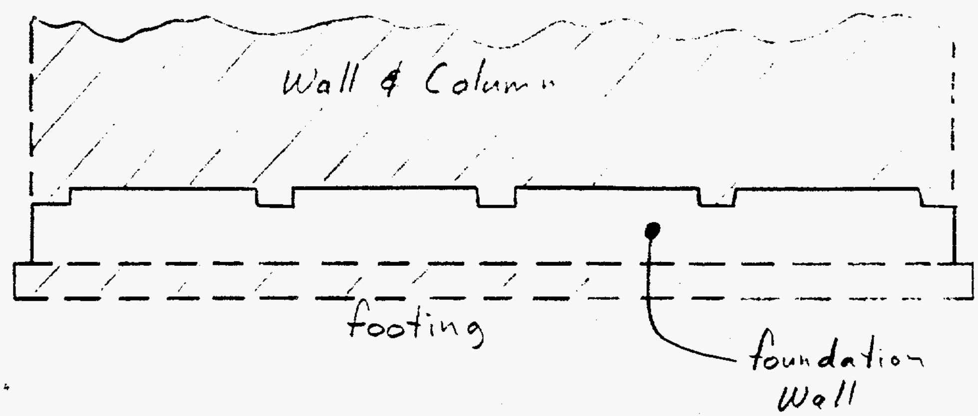
4.) $D + L + E$

$947 + 75 + 237 = 1259^K$

vertical loads

4.) $E = 519^K$ @ Top of Found

583^K @ Footing Base



N.S. Wall Line

Typical 4 Places

DESIGN CALCULATION

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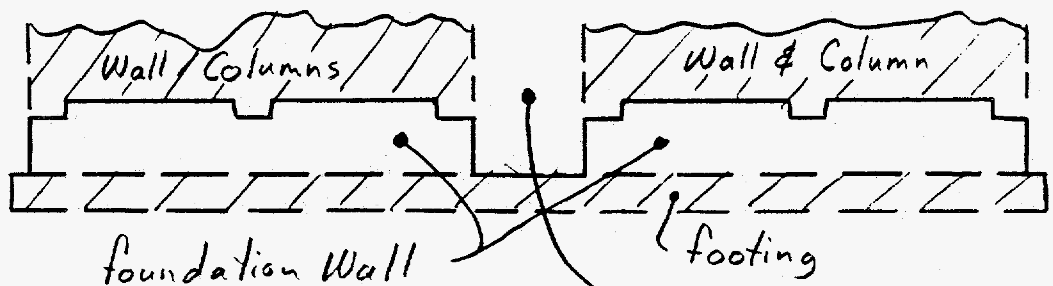
(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10)



E.W. Wall Line

1 place as shown, 2 places with center section filled in.

Vertical load;

$= 1259^K / (4 \text{ walls} \times 57.2' + 6 \text{ walls} \times 29.5')$

$= 3.1\ ^K/ft$

Horizontal load;

N.S. $= 519^K / (4 \times 57.2') = 2.27\ ^K/ft$ F. Wall Top

$583^K / (4 \times 57.2) = 2.55\ ^K/ft$ Footing Base

EW $= 519^K / (6 \times 29.5) = 2.93\ ^K/ft$ F. Wall Top

$583^K / (6 \times 29.5) = 2.55\ ^K/ft$ Footing Base

DESIGN CALCULATION

(1) Drawing ______ (2) Doc. No. ______ (3) Page A6-6 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator R.A. Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) COMPRESSIVE STRENGTH ANALYSIS

ACI-349 Section 14.5

width of bearing under columns

$b_w = 4t = 4 \times 8'' = 32''$

$$\phi P_n = .55 \phi f_c' A_g \left[1 - \left(\frac{k \ell_c}{32h}\right)^2\right] \qquad \phi = .7 \quad f_c' = 3\text{ ksi}$$

$A_g = 8'' \times 32'' \times 26$ columns $= 6656$ si

$k = 1.0$

$\ell_c = 1.2'$ to $4.2'$

$h = 8''$

$$= .55(.7)(3)(6656)\left[1 - \left(\frac{1.0 \times 4.2 \times 12''}{32 \times 8''}\right)^2\right]$$

$= 7390^K > 1259^K$ O.K.

DESIGN CALCULATION

(1) Drawing ____ (2) Doc. No. ____ (3) Page A6-7 of ____

(4) Building ____ (5) Rev. ____ (6) Job No. ____

(7) Subject ____

(8) Originator R.A. Diller Date 2-8-94

(9) Checker EO Weiner Date 2-5-94

(10) SHEAR STRENGTH ANALYSIS

N.S. Direction :

$583^K \div 4 \text{ walls} = 146^K/\text{wall}$

$V_c = 2\sqrt{f'_c}\,bh = 2\sqrt{3000}(8'')(.8 \times 57.2' \times 12'')$

$= 481^K$

$V_s = \dfrac{A_v f_y d}{s} = .11(40)$ a one bar wall segment

$= 4^K$

$\phi V_n = .85(481+4) = 412^K > 146^K$ O.K.

E.W. Direction :

$583^K \div 6 \text{ walls} = 97^K$

$V_c = 2\sqrt{f'_c}\,bh = 2\sqrt{3000}(8'')(.8 \times 31.7' \text{ ave.} \times 12'')$

$= 267^K$

V_c for one bar $= 4^K$

$\phi V_n = .85(267+4) = 230^K > 97^K$ O.K.

(1) Drawing ______ (2) Doc. No. ______ (3) Page A6-8 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator R.A. Stiller Date 2-8-94
(9) Checker Ed Weimer Date 2-8-94

(10) Check Shear Friction

@ Fnd Wall to Footing

N.S. Direction :

$V_s = A_v f_y \mu$ A_v (20-#5bars & 41-#4 bars)

$A_v = 14.4$ si

$f_y = 40$ ksi $\mu = 0.6$

$\phi V_n = .85(14.4)(40)\,0.6$

$= 294^K > 146^K$ O.K.

E.W. Direction :

$V_s = A_v f_y \mu$ A_v (12-#5bars & 20-#4 bars)

$A_v = 7.72$ si

$f_y = 40$ ksi $\mu = 0.6$

$\phi V_n = .85(7.72)(40)\,0.6$

$= 157^K > 97^K$ O.K.

Margins of Safety

$$MS = \frac{\text{allowable}}{\text{actual}} - 1$$

Wall compression $\frac{7390 \text{ kips}}{1259 \text{ kips}} - 1 = 4.86$

Wall shear $\frac{230 \text{ kips}}{97 \text{ kips}} - 1 = 1.37$

Shear friction $\frac{157 \text{ kips}}{97 \text{ kips}} - 1 = 0.62$

REINFORCING CHECK:

Minimum reinforcing:

Since the shear capacity of the concrete (Vc) is approximatly twice the value of the required strength, ACI-349 section 7.12 rather than 11.10.9 controls the minimum reinforcment.

Section 7.12 reinforcing;

$\rho_{min.}$ = .0012

ρ_{vert} = A_s/bs_o = .2 si / (8" X 15") = .0017 > .0012 O.K.

ρ_{horz} = .0017 > .0012 O.K.

A7.0 FOOTING EVALUATION

Orig RA Hiller 2-8-94 Checker EO Weiner 2-8-94

The 2736Z building footings are wall and column footings constructed of reinforced concrete poured monlithic with the foundation wall-column pedistal. The footings must be designed to resist shear and bending due to vertical loads, shear and bending due to horizontal loads, and to resist sliding and overturning through interaction surrounding soil.

- **DESIGN LOADS AND LOAD COMBINATIONS**

  Design loads are specfied in SDC-4.1 ~~rev.11~~ rev.12, UCRL-15910 and ANSI A58.1. Load combinations are listed in UCRL-15910 and ACI-349.

- **BENDING AND SHEAR STRENGTH ANALYSIS**

  Vertical and lateral building loads produce bending and shear due to bending in the footings. The design shall be in accordance with ACI-349 chapters 10, 11 and 15 requirements.

- **OVERTURNING, SLIDING RESISTANCE AND SOIL BEARING**

  The design of footings or foundations for overturning, sliding and soil bearing is based on UBC 1991 and the Structural Engineers Hanbook. These documents provide factors of safety for different loading conditions

DESIGN LOADS:

Vertical Loads:

Dead Loads

roof =	325 kips		
beams =	40 kips		
8" walls =	402 kips		
6" walls =	153 kips		
columns =	44 kips		
foundation walls =	136 kips		
wall footing =	90 kips		
column footing =	79 kips		
column Ped. =	8 kips	Total =	1277 kips

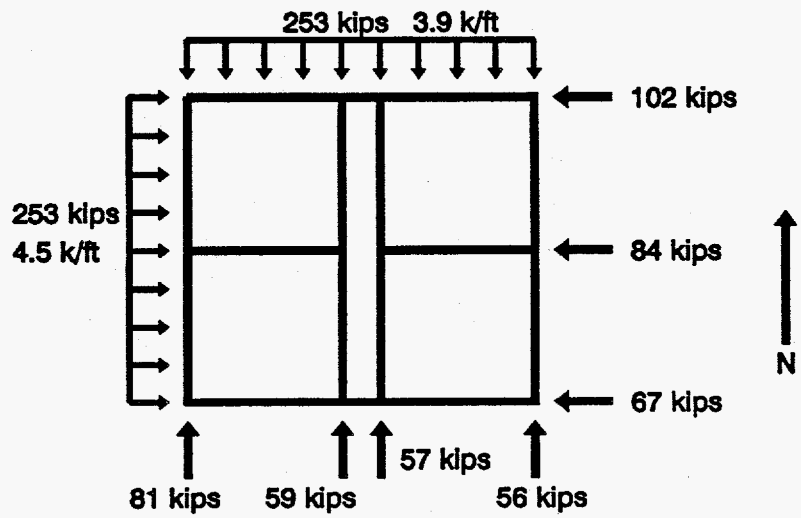
Live Loads, roof =	75 kips	Total =	75 kips

Earthquake Loads

roof =	325 X .25 =	81 kips		
beams =	40 X .25 =	10 kips		
8" walls =	402 X .25 =	101 kips		
6" walls =	153 X .25 =	38 kips		
columns =	44 X .25 =	11 kips		
fdn wall =	136 X .25 =	34 kips		
wall ftng =	90 X .25 =	23 kips		
col. ftng =	79 X .25 =	20 kips		
column Ped.	8 X .25 =	2 kips	Total =	320 kips

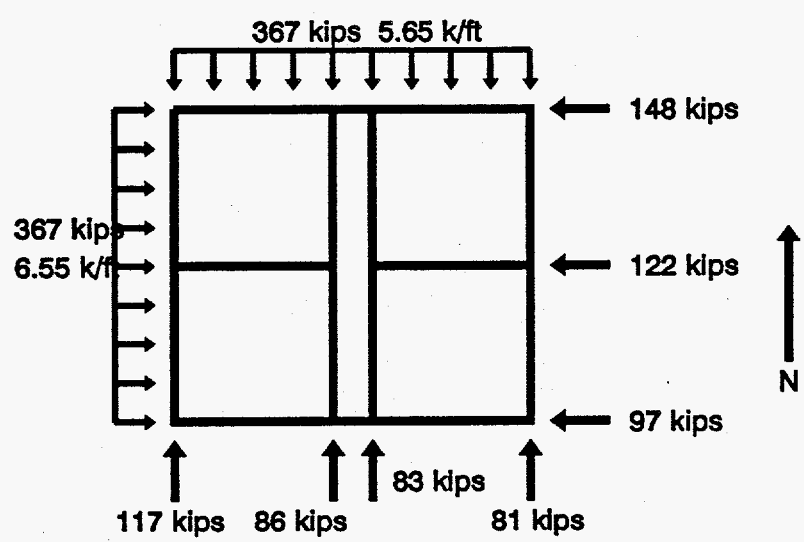
Horizontal Loads:

Earthquake Loads at the roof level (EL = 11')



Building Plan

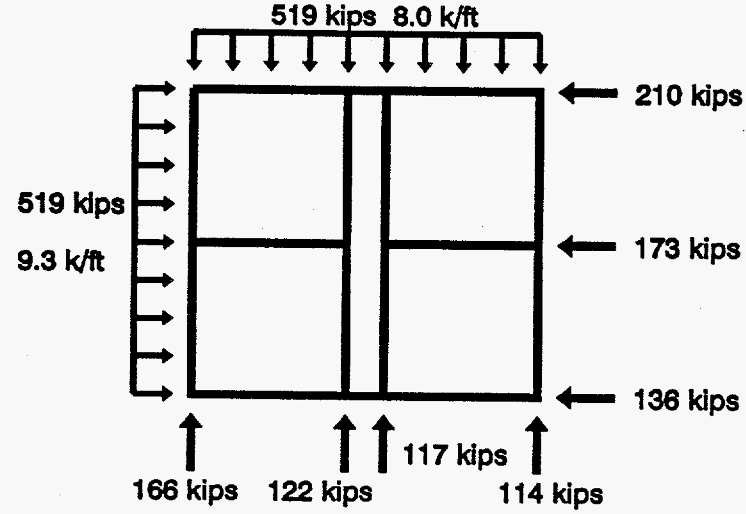
Earthquake Loads at the Wall Bottom (EL = 0.22'), (includes roof level loads)



Building Plan

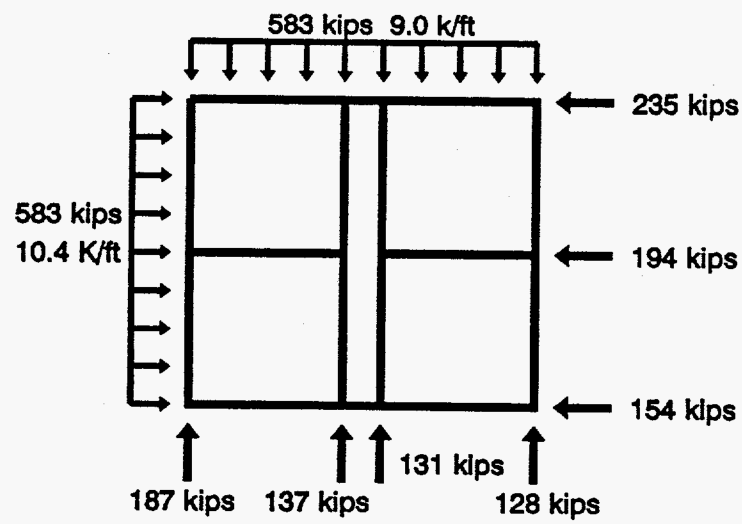
Orig R.A. Hiller 2-8-94 Checker EO Weiner 2-8-94

Earthquake Loads at the Foundation Wall Top (EL = 0.2′)
(Includes the Roof level and Wall Bottom level loads)



Building Plan

Earthquake Loads at the Footing Base (EL = -2.75′)
(Includes the loads form the Roof, Wall Bottom, and Foundation Wall levels)



Building Plan

LOAD COMBINATIONS: (ACI-349 Section 9.2)

Vertical Loads:

Note; Live, roof, beams, 8" wall, columns, column pedistal, column footing and associated earthquake vertical loads are transfered into the column footings because the 8" walls do not rest on the foundation walls.

The 6" walls, foundation walls, wall footing, and associated earthquake vertical loads are transfered into the wall footings.

1.) 1.4D + 1.7L

$$\frac{1.4(325 + 40 + 402 + 44 + 79 + 8) + 1.7(75)}{30 \text{ columns}} = \quad 46 \text{ kips/column}$$

$$\frac{1.4(153 + 136 + 90)}{297' \text{ of wall footing}} = \quad 1.8 \text{ kips/ft}$$

4.) D + L + E

$$\frac{(898) + 75 + (81 + 10 + 101 + 11 + 20 + 2)}{30 \text{ columns}} = \quad 40 \text{ kips/column}$$

$$\frac{(379) + (38 + 34 + 23)}{297' \text{ of wall footing}} = \quad 1.6 \text{ kips/ft}$$

Horizontal Loads:

4.) E

583 kips — 20 columns resist loads in the NS direction

18 columns resist loads in the EW direction

583 kips / 20 columns = 29 kips / column

583 kips / 18 columns = 32 kips / column

OVERTURNING, SLIDING, AND SOIL BEARING:

References: SDC-4.1 ~~Rev. 11~~ Rev. 12
UBC 1991
Structural Engineers Handbook (SEH)

Acronyms:

OTM =	Overturning Moment
SF =	Sliding Force
DLRM =	Dead Load resisting Moment
DLSR =	Dead Load Shear Resistance
SBL =	Soil Bearing Load
ASB =	Allowable Soil Bearing

Criteria: For sustained loads such as a retaining wall resisting soil loads or a building structure resisting wind loads, UBC and SEH recommend or require the following.

DLRM $\geq$ 1.5 (OTM)

DLSR $\geq$ 1.5 (SF)

SBL $\geq$ ASB Note; a 1/3 increase in ASB is allowed when resistance is being provided for wind loads.

For non-sustained seismic loads UBC recommends the following criteria.

DLRM $\geq$ OTM

DLSR $\geq$ SF

SBL $\geq$ 1.33 (ASB)

Allowable Soil Bearing pressure:

From the ~~2725~~ 2425 evaporator building soil report by Shannon & Wilson

ASB = 5,000 psf at 3' depth and less than 4' wide footing.

The 2736Z building footings are 2.75' deep which will reduce the ASB to about 4,500 psf.

Use ASB = 4,500 psf
seismic ASB = 1.33 X 4,500 psf = 6,000 psf

BENDING AND SHEAR STRENGTH ANALYSIS:

Vertical load = 40 kips / col. and 1.6 kips / ft

Horizontal load = 32 kips / col.

Overturning Moment:

NS direction, worst case wall

$(81^k)14' + (166^k - 81^k)3' = 1389$ ft-kips

EW direction, worst case wall

$(102^k)14' + (210^k - 102^k)3' = 1752$ ft-kips

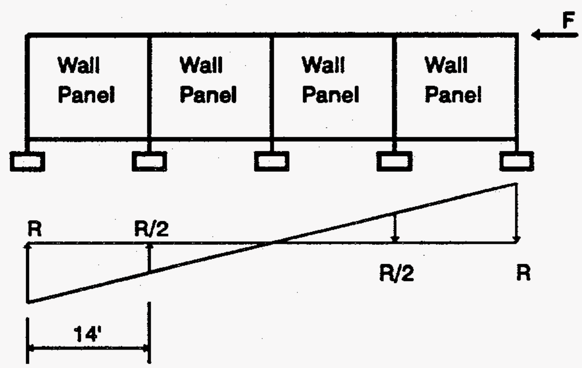
Additional vert. load due to OTM:

NS direction

OTM = R(56') + R/2(28')

OTM = 70R

$R = \dfrac{1389 \text{ ft-kips}}{70} = 19.8$ kips

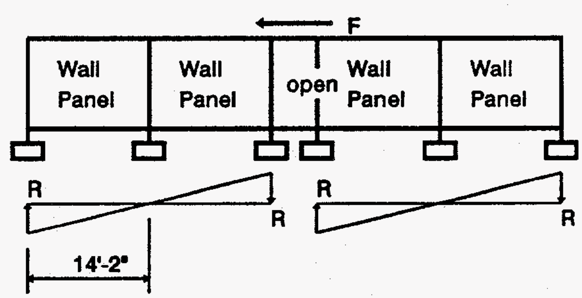


EW direction

OTM = R(14.17')2

OTM = 28.34R

$R = \dfrac{1752 \text{ ft-kips}}{28.34(2 \text{ walls})} = 30.9$ kips



DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A7-8 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Miller Date 2-8-94

(9) Checker Ed Weiner Date 2-8-94

(10) Total Vertical Load use 100-40-40

Column Footing:

$$= \frac{D + L + .4E_V}{30} + E_{EW} + .4E_{NS}$$

$$= \frac{298 + 75 + .4(225)}{30} + 30.9 + .4(19.8)$$

$= 74^K$ with seismic loads

46^K w/o seismic loads

DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. 0/1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A7-9 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

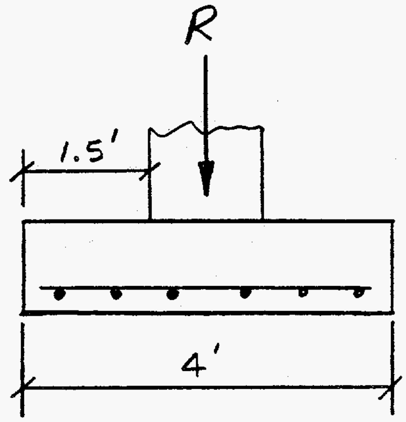
(8) Originator R. A. Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10)

Column Footing Strength:

No applied base moment
(18 places)

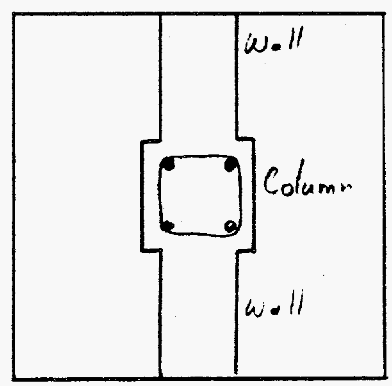


6 - #4 bars each way

$$A_s = \frac{6(.2)}{4'} = .3\ \text{si}/\text{ft}$$

$t = 12''$

$d = 12'' - 3'' - .75'' \simeq 8''$



PLAN

OR

Soil Bearing Pressure:

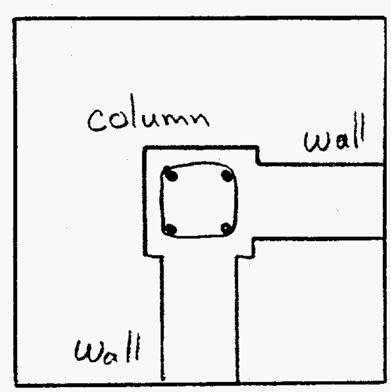
$46^{K}/16\text{sf} = 2.9\ \text{ksf} < 4.5\ \text{ksf}$

O.K.

$74^{K}/16\text{sf} = 4.625$

$4.625 < 6.0$

O.K.



PLAN

DESIGN CALCULATION WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A7-10 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Miller Date 2-8-94

(9) Checker EO Werner Date 2-9-94

(10) Column Footing Strength:

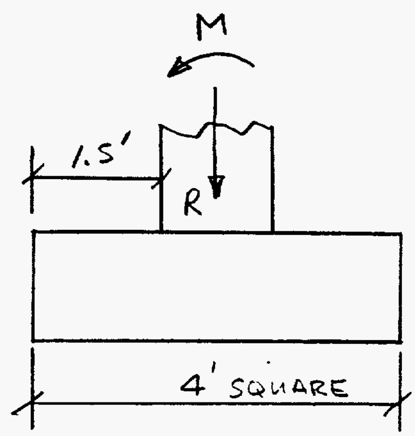
with applied moment in one direction (8 places)

vertical load = 74^K

applied moment = $166''^K$ from column evaluation

Soil Bearing Pressure:

$e = \frac{M}{R} = \frac{166}{74} = 2.24'' = .187'$

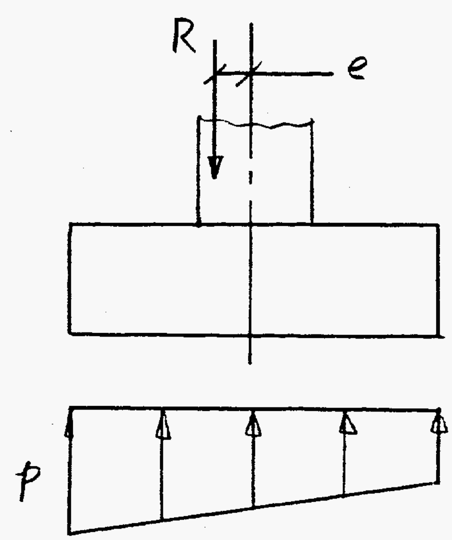


$p = \frac{R}{A}\left(1 + \frac{6e}{l}\right)$

$p = \frac{74^K}{16 sf}\left(1 + \frac{6 \times .187'}{4'}\right)$

$p = 5.9$ ksf < 6.0 ksf

O.K.



(1) Drawing ______ (2) Doc. No. ______ (3) Page A7-11 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator R. A. Hiller Date 2-8-94
(9) Checker E O Weiner Date 2-8-94

(10) Column Footing Strength:

with applied moment in 2-directions (4 places)

vertical load = 46^K

applied moments = $\frac{86}{2} = 43^{''\cdot K}$ & $\frac{97}{2} = 49^{''\cdot K}$

Soil Bearing Pressure:

$$e = \frac{49^{''\cdot K}}{46^K} = 1.06'' = .089'$$

$$P = \frac{P}{A}\left(1 \pm \frac{6e}{l}\right) \qquad P_{ave} = \frac{46^K}{16sf} = 2.875 \text{ ksf}$$

$$= \frac{46}{4' \times 4'}\left(1 \pm \frac{6 \times .089}{4'}\right)$$

3.26 ksf for one direction bending

pressure increase due applied moment in one direction = 3.26 − 2.875 = .385

Total soil pressure:

$$P = 3.26 + .385 = 3.645 \text{ ksf}$$

$$3.645 < 4.5$$

footing O.K.

DESIGN CALCULATION

WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A7-12 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator R. A. Hiller Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10) Bending Strength:

$$M_u = \begin{cases} \text{Type III} & \frac{wl^2}{2} = \frac{3.645(1.67')^2}{2} = 5.08^{'\cdot K} = 61^{''\cdot K} \\ \text{Type II} & \frac{wl^2}{2} = \frac{5.9(1.5)^2}{2} = 6.64^{'\cdot K} = 80^{''\cdot K} \end{cases}$$

$$A_sF_y = .85 f_c' ab$$

$$a = \frac{A_sF_y}{.85 f_c' b} = \frac{.3(40)}{.85(3)(12'')} = .392''$$

$$\phi M_n = .9 A_sF_y(d - a/2) = .9(.3)(40)(8 - \frac{.392}{2})$$

$$= 84^{''\cdot K} > 80^{''\cdot K} \quad \text{O.K.}$$

Shear Strength:

for this footing-column-wall configuration only beam shear is applicable

$$V_u = wl = 6.0(0.8') = 4.8^K$$

$$\phi V_c = (.85)2\sqrt{f_c'}\, bd = \frac{(.85)2\sqrt{3000}\ 12''\cdot 8''}{1000} = 8.9^K$$

$$8.9 > 4.8 \quad \text{O.K.}$$

DESIGN CALCULATION

(1) Drawing ______ (2) Doc. No. ______ (3) Page A7-13 of ______
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator R. A. Gillen Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10) Margins of Safety

D & L load Soil Bearing $\frac{4.5}{3.645} - 1 = 0.23$

D + L + E Soil Bearing $\frac{6.0}{5.9} - 1 = 0.017$

Footing Bending $\frac{84^{\text{''}\cdot K}}{81} - 1 = 0.037$

Footing Shear $\frac{8.9}{4.8} - 1 = 0.854$

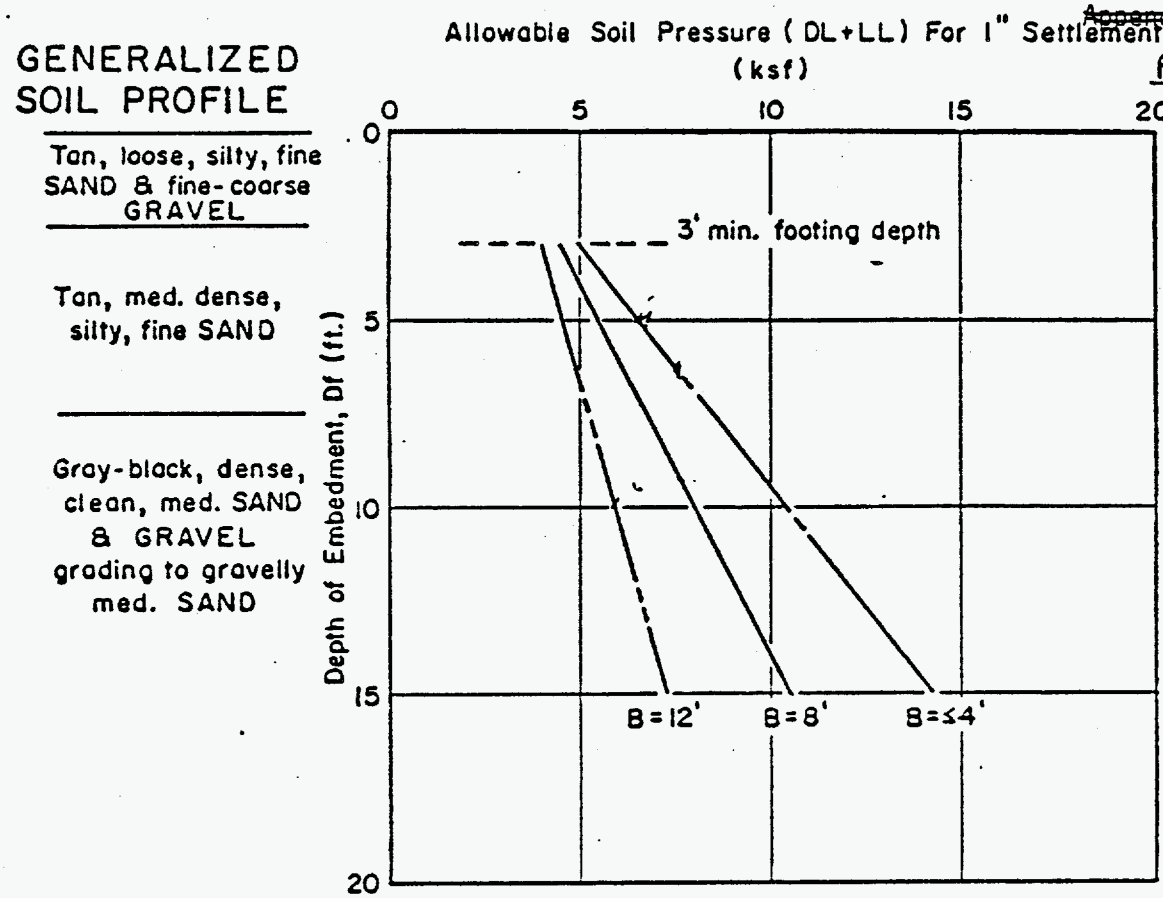
Wall Footing

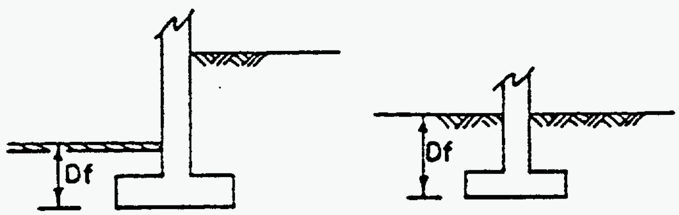
Soil Bearing $\frac{4.5}{0.9} - 1 = 4.0$

w & w/o seismic

Bending $= \infty$

Shear $= \infty$





Embedment

Note:

For footings in fill or backfill use allowable bearing pressure for B=12' regardless of footing size.

B is the least dimension of continuous or square footing.

HANFORD ATOMIC ENERGY RESERVATION
EVAPORATOR FACILITY
BLDG. 242-S

ALLOWABLE BEARING PRESSURES
UNDISTURBED SUBGRADE

C-301 OCT, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 5

A8.0 DEFLECTION CALCULATION

Orig R. A. Diller 2-8-94 Checker EO Weiner 2-8-94

WHC-SD-CP-SA-023
Rev. ~~0~~ 1
Page A8-1

- **SHEAR WALL DEFLECTIONS**

  Shear wall deflections result from shear and flexural deformations of the concrete shear walls with respect to a fixed base.

- **OUT-OF-PLANE DEFLECTIONS**

  Out-of-plane deflections results from the horizontal deflection of the wall panels due to slab type flexure (out-of-plane flexure).

- **SOIL SPRING DEFLECTIONS**

  Soil spring deflections result from the rocking type motion of a rigid shear wall on a flexable soil base.

(1) Drawing ______ (2) Doc. No. ______ (3) Page A8-2 of ______

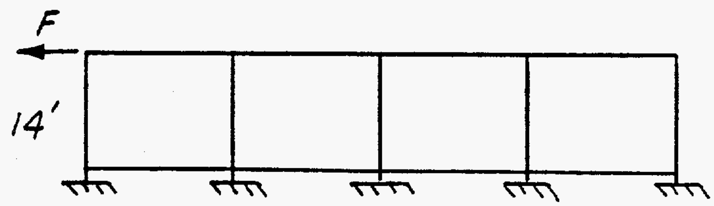
(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator R.A. Hiller Date 2-8-94

(9) Checker Ed Wemer Date 2-8-94

(10) SHEAR WALL DEFLECTION



Shear Deflection:

$F = 102\ ^{K}/\text{wall}$ maximum

$\delta_s = \dfrac{PL\alpha}{AE_c}$

$P = F = 102^{K}$ $L = 14' = 168''$

$\alpha = 1.2$ for rectangular members

$A = (56' \times 12) \times 8'' = 5376$ si

$E_c = 1{,}000{,}000$ psi

$\delta_s = .0038$

Bending Deflection:

$\delta_B = \dfrac{PL^3}{3EI}$

$P = 102^{K}$ $L = 168''$

$E = 3{,}000{,}000$ psi

$I = \dfrac{bh^3}{12} = \dfrac{8''(672'')^3}{12} = 202{,}309{,}632\ \text{in}^4$

$\delta_B = .0003$

(1) Drawing ______ (2) Doc. No. ______ (3) Page A8-3 of ______

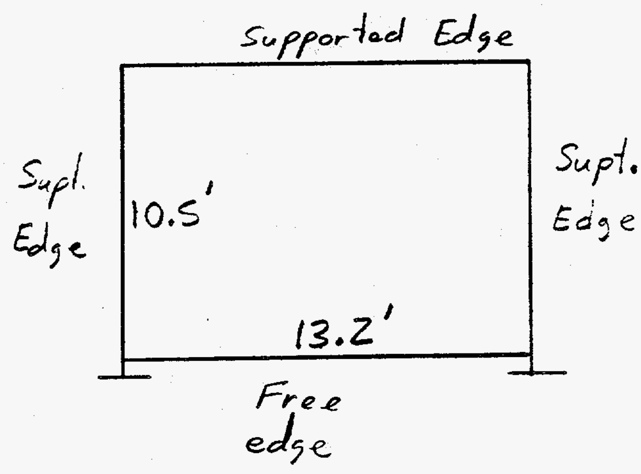
(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) OUT-OF-PLANE DEFLECTIONS



From "Roark" page 461 2a.

$$\max y = \frac{\alpha q b^4}{Et^3}$$

$\max y = \delta_0$

$q = w = .0665$ ksf $= .000462$ ksi

$b = 13.2' = 158''$ $a = 10.5' = 126''$

$E = 3{,}000{,}000$ psi

$t = 8''$

$a/b = .80$

$\alpha = 0.120$

$\max y = \delta_0 = .02249''$

(1) Drawing ______ (2) Doc. No. ______ (3) Page A8-4 of ____
(4) Building ______ (5) Rev. ______ (6) Job No. ______
(7) Subject ______
(8) Originator R.A. Gilla Date 2-8-94
(9) Checker EO Weiner Date 2-8-94

(10) SOIL SPRING DEFLECTIONS

Vertical Spring Rates from UCRL-15910:

$$K_v = \frac{G}{1-\nu} \beta_z \sqrt{BL}$$

for square 4' column footings

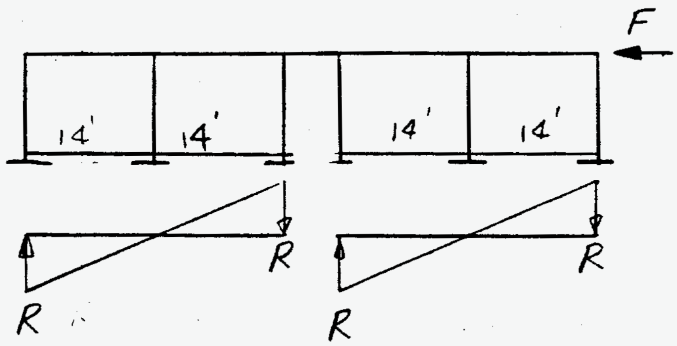
$B = L = 4'$

$\beta_z = 2.2$

From Geotechnical Investigation of Z-Plant dated January 1980

$G = 1,700,000$ psf
$\nu = .37$
} 0 to 10' below grade

$K_v = 23746$ K/ft



$F = 102^K$

$R = 31^K$ from footing analysis

DESIGN CALCULATION WHC-SD-CP-SA-023 Rev. ~~0~~ 1

(1) Drawing ______ (2) Doc. No. ______ (3) Page A8-5 of ______

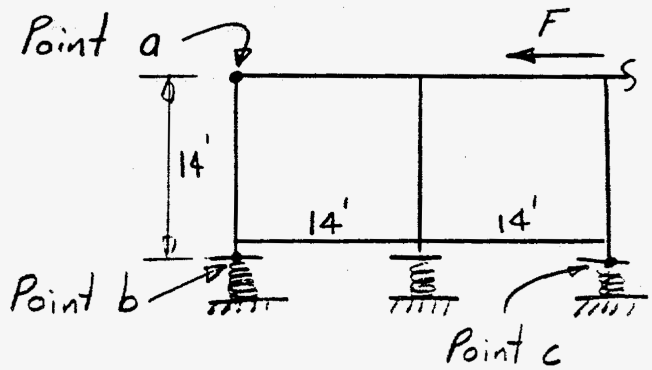
(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Hiller Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10)



Vertical displacement @b due to F :

$$\delta_b = \frac{31^K}{K_v} = \frac{31}{23746} = .0013' = .01567''$$

$$\delta_c = -\delta_b = -.01567$$

Horizontal displ. @ a due to δ_b & δ_c :

$$\delta_a = \frac{h}{L}\delta_b = \frac{14'}{14'}(.01567'') = .01567''$$

Horizontal Spring Rate from UCRL-15910 :

$$K_h = 2(1+\nu)G\,\beta_x\sqrt{BL} \qquad \beta_x = 1.0$$

$$K_h = 18632\ ^{K}/_{ft}$$

$$\delta_h = \frac{F}{K_h} = \frac{102}{18632} = .00547' = .06569''$$

DESIGN CALCULATION

(1) Drawing ______ (2) Doc. No. ______ (3) Page A8-6 of ______

(4) Building ______ (5) Rev. ______ (6) Job No. ______

(7) Subject ______

(8) Originator RA Gilles Date 2-8-94

(9) Checker EO Weiner Date 2-8-94

(10) Total Deflections:

$\delta_S = .0038''$

$\delta_B = .0003''$

$\delta_O = .0225''$

$\delta_o = .0157''$

$\delta_h = .0657''$

$0.1080''$