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A REPORT ON
THE SEISMIC CAPACITY OF THE GENERAL LABORATORY AND
ADMINISTRATION BUILDING AT LOS ALAMOS NATIONAL LABORATORY

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Abstract

A seismic analysis of the General Laboratory and Administration Building at Los Alamos National Laboratory is performed. The analyses are performed in detail for one portion of the building and then qualitatively extrapolated to other portions of the building. Seismic capacities are evaluated based on two sets of acceptance criteria. The first is based on Code-type criteria and is associated with a low probability of failure. This capacity is found to be in the 0.04 - 0.06 G ZPA range (the free field seismic motion is defined with a NUREG 0098 response spectrum). The second capacity is based on much less conservative criteria such as might be associated with a high probability of failure. This capacity is found to be about 0.15 G. Finally structural modifications are proposed that would increase the low probability of failure capacity to 0.15 G ZPA. These modifications consist of steel double angle braces or concrete shear walls placed at some of the frames in the building.

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Executive Summary

The General Laboratory and Administration Building (GLAB) at Los Alamos National Laboratory was constructed during the 1950's and is now used primarily for office space. The building consists of 6 units forming an "H" shape with each of the units separated by 1-inch construction joints. The building has about 64,000 square feet of plan area and 3-4 stories above grade with 1-2 stories below grade. The building is a reinforced concrete frame with stiff beams and flexible columns, and is founded on individual column footings. Most of the interior partitions are unreinforced masonry. The soil is a rather competent tuff.

The objective of this study was to evaluate the seismic capacity of the structure and in particular to determine the level of earthquake, defined with a NUREG 0098 soil spectra, that causes the structure to fail Code acceptance criteria. A preliminary review of the building details indicated that its seismic capacity is likely to be low; the details of the design indicate that it was constructed primarily to resist gravity loading and does not contain many of the details required of frames that must resist lateral loading. Two analyses were performed to evaluate its seismic capacity. The first used standard evaluation methods with Code material properties and determined the seismic level that would be required to exceed Code acceptance criteria. This capacity was found to be in the range of 0.04 - 0.06 G and is associated with a low probability of failure. The second analysis used optimistic assumptions (for material properties, ductility effects, and load redistribution) to obtain an estimate of the earthquake that would be associated with a high probability of failure. This result was found to be about 0.15 G. An estimate of the median seismic capacity of the structure is about 0.10 G. It should be cautioned that these results are associated with damage to the structural frame. Other damage such as cracking and partial collapse of the unreinforced masonry walls in the building could occur at lower earthquake levels.

A preliminary study was performed defining the type of structural modifications that would be required to improve the buildings Code capacity to 0.15 G ZPA. Either steel bracing or concrete shear walls could be used to increase the capacity. The steel bracing system is probably easier to construct, but the concrete shear walls are more likely to provide the stiffness required to protect the masonry walls.

Acknowledgement

The building under consideration is quite old and the assistance of D. Volkman, D. Keller, and B. Sanchez of LANL was essential to obtaining the information required to conduct the study. The suggestions offered by LANL staff and their consultant, Dr. Loring Wyllie, for improving the draft report are acknowledged. Finally the support and work of J. Kimball and M. Davister of DOE-DP are acknowledged.

1.0 Introduction

This report describes seismic analyses performed on the General Laboratory and Administration Building (GLAB) at Los Alamos National Laboratory (LANL). The work was supported on a contract from the Office of Defense Programs, Department of Energy (DOE). Many of the calculations described in this report were performed jointly by The City College of New York (CCNY) and BNL staff using the ANSYS computer program available at BNL. The work began in November 1993 with an inspection of the GLAB at LANL. Engineering drawings and design calculations were supplied by LANL staff at this meeting. A draft report was submitted during July 1994, and revisions have been made to that draft based on discussions with DOE, and LANL staff (including LANL consultants).

1.1 Objectives

The objective of the project is to provide data to DOE that can be used to develop an appropriate response for the GLAB to the National Earthquake Hazard Reduction Program (NEHRP) which requires that the seismic adequacy of all government facilities should be determined. The following two specific project objectives are established:

- The first specific project objective is to estimate a seismic fragility curve for the GLAB. The GLAB is a large structure contains several different sections. The fragility curve is considered in detail for "typical" portions of the structure and these results are then extrapolated to the remaining parts of the structure.
- The second specific project objective is to investigate the potential for increasing the seismic capacity to at least 0.15 G (ZPA). The seismic upgrade studies are performed with the intent of demonstrating the feasibility of potential structural concepts for portions of the structure. Complete upgrade designs would have to be developed if DOE decides to enhance the seismic capacity of the building.

The fragility curve is estimated by deterministic evaluations of structural capacities associated with two significantly different probabilities of failure, using two different acceptance criteria. The first evaluation makes use of Code type acceptance criteria, and leads to a capacity estimate associated with a low probability of failure (LPF). The second evaluation utilizes more liberal estimates of structural capacity (based on higher percentile material properties than permitted by Code criteria, consideration of ductility effects, and redistribution of loads after local failures), leading to a capacity estimate associated with what is estimated to be a high probability of failure

(HPF). The mean seismic capacity of the structure is then estimated to be the average of the capacities associated with the LPF and the HPF estimates. This is formally true if the fragility curve can be represented with a normal distribution and if the sum of the probabilities associated with the LPF and HPF equals unity. While the estimate of failure probabilities associated with either the LPF or HPF is subjective, it is expected that the LPF is less than 10 % , the HPF is more than 90 % and the sum of the two is unity.

The evaluations of the LPF and HPF points on the fragility curve are based on a level of damage associated with collapse of major portions of the structure. Of course, fragility curves associated with lower levels of damage (e.g., development of structural cracks, cracking or even partial collapse of masonry walls, breakage of glass, etc.) would be related to significantly lower seismic loads than the fragility data developed in this study. The GLAB contains many unreinforced masonry walls. It is important to note that damage (or even collapse) of these walls is not included within the definition of damage for which the fragility data are defined.

Several types of seismic upgrades are investigated to achieve the second objective. The addition of a steel bracing system or concrete shear walls (or external diaphragms) is considered to increase the seismic capacity of the structure. These upgrades are designed based on Code acceptance criteria to achieve a LPF seismic capability of 0.15 G ZPA.

1.2 Structural Description

The GLAB was constructed in the late 1950's. A sketch of the plan of the GLAB is shown on Fig. 1.1. The column lines (A through V in the E-W direction and 1 through 25 in the N-S direction) are spaced at about 20' in each direction so that the overall plan dimensions are about 320' by 440'. The N-S legs on the "H" shape are called wings with the west wings labeled "A" and the east wings labeled "D". The cross of the "H" is called the main structure and divided into two pieces labeled "B" and "C". Expansion joints separate the wings from the main core and divide the main structure into the "B" and "C" pieces as shown. One inch gaps exist at all of the expansion joints. The numbers shown on Fig. 1.1 indicate: the number of stories above grade / number of stories below grade. Each story height is about 11.65'. The roof elevations of the wings "A" and "D" are the same with four stories contained from the roof elevation to the ground. Wing "A" contains one basement story while wing "D" contains a sub-basement over a portion of its area. The main building ("B" and "C") has two stories above grade (i.e., roof is two stories below the roof elevation of the wings) although there are several penthouses located on the roof of the main building. The main buildings contain four stories above grade over column lines A-C and T-V; in fact, the structure of these portions of the main buildings "B" and "C" are identical to the

respective wing structures "A" and "D".

The primary structural system is shown on Fig. 1.2 . Wing "A" is used for illustrative purposes but the other buildings are similar. The frame consists of columns connected with stiff beams at the floor elevations. The columns are lightly reinforced and on the order of 18" by 18" in cross section. The beams are about 19" deep and 4' wide. One way floor slabs about 6" thick span between the 4' wide beams. Spandrel beams span between the columns in the direction of the floor slabs. As shown on Fig. 1.1, the 4' beams span in the E-W direction of the wing structures. The first floor beams span in the N-S direction and the upper story beams span in the E-W direction of the "C" section of the main building. The beams span in both directions (for different locations) in the "B" section. The "B" section of the main building contains an auditorium area between column lines E and H, and 11 and 15. A open area from the first to roof is created with N-S steel trusses located on column lines E and H.

The columns are founded on individual footings about 8' by 8'. Foundation walls have minimal reinforcement and do not have good structural connections with the columns or floor slabs. These walls provide in-plane stiffness to the frame but not significant out-of-plane stiffness.

Interior partitions are of unreinforced masonry. The exterior walls, with the exception of the ends of the wings, are of transite and glass. The ends of the wings (along column lines 1 and 25) are filled with unreinforced masonry. The primary use of the GLAB is for office space with a few laboratory and storage spaces provided.

1.3 Report Organization

The E-W frame spanning between column lines A-B-C and located at column line 7 is selected as representative of frames containing the beams. The seismic analyses for this frame is described in Section 2. The N-S frame in the same wing (spanning from column line 1 to 9) is selected as typical of a frame spanning in the direction of the floor slabs. The seismic analyses for this frame is described in Section 3. The data and conclusions obtained from these analyses are then applied to the remainder of the structural elements (wing "D" and main structural sections "B" and "C"). These analyses are presented in Section 4. The results are summarized in Section 5.

Fig. 1.1 GENERAL LABORATORY AND ADMINISTRATION BUILDING PLAN

Fig. 1.2 TYPICAL WING STRUCTURAL FRAMING SYSTEM

2.0 Transverse (E-W) Frame Model of Wing "A" Structure

The response of a typical (E-W) transverse frame in the wing structures to the applied loading is discussed in this section of the report. This section contains: discussion of the data used in the analyses, description of the model, discussion of the frame capacity (based on code-like acceptance criteria utilizing lower bound estimates for capacity) for both static and seismic load cases, an evaluation of the increased capacity that would occur if less conservative acceptance criteria are used, and an evaluation of potential fixes to increase the frame capacity.

2.1 Data Used in the Analyses

The drawings for the wing structures were reviewed and the E-W frame along column line 7 (see Fig. 1.1) was selected as the typical frame to be used for the analyses. The frame spans across column lines A-B-C. A sketch of the frame with pertinent dimensions, gross member properties, material descriptions, and general notes of interest is shown on Fig. 2.1. Reinforcement for the columns is also shown on Fig. 2.1. Beam dimensions and reinforcement details are shown on Fig. 2.2. These data are obtained from the following GLAB drawings: S-1.2, S-5.1, S-6.2, S-7.1, S-8.1, S-9.1, S-13.1, S-20.2, S-23.2, S-24.2, and S-25.2 .

The following characteristics of these data are important in assessing the seismic capacity of the frame:

- The only lateral resistance (such as required to resist horizontal seismic loading) comes from the beam/column frame shown on Fig. 2.1. Interior E-W masonry block walls exist but they are unreinforced and would not be expected to contribute to the lateral load capacity of the frame.
- The bottom reinforcement in the beams is only embedded 6" into the columns. This is a standard detail for beams subjected to gravity loads when only negative moments (tension at the top of the beam) occur at the beam/column connection. However, seismic loads will develop positive moments at alternating ends of the beam as the frame sways from one side to the other. The "gravity load detail" therefore degrades the seismic capacity.
- There is very little shear reinforcement in the beams and it does not satisfy the minimal transverse reinforcement requirements for seismic details (see Section 21.3.3 of ACI 318-89).
- The columns are lightly reinforced with typical primary reinforcement ratios in the upper story columns being about 0.008. This does not satisfy the current ACI Code requirement of a minimum gross reinforcement ratio of 0.01.
- Ties in the columns are minimal and in most cases do not satisfy ACI Code minimum

requirements even for non seismic loads.

The following loads are used in the analyses in addition to the structural weight:

- Live load (LL) - 100 psf on the floors and 25 psf on the roof. When LL and seismic loads are combined one quarter of the LL is used. In some cases the live loading increases the seismic capacity of the frame so that too large of a live load may be unconservative. The live loading ($100/4 = 25$ psf) however is small as compared with the dead loading (6" slab plus 31 psf superimposed DL ~ 105 psf) so that further reductions in live load will not seriously effect the results.
- Exterior wall panel weight - 280 lbs/ft
- Superimposed dead load (DL) on floors - 31 psf
- Superimposed DL on roof - 23.5 psf
- Seismic load (EQ) is defined with a NUREG 0098 median soil spectra (spectral shape is shown on Fig. 2.3). The spectral shape is anchored to different peak accelerations (ZPA) and the magnitude of the ZPA is used to define the earthquake.

Soil data was obtained from available nearby boreholes and the soil was found to be rather competent. The soil analysis is shown in Appendix A.

2.2 Description of Model

The frame model used for the analyses is shown on Fig. 2.4. Separate nodes are used for the beam and columns at the beam/column joints so that it is possible to specify shear or moment connections between the two elements. The connection is rigid (i.e., capable of transmitting both moment and shear forces) when tensile stresses occur at the top of the beam but is close to a shear connection when tensile stresses occur at the bottom of the beam (since the bottom reinforcement is not sufficiently embedded into the column to develop a moment). The two conditions are handled by identifying a node at the end of the beam and a colocated node on the column and by coupling the displacements and rotations of the two nodes for the moment connection or by coupling only the displacements of the two nodes for the shear connection. Nodes are placed at the center of the beams for convenience so that bending moments can be determined directly at these locations. The base of the columns are connected to the soil with rotation springs representing the fixity provided by the footings. The characteristics of this spring are discussed below.

The following load combinations are discussed relative to the appropriate beam/column connection to use for the model:

- (DL + LL) loading produces negative moments at the connection (tensile stresses at the top) so that the joints are treated as continuous for this load case.
- Treatment of the (DL + LL/4 + EQ) load case is more complex. Typical beam bending moment diagrams for the (DL + LL/4) and the seismic (EQ) are shown below. As stated above the first load case results in negative moments at both ends of the beam while the second load case results in a negative moment at one end of the beam and a positive moment at the opposite end of the beam. If the magnitude of the (EQ) end moments are less than the magnitude of the (DL + LL/4) moments (Case 1) then the joint at both ends of the beam can be treated as continuous, but if the the (EQ) end moments are larger than the (DL + LL/4) moments (Case 2) then one of the end joints will be continuous and the other will be a shear connection. Of course the sense of the seismic moments vary as the structure sways in the east or west directions. An exact treatment of this varying connection detail can be handled with the use of a nonlinear rotational spring connecting the end of the beam to the column. Unfortunately this connection detail requires that time history analyses be used to evaluate the seismic response of the structure. The following alternative representations of the connection is used so that the structure can be analyzed with linear response spectrum analyses methods. First all joints are modeled as rigid and the seismic capacity is determined such that Case 1 applies (if the joint can be shown to have a positive moment capacity greater than zero then the seismic moments are restricted to the sum of the DL + LL/4 moments + the positive moment capacity). Second the joint at one end of the beam is modeled as pinned and the opposite end joint is modeled as rigid if the Case 1 results show that the positive moment capacity of the joint is limiting. The total bending moment at the rigid end is the sum of (DL + LL/4 + EQ) and must be less than the negative moment capacity of the joint.

The stiffness of the structural members (moments of inertia and cross sectional area) are determined based on gross properties using uncracked properties. This is not likely to be significant since the beams are very stiff as compared to the columns (about 3 times). The response of the frame to any of the load cases is dependent on the relative stiffness of the beams and columns and the beams are effectively rigid as compared to the columns. Cracking will reduce the moment of inertia of the columns more than the beams so that the difference in stiffness is likely to increase as cracks occur. The reduced stiffness of the beams and columns will reduce the frequency of the frame, but as will be shown below the frame frequency places it in the displacement sensitive portion of the criteria response spectra. Any reduction in frequency will therefore reduce the loading.

Most of the interior partitions are made of 4" unreinforced masonry units placed along column lines. The stiffness of these walls is neglected in the analyses although their mass is included in the models. The walls are likely to cause significant increases in the fundamental frequency of the E-W frames (probably by a factor of 3) until the walls crack as a result of the seismic loading. Relatively low seismic input will cause this cracking so that the frame model (neglecting the masonry wall stiffness) is appropriate at the seismic levels predicted to be the frame capacity. However, it is likely that the masonry walls will be seriously cracked (with the possibility of local failures) when the seismic capacity of the frame is reached. As stated in the introduction however the objective of this study is to predict a seismic fragility of the GLAB based of significant structural damage.

The lower story of the frame is embedded in soil. This soil bears against the foundation wall (10" thick and reinforced with # 5 @ 14" vertically and #5 @ 15" horizontally). This steel is not anchored in the footing, first floor slab or into the columns. The effect of this side soil on providing lateral support for the frame is therefore neglected. The critical portions of the frame (for seismic loading) are found to be above the first floor where the consequences of this assumption are small.

The footings along the outer columns are 7.25' square and those under the center column are 8.67' square. These footings rest on a soil which has a foundation modulus of at least 300 kcf (see Appendix A). If the footing is assumed to be rigid as compared to the soil the resulting rocking stiffness of the outer and inner footings can be shown to be 0.83×10^6 k in and 1.70×10^6 k in respectively. This degree of fixity at the base of the columns can be shown to result in bending moments at the base of the columns equal to about 60 % of those obtained using a fixed base assumption. The actual springs are used in the model. The vertical stiffness of the outer and inner footings can be shown to be 1314 k/in and 1879 k/in respectively. It can be shown that this vertical

foundation flexibility can result in beam bending moments about 20% larger than those which are found assuming the base of the columns to be fixed in the vertical direction. The vertical footing flexibility is not included in the model.

The ASCE 4-86 soil-structure-interaction (SSI) coefficients are used to evaluate the extent to which SSI effects need to be considered in the analysis. The soil is modeled with a shear wave velocity of 560 fps, Poisson's ratio of 0.3, and density of 110 pcf. The horizontal SSI coefficient are computed for the entire footprint under the wing (44.5' by 442'). The resulting horizontal SSI spring and damper are found to be 392,700 k/ft and 31,8900 k sec/ft respectively. The total mass of the wing (DL + LL/4) is found to be 640.8 k sec²/ft. The horizontal SSI frequency and damping are found to be 4 cps and 100 % of critical respectively. Because of the large damping horizontal SSI effects are not significant. Rocking SSI effects are evaluated based on the vertical SSI effects under each of the footings assumed to act independently of the other footings. The vertical SSI spring and damper under the two outer footing are found (from ASCE 4-86) to be 25171 k/ft and 155.8 k sec/ft respectively. These parameters result in a rocking frequency of 4 cps and a damping of 8 % of critical. The rocking SSI effects would be significant except for the fact that the structural frequency is less than 1 cps. The SSI rocking frequency is sufficiently above the structural frequency so that SSI rocking effects would not be expected to be significant. This effect is evaluated by obtaining seismic solutions including and neglecting the rocking SSI effects. The resulting column moments are shown on Table 2.1. It may be seen that SSI effects are negligible.

Table 2.1
SSI EFFECTS ON SEISMIC INDUCED COLUMN MOMENT

Column #	Moments (K in)	
	Neglecting SSI Effects	Including SSI Effects
A7-1	1377	1338
B7-1	2895	2823
C7-1	1338	1313
A7-2	1121	1092
B7-2	2381	2337
C7-2	1077	1068
A7-3	984	961
B7-3	1833	1802
C7-3	955	949
A7-4	805	788
B7-4	1173	1159
C7-4	790	789
A7-5	377	368
B7-5	605	600
C7-5	376	384

2.3 Frame Capacity (LPF Acceptance Criteria)

The capacity of the E-W frame to both the $(1.4 \text{ DL} + 1.7 \text{ LL})$ and $(\text{DL} + \text{LL}/4 + \text{EQ})$ load combinations are reviewed in this section of the report based on criteria associated with a high confidence low probability of failure (LPF) acceptance criteria. These acceptance criteria are those contained in the ACI 318-89 code. The member capacities are first developed followed with a discussion for each of the load cases.

2.3.1 Member Capacities

Interaction charts are developed for each of the frame columns based on the ultimate load provisions contained in ACI 318-89 criteria. A review of the column details indicate that the column splices are for compressive loads (24 bar diameters) and not adequate, in many cases, for

the tensile stresses which will develop as a result of the seismic loads. This deficiency is handle by reducing the yield stress of the reinforcing steel so that the "effective yield stress" equals the actual yield stress times the splice length supplied and divided by the required yield stress. The required splice length is determined from the ACI Code to be the basic development length ($0.04 A_b f_y / f'_c{}^{1/2}$) times a factor of 1.3 since the column splices are Class B according to the Code definitions. The resulting "effective yield stresses" are shown in Table 2.2 for the bar sizes used in the columns. The column interaction charts are developed using the following material properties: concrete compressive strength = 3 ksi, steel compressive yield stress = 50 ksi, steel tensile yield stress = effective yield stress from Table 2.2, and ultimate compressive strain in the concrete = 0.003. The resulting interaction charts are shown in Appendix B. The capacity reduction factors (ϕ) are not included in these charts and must be added when applying these data for the column reviews.

Table 2.2
EFFECTIVE YIELD STRESS IN COLUMN REINFORCEMENT

Bar	Splice Length (in)		Effective Yield Stress (ksi)
	Supplied	Required	
#6	18	20.9	43.0
#7	21	28.5	36.9
#8	24	37.5	32.0
#9	27	47.5	28.5
#10	30.5	60.3	25.3
#11	33.8	74.1	22.8

The moment capacity of the beams are determined again based on the ultimate load provisions of the ACI 318-89. The material properties used is these evaluations are: concrete strength = 3 ksi, steel yield stress = 40 ksi, and ultimate compressive strain in the concrete = 0.003. The moment capacities are evaluated at the center and end of each beam, and at each location for the cases where the moment is positive (tension at bottom of beam) and where the moment is negative. The capacity reduction factors ($\phi = 0.9$) are not included in these evaluations and will be added when the frame evaluations are done. For the case where the steel development length required in ACI 318-89 are not provided (all positive moments at the ends of the beams) the moment capacity is evaluated by reducing the steel yield stress in proportion to the ratio of the provided to the required development

length. The resulting beam capacities are tabulated in Table 2.3. The beams have no top reinforcement at the center span so that the negative moment capacity of the beams is zero at midspan. This will not create a problem since the DL and LL mid span moments are positive and the EQ moments are close to zero at midspan.

Table 2.3
BEAM MOMENT CAPACITIES (Kip Inches) CODE MATERIAL PROPERTIES

Beam No.	Left End		Center		Right End	
	M _{max}	M _{min}	M _{max}	M _{min}	M _{max}	M _{min}
B-104	708.	-3080.	3190.	0.	720.	-6190.
B-103	720.	-6190	4140.	0.	957.	-3080.
B-4	583.	-2980.	3110.	0.	595.	-6000.
B-3	719.	-6000.	4050.	0.	711.	-3560.
B-504	512.	-2180.	2280.	0.	628.	-4500.
B-503	628.	-4500.	2590.	0.	515.	-2480.

2.3.2 Dead Load + Live Load (DL + LL) Combination

Solutions are obtained for the two loads (DL and LL) using the ANSYS code and the model with the rigid beam / column joints. A preliminary analysis indicated that the top story exterior columns are overstressed for this load case (Capacity / Demand ratios equal to about 0.6). As a result the model is changed assuming that hinges develop between the roof beam and the outer columns (the rotational degrees of freedom are uncoupled between nodes 43 and 50 and between nodes 49 and 51).

The beam evaluation for this load case is shown on Table 2.4. The first column of the table indicates the node number as shown on Fig. 2.3. The second column indicates the beam number (see Fig. 2.2) with the letter L,C, or R appended to the beam number indicating left end, center, or right end location on the beam respectively. The moments at the ends and centers of each of the beams are tabulated in the third and fourth columns of Table 2.4. Note that all of the moments in this table are in kip inch units and carry the beam sign convention (positive moments give tensile stresses on the bottom of beam). The load factors of 1.4 and 1.7 are applied to the DL and LL respectively and the two moments combined to give the moment demand (D) tabulated in the fifth column of the table. The appropriate allowable moments are determined from Table 2.3, multiplied

by ($\phi=0.9$), and tabulated in the sixth column of the table. Finally, the capacity to demand ratios are tabulated in the seventh column of the table. As may be seen all of the beams are adequate for this load combination with a minimum C/D ratio equal to 1.19.

Shear loads are evaluated at each end of the beams and the results are tabulated in Table 2.5. The shear load demand is calculated at a distance "d" from the face of the beam support and compared with an ACI allowable shear of $(2 \phi f'_c)^{1/2} b d$ where $\phi = 0.85$ is the capacity reduction factor, b is the beam width, and d is the beam depth. As may be seen the shear demand loads exceed the capacity at all interior beam supports (shaded entries in Table 2.5) except for the roof beams. There is some bent up steel in the beams but no stirrups so that sufficient shear reinforcement is not provided to accommodate the excess shear. It should be noted that the ACI 318-89 also requires minimum shear reinforcement unless the C/D ratio exceeds 2. The beams do not have shear reinforcement (other than a few bent up bars) so the ACI requirements are not satisfied.

The results for the review of the columns are contained on Table 2.6. The designation used for the columns is E-W column line (A, B, or C), N-S column line 7 - story (1 through 5). The axial load in the column for the two load cases is tabulated in the second and third columns of the table, and the maximum bending moment in the columns for the two load cases are tabulated in the fourth and fifth columns. A review of the column data indicate that the columns are bent in double curvature with an inflection point (zero moment) located close to midheight of each column. The column capacity can be determined from the interaction charts of Appendix B except that a capacity reduction factor (ϕ) must be applied to these data. The ACI code requires that a factor equal to 0.7 be used for $P > P_{bal}$, a factor of 0.9 can be used for $P = 0$, and linear interpolation can be used for $0 < P < P_{bal}$. The balanced loads (P_b) have been computed and are tabulated at the top of the interaction charts contained in Appendix B. The computation of (ϕ) is carried out in columns 6, 7, and 8 of the table. The balanced load (P_b) is copied into column 6 from the data in Appendix B, the load factors are applied to the DL and LL ($P_u = 1.4 P_{DL} + 1.7 P_{LL}$) and the results copied into column 7. Finally, the factor is listed in column 8 and found from :

$$\phi = 0.9 - 0.2 P_u / P_b ; P_u < P_b$$

$$\phi = 0.7 ; P_u > P_b$$

The load factors are applied to the DL and LL moments and the demand moment taken as the factored moment divided by (ϕ). This demand moment is tabulated in column 9. The demand axial

load is equal to (P_u / ϕ) and is tabulated in column 10. The moment capacity is then evaluated from the interaction curves of Appendix B by entering the curves with the demand axial load and reading off the moment capacity. This is tabulated in column 11. The capacity demand ratio is listed in column 12. As may be seen the capacity / demand ratios for the center columns (line B) are large but that the capacity / demand ratios for the outside columns (line A and C) are relatively small.

The C/D ratios are less than unity (indicating unacceptable performance) for the fourth and fifth story outer columns on line A and for the fourth story outer column on line C. The moments in the columns are large and cause this unacceptable performance. The result of yielding of these outer columns would be to increase the moments of the center columns and to increase the positive moments of the fourth and fifth floor beams. It should be noted, however, that the fourth and fifth story center columns (line B) have rather large C/D ratios as do the upper story beams. Loads carried by the outer columns could, therefore, be transferred to the center columns if the outer columns should begin to fail.

Slenderness effects are investigated for the columns and the moment magnifier (of the ACI 318-89) are found to be equal to unity. This occurs because the columns are bent in double curvature for this load case.

In summary, this E-W frame is adequate for the applied static loading except for the shear stresses in the beams at the inner column supports. The upper story outside columns do not satisfy the acceptance criteria but alternative load paths are available to carry the load.

2.3.3 Dead + Live + Seismic Load (DL + LL/4 + EQ) Capacities

The same model is used to solve the DL + LL/4 + EQ load case. Load factors of unity are used and, as discussed above, it is assumed that only 25 % of the LL is acting at the time of the earthquake. The ANSYS solutions for the DL and LL cases from the previous section are used for this evaluation. The EQ loading is developed using the response spectrum methodology of ANSYS. Solutions are obtained for the EQ defined with the spectra of Fig. 2.4 anchored to 0.1 G. This solution is scaled in proportion to the ZPA to obtain solutions for earthquakes at other levels.

The response spectrum analysis indicates that the fundamental frequency of the frame is about 0.7 cps and that the peak displacement at the roof is 2.1" for the 0.1 G earthquake.

The evaluation of the beams for this load case is presented in Table 2.7. The same notation as discussed above for Table 2.4 is used in this table. The midspan locations for the beams are not

included because seismic moments are close to zero at midspan of the beams and the static load case will therefore be more critical than the seismic load case. Two beam evaluations are performed at the ends of the beams where the DL and LL moments are negative. The first (shown in columns 6, and 7) examines the situation where the seismic moment is positive and therefore limited to the sum of the DL and LL negative moments plus the positive moment capacity of the beam. The positive moment capacity of the beam (from Table 2.3) is then used to determine the maximum ZPA that can be sustained. The second evaluation (columns 8 and 9 of Table 2.7) occurs at the end of the beam where the seismic moment is negative and is additive to the static moments. In this case the allowable earthquake is determined so that the total of the seismic and static moments are less than the allowable negative moment capacity of the beam.

A comparison of the allowable earthquake for the two cases, as shown on columns 7 and 9, indicates that the first case limited by the positive moment capacity of the beam controls. This moment capacity is of course governed by the short embedment lengths of the bottom reinforcement in the beams into the columns. The allowable earthquake can be seen to be controlled by the beams at the lower floors and to be equal to about 0.04 G.

The beams are also evaluated at the points where the top bars are cutoff (these locations are shown on Fig. 2.2). The development length for the #8 top reinforcement is 30" so that the full negative moment capacity of the beam is developed 30" closer to the support than the cutoff point shown in Fig. 2.2. If it is assumed however that the moment capacity varies from zero at the cutoff point to the full capacity at a location 30" closer to the support, it can be shown that the critical location is the point where the moment capacity is zero. At this location the seismic plus gravity moments are limited to be positive. This analysis is carried out in Table 2.8. The critical location is found to be at the exterior cutoff points and for the lower level beams and to result in a seismic capacity of 0.06 G. This is not as critical as the seismic capacity governed by the beam behavior at the column supports.

The evaluation of the columns for this load combination is presented in Table 2.9. The process used to conduct the evaluation is the same as that discussed for the static load case (Table 2.6). The allowable earthquake is determined so that the seismic moments in the columns bring the total moment to the capacity. It is assumed that the seismic axial loads do not change as the magnitude of the earthquake is varied. This is not a serious limitation since the seismic induced axial loads account for less than 10 % of the total axial load in the column. The fourth floor columns can be seen to limit the seismic capability of the frame to 0.06 G. Note that the top story columns are not critical for the seismic load case

In summary the LPF seismic capacity of the frame is 0.04 G and is limited by the first floor beams. The fourth story columns limit the seismic capacity to 0.06 G. The interstory drifts for the 0.04 G earthquake are found to be: 0.45", 0.14", 0.11", 0.08", and 0.07" respectively for stories one through five. These drifts represent 0.29 %, 0.10 %, 0.08 %, 0.06 %, and 0.05 % of the story heights. The masonry walls would be expected to be cracked with this level of response but one would not expect major collapse of the walls.

2.4 Frame Capacity (HPF Acceptance Criteria)

The previous seismic analysis is based on code type acceptance criteria and leads to a seismic capacity estimation with a high confidence that it will not cause collapse of the structure. The objective of the work reported in this section is to develop an estimate of seismic load that one has a high confidence that collapse of the structure will occur. These two data points will provide the user with a basis for estimating the range of earthquakes that may cause serious damage to the structure.

Three factors are considered in arriving at this high probability of failure estimate: material properties, redistribution of load within the structure, and ductility effects. Each of these are discussed. The capacity reduction factors (ϕ) are set equal to unity for all of the results presented in this section.

2.4.1 Material Properties

The design material properties used in the analyses of the previous section are "guaranteed" minimum values so that average values are usually considerably higher. Since the objective of this section is to obtain upper bound seismic capacities, the goal is to use eighty fifth percentile properties. Such data is not directly available so that estimates are required to be made based on available generic data. It is important to recognize that the material properties used in these analyses are based on general trends and are not derived from project specific test data.

The design concrete strength is 3000 psi, defined as the "guaranteed" compressive strength at 28 days. Since concrete gains strength with age, it is estimated that long term strength of Type I cement concrete is 25% higher (see MacGregor, "Reinforced Concrete") than the 28 day strength. The current design strength is therefore estimated to be 3750 psi. The current ACI code requires that the average concrete strength of a mix be equal to the design strength plus 1.34 times the standard deviation of the concrete strength or equal to the design strength plus 1000 psi. These requirements imply a standard deviation of $(1000/1.34 = 746 \text{ psi})$ with a mean strength of 4750 psi

for the Administration building concrete. The eighty fifth percentile strength would be about 5500 psi. This concrete strength is used for this analysis.

MacGregor also contains data showing the distribution of the failure concrete compressive strain. This would indicate that a failure strain equal to 0.005 is likely to be an eighty fifth percentile value as compared with the value of 0.003 used in the previous analyses.

The design reinforcing steel yield stress is 40 ksi and 50 ksi for the steel in the beams and columns respectively. Eighty fifth percentile yield stresses are assumed to be 25 % higher resulting in 50 ksi for the beam steel and 62.5 ksi for the column steel.

The member capacities are computed using these properties. The beam capacities are shown on Table 2.10. The steel yield stress has the most effect of the three material property changes on these results.

Table 2.10
BEAM MOMENT CAPACITIES (Kip Inches)
MAXIMUM MATERIAL PROPERTIES

Beam No.	Left End		Center		Right End	
	M _{max}	M _{min}	M _{max}	M _{min}	M _{max}	M _{min}
B-104	1000.	-3920.	4030.	0.	1030.	-7960.
B-103	1030.	-7060.	5270.	0.	1340.	-3930.
B-4	833.	-3800.	3940.	0.	863.	-7700.
B-3	1030.	-7700.	5150.	0.	1010.	-4530.
B-504	734.	-2790.	2880.	0.	906.	-5790.
B-503	906.	-5790.	3280.	0.	741.	-3170.

Interaction diagrams are calculated for the higher material properties and the results are contained in Appendix C. Since this analysis has the objective of determining upper bound seismic capacities, the reduced steel yield stresses accounting for short splice lengths are not included in the evaluation of the interaction charts.

The seismic evaluation is repeated for these higher member capacities and setting the strength

reduction factors equal to unity. The evaluation is shown on Tables 2.11 for the beams and 2.12 for the columns. The data contained on the two tables are identical to that contained on Tables 2.7 and 2.9 except for the revised capacities. Once again the upper story outer columns and the lower story beams are critical. The seismic capacity of the is increased from the 0.04 G to 0.06 G. The increase in material properties results in only a modest increase in the seismic capacity.

2.4.2 Redistribution of Loads

The member forces used in the stress evaluations shown on Tables 2.11 and 2.12 are those found from the elastic frame solutions using the ANSYS code. When a member fails the stress evaluation it only means that no additional load can be applied to that member. It is possible for the additional load to be carried through alternative load paths so that the "collapse" capacity of the frame may be larger than that found in the tables. This redistribution of forces is considered in this section. The frame must still carry the total seismic shears and moments down to the foundation, but one now looks for the maximum shears and moments that can be carried through each of the floors with little concern for the relative distribution of these forces between the members.

The total seismic shears and moments, obtained from the ANSYS results for the 0.1 G earthquake, are shown on Fig. 2.5. These forces, shear, and moments are compatible with the ANSYS results. The following assumptions are made in evaluating the extent to which the seismic loads can be increased before the total loads acting at any floor exceed the total capacity:

- The static DL and LL problem is symmetric while the seismic problem is asymmetric. Therefore if seismic and static loads add on one side of the frame they will cancel each other on the opposite side. Static shears and moments are therefore not added and the evaluation uses only the seismic shear and moments. The axial loads are additive however.
- The total shear force at a story must be carried by the three columns. The allowable shear stress is $(2 f_c^{1/2} = 0.148 \text{ ksi})$ so that the story seismic shear is limited to $0.148 * \text{total column area}$.
- The column moment capacity is determined from the Appendix C interaction charts for the axial loads acting on the columns. The total seismic moment resistance is then the sum of the three column capacities.
- Plastic analysis methods are used to evaluate the moment capacity of the frame.

The evaluation of shear forces in the columns is given on Table 2.13. As may be seen the

seismic capacity is limited to 0.28 G based on this shear stress evaluation and the critical columns are those at the first floor.

A plastic analysis is performed to evaluate the potential for moment collapse of the frame. The model used for the analysis is shown on Fig. 2.7. Possible collapse mechanisms are considered within each of the floors. This analysis is modified for the upper story since a hinge is assumed to exist at the connection between the roof beams and the outer columns as a result of the gravity loading. Note that the sum of the forces term appearing in the equations is equal to the seismic shear at the story and may be obtained for any seismic input by scaling the shears on Fig. 2.5. The results of the analysis are shown on Table 2.14, and indicate that the seismic capacity is 0.23 G and limited by the fifth story collapse mechanism.

The consideration of redistribution of seismic forces within the frame together with the increased material properties increases the frame capacity to 0.23 G.

2.4.3 Ductility Effects

The above analysis assumes that the seismic loads are statically applied and one would expect that somewhat higher loads could be applied if they were of short duration. Newmark has shown that the yield capacity of a ductile structure, such as this frame, can be increased by a factor of $(2\mu - 1)^{1/2}$ where μ is the ratio of failure displacement (or strain) to yield displacement. It is recognized that the details of this structure do not satisfy the requirements for ductile behavior. This analysis is performed however to determine the seismic load that is associated with a high probability of failure so that high estimates of the effect of ductility should be used.

The yield capacity of the frame using the increased material properties are given on Tables 2.11 and 2.12. Since the fourth story columns control the frame capacity, the ductility of these columns is investigated.

A moment curvature relationship is generated using a stress strain law for the concrete given by:

$$f_c = a_1 \epsilon + a_2 \epsilon^2 + a_3 \epsilon^3$$

where, $a_1 = E_c = 57000 [f'_c]^{1/2}$

$$a_2 = [\beta f'_c - E_c \epsilon^* - f'_c (\epsilon^*/\epsilon_m)^2 + E_c \epsilon^{*2}/\epsilon_m] / [\epsilon^{*3} - \epsilon^* \epsilon_m]$$

$$a_3 = f'_c / \epsilon_m^2 - E_c / \epsilon_m - \epsilon_m a_2$$

ϵ_m = strain when stress is maximum, taken as 0.002

β = reduced stress after maximum taken as 0.85

ϵ^* = strain at which stress is $0.85 f'_c$, taken as 0.003

The steel is modeled as elastic perfectly plastic and the increased material properties are used for the analysis. The equilibrium equations are solved for the fifth story columns for the axial loads shown on Table 2.12. The solution is carried out for peak concrete strains varying from 0 to 0.005. The results are shown on Fig. 2.7 for columns A7-4 and B7-4. If the peak concrete strain is allowed to reach 0.005 then the ductility factors are 3.6 and 5.4 for columns B7-4 and A7-4 respectively. If the peak concrete strain is restricted to 0.003 the ductility factors are 2.3 and 3.6 for columns B7-4 and A7-4 respectively. The ductility factors for column B7-4 are used since that column would lose all of its capacity once this strain occurs. A ductility factor of 3.6 is used to obtain:

$$[2 * 3.6 - 1]^{1/2} = 2.5$$

This then results in a frame seismic capacity of $2.5 * 0.06 = 0.15$ G.

2.4.4 Combined Effects

The elastic analyses including increased material properties and ductility effects result in an estimate of frame capacity of 0.15 G while the plastic analysis considering redistribution of forces and the increased material properties result in the frame capacity of 0.23 G. Since the analysis of frame capacity based on redistribution of forces requires ductility for the plastic hinges to develop, the lower limit of 0.15 G from the ductility analysis is taken as the HPF frame capacity. The mean seismic capacity is therefore 0.1 G $([0.04 + 0.15]/2)$.

2.5 Structural Modifications to Increase Seismic Frame Capacity

This section considers two preliminary designs to enhance the seismic capacity of the frame.

The first is based on the addition of a steel bracing system designed so that the LPF seismic capacity of the frame is at least 0.15 G fitted to a NUREG 0098 spectra. The required modification is designed so that the entire seismic shear loads are carried by the added members and so that the fundamental frequency of the frame is at least 1.5 cps. The frame would be sufficiently flexible, however, so that damage to the masonry walls would be expected for the 0.15 G earthquake. The second design is based on the use of concrete shear walls proportioned so that the frame capacity is 0.15 G and additionally the frame frequency is increased to 4 cps. This fix should protect the masonry walls from damage.

2.5.1 Steel Bracing

It is proposed that cross steel diagonal braces be included between column lines A and B (see Fig. 2.8) at every floor level. One of the braces will be subjected to tensile loads and the other to compressive loads. The brace acting in compression is neglected in the following analyses so that slenderness effects do not have to be considered. It should be noted that the actual modification design can be done for any desired bracing configuration following the methodology discussed here. This analysis is also done for a single bay (e.g., column line 7). It can easily be modified so that one set of bracing will carry the loading of (m) frames. The bracing areas found here need to be multiplied by (m) to obtain a design with bracing at every (m) frames.

The seismic shear force applied at each floor is fairly uniform so that the lateral shear load increases linearly with height in the frame. The bracing stiffness is selected to match this variation so that the stiffness is 5k, 4k, 3k, 2k, and k from the first to top floors respectively. The stiffness can be related to the bracing member areas as:

$$k = A_5 E / 31.47$$

where k = fifth story stiffness (k/ft)

A_5 = bracing area in the fifth floor (sq. in.)

E = Young's modulus = 29,000 ksi

The bracing area in floor (n) will be (6-n) A_5 to achieve the assumed stiffness.

The stiffness matrix for the bracing system is then:

$$K = k \begin{bmatrix} 9 & -4 & & & \\ -4 & 7 & -3 & & \\ & -3 & 5 & -2 & \\ & & -2 & 3 & -1 \\ & & & -1 & 1 \end{bmatrix}$$

The mass matrix for the frame is:

$$M = 5.15 \begin{bmatrix} 1 & & & & \\ & 1 & & & \\ & & 1 & & \\ & & & 1 & \\ & & & & 1 & 0.75 \end{bmatrix}$$

The expected deformation of the frame relative to the base is linear so that the fundamental mode shape (X) should be:

$$X' = [0.2, 0.4, 0.6, 0.8, 1.0]$$

The bracing system is designed so that the fundamental frequency is at least 1.5 cps. The frequency of 1.5 cps corresponds to peak roof displacements and accelerations of 1.35" and 0.3 G respectively for the 0.15 G ZPA earthquake (see Fig. 2.4) The frequency of the bracing system (f) can be estimated from:

$$f = [X' K X / X' M X]^{1/2} / 2\pi$$

If these operations are performed the result is that:

$$f = [0.6 k / 1.95 * 5.15]^{1/2} / 2\pi$$

The stiffness factor required to obtain a frequency of 1.5 cps is:

$$k = 4 \pi^2 * 1.5^2 * 1.95 * 5.15 / 0.6 = 1486.7 \text{ k/ft}$$

This stiffness corresponds to a fifth story bracing area of 1.61 sq inches.

The stresses in this bracing system are evaluated next. The floor accelerations vary linearly from 0.3 G at the roof to 0.15 G at the foundation. The stresses in the braces are found by: computing the forces acting on each of the floors by multiplying the accelerations by the floor mass, summing the inertial forces to obtain the story shears, resolving the story shear to its diagonal component to obtain brace forces, and finally dividing this force by the brace area. The resulting stresses in the braces are: 26.5 ksi, 29.1 ksi, 28.8 ksi, 27.8 ksi, and 26.4 ksi respectively from the top story the first story. These stresses are adequate if A36 steel is used.

It is proposed that a reasonable option for bracing the frame would be to add the bracing system shown on Fig. 2.8 to every third frame. Double angles are used. If an existing wall makes the double angle arrangement difficult, one angle could be placed at each side of the wall. These angles must have connections to the columns that are sufficient to develop the full capacity of the brace.

It can be shown that this bracing system would result in seismic induced tensile loads in the outer columns equal to about 330 k. The inner columns will not control since they are subjected to larger compressive gravity loads than are the outer columns. If the dead load column compressive load is included, the net tensile load in the column is 150 k. This results in a tensile stress acting on the column area of 0.5 ksi. If the concrete is assumed to be cracked under this tensile stress, the column main reinforcement would be stressed to 25 ksi. The effective yield stress (accounting for the short splice lengths) is 25.3 ksi for the # 10 bars (see Table 2.2) and higher for the smaller bars. Since the largest bar size for the outer columns is #10, the columns can carry this additional tensile load.

2.5.2 Concrete Walls

Concrete shear walls (or buttresses on the outside) can be used to accomplish the same as the steel bracing system. The concrete walls are designed to increase the frame frequency to at least 4 cps to protect the masonry walls. The peak acceleration is 0.4 G for the 0.15 G ZPA. It is found that the wall stress is more critical than stiffness, however, so that the walls are designed to carry the load and the frequency of the system is then checked.

The seismic acceleration varies from 0.4 G at the roof to 0.15 G at the foundation for frame frequencies ranging from 2 to 9 cps. The resultant story shears are: 49.7 k, 109.4 k, 162.4 k, 208.8 k, and 248.6 k respectively for the top story down to the first story. These shear loads arise

from the mass of a single frame. The shear capacity of a shear wall of thickness (t) and width (l_w) is taken as:

$$V_{cap} = 3.3 * 0.85 * [f'_c]^{1/2} * t * (0.8 l_w)$$

The resultant wall thickness requirements for ($l_w = 21.36'$) are: 1.56", 3.45", 5.12", 6.58", and 7.84" respectively for the top story down to the first story. If the walls are placed every third frame as done with the steel system, these thickness requirements must be tripled. Of course, shear reinforcement could be used to reduce the wall thickness somewhat.

The resultant frequency of the shear wall system can be found using the same methodology as for the steel frame. The top floor wall stiffness is:

$$k = A_w G / 1.2 h = 21.36 * (1.56 / 12) * 1500 * 144 / 1.2 * 11.65 = 42,900 \text{ k / ft}$$

The frequency is therefore:

$$f = [0.6 k / 1.95 * 5.15]^{1/2} / 2 \pi = 8.1 \text{ cps}$$

Table 2.4

BEAM EVALUATION FOR (1.4 DL + 1.7 LL)

Node	Beam #	Moments (K-In)		1.4 DL + 1.7 LL		
		DL	LL	D	C	C/D
1	2	3	4	5	6	7
8	104L	-518	-388	-1384.8	-2772	2.00
9	104C	755	561	2010.7	2871	1.43
11	104R	-1589	-1172	-4217	-5571	1.32
12	103L	-1762	-1301	-4678.5	-5571	1.19
13	103C	979	727	2606.5	3726	1.43
15	103R	-696	-519	-1856.7	-2772	1.49
17	4L	-661	-493	-1763.5	-2682	1.52
18	4C	749	556	1993.8	2799	1.40
20	4R	-1457	-1076	-3869	-5400	1.40
21	3L	-1650	-1219	-4382.3	-5400	1.23
22	3C	957	710	2546.8	3645	1.43
24	3R	-852	-634	-2270.6	-3204	1.41
26	4L	-720	-530	-1909	-2682	1.40
27	4C	752	557	1999.7	2799	1.40
29	4R	-1392	-1037	-3711.7	-5400	1.45
30	3L	-1569	-1169	-4183.9	-5400	1.29
31	3C	976	722	2593.8	3645	1.41
33	3R	-897	-661	-2379.5	-3204	1.35
35	4L	-600	-445	-1596.5	-2682	1.68
36	4C	796	582	2103.8	2799	1.33
38	4R	-1425	-1073	-3819.1	-5400	1.41
39	3L	-1554	-1176	-4174.8	-5400	1.29
40	3C	1030	762	2737.4	3645	1.33
42	3R	-803	-574	-2100	-3204	1.53
43	504L	0	0	0	-1962	
44	504C	791	190	1430.4	2052	1.43
46	504R	-1623	-279	-2746.5	-4050	1.47
47	503L	-1701	-307	-2903.3	-4050	1.39
48	503C	1101	249	1964.7	2331	1.19
49	503R	0	0	0	-2232	

Table 2.5

BEAM SHEAR EVALUATION FOR (DL + LL)
CODE MATERIAL PROPERTIES

Nodes	DL Shear		LL Shear		Dem'd		Capac
	Ext	Int	Ext	Int	Ext	Int	
1	2	3	4	5	6	7	8
8,11	24.3	32.7	18.0	24.2	60.3	82.6	76.0
17,20	25.4	31.7	18.8	23.4	63.0	79.6	76.0
26,29	25.9	31.2	19.1	23.1	64.1	78.3	76.0
35,38	25.3	31.8	18.7	23.6	62.7	80.1	76.0
50,46	8.9	31.7	4.1	6.3	18.3	53.9	67.0
12,15	27.7	35.3	20.6	26.1	69.3	89.3	76.0
21,24	28.7	34.4	21.3	25.4	71.8	86.7	76.0
30,33	29.1	33.9	21.6	25.2	72.8	85.6	76.0
39,42	28.8	34.2	21.2	25.5	71.7	86.6	76.0
47,51	21.9	34	4.6	6.8	36.4	57.1	67.0

Table 2.6

COLUMN EVALUATION FOR (1.4 DL + 1.7 LL)

Col #	Axial Load (K)		Moment (K-In)		Computation of Phi			1.4 DL + 1.7 LL			
	DL	LL	DL	LL	Bal Load	Ult Load	ϕ	D	C	C/D	
								M/Phi	P/Phi	Moment	
1	2	3	4	5	6	7	8	9	10	11	12
A7-1	156	92.4	187	140	418	375.5	0.72	694	521.3	1870	2.70
B7-1	349	228	65	49	812	876.2	0.70	249	1251.7	2750	11.04
C7-1	172	103	252	188	459	415.9	0.72	935	578.6	1980	2.12
A7-2	123	71	332	248	360	292.9	0.74	1202	397.3	1730	1.44
B7-2	276	173	108	80	631	680.5	0.70	410	972.1	2430	5.92
C7-2	136	79.1	444	331	431	324.9	0.75	1581	433.6	2079	1.32
A7-3	89.1	49.7	340	252	316	209.2	0.77	1178	272.6	1680	1.43
B7-3	206	120	94	69	539	492.4	0.72	347	686.5	2310	6.66
C7-3	98.9	55.3	432	320	337	232.5	0.76	1508	305.1	1810	1.20
A7-4	55	28	414	298	294	124.6	0.82	1332	152.8	1300	0.98
B7-4	138	68.3	88	67	381	309.3	0.74	321	419.3	1870	5.82
C7-4	61.5	31.1	498	365	316	139.0	0.81	1623	171.1	1435	0.88
A7-5	21.5	6.7	296	147	294	41.5	0.87	762	47.6	777	1.02
B7-5	68.6	16.1	77	35	381	123.4	0.84	200	147.8	1420	7.09
C7-5	24.5	7.3	305	209	294	46.7	0.87	901	53.8	812	0.90

Table 2.7

BEAM EVALUATION FOR (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES

Node	Beam #	Moments (K-In)			DL+LL+EQ (+Mom)		DL+LL+EQ (-Mom)		Mimimum
		DL	LL	EQ (.1 G)	C	ZPA	C	ZPA	ZPA
1	2	3	4	5	6	7	8	9	10
8	104L	-518	-388	2953	637	0.04	-2772	0.07	0.04
11	104R	-1589	-1172	3188	648	0.08	-5571	0.12	0.08
12	103L	-1762	-1301	2988	648	0.09	-5571	0.12	0.09
15	103R	-696	-519	2850	861	0.06	-2772	0.07	0.06
17	4L	-661	-493	1643	525	0.08	-2682	0.12	0.08
20	4R	-1457	-1076	1634	536	0.14	-5400	0.22	0.14
21	3L	-1650	-1219	1547	647	0.17	-5400	0.22	0.17
24	3R	-852	-634	1579	640	0.10	-3204	0.14	0.10
26	4L	-720	-530	1260	525	0.11	-2682	0.15	0.11
29	4R	-1392	-1037	1097	536	0.20	-5400	0.34	0.20
30	3L	-1569	-1169	1054	647	0.24	-5400	0.34	0.24
33	3R	-897	-661	1227	640	0.14	-3204	0.17	0.14
35	4L	-600	-445	847	525	0.15	-2682	0.23	0.15
38	4R	-1425	-1073	741	536	0.30	-5400	0.50	0.30
39	3L	-1554	-1176	732	647	0.34	-5400	0.49	0.34
42	3R	-803	-574	834	640	0.19	-3204	0.27	0.19
43	504L	0	0	0	461		-1962		
46	504R	-1623	-279	320	565	0.71	-4050	0.74	0.71
47	503L	-1701	-307	303	565	0.77	-4050	0.75	0.77
49	503R	0	0	0	464		-2232		

Table 2.8

BEAM EVALUATION AT TOP BAR CUTOFF POINTS FOR (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES

Nodes	DL + LL/4 Mom (K-In) @					0.1 G Seismic Mom @				ZPA @	
	Ext	Center	Int	Ext C	Int C	Ext	Int	Ext C	Int C	Ext C	Int C
1	2	3	4	5	6	7	8	9	10	11	12
8,11	-615	895	-1882	771	591	2953	3188	-1204	-906	0.06	0.07
17,20	-826	1161	-2087	948	826	2850	2988	-1187	-819	0.08	0.10
26,29	-784	888	-1726	694	626	1643	1634	-709	-416	0.10	0.15
35,38	-1010	1135	-1955	854	841	1579	1547	-688	-385	0.12	0.22
50,46	-853	891	-1651	666	647	1260	1087	-591	-215	0.11	0.30
12,15	-853	1157	-1861	879	712	1227	1054	-597	-292	0.15	0.24
21,24	-711	942	-1693	732	847	847	741	-408	-211	0.18	0.40
30,33	-947	1221	-1848	898	782	834	732	-401	-209	0.22	0.37
39,42	0	839	-1693	880	372	0	320	88	-213		0.17
47,51	0	1163	-1778	1150	641	0	303	84	-202		0.32

Table 2.9

COLUMN EVALUATION FOR (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES

Col #	Axial Load (K)			Moment (K-In)			Computation of Phi			DL+LL+EQ		
	DL	LL	EQ	DL	LL	EQ	Bal	Ult	ϕ	C	ZPA	
			0.1 G			0.1 G	Load	Load		P/Phi	Moment	
1	2	3	4	5	6	7	8	9	10	11	12	13
A7-1	156	92.4	53	187	140	2126	418	232.1	0.79	294.2	2130	0.07
B7-1	349	228	7	65	49	4659	812	413.0	0.80	517.4	4820	0.08
C7-1	172	103	46	252	188	2083	459	243.8	0.79	307.1	2310	0.07
A7-2	123	71	29	332	248	938	360	169.8	0.81	210.7	1730	0.11
B7-2	276	173	4	108	80	1940	631	323.3	0.80	405.3	3980	0.16
C7-2	136	79.1	26	444	331	896	431	181.8	0.82	222.9	2000	0.12
A7-3	89.1	49.7	17	340	252	732	316	118.5	0.82	143.7	1330	0.09
B7-3	206	120	2	94	69	1372	539	238.0	0.81	293.2	2700	0.15
C7-3	98.9	55.3	15	432	320	712	337	127.7	0.82	155.0	1420	0.09
A7-4	55	28	7	414	298	585	294	69.0	0.85	80.9	957	0.06
B7-4	138	68.3	1	88	67	810	381	156.1	0.82	190.8	1620	0.15
C7-4	61.5	31.1	7	498	365	576	316	76.3	0.85	89.6	1070	0.06
A7-5	21.5	6.7	1	296	147	271	294	24.2	0.88	27.4	665	0.09
B7-5	68.6	16.1	0	77	35	674	381	72.6	0.86	84.3	1110	0.13
C7-5	24.5	7.3	1	305	209	268	294	27.3	0.88	31.0	685	0.09

Table 2.11

BEAM EVALUATION FOR (DL + LL/4 + EQ)
MAX MATERIAL PROPERTIES

Node	Beam #	Moments (K-In)			DL+LL+EQ (+Mom)		DL+LL+EQ (-Mom)		Minimum
		DL	LL	EQ (.1 G)	C	ZPA	C	ZPA	
1	2	3	4	5	6	7	8	9	10
8	104L	-518	-388	2953	1000	0.05	-3920	0.11	0.05
11	104R	-1589	-1172	3188	1030	0.09	-7960	0.19	0.09
12	103L	-1762	-1301	2988	1030	0.10	-7960	0.20	0.10
15	103R	-696	-519	2850	1340	0.08	-3930	0.11	0.08
17	4L	-661	-493	1643	833	0.10	-3800	0.18	0.10
20	4R	-1457	-1076	1634	863	0.16	-7700	0.37	0.16
21	3L	-1650	-1219	1547	1030	0.19	-7700	0.37	0.19
24	3R	-852	-634	1579	1010	0.13	-4530	0.22	0.13
26	4L	-720	-530	1260	833	0.13	-3800	0.23	0.13
29	4R	-1392	-1037	1097	863	0.23	-7700	0.55	0.23
30	3L	-1569	-1169	1054	1030	0.27	-7700	0.55	0.27
33	3R	-897	-661	1227	1010	0.17	-4530	0.28	0.17
35	4L	-600	-445	847	833	0.18	-3800	0.36	0.18
38	4R	-1425	-1073	741	863	0.34	-7700	0.81	0.34
39	3L	-1554	-1176	732	1030	0.39	-7700	0.80	0.39
42	3R	-803	-574	834	1010	0.23	-4530	0.43	0.23
43	504L	0	0	0	734		-2790		
46	504R	-1623	-279	320	906	0.81	-5790	1.28	0.81
47	503L	-1701	-307	303	906	0.89	-5790	1.32	0.89
49	503R	0	0	0	741		-3170		

Table 2.12

COLUMN EVALUATION FOR (DL + LL/4 + EQ)
MAX MATERIAL PROPERTIES

Col #	Axial Load (K)			Moment (K-In)			Computation of Phi			DL+LL+EQ						
	DL	LL	EQ	DL	LL	EQ	Bal	Ult	φ	C		ZPA				
										0.1 G	0.1 G		Load	Load	P	Moment
1	2	3	4	5	6	7	8	9	10	11	12	13				
A7-1	156	92.4	53	187	140	2126	295	232.1	1.00	232.1	1760	0.07				
B7-1	349	228	7	65	49	4659	512	413.0	1.00	413.0	5251	0.11				
C7-1	172	103	46	252	188	2083	294	243.8	1.00	243.8	2080	0.09				
A7-2	123	71	29	332	248	938	273	169.8	1.00	169.8	1640	0.13				
B7-2	276	173	4	108	80	1940	412	323.3	1.00	323.3	3260	0.16				
C7-2	136	79.1	26	444	331	896	272	181.8	1.00	181.8	1895	0.15				
A7-3	89.1	49.7	17	340	252	732	272	118.5	1.00	118.5	1150	0.10				
B7-3	206	120	2	94	69	1372	362	238.0	1.00	238.0	2510	0.17				
C7-3	98.9	55.3	15	432	320	712	268	127.7	1.00	127.7	1350	0.12				
A7-4	55	28	7	414	298	585	273	69.0	1.00	69.0	851	0.06				
B7-4	138	68.3	1	88	67	810	306	156.1	1.00	156.1	1580	0.18				
C7-4	61.5	31.1	7	498	365	576	272	76.3	1.00	76.3	939	0.06				
A7-5	21.5	6.7	1	296	147	271	273	24.2	1.00	24.2	645	0.12				
B7-5	68.6	16.1	0	77	35	674	306	72.6	1.00	72.6	1165	0.16				
C7-5	24.5	7.3	1	305	209	268	273	27.3	1.00	27.3	544	0.07				

Table 2.13

ALLOWABLE EARTHQUAKE BASED ON SHEAR CAPACITY

Story	Column Areas			Total Area	Shear C	Shr Stl Capac	Shear 0.1 G	ZPA
	Col A	Col B	Col C					
5	224	256	224	704	104.2	24.4	13.3	0.97
4	224	256	224	704	104.2	24.4	27.3	0.47
3	224	288	224	736	108.9	28.7	39.1	0.35
2	224	320	224	768	113.7	33.0	49.4	0.30
1	238	400	238	876	129.6	33.7	58.0	0.28

Table 2.14

ALLOWABLE EARTHQUAKE BASED ON MOM COLLAPSE OF COLUMNS

Floor	Seismic Shear 0.1 G	Column Capacity						Total Mom Capac	ZPA
		Column A		Column B		Column C			
		P	MC	P	MC	P	MC		
5	13.3	24	858	72	1680	27	895	3433	0.37
4	14.0	69	1210	156	2100	76	1400	4710	0.48
3	11.8	119	1660	238	4350	128	2200	8210	1.00
2	10.3	170	2790	323	6480	182	3300	12570	1.75
1	9.4	232	3350	243	7950	244	3750	15050	2.29

NOTES

- $f'_c = 3 \text{ ksi}$
- f_y (COLUMNS) = 50 ksi
- f_y (OTHER) = 40 ksi
- FIRST COL DIMEN IN N-S
- REINF IN COLS

4 BAR

6 BAR

8 BAR

Fig. 2.1 E-W WING "A" FRAME

BEAMS	REINFORCEMENT LABEL						
	A	B	C	D	E	F	G
503-504	4#7	2#8	5#7	2#8 3#7	3#7	3#7	2#8
3-4	5#7	2#8	5#8	2#8 2#7	3#7	4#7	2#8
103-104	4#8	2#8	6#8	2#8	4#6	4#6	2#8

Fig. 2.2 WING "A" BEAM DETAILS

NOTES:

- NODE NUMBERS FOR ANSYS MODEL
- ALL ELEMENTS MODELED WITH 2-D BEAM ELEMENTS
- ROTATIONAL SPRINGS PLACED BETWEEN NODES 1-2, 3-4, 5-6 TO SIMULATE ROTATIONAL STIFFNESS OF FOOTINGS
- DUPLICATE NODES AT JOINTS COUPLED IN ALL DOF TO SIMULATE MOMENT CONN.... ONLY DISPL COUPLED TO SIMULATE SHEAR CONN

Fig. 2.3 E-W STRUCTURAL MODEL

Fig. 2.4 GROUND RESPONSE SPECTRA

EQUIVALENT SEISMIC
SHEAR FORCES

SHEAR (k)

MOMENT (k ft)

Fig. 2.5 SEISMIC SHEARS AND MOMENTS (0.1 G)

$$\Delta = \theta H$$

$$\text{EXT WORK} = \sum_{i=j}^5 \theta P_i H$$

$$\text{INT WORK} = 2 (MCA + MCB + MCC) \theta$$

$$2 (MCA + MCB + MCC) = \sum_{i=j}^5 P_i H$$

j = STORY UNDER INVESTIGATION

Fig. 2.6 PLASTIC ANALYSIS MODEL

Fig. 2.7 Ductility of Fourth Floor Columns

	7F 4 x 3 1/2 x 3/8
	7F 6 x 4 x 9/16
	7F 6 x 4 x 7/8
	7F 8 x 4 x 7/8
	7F 8 x 6 x 1

NOTE: CROSS BRACES PLACED IN ONE BAY OF EVERY THIRD FRAME
 EACH DIAGONAL IS DOUBLE ANGLE MEMBER MADE AS SHOWN
 BRACES DESIGNED FOR TENSION ONLY

Fig. 2.8 STEEL BRACING :LOCATED EVERY THIRD FRAME

3.0 Longitudinal (N-S) Frame Model of Wing A Structure

The response of a typical longitudinal (N-S) frame in the wing structures to the applied loading is discussed in this section of the report. This section contains: discussion of the data used in the analyses, description of the model, discussion of the frame capacity (based on code-like acceptance criteria utilizing lower bound estimates for capacity) for both static and seismic load cases, an evaluation of the increased capacity that would occur if less conservative acceptance criteria are used, and an evaluation of potential fixes to increase the frame capacity.

3.1 Data Used in the Analyses

The wing structures are eight bays long in the N-S direction with each bay having a span of about 20'. Since the structural configuration of all of the bays is about the same it is reasonable to consider a single bay spanning from the middle of one bay to the middle of the adjacent bay. Symmetric or asymmetric boundary conditions at the mid-bay divisions can be applied for the static and seismic load cases respectively. The frame centered about column line 7 is selected for analysis. The results for this analysis can then be combined with the transverse frame analysis discussed in Section 2. A sketch of the frame with pertinent dimensions, gross column properties, material descriptions, and general notes of interest is shown on Fig. 2.1. Of course the N-S frame spans in the direction normal to the frame shown on Fig. 2.1 with the symmetric or asymmetric boundary conditions applied 10' north and south of frame 7. Reinforcement for the columns is shown on Fig. 2.1. The members spanning in the N-S direction between the columns consist of a slab and outside spandrel beams. The slabs consist of a 19" deep (16.25" at the roof) section 4' long and centered about the column lines, and a 16' long slab about 6" deep that fills the area between the deeper sections. The deeper slab sections are the beams that span in the E-W direction. The slabs were designed as one way slabs spanning in the N-S direction. The beam and slab dimensions and reinforcement are shown on Fig. 3.1. These data are obtained from the following drawings: S-1.2, S-5.1, S-6.2, S-7.1, S-8.1, S-9.1, S-13.1, S-20.2, S-23.2, S-24.2, and S-25.2.

The following characteristics of these data are important in assessing the seismic capacity of the frame:

- The only lateral resistance comes from the beam (or slab)/column frame. Interior N-S masonry block walls exist but they are unreinforced and would not be expected to contribute to the lateral load capacity of the frame. These walls will increase the frequency of the frame and will attract seismic loads until they crack. One would therefore expect these walls to be

seriously cracked at the seismic levels required to cause structural damage to the N-S frames.

- The bottom steel in the spandrel beams and the 6" slabs are only embedded 6" into the columns and 19" slabs. This is a standard detail when only negative moments (tension at the top of the beam) occur at the beam/column connection. Seismic loads will develop positive moments at one end of the beam so that this detail degrades the seismic capacity. It is also important to note that there is no bottom reinforcement across the 4' wide beams centered on the columns.
- There is very little shear reinforcement in the beams and it does not satisfy the minimal transverse reinforcement requirements for seismic details (see Section 21.3.3 of ACI 318-89).

The same loads are used in the analyses as discussed in Section 2.

3.2 Description of Model

The static loads are distributed in the N-S direction through the floor slabs and then in the E-W direction through the 4' wide beams to the columns. The E-W load path is discussed in Section 2 and does not need to be discussed further. The N-S dead load path through the floor slabs can be treated simply by modeling each of the slabs as a fixed-fixed beam.

A preliminary review of the N-S frame indicates that the foundation walls along column lines A and C together with the first floor slab form a rigid box so that essentially all of the N-S deformations occur above the first floor. Diaphragm action of the upper floor slabs also restrain relative N-S deformation of the three columns. The seismic model is therefore fixed at the first floor elevation and the flexural stiffness of the columns and total E-W width of the floor slabs are combined into single elements. The torsional rigidity of the 19" x 48" E-W beam is about nine times the flexural rigidity of the N-S 6" deep slabs. It is therefore assumed that the rotation along the E-W column line is a constant value so that the full width (in the E-W direction) of the slabs are effective.

The seismic model for the frame based on these assumptions is shown on Fig. 3.2 a. The simple supports for the beams are at midspan between column lines and represent inflection points for the beam/slab elements. Two acceptance criteria need to be applied for the seismic loading. First, the sum of the seismic and gravity (DL+LL/4) moments must be less than the negative moment capacity of the slab at its support from the 4' beam. Second the difference between the seismic and gravity moments must be less than the positive moment capacity (relatively small because of the short embedment lengths of the bottom slab steel) of the slab at the support.

A second model, shown on Fig. 3.2 b is also used. This model addresses the case when the seismic end moments in the beam, as determined from the first model, exceed the positive moment capacity of the beams. Since the beam moment capacity is different for positive and negative moments, the beam failure will occur at the end where the seismic moments are of the same sign as the critical beam capacity. The seismic moments will be of the opposite sign at the other end of the beam so that failures will not occur at that end. The model shown on Fig. 3.2 b represents this condition.

Soil structure interaction effects are neglected in this analysis as in the E-W frame model. They can be shown to be less important for this problem than for the E-W model since the frame is longer in this direction than in the E-W direction.

3.3 Frame Capacity (LPF Acceptance Criteria)

The capacity of the N-S frame to both the $(1.4 \text{ DL} + 1.7 \text{ LL})$ and $(\text{DL} + \text{LL}/4 + \text{EQ})$ load combinations are reviewed in this section of the report against criteria associated with a high confidence low probability of failure (LPF) acceptance criteria. These acceptance criteria are taken from the ACI 318-89 code. The member capacities are first developed followed with a discussion for each of the load cases.

3.3.1 Member Capacities

Interaction charts are developed for each of the frame columns based on the ultimate load provisions contained in ACI 318-89 criteria. The column interaction charts are developed using the following material properties: concrete compressive strength = 3 ksi, steel compressive yield stress = 50 ksi, steel tensile yield stress is taken from Table 2.2 to account for the short splice lengths, and ultimate compressive strain in the concrete = 0.003. The resulting interaction charts are shown in Appendix D. The capacity reduction factors (ϕ) are not included in these charts and must be added when applying these data for the column reviews.

The moment capacity of the beams are determined again based on the ultimate load provisions of the ACI 318-89. The material properties used in these evaluations are: concrete strength = 3 ksi, steel yield stress = 40 ksi, and ultimate compressive strain in the concrete = 0.003. The moment capacities are evaluated at the center and end of each beam, and at each location for the cases where the moment is positive (tension at bottom of beam) and where the moment is negative. The capacity reduction factors ($\phi = 0.9$) are not included in these evaluations and will be added when the frame

evaluations are done. For the case where the steel development length required in ACI 318-89 are not provided (all positive moments at the ends of the beams and slabs) the moment capacity is evaluated by reducing the steel yield stress in proportion to the ratio of the provided to the required development length. The resulting beam capacities are tabulated in Table 3.1. The beams and slabs have no top reinforcement at the center span so that the negative moment capacity of the beams is zero at midspan. This will not create a problem since the DL and LL mid span moments are positive and the EQ moments are close to zero at midspan. The slab moments are given for a 12" wide segment. It should be noted that the slab frames into the 19" wide beams and that the beams have no bottom reinforcement in the E-W direction. The positive end slab moments result in tensile stresses in the 19" beam which are less than the modulus of rupture. These moment capacities may be optimistic. It will be shown below, however, that the frame capacity is governed by "pounding" limitations and that the assumed seismic capacity does not result in net positive moments at the ends of the slabs.

Table 3.1
N-S BEAM MOMENT CAPACITIES (Kip Inches)

Beam No.	Location	Center		End	
		M _{max}	M _{min}	M _{max}	M _{min}
B-5	Spandrel - Flr 1,2,3,4	687	0	288	-821
B-505	Spandrel - Roof	472	0	280	-667
S-551	Slab - Flr 1,2,3,4				
	Cols 7-6	92	0	32	-133
S-601	Slab - Flr 1,2,3,4				
	Cols 7-8	105	0	34	-148
S-554	Slab - Roof	58	0	28	-86

3.3.2 Dead Load + Live Load (DL + LL) Combination

The dead and live loads travel in the N-S direction through the slabs to the E-W beams and then in the E-W direction through the beams and columns to the foundation. The latter load path is considered in Section 2 so that the analysis presented here deals only with the transfer across the slabs. Two slabs must be considered; the roof slab (S-554) and the thinner slab at the other floors (S-551).

The DL acting on the roof consists of the weight of the 5.5" slab plus a superimposed DL of 23.5 psf resulting in a load of 92.25 pounds per foot acting on a 12" slab segment. The roof LL is 25 psf. The resulting factored load acting on a 12" segment of the slab is 0.172 k/ft. The positive moments and negative moment at the end of the slab are:

$$M^- = w L^2 / 12 = 0.172 * 16^2 / 12 = 3.66 \text{ k/ft} = 43.9 \text{ k in}$$

$$M^+ = w L^2 / 24 = 22.0 \text{ k in}$$

The moment capacities are $(0.9 * 86.4 = 77.8 \text{ k in})$ for the negative moment region and $(0.9 * 57.5 = 51.8 \text{ k in})$ for the positive moment region. The capacities therefore exceed the demand.

The DL acting on the other floors consists of the weight of the 5.5" slab plus a superimposed DL of 31 psf resulting in a load of 99.75 pounds per foot acting on a 12" slab segment. The LL is 100 psf. The resulting factored load acting on a 12" segment of the slab is 0.31 k/ft. The positive moments and negative moment at the end of the slab are:

$$M^- = w L^2 / 12 = 79.3 \text{ k in}$$

$$M^+ = w L^2 / 24 = 39.7 \text{ k in}$$

The moment capacities are $(0.9 * 133 = 119.7 \text{ k in})$ for the negative moment region and $(0.9 * 92 = 82.8 \text{ k in})$ for the positive moment region. The capacities therefore exceed the demand. The shear load demand at a distance "d" from the slab support is $(0.31 * 16 / 2 - 0.31 * 4.5 / 12 = 2.36 \text{ k})$ as compared with a shear capacity of $(2 * 0.85 * 3000^{1/2} * 12 * 4.5 / 1000 = 5.0 \text{ k})$.

3.3.3 Dead + Live + Seismic Load (DL + LL/4 + EQ) Capacities

The seismic model shown on Fig. 3.1 a is first used and a seismic response determined using ANSYS for the 0.1 G NUREG 0098 motion (see Fig. 2.4) used as input. The fundamental frequency of the frame is found to be 0.9 cps and the peak displacement at the roof is 1.64". The expansion joint at the end of the wing structure only has a 1" gap so that pounding of the structures is expected when the seismic input exceeds $(1 * 0.1 / 1.64 = 0.06 \text{ G})$. There is considerable uncertainty in evaluation of the effects of pounding. No inspection of the gaps has been performed so that the actual gaps may be different from the 1". Damage caused by pounding is likely to be local however since the floors in the two structures are at the same elevation and one floor slab will impact on the adjacent floor slab. For this reason, the seismic capacity is based on the assumption

that the adjacent frames responses are in phase.

The stress evaluation of the beams is carried out in Table 3.2. The DL and LL moments are obtained from Section 3.3.2 and multiplied by the width of the structure (44.5') to obtain the total moments. The seismic loads (in column 4) are obtained from the ANSYS results. Two evaluations are then made: one when the seismic moments are of the opposite sign from the DL and LL moments (columns 5-7) and the other when the moments are of the same sign (columns 8-10). The results indicate that the first condition governs, that the second floor slabs are critical, and that the seismic capacity is 0.1 G. This input is greater than required to close the gap in the expansion joint.

The stress evaluation of the columns is carried out in Table 3.3. The axial loads are taken from column 9 of Table 2.9 since the axial column loads arise from the DL+LL and are treated in Section 2. The balanced load of column 3 is taken from the Appendix D interaction charts, and the strength reduction factor determined as discussed in Section 2. The column capacities of (column 5) is found from the Appendix D interaction charts at the magnitude of the axial load listed in column 2. The analysis is done by adding the stiffness of the three columns at a given floor and the total seismic moment acting on the three columns is determined from the ANSYS model. The capacity of the three columns is tabulated in column 6, the total seismic moment for a 0.1 G input is shown on column 7 and the capacity ZPA shown on column 8. The fourth floor column is critical and limits the seismic load to 0.10 G.

It is interesting to compare the effect that biaxial bending may have on the column strength. The critical column for the E-W response is A7-4 and the fourth floor columns are also critical for the N-S response. Column A7-4 is selected for consideration. The seismic capacity of the frame is 0.04 G as limited by the first floor beams in the E-W response. The E-W seismic moment for this column is $(0.04 \times 585 / 1 = 234 \text{ k in})$. This column is also subjected to a DL+LL/4 moment of 489 k in. The N-S moment is obtained by distributing the total seismic moment $(0.04 \times 2621 / 0.1 = 1048 \text{ k in})$ in proportion to the stiffness of the three columns. The seismic moment carried by column A7-4 is therefore $(0.29 \times 1048 = 304 \text{ k in})$. It is normal to perform the load combination by taking 100% of the load in one direction in combination with 40% of the load in the other direction (ASCE 4-86). The N-S moment can then be reduced to $(0.4 \times 304 = 122 \text{ k in})$. The column capacities at the axial load of 69 k are 957 k in and 781 k in respectively for E-W and N-S bending respectively. The capacities shown in the tables are multiplied by the capacity reduction factor to obtain these results. Biaxial effects are investigated using the load contour method recommended in ACI Notes on 318. The value of (β) is found to be 0.71 from this reference. The ratios of applied moment to uniaxial moment capacities in the E-W and N-S directions are found to be $([234+489]/957 = 0.76)$ and $(122/781 = 0.16)$ respectively. These points are plotted on the load

contour shown on Fig. 3.3 and can be seen to be within acceptable limits for the value of ($\beta = 0.71$).

Since the beams are critical a seismic load solution is obtained using the second model on Fig. 3.2. This model assumes that the beam has "failed" at the end where the seismic moments are positive, and places a hinge at that location. The seismic solution for this case indicates that the fundamental frequency of the model is 0.6 cps and the peak displacement for the 0.1 G input is 2.45". The evaluation for the beams and columns are shown on Tables 3.4 and 3.5 respectively. The same procedure is used as discussed above except that beam moments at the hinged end do not need to be evaluated. The seismic capacity of the frame can be seen to increase to 0.14 G based on the lower story beams and to 0.13 G based on the lower story column. There is some question whether the "failed" end of the beam can transmit shear as required to achieve this seismic capacity. This is somewhat academic however since the frame capacity is governed by pounding.

In summary the LPF seismic capacity of the frame, as limited by the N-S response, is 0.10 G except for the displacement limitation at the expansion joint. This displacement limits the capacity to 0.06 G.

3.4 Frame Capacity (HPF Acceptance Criteria)

The previous seismic analysis is based on code type acceptance criteria and leads to a seismic capacity estimation with a high confidence that it will not cause collapse of the structure. The objective of the work reported in this section is to develop an estimate of seismic load that one has a high confidence that collapse of the structure will occur. These two data points will provide the user with a basis for estimating the range of earthquakes that may cause serious damage to the structure.

Three factors are considered in arriving at this high probability of failure estimate: material properties, redistribution of load within the structure, and ductility effects. Each of these are discussed. The capacity reduction factors (ϕ) are set equal to unity for all of the results presented in this section.

As discussed in Section 2.4.1 the following "upper bound" material properties are used: concrete compressive strength = 5.5 ksi; ultimate concrete compressive strain = 0.005; column reinforcing steel yield stress = 62.5 ksi; and beam / slab reinforcing steel yield stress = 50 ksi.

The 1.83 factor increasing the concrete compressive strength would be expected to increase the concrete modulus by a factor of $1.83^{1/2} = 1.35$. This would then increase the frame stiffness by 1.35 and reduce the 0.1 G peak displacement to 1.21". The ZPA required to close the 1" expansion joint gaps is therefore $(0.1 * 1/1.21 = 0.08 \text{ G})$. There is not much data available to assess the needed additional seismic input required to cause serious structural damage caused by pounding. It is assumed that a seismic input of twice that required to close the gap would result in serious damage. Therefore a ZPA of 0.16 G could be expected to cause major damage.

The member capacities are computed for these increased properties. The beam capacities are shown on Table 3.6. The column capacities are determined for the higher material properties and the interaction charts for the columns are shown in Appendix E.

Table 3.6
N-S BEAM MOMENT CAPACITIES (Kip Inches)
MAXIMUM MATERIAL PROPERTIES

Beam No.	Location	Center		End	
		M _{max}	M _{min}	M _{max}	M _{min}
B-5	Spandrel - Flr 1,2,3,4	871	0	400	-1040
B-505	Spandrel - Roof	600	0	361	-847
S-551	Slab - Flr 1,2,3,4				
	Cols 7-6	119	0	45	-174
S-601	Slab - Flr 1,2,3,4				
	Cols 7-8	134	0	49	-193
S-554	Slab - Roof	73	0	37	-112

The seismic capacity of the frame is again evaluated based on the model shown in Fig. 3.2 b since that is the analysis that controls the previous evaluation (the seismic capacity is the larger of that determined from the two models). The evaluation of the beams and columns are shown on Tables 3.7 and 3.8 respectively. As may be seen the frame capacity is 0.23 G based on the lower floor beams and fourth floor column capacities.

The same increase in capacity due to frame ductility can be used as discussed in Section

2.4.3. The frame capacity increases to $2.5 * 0.23 = 0.58$ G.

The capacity is however restricted by the deformation (and closing of the expansion joint 1" gap) to 0.16 G. This results in a mean capacity of $([0.06+0.16]/2 = 0.11$ G).

3.5 Structural Modifications to Increase Frame Capacity

Several options are available to increase the frame capacity to 0.15 G (based on LPF acceptance criteria). The gap at the expansion joint can be increased to $(1.64 * 1.5 = 2.5")$ so that the gap does not close under the 0.15 G seismic input. A second option is to tie the buildings together at the expansion joints so as to eliminate the possibility of relative motion between the various parts of the building. The N-S frame capacity is then 0.10 G and may be adequate without additional modifications. The consequences of thermal loads introduced by the elimination of the expansion joints must be evaluated if this option is selected. The third option is to provide bracing so as to reduce the peak displacement.

Bracing is designed so that the peak displacement at the top of the frame is no larger than one half of the gap of 1" when the ZPA equals 0.15 G. The criteria response spectra (Fig. 2.4) indicates that this requires the structural frame to have a fundamental frequency of at least 3 cps. At this frequency the peak acceleration is 0.41 G. The bracing system is designed as outlined in Section 2.5 except that no bracing is provided between the foundations and the first floor because of the available stiffness at this level in the existing frame. Following the procedures of Section 2.5 it is found that the following bracing areas per bay are needed at the second through fifth levels respectively: 3.46 sq. in, 6.92 sq. in., 10.38 sq. in., and 13.83 sq in. The corresponding bracing stresses for the 0.15 G earthquake are 16.8 ksi, 18.3 ksi, 17.8 ksi, and 16.9 ksi respectively. It is recommended that the bracing be provided along column line B with four out of the eight bays braced. This requires that the above areas be increased by a factor of two. The following sets of double angles are required for the four floors from the top down: 2L 6x4x3/8; 2L 6x4x3/4; 2L 8x4x1; and 2L 8x8x1. They should be arranged as counters (see Fig. 2.8 so that only the tensile brace is considered to be acting. Other arrangements are possible and may be designed by multiplying the number of bays allocated to the bracing by the areas/bay listed above to obtain the required area. The bracing should be placed either along column line B or along both column lines A and C so that no torsional response is introduced.

Table 3.2

BEAM / SLAB EVALUATION FOR (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES...MOMENT CONNECTIONS

Floor	Moments (k in)			DL+LL/4+EQ (+Mom)		DL+LL+EQ (-Mom)		Min ZPA		
	DL	LL	EQ(.1G)	Capacity		ZPA	Capacity			
				Spandre	Slab		Spandre		Slab	
1	2	3	4	5	6	7	8	9	10	11
Roof	-1051	-285	908	560	1230	0.30	-1334	-3840	0.37	0.30
4	-1136	-1139	1955	576	1402	0.16	-1642	-5915	0.23	0.16
3	-1136	-1139	2850	576	1402	0.11	-1642	-5915	0.16	0.11
2	-1136	-1139	3174	576	1402	0.10	-1642	-5915	0.14	0.10

Table 3.3

COLUMN EVALUATION FOR (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES...MOMENT CONNECTIONS

Column	Axial Load	Bal Load	ϕ	Capacity	Total Flr C	EQ Moment	ZPA
1	2	3	4	5	6	7	8
A7-5	24	448	0.89	574			
B7-5	72	585	0.88	1064			
C7-5	27	448	0.89	575	2213	1520	0.13
A7-4	69	448	0.87	781			
B7-4	156	585	0.85	1488			
C7-4	76	476	0.87	863	3132	2621	0.10
A7-3	119	476	0.85	1063			
B7-3	238	750	0.84	2283			
C7-3	128	508	0.85	1224	4570	3298	0.12
A7-2	170	544	0.84	1504			
B7-2	323	869	0.83	2936			
C7-2	182	588	0.84	1646	6086	4384	0.12

Table 3.4

BEAM / SLAB EVALUATION (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES...SHEAR CONNECTIONS

Floor	Moments (k in)			DL+LL+EQ (-Mom)		
	DL	LL	EQ(.1G)	Capacity	ZPA	
				Spandrel	Slab	
1	2	3	4	5	6	7
Roof	-1051	-285	1757	-1334	-3840	0.19
4	-1136	-1139	2608	-1642	-5915	0.17
3	-1136	-1139	3324	-1642	-5915	0.14
2	-1136	-1139	3208	-1642	-5915	0.14

Table 3.5

COLUMN EVALUATION FOR (DL + LL/4 + EQ)
CODE MATERIAL PROPERTIES...SHEAR CONNECTIONS

Column	Axial Load	Bal Load	ϕ	Capacity	Total Flr C	EQ Moment	ZPA
1	2	3	4	5	6	7	8
A7-5	24	448	0.89	574			
B7-5	72	585	0.88	1064			
C7-5	27	448	0.89	575	2213	1320	0.15
A7-4	69	448	0.87	781			
B7-4	156	585	0.85	1488			
C7-4	76	476	0.87	863	3132	1931	0.14
A7-3	119	476	0.85	1063			
B7-3	238	750	0.84	2283			
C7-3	128	508	0.85	1224	4570	2138	0.18
A7-2	170	544	0.84	1504			
B7-2	323	869	0.83	2936			
C7-2	182	588	0.84	1646	6086	3831	0.13

Table 3.7

BEAM / SLAB EVALUATION (DL + LL/4 + EQ)
MAX MATERIAL PROPERTIES...SHEAR CONNECTIONS

Floor	Moments (k in)			DL+LL+EQ (-Mom)		ZPA
	DL	LL	EQ(.1G)	Capacity		
				Spandrel	Slab	
1	2	3	4	5	6	7
Roof	-1051	-285	1757	-1694	-4984	0.30
4	-1136	-1139	2608	-2080	-7722	0.29
3	-1136	-1139	3324	-2080	-7722	0.23
2	-1136	-1139	3208	-2080	-7722	0.23

Table 3.8

COLUMN EVALUATION FOR (DL + LL/4 + EQ)
MAX MATERIAL PROPERTIES...SHEAR CONNECTIONS

Column	Axial Load	Capacity	Total Flr C	EQ Moment	ZPA
1	2	3	4	5	6
A7-5	30	827			
B7-5	72	1680			
C7-5	33	830	3337	1320	0.25
A7-4	76	1040			
B7-4	159	2120			
C7-4	82	1230	4390	1931	0.23
A7-3	117	1400			
B7-3	249	3970			
C7-3	128	1860	7230	2138	0.34
A7-2	157	2190			
B7-2	339	4890			
C7-2	172	2900	9980	3831	0.26

BEAMS	REINFORCEMENT LABEL		
	A	B	C
ROOF	1#5	2#4	2#4
OTHER FLOORS	1#6	2#5	2#4

SPANDREL BEAM DETAILS

SLAB	THICKNESS	POSITIVE MOMENT STEEL
ROOF	5-1/2"	#4@8
COL 7-8	6"	#5@6-1/2
COL 6-7	5-1/2"	#5@6-1/2

SLAB DETAILS

Fig. 3.1 N-S WING "A" BEAM AND SLAB DETAILS

Fig. 3.2 N-S STRUCTURAL MODELS

COLUMN (A7-4) STRENGTHS:

$$P_o = 655 \text{ k}$$

$$M_{oNS} = 781 \text{ k in} \quad M_{oEW} = 957 \text{ k in}$$

LOADS:

$$P = 69 \text{ k}$$

$$M_{NS} = 234 \text{ k in}; M_{EW} = 304 \text{ k in} (.04 \text{ G E})$$

$$M_{NS \text{ DL+LL}} = 489 \text{ k in}$$

PARAMETER (β) DEFINING LOAD CONTOUR:

$$\omega = \rho f'_c / f_y = 0.13$$

$$\beta = 0.72 \quad \beta = 0.68 \text{ USE } 0.70$$

40 ksi 60 ksi

$$M_{EW} / M_{oEW} = (234 + 489) / 957 = 0.76$$

$$M_{NS} / M_{oNS} = 122 / 781 = 0.16$$

Fig. 3.3 BIAxIAL BENDING EFFECTS

4.0 Extrapolation Of Unit "A" Results To Units "B", "C", And "D"

The results obtained for Unit "A" and presented in Sections 2 and 3 of this report are extrapolated to the remaining three Units. As discussed in Section 1, the general structural configuration is similar for all of the units but many differences in specific geometric proportions exist between the four Units of the building. The objective in performing this extrapolation of the Unit "A" results is to ensure that the general conclusions drawn from the Unit "A" work are applicable, and is not to obtain precise values of seismic capacity or member areas required for upgrading the capacity. For example, it is desired to identify those Units where the median seismic capacity may be significantly less (e.g., 0.05 G) than that found for Unit "A" (about 0.10 G) or to identify those Units where the LPF capacity approaches 0.15 G so that no fixes are required.

4.1 Unit "B"

Unit "B" is the western portion of the main structure (see Fig. 1.1) and is included within column lines A through K and 10 through 16. Its plan dimensions are about 160' (EW) by 120' (NS). There are two stories above grade with one basement level. There are some penthouse structures that carry one story above the main roof. A feature of this Unit that differs from other Units is the large auditorium that is about 60' by 80' and includes space from the first floor to the roof. Vertical roof loading is carried through E-W steel joists to large N-S steel trusses running along column lines E and H. One foot thick reinforced concrete shear walls frame several bays around the auditorium providing significant seismic capability.

Unit "B" also contains shear walls placed around the periphery of the auditorium as shown on Fig. 4.1 a. The walls are continuous through the three stories. There are about 63' and 40' of twelve inch shear walls in the N-S and E-W directions respectively. The walls are reinforced with #4 @ 12" in each direction at each face. The simple models shown on Fig. 4.1 b are used to determine whether the shear walls can carry the seismic load associated with a 0.15 G ZPA input. A simple analysis is used to show that the frequencies are 6.0 cps and 4.8 cps respectively in the N-S and E-W directions. The shear loads in the N-S walls are 1392 k, 1916 k, and 2383 k for a 0.1 G ZPA input motion. The capacity of the three N-S walls is governed by flexural stresses and found to be less than 1000 k so that the walls will not carry a significant portion of the seismic loading. The LPF capacity may be estimated to be $(1000 \times 0.1 / 2383 = 0.04 \text{ G})$.

The Unit "B" conclusions regarding median seismic capacity and required fixes are therefore expected to be similar to those developed for Unit "A".

4.2 Unit "C"

Unit "C" is the same in plan as Unit "B" and comprises the eastern portion of the main structure. It has no auditorium nor shear walls comparable to those contained in Unit "B".

The primary structure is very similar to the Unit "A" structure with the following exceptions:

- Unit "C" has two stories above grade while Unit "A" has four stories above grade.
- The four feet wide beams of Unit "A" all span in the E-W direction. The Unit "C" beams span in the E-W direction for the second and third stories but span in the N-S direction for the first floor.
- Unit "A" has two spans in the E-W direction and eight spans in the N-S direction while Unit "C" has eight spans in the E-W direction and six spans in the N-S direction.

The net result of these differences is not expected to significantly alter the conclusions drawn for Unit "A".

4.3 Unit "D"

The structure of Unit "D" is the same as Unit "A" except that a sub basement exist under a portion of Unit "D". In the areas of the sub basement the height of the lowest columns changes from about 12.75' in Unit "A" to a maximum of about 17.75'. The critical columns are located in the upper stories, however, so that this difference is not expected to seriously change the Unit "A" conclusions.

Fig. 4.1 UNIT "B" PLAN AND MODEL

5.0 Summary

The seismic capacity of the Administration Building at Los Alamos is considered in this report. Two types of acceptance criteria are used in these evaluations. Seismic capacities based on code type acceptance criteria are used first and result in a seismic capacity associated with a very low probability of failure. A second evaluation is then performed with the objective of defining a seismic loading that corresponds to a high probability of failure. The mean seismic capacity may be estimated as midway between these two values. When this mean seismic capacity is found to be less than an earthquake having a 0.15 G ZPA, fixes to the structure are investigated that would raise its seismic capacity (based on the code type acceptance criteria) to an earthquake defined with a NUREG 0098 spectra anchored at 0.15 G. It should be noted that this work is based on the NUREG 0098 spectral shape but can be extrapolated to other spectra by recognizing that the significant amplitudes of the spectra are at frequencies between about 0.7 cps and 2.2 cps. It is also important to note that the fragility levels are associated with a damage state in which significant portions of the structural frames would be expected to collapse. Other damage states, especially cracking and perhaps even partial collapse of the masonry walls, are likely to occur at seismic levels below those estimated for the frames. Conclusions of the study are divided into three parts: observations regarding the structural configuration, seismic capacity of the building, and fixes required to increase the building's seismic capacity.

A review of the structural configuration indicate that, as a result of its vintage and the treatment of seismic loads at the time, the building contains many poor details that have negative impact on its seismic capacity as evidenced by the following observations:

1. The top story outside columns of the wing Units do not satisfy the static load requirements. Alternate loads paths are available to carry the DL but the structure does not formally satisfy the ACI code requirements.
2. The shear loads in the main beams of the E-W frames of the wing Units exceed the ACI code allowable values.
3. The longitudinal reinforcement in some of the columns do not meet ACI code minimum specifications (at least 1 %).
4. Many of the ties in the columns do not satisfy ACI code standards.

5. The transverse steel in both the columns and beams are far below the ACI standards for seismic loads.
6. The original design appears to have provided no system to carry lateral load. The bottom beam reinforcement only extends into the columns 6". This is a code requirement when the beam end moments are negative. Positive moments occur, however, when the frame is subjected to lateral loads.
7. The floor slabs do not satisfy ACI code minimum thickness requirements.

The following results are found from the seismic analyses:

1. A code type seismic assessment (LPF) of an E-W frame in the wing structures indicates that the frame can carry a 0.04 G NUREG 0098 earthquake. The fundamental frequency is 0.7 cps so that portion of the spectra is of most interest. This capacity is limited by the lower story beams and the fourth story columns. As indicated above, some damage to the masonry partitions is expected at somewhat lower seismic levels.
2. A high probability of failure type analysis for the same frame indicates that the structure would collapse at 0.15 G. This is derived by using "84% " material strengths, considering ductility effects, and considering the possibility of load redistribution. The critical response is found to be collapse of the fourth floor columns. It is important to note that this fragility level assumes some frame ductility even though it does not contain many of the detailing requirements for ductile behavior. Some ductile behavior is assumed, however, since the objective of the calculation is to evaluate a seismic capacity that is associated with a high probability (>90 %) of major frame collapse.
3. The previous two results combine to give a mean seismic capacity of 0.10 G.
4. An assessment of the N-S frame of the wing Units indicate similar results with the LPF capacity found to be 0.06 G limited by closure of the expansion joints so that pounding of the structures would be expected for higher acceleration levels. The HPF capacity is found to be 0.16 G resulting in a mean capacity of 0.11 G.
5. A review of the structural configuration of the other Units making up the building suggest that these seismic capacities are applicable.

Building modifications are investigated to increase the LPF capacity to 0.15 G. Both concrete shear walls (either internal walls or external buttresses) and steel cross bracing are investigated. The steel bracing is likely to be the easier fix but is likely to be too flexible to protect the masonry walls from some damage. The concrete shear walls can be designed to raise the frame stiffness to the level required to minimize damage to the masonry walls at the 0.15 G level earthquake.

Appendix A

Soils Data

Footing Modulus of Subgrade Reaction

From D. Keller report on seismic tests in Bandolier tuff:

Modulus of upper tuff unit (softer rock) $E \sim 1400 \text{ tsf} = 2800 \text{ ksf}$

Modulus of subgrade reaction $k = 0.65 [E_s B^4 / E_f I_f]^{1/2} E_s / B (1 - \mu^2)$

if $[E_s B^4 / E_f I_f] \sim 1$

$E_s = 2800 \text{ ksf}$

$B = 3'$

$\mu^2 \sim 0.1$

$k = 700 \text{ ksf}$

If the material were a stiff sand one would expect $k = 300 \text{ ksf}$

Therefore, $300 < k < 700 \text{ ksf}$

Soils Boring Data

Boring data from the OTOWI Bldg 261 and Library Study Center Bldg 207 are shown on the following pages.

AVERAGE SITE SPT DATA LANL-OTOWI BLDG 261

DATE OF COMPUTATION: 12/22/93

*** SOIL LAYER DATA:

ORIGINAL NO. OF SOIL LAYERS = 2
 NO. LAYERS INCL GWT EFFECTS = 2
 DEPTH TO GWT (FT) = -1.0

LAYER TYPE	CLASS	DEPTH TO TOP	GSAT (KCF)	GDRY (KCF)	GAMMA (KCF)
1 5	SANDY SILT	.0	.132	.110	.110
2 1	VOLC TUFF	12.0	.132	.110	.110

*** SOIL SAMPLE DATA :

BORING NUMBER = B1
 NO. OF SOIL SAMPLES = 8

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	31.0	.059	45.4
2	1	SANDY SILT	2.5	52.0	.275	76.2
3	1	SANDY SILT	5.0	51.0	.550	73.1
4	1	SANDY SILT	10.0	55.0	1.100	66.1
5	2	VOLC TUFF	15.0	100.0	1.650	106.6
6	2	VOLC TUFF	20.0	100.0	2.200	97.0
7	2	VOLC TUFF	25.0	120.0	2.750	107.4
8	2	VOLC TUFF	30.0	300.0	3.300	250.3

*** SOIL SAMPLE DATA :

BORING NUMBER = B2
 NO. OF SOIL SAMPLES = 8

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	8.0	.055	11.7
2	1	SANDY SILT	2.5	22.0	.275	32.2
3	1	SANDY SILT	5.0	36.0	.550	51.6
4	1	SANDY SILT	10.0	84.0	1.100	100.9
5	1	SANDY SILT	15.0	108.0	1.650	115.1
6	2	VOLC TUFF	20.0	150.0	2.200	145.5
7	2	VOLC TUFF	25.0	200.0	2.750	179.1
8	2	VOLC TUFF	30.0	200.0	3.300	166.9

AVERAGE SITE SPT DATA LANL-LIBRARY STUDY CENTER BLDG 207

DATE OF COMPUTATION: 12/22/93

*** SOIL LAYER DATA:

ORIGINAL NO. OF SOIL LAYERS = 2
 NO. LAYERS INCL GWT EFFECTS = 2
 DEPTH TO GWT (FT) = -1.0

LAYER TYPE	CLASS	DEPTH TO TOP	GSAT (KCF)	GDRY (KCF)	GAMMA (KCF)
1	5 SANDY SILT	.0	.132	.110	.110
2	1 VOLC TUFF	12.0	.132	.110	.110

*** SOIL SAMPLE DATA :

BORING NUMBER = B1
 NO. OF SOIL SAMPLES = 12

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	1.5	15.0	.165	22.0
2	1	SANDY SILT	5.0	13.0	.550	18.6
3	1	SANDY SILT	10.0	16.0	1.100	19.2
4	2	VOLC TUFF	15.0	150.0	1.650	159.9
5	2	VOLC TUFF	20.0	100.0	2.200	97.0
6	2	VOLC TUFF	25.0	120.0	2.750	107.4
7	2	VOLC TUFF	30.0	150.0	3.300	125.1
8	2	VOLC TUFF	35.0	190.0	3.850	148.7
9	2	VOLC TUFF	40.0	300.0	4.400	221.4
10	2	VOLC TUFF	45.0	350.0	4.950	244.6
11	2	VOLC TUFF	50.0	1200.0	5.500	796.2
12	2	VOLC TUFF	55.0	1500.0	6.050	947.5

*** SOIL SAMPLE DATA :

BORING NUMBER = B2
 NO. OF SOIL SAMPLES = 13

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	1.5	10.0	.165	14.7
2	1	SANDY SILT	5.0	9.0	.550	12.9
3	2	VOLC TUFF	10.0	76.0	1.100	91.3
4	2	VOLC TUFF	15.0	100.0	1.650	106.6
5	2	VOLC TUFF	20.0	120.0	2.200	116.4
6	2	VOLC TUFF	25.0	100.0	2.750	89.5
7	2	VOLC TUFF	30.0	150.0	3.300	125.1
8	2	VOLC TUFF	35.0	300.0	3.850	234.8
9	2	VOLC TUFF	40.0	300.0	4.400	221.4
10	2	VOLC TUFF	45.0	1500.0	4.950	1048.1
11	2	VOLC TUFF	50.0	1500.0	5.500	995.3
12	2	VOLC TUFF	55.0	1200.0	6.050	758.0
13	2	VOLC TUFF	60.0	1200.0	6.600	723.0

*** SOIL SAMPLE DATA :

BORING NUMBER = B3
NO. OF SOIL SAMPLES = 13

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	1.5	14.0	.165	20.5
2	1	SANDY SILT	5.0	16.0	.550	22.9
3	2	VOLC TUFF	10.0	100.0	1.100	120.2
4	2	VOLC TUFF	15.0	100.0	1.650	106.6
5	2	VOLC TUFF	20.0	100.0	2.200	97.0
6	2	VOLC TUFF	25.0	150.0	2.750	134.3
7	2	VOLC TUFF	30.0	2400.0	3.300	2002.4
8	2	VOLC TUFF	35.0	2400.0	3.850	1878.7
9	2	VOLC TUFF	40.0	600.0	4.400	442.9
10	2	VOLC TUFF	45.0	600.0	4.950	419.2
11	2	VOLC TUFF	50.0	2400.0	5.500	1592.4
12	2	VOLC TUFF	55.0	2400.0	6.050	1515.9
13	2	VOLC TUFF	60.0	300.0	6.600	180.8

*** SOIL SAMPLE DATA :

BORING NUMBER = B4
NO. OF SOIL SAMPLES = 13

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	1.5	9.0	.165	13.2
2	1	SANDY SILT	5.0	9.0	.550	12.9
3	2	VOLC TUFF	10.0	150.0	1.100	180.3
4	2	VOLC TUFF	15.0	150.0	1.650	159.9
5	2	VOLC TUFF	20.0	100.0	2.200	97.0
6	2	VOLC TUFF	25.0	150.0	2.750	134.3
7	2	VOLC TUFF	30.0	600.0	3.300	500.6
8	2	VOLC TUFF	35.0	600.0	3.850	469.7
9	2	VOLC TUFF	40.0	1200.0	4.400	885.8
10	2	VOLC TUFF	45.0	2400.0	4.950	1677.0
11	2	VOLC TUFF	50.0	2400.0	5.500	1592.4
12	2	VOLC TUFF	55.0	2400.0	6.050	1515.9
13	2	VOLC TUFF	60.0	2400.0	6.600	1446.1

*** SOIL SAMPLE DATA :

BORING NUMBER = B5
NO. OF SOIL SAMPLES = 13

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	1.5	18.0	.165	26.4
2	1	SANDY SILT	5.0	11.0	.550	15.8
3	1	SANDY SILT	10.0	32.0	1.100	38.5
4	2	VOLC TUFF	15.0	192.0	1.650	204.7
5	2	VOLC TUFF	20.0	160.0	2.200	155.2
6	2	VOLC TUFF	25.0	160.0	2.750	143.2
7	2	VOLC TUFF	30.0	240.0	3.300	200.2
8	2	VOLC TUFF	35.0	240.0	3.850	187.9
9	2	VOLC TUFF	40.0	300.0	4.400	221.4
10	2	VOLC TUFF	45.0	2400.0	4.950	1677.0
11	2	VOLC TUFF	50.0	2400.0	5.500	1592.4
12	2	VOLC TUFF	55.0	2400.0	6.050	1515.9
13	2	VOLC TUFF	60.0	2400.0	6.600	1446.1

*** SOIL SAMPLE DATA :

BORING NUMBER = B6
NO. OF SOIL SAMPLES = 14

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	1.5	10.0	.165	14.7
2	1	SANDY SILT	2.5	24.0	.275	35.2
3	1	SANDY SILT	5.0	25.0	.550	35.8
4	1	SANDY SILT	10.0	109.0	1.100	131.0
5	2	VOLC TUFF	15.0	51.0	1.650	54.4
6	2	VOLC TUFF	20.0	120.0	2.200	116.4
7	2	VOLC TUFF	25.0	150.0	2.750	134.3
8	2	VOLC TUFF	30.0	200.0	3.300	166.9
9	2	VOLC TUFF	35.0	240.0	3.850	187.9
10	2	VOLC TUFF	40.0	343.0	4.400	253.2
11	2	VOLC TUFF	45.0	2400.0	4.950	1677.0
12	2	VOLC TUFF	50.0	480.0	5.500	318.5
13	2	VOLC TUFF	55.0	2400.0	6.050	1515.9
14	2	VOLC TUFF	60.0	2400.0	6.600	1446.1

NSMPS	DEPTH	BPFMX	BPFAV	BPFMN	BPFCTX	BPFCAV	BPFCTXN
6	1.50	18.00	12.67	9.00	26.38	18.56	13.19
1	2.50	24.00	24.00	24.00	35.17	35.17	35.17
6	5.00	25.00	13.83	9.00	35.84	19.83	12.90
6	10.00	150.00	80.50	16.00	180.26	96.74	19.23
6	15.00	192.00	123.83	51.00	204.70	132.02	54.37
6	20.00	160.00	116.67	100.00	155.19	113.16	96.99
6	25.00	160.00	138.33	100.00	143.25	123.85	89.53
6	30.00	2400.00	623.33	150.00	2002.39	520.07	125.15
6	35.00	2400.00	661.67	190.00	1878.68	517.94	148.73
6	40.00	1200.00	507.17	300.00	885.75	374.35	221.44
6	45.00	2400.00	1608.33	350.00	1676.98	1123.81	244.56
6	50.00	2400.00	1730.00	480.00	1592.42	1147.87	318.48
6	55.00	2400.00	2050.00	1200.00	1515.92	1294.85	757.96
5	60.00	2400.00	1740.00	300.00	1446.09	1048.41	180.76

I	DEPTH	DRAV	PHIAV	GAMAV	DRCV	PHICV	GAMCV
1	1.50	49.12	30.89	87.86	59.66	31.77	89.55
2	2.50	66.36	32.31	90.62	78.06	33.43	92.74
3	5.00	51.25	31.10	88.20	61.31	31.90	89.81
4	10.00	100.00	36.50	98.00	100.00	36.50	98.00
5	15.00	100.00	41.00	124.00	100.00	41.00	124.00
6	20.00	100.00	41.00	124.00	100.00	41.00	124.00
7	25.00	100.00	41.00	124.00	100.00	41.00	124.00
8	30.00	100.00	41.00	124.00	100.00	41.00	124.00
9	35.00	100.00	41.00	124.00	100.00	41.00	124.00
10	40.00	100.00	41.00	124.00	100.00	41.00	124.00
11	45.00	100.00	41.00	124.00	100.00	41.00	124.00
12	50.00	100.00	41.00	124.00	100.00	41.00	124.00
13	55.00	100.00	41.00	124.00	100.00	41.00	124.00
14	60.00	100.00	41.00	124.00	100.00	41.00	124.00

SOIL DATA FILE NAME = LIBRARY.BORING

Appendix B

E-W Frame Column Interaction Diagrams Code Material Properties

C O L U M N I N T E R A C T I O N D I A G

COL A7 1

WIDTH = 14.000 IN HEIGHT = 17.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 28.500 KSI EU = .0030

DIST FROM CG AREA

5.560 2.000

.000 2.000

-5.560 2.000

P0 = 0.892E+03 K

PB = 0.418E+03 MB = 0.214E+04 K IN CB = 0.106E+02 IN

LOAD/LINE = 23 K MOMENT/SP = 54 K IN

P (KIPS)

MOMENT

LOAD

920	+	-----	+	-----	+	-----	+	-----	+
:	:	:	:	:	:	:	:	:	:
:	*	:	:	:	:	:	:	0.187E+03	0.874E+03
:	*	:	:	:	:	:	:	0.291E+03	0.851E+03
:	*	:	:	:	:	:	:	0.441E+03	0.828E+03
805	+	-----	+	-----	+	-----	+	0.599E+03	0.805E+03
:	:	:	*	:	:	:	:	0.744E+03	0.782E+03
:	:	:	*	:	:	:	:	0.880E+03	0.759E+03
:	:	:	*	:	:	:	:	0.101E+04	0.736E+03
:	:	:	*	:	:	:	:	0.113E+04	0.713E+03
690	+	-----	+	-----	+	-----	+	0.124E+04	0.690E+03
:	:	:	:	*	:	:	:	0.135E+04	0.667E+03
:	:	:	:	*	:	:	:	0.145E+04	0.644E+03
:	:	:	:	*	:	:	:	0.154E+04	0.621E+03
:	:	:	:	*	:	:	:	0.162E+04	0.598E+03
575	+	-----	+	-----	+	-----	+	0.171E+04	0.575E+03
:	:	:	:	:	*	:	:	0.178E+04	0.552E+03
:	:	:	:	:	*	:	:	0.185E+04	0.529E+03
:	:	:	:	:	*	:	:	0.192E+04	0.506E+03
:	:	:	:	:	*	:	:	0.198E+04	0.483E+03
460	+	-----	+	-----	+	-----	+	0.204E+04	0.460E+03
:	:	:	:	:	*	:	:	0.210E+04	0.437E+03
:	:	:	:	:	*	:	:	0.215E+04	0.414E+03
:	:	:	:	:	*	:	:	0.216E+04	0.391E+03
:	:	:	:	:	*	:	:	0.216E+04	0.368E+03
345	+	-----	+	-----	+	-----	+	0.216E+04	0.345E+03
:	:	:	:	:	*	:	:	0.215E+04	0.322E+03
:	:	:	:	:	*	:	:	0.214E+04	0.299E+03
:	:	:	:	:	*	:	:	0.212E+04	0.276E+03
:	:	:	:	:	*	:	:	0.210E+04	0.253E+03
230	+	-----	+	-----	+	-----	+	0.207E+04	0.230E+03
:	:	:	:	:	*	:	:	0.204E+04	0.207E+03
:	:	:	:	:	*	:	:	0.199E+04	0.184E+03
:	:	:	:	:	*	:	:	0.190E+04	0.161E+03
:	:	:	:	:	*	:	:	0.180E+04	0.138E+03
115	+	-----	+	-----	+	-----	+	0.170E+04	0.115E+03
:	:	:	:	:	*	:	:	0.159E+04	0.920E+02
:	:	:	:	*	:	:	:	0.147E+04	0.690E+02
:	:	:	:	*	:	:	:	0.135E+04	0.460E+02
:	:	:	:	*	:	:	:	0.122E+04	0.230E+02
0	+	-----	+	-----	+	-----	+	0.109E+04	0.000E+00

0 540 1080 1620 2160

MOMENT (K IN)

COL A7 2

STL YIELD T= 32.000 KSI

EU = .0030

5.130	1.580
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.000	1.580
------	-------

-5.130	1.580
--------	-------

P0 = 0.796E+03 K

PB = 0.360E+03 MB = 0.180E+04 K IN CB = 0.960E+01 IN

LOAD/LINE = 20 K MOMENT/SP = 45 K IN

P (KIPS)

MOMENT

LOAD

800	+	+	+	+	+	0.149E+03	0.780E+03
:	*	:	:	:	:	0.226E+03	0.760E+03
:	*	:	:	:	:	0.368E+03	0.740E+03
:	:	*	:	:	:	0.495E+03	0.720E+03
:	:	*	:	:	:	0.615E+03	0.700E+03
700	+	+	+	+	+	0.727E+03	0.680E+03
:	:	:	:	:	:	0.833E+03	0.660E+03
:	:	:	:	:	:	0.932E+03	0.640E+03
:	:	:	:	:	:	0.103E+04	0.620E+03
:	:	:	:	:	:	0.111E+04	0.600E+03
600	+	+	+	+	+	0.119E+04	0.580E+03
:	:	:	:	:	:	0.127E+04	0.560E+03
:	:	:	:	:	:	0.134E+04	0.540E+03
:	:	:	:	:	:	0.141E+04	0.520E+03
:	:	:	:	:	:	0.147E+04	0.500E+03
500	+	+	+	+	+	0.153E+04	0.480E+03
:	:	:	:	:	:	0.158E+04	0.460E+03
:	:	:	:	:	:	0.163E+04	0.440E+03
:	:	:	:	:	:	0.168E+04	0.420E+03
:	:	:	:	:	:	0.172E+04	0.400E+03
400	+	+	+	+	+	0.176E+04	0.380E+03
:	:	:	:	:	:	0.180E+04	0.360E+03
:	:	:	:	:	:	0.181E+04	0.340E+03
:	:	:	:	:	:	0.181E+04	0.320E+03
:	:	:	:	:	:	0.180E+04	0.300E+03
300	+	+	+	+	+	0.179E+04	0.280E+03
:	:	:	:	:	:	0.178E+04	0.260E+03
:	:	:	:	:	:	0.176E+04	0.240E+03
:	:	:	:	:	:	0.174E+04	0.220E+03
:	:	:	:	:	:	0.171E+04	0.200E+03
200	+	+	+	+	+	0.167E+04	0.180E+03
:	:	:	:	:	:	0.163E+04	0.160E+03
:	:	:	:	:	:	0.157E+04	0.140E+03
:	:	:	:	:	:	0.149E+04	0.120E+03
:	:	:	:	:	:	0.140E+04	0.100E+03
100	+	+	+	+	+	0.131E+04	0.800E+02
:	:	:	:	:	:	0.122E+04	0.600E+02
:	:	:	:	:	:	0.112E+04	0.400E+02
:	:	:	:	:	:	0.102E+04	0.200E+02
:	:	:	:	:	:	0.915E+03	0.000E+00
0	+	+	+	+	+		

MOMENT (K IN)

COLUMN INTERACTION DIAG

COL A7 3 C7 4

WIDTH = 14.000 IN HEIGHT = 16.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 36.900 KSI EU = .0030

DIST FROM CG AREA

5.310 1.200

-5.310 1.200

P0 = 0.685E+03 K

PB = 0.316E+03 MB = 0.169E+04 K IN CB = 0.935E+01 IN

LOAD/LINE = 18 K MOMENT/SP = 43 K IN

P (KIPS)					MOMENT	LOAD
720	+	-----	+	-----	+	-----
:	:	:	:	:	:	:
:	*	:	:	:	0.452E+02	0.684E+03
:	*	:	:	:	0.126E+03	0.666E+03
:	*	:	:	:	0.250E+03	0.648E+03
630	+	-----	+	-----	+	-----
:	:	*	:	:	0.382E+03	0.630E+03
:	:	*	:	:	0.506E+03	0.612E+03
:	:	*	:	:	0.623E+03	0.594E+03
:	:	*	:	:	0.733E+03	0.576E+03
:	:	*	:	:	0.836E+03	0.558E+03
540	+	-----	+	-----	+	-----
:	:	:	*	:	0.933E+03	0.540E+03
:	:	:	*	:	0.102E+04	0.522E+03
:	:	:	*	:	0.111E+04	0.504E+03
:	:	:	*	:	0.119E+04	0.486E+03
:	:	:	*	:	0.126E+04	0.468E+03
450	+	-----	+	-----	+	-----
:	:	:	:	*	0.133E+04	0.450E+03
:	:	:	:	*	0.139E+04	0.432E+03
:	:	:	:	*	0.145E+04	0.414E+03
:	:	:	:	*	0.150E+04	0.396E+03
:	:	:	:	*	0.155E+04	0.378E+03
360	+	-----	+	-----	+	-----
:	:	:	:	*	0.160E+04	0.360E+03
:	:	:	:	*	0.164E+04	0.342E+03
:	:	:	:	*	0.168E+04	0.324E+03
:	:	:	:	*	0.170E+04	0.306E+03
:	:	:	:	*	0.169E+04	0.288E+03
270	+	-----	+	-----	+	-----
:	:	:	:	*	0.168E+04	0.270E+03
:	:	:	:	*	0.166E+04	0.252E+03
:	:	:	:	*	0.163E+04	0.234E+03
:	:	:	:	*	0.159E+04	0.216E+03
:	:	:	:	*	0.154E+04	0.198E+03
180	+	-----	+	-----	+	-----
:	:	:	:	*	0.147E+04	0.180E+03
:	:	:	:	*	0.140E+04	0.162E+03
:	:	:	:	*	0.133E+04	0.144E+03
:	:	:	:	*	0.124E+04	0.126E+03
:	:	:	:	*	0.116E+04	0.108E+03
90	+	-----	+	-----	+	-----
:	:	:	*	:	0.107E+04	0.900E+02
:	:	:	*	:	0.974E+03	0.720E+02
:	:	:	*	:	0.878E+03	0.540E+02
:	:	:	*	:	0.779E+03	0.360E+02
:	:	:	*	:	0.690E+03	0.180E+02
0	+	-----	+	-----	+	-----
:	:	:	:	:	0.584E+03	0.000E+00
0		430	860	1290	1720	
			MOMENT	(K IN)		

C O L U M N I N T E R A C T I O N D I A G

COL A7 4 A7 5 C7 5

WIDTH = 14.000 IN HEIGHT = 16.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 43.000 KSI EU = .0030

DIST FROM CG AREA

5.380 .880

-5.380 .880

P0 = 0.655E+03 K

PB = 0.294E+03 MB = 0.158E+04 K IN CB = 0.895E+01 IN

LOAD/LINE = 17 K MOMENT/SP = 40 K IN

P (KIPS)	MOMENT	LOAD
680 +-----+-----+-----+-----+-----+		
: : : : :		
:* : : : :	0.796E+02	0.646E+03
: * : : : :	0.165E+03	0.629E+03
: : * : : :	0.300E+03	0.612E+03
595 +-----+-----+-----+-----+-----+	0.421E+03	0.595E+03
: : * : : :	0.534E+03	0.578E+03
: : * : : :	0.641E+03	0.561E+03
: : * : : :	0.741E+03	0.544E+03
: : * : : :	0.834E+03	0.527E+03
510 +-----+-----+-----+-----+-----+	0.922E+03	0.510E+03
: : : * : :	0.100E+04	0.493E+03
: : : * : :	0.108E+04	0.476E+03
: : : * : :	0.115E+04	0.459E+03
: : : * : :	0.121E+04	0.442E+03
425 +-----+-----+-----+-----+-----+	0.127E+04	0.425E+03
: : : : * :	0.133E+04	0.408E+03
: : : : * :	0.138E+04	0.391E+03
: : : : * :	0.142E+04	0.374E+03
: : : : * :	0.146E+04	0.357E+03
340 +-----+-----+-----+-----+-----+	0.150E+04	0.340E+03
: : : : * :	0.153E+04	0.323E+03
: : : : * :	0.156E+04	0.306E+03
: : : : * :	0.158E+04	0.289E+03
: : : : * :	0.158E+04	0.272E+03
255 +-----+-----+-----+-----+-----+	0.156E+04	0.255E+03
: : : : * :	0.154E+04	0.238E+03
: : : : * :	0.151E+04	0.221E+03
: : : : * :	0.147E+04	0.204E+03
: : : : * :	0.142E+04	0.187E+03
170 +-----+-----+-----+-----+-----+	0.136E+04	0.170E+03
: : : : * :	0.130E+04	0.153E+03
: : : : * :	0.123E+04	0.136E+03
: : : : * :	0.115E+04	0.119E+03
: : : : * :	0.107E+04	0.102E+03
85 +-----+-----+-----+-----+-----+	0.979E+03	0.850E+02
: : : * : :	0.889E+03	0.680E+02
: : : * : :	0.796E+03	0.510E+02
: : : * : :	0.700E+03	0.340E+02
: : : * : :	0.614E+03	0.170E+02
0 +-----+-----+-----+-----+-----+	0.511E+03	0.000E+00
0 400 800 1200 1600		

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 1

WIDTH = 20.000 IN HEIGHT = 20.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 22.800 KSI EU = .0030
DIST FROM CG AREA

6.920 4.680
.000 3.120
-6.920 4.680

P0 = 0.161E+04 K

PB = 0.812E+03 MB = 0.480E+04 K IN CB = 0.134E+02 IN

LOAD/LINE = 41 K MOMENT/SP = 120 K IN

P (KIPS)					MOMENT	LOAD
1640	+	-----	+	-----	+	-----
:	*	:	:	:	0.327E+03	0.160E+04
:	*	:	:	:	0.588E+03	0.156E+04
:	*	:	:	:	0.864E+03	0.152E+04
:	*	:	:	:	0.114E+04	0.148E+04
1435	+	-----	+	-----	+	-----
:	:	*	:	:	0.151E+04	0.144E+04
:	:	*	:	:	0.180E+04	0.139E+04
:	:	*	:	:	0.209E+04	0.135E+04
:	:	*	:	:	0.237E+04	0.131E+04
:	:	*	:	:	0.263E+04	0.127E+04
1230	+	-----	+	-----	+	-----
:	:	:	*	:	0.288E+04	0.123E+04
:	:	:	*	:	0.312E+04	0.119E+04
:	:	:	*	:	0.334E+04	0.115E+04
:	:	:	*	:	0.355E+04	0.111E+04
:	:	:	*	:	0.375E+04	0.107E+04
1025	+	-----	+	-----	+	-----
:	:	:	:	*	0.394E+04	0.102E+04
:	:	:	:	*	0.412E+04	0.984E+03
:	:	:	:	*	0.430E+04	0.943E+03
:	:	:	:	*	0.446E+04	0.902E+03
:	:	:	:	*	0.462E+04	0.861E+03
820	+	-----	+	-----	+	-----
:	:	:	:	*	0.477E+04	0.820E+03
:	:	:	:	*	0.484E+04	0.779E+03
:	:	:	:	*	0.488E+04	0.738E+03
:	:	:	:	*	0.490E+04	0.697E+03
:	:	:	:	*	0.491E+04	0.656E+03
615	+	-----	+	-----	+	-----
:	:	:	:	*	0.490E+04	0.615E+03
:	:	:	:	*	0.487E+04	0.574E+03
:	:	:	:	*	0.484E+04	0.533E+03
:	:	:	:	*	0.479E+04	0.492E+03
:	:	:	:	*	0.474E+04	0.451E+03
410	+	-----	+	-----	+	-----
:	:	:	:	*	0.468E+04	0.410E+03
:	:	:	:	*	0.451E+04	0.369E+03
:	:	:	:	*	0.429E+04	0.328E+03
:	:	:	:	*	0.405E+04	0.287E+03
:	:	:	*	:	0.381E+04	0.246E+03
205	+	-----	+	-----	+	-----
:	:	:	*	:	0.356E+04	0.205E+03
:	:	:	*	:	0.330E+04	0.164E+03
:	:	:	*	:	0.302E+04	0.123E+03
:	:	*	:	:	0.275E+04	0.820E+02
:	:	*	:	:	0.250E+04	0.410E+02
0	+	-----	+	-----	+	-----
0		1200	2400	3600	4800	

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 3

WIDTH = 16.000 IN HEIGHT = 18.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 25.300 KSI EU = .0030
DIST FROM CG AREA

5.990 2.540

.000 2.540

-5.990 2.540

P0 = 0.110E+04 K

PB = 0.539E+03 MB = 0.276E+04 K IN CB = 0.116E+02 IN

LOAD/LINE = 28 K MOMENT/SP = 69 K IN

P (KIPS)					MOMENT	LOAD
1120	+	-----	+	-----	+	-----
:	*	:	:	:	0.151E+03	0.109E+04
:	*	:	:	:	0.302E+03	0.106E+04
:	*	:	:	:	0.435E+03	0.104E+04
:	*	:	:	:	0.650E+03	0.101E+04
980	+	-----	+	-----	+	-----
:	:	*	:	:	0.846E+03	0.980E+03
:	:	*	:	:	0.103E+04	0.952E+03
:	:	*	:	:	0.121E+04	0.924E+03
:	:	*	:	:	0.137E+04	0.896E+03
:	:	*	:	:	0.152E+04	0.868E+03
840	+	-----	+	-----	+	-----
:	:	:	*	:	0.166E+04	0.840E+03
:	:	:	*	:	0.180E+04	0.812E+03
:	:	:	*	:	0.193E+04	0.784E+03
:	:	:	*	:	0.204E+04	0.756E+03
:	:	:	*	:	0.216E+04	0.728E+03
700	+	-----	+	-----	+	-----
:	:	:	:	*	0.226E+04	0.700E+03
:	:	:	:	*	0.236E+04	0.672E+03
:	:	:	:	*	0.245E+04	0.644E+03
:	:	:	:	*	0.254E+04	0.616E+03
:	:	:	:	*	0.262E+04	0.588E+03
560	+	-----	+	-----	+	-----
:	:	:	:	*	0.270E+04	0.560E+03
:	:	:	:	*	0.276E+04	0.532E+03
:	:	:	:	*	0.278E+04	0.504E+03
:	:	:	:	*	0.280E+04	0.476E+03
:	:	:	:	*	0.280E+04	0.448E+03
420	+	-----	+	-----	+	-----
:	:	:	:	*	0.279E+04	0.420E+03
:	:	:	:	*	0.278E+04	0.392E+03
:	:	:	:	*	0.276E+04	0.364E+03
:	:	:	:	*	0.274E+04	0.336E+03
:	:	:	:	*	0.271E+04	0.308E+03
280	+	-----	+	-----	+	-----
:	:	:	:	*	0.268E+04	0.280E+03
:	:	:	:	*	0.263E+04	0.252E+03
:	:	:	:	*	0.253E+04	0.224E+03
:	:	:	:	*	0.240E+04	0.196E+03
:	:	:	:	*	0.227E+04	0.168E+03
140	+	-----	+	-----	+	-----
:	:	:	*	:	0.213E+04	0.140E+03
:	:	:	*	:	0.198E+04	0.112E+03
:	:	:	*	:	0.182E+04	0.840E+02
:	:	:	*	:	0.166E+04	0.560E+02
:	:	:	*	:	0.149E+04	0.280E+02
0	+	-----	+	-----	+	-----
:	:	:	:	:	0.134E+04	0.000E+00
0		690	1380	2070	2760	

MOMENT (K IN)

COL B7 4 B7 5

CITI. V

DIST FROM CG	AREA
0	0
1	1
2	4
3	9
4	16
5	25
6	36
7	49
8	64
9	81
10	100
11	121
12	144
13	169
14	196
15	225
16	256
17	289
18	324
19	361
20	400
21	441
22	484
23	529
24	576
25	625
26	676
27	729
28	784
29	841
30	900
31	961
32	1024
33	1089
34	1156
35	1225
36	1296
37	1369
38	1444
39	1521
40	1600
41	1681
42	1764
43	1849
44	1936
45	2025
46	2116
47	2209
48	2304
49	2401
50	2500
51	2601
52	2704
53	2809
54	2916
55	3025
56	3136
57	3249
58	3364
59	3481
60	3600
61	3721
62	3844
63	3969
64	4096
65	4225
66	4356
67	4489
68	4624
69	4761
70	4900
71	5041
72	5184
73	5329
74	5476
75	5625
76	5776
77	5929
78	6084
79	6241
80	6400
81	6561
82	6724
83	6889
84	7056
85	7225
86	7396
87	7569
88	7744
89	7921
90	8100
91	8281
92	8464
93	8649
94	8836
95	9025
96	9216
97	9409
98	9604
99	9801
100	10000

-5.130 1.580

PB = 0.381E+03 MB = 0.196E+04 K IN CB = 0.960E+01 IN

P (KIPS)

LOAD

MOMENT (K IN)

COLUMN INTERACTION DIAG

COL C7 1

WIDTH = 14.000 IN HEIGHT = 17.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 25.300 KSI EU = .0030

DIST FROM CG AREA

5.490 2.540

.000 2.540

-5.490 2.540

P0 = 0.968E+03 K

PB = 0.459E+03 MB = 0.231E+04 K IN CB = 0.108E+02 IN

LOAD/LINE = 25 K MOMENT/SP = 58 K IN

P (KIPS)					MOMENT	LOAD
1000	+	-----	+	-----	+	-----
:	:	:	:	:	:	:
:	*	:	:	:	:	:
:	:	:	:	:	:	:
:	*	:	:	:	:	:
:	:	*	:	:	:	:
875	+	-----	+	-----	+	-----
:	:	:	:	:	:	:
:	:	*	:	:	:	:
:	:	:	*	:	:	:
:	:	:	*	:	:	:
:	:	:	*	:	:	:
750	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
625	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
500	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
375	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
250	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
125	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
0	+	-----	+	-----	+	-----
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
:	:	:	:	*	:	:
0	0	580	1160	1740	2320	

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL C7 3

WIDTH = 14.000 IN HEIGHT = 16.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 32.000 KSI EU = .0030
DIST FROM CG AREA

5.130 1.580

-5.130 1.580

P0 = 0.721E+03 K

PB = 0.337E+03 MB = 0.180E+04 K IN CB = 0.960E+01 IN

LOAD/LINE = 19 K MOMENT/SP = 45 K IN

P (KIPS)					MOMENT	LOAD
760	+	-----	+	-----	+	-----
:	:	:	:	:	:	:
:	:	:	:	:	:	:
:	*	:	:	:	0.143E+03	0.703E+03
:	*	:	:	:	0.234E+03	0.684E+03
665	+	-----	+	-----	+	-----
:	:	*	:	:	0.376E+03	0.665E+03
:	:	*	:	:	0.509E+03	0.646E+03
:	:	*	:	:	0.633E+03	0.627E+03
:	:	*	:	:	0.750E+03	0.608E+03
:	:	*	:	:	0.861E+03	0.589E+03
570	+	-----	+	-----	+	-----
:	:	:	*	:	0.964E+03	0.570E+03
:	:	:	*	:	0.106E+04	0.551E+03
:	:	:	*	:	0.115E+04	0.532E+03
:	:	:	*	:	0.124E+04	0.513E+03
:	:	:	*	:	0.132E+04	0.494E+03
475	+	-----	+	-----	+	-----
:	:	:	:	*	0.139E+04	0.475E+03
:	:	:	:	*	0.146E+04	0.456E+03
:	:	:	:	*	0.152E+04	0.437E+03
:	:	:	:	*	0.158E+04	0.418E+03
:	:	:	:	*	0.164E+04	0.399E+03
380	+	-----	+	-----	+	-----
:	:	:	:	*	0.169E+04	0.380E+03
:	:	:	:	*	0.174E+04	0.361E+03
:	:	:	:	*	0.179E+04	0.342E+03
:	:	:	:	*	0.181E+04	0.323E+03
:	:	:	:	*	0.181E+04	0.304E+03
285	+	-----	+	-----	+	-----
:	:	:	:	*	0.180E+04	0.285E+03
:	:	:	:	*	0.177E+04	0.266E+03
:	:	:	:	*	0.174E+04	0.247E+03
:	:	:	:	*	0.169E+04	0.228E+03
:	:	:	:	*	0.163E+04	0.209E+03
190	+	-----	+	-----	+	-----
:	:	:	:	*	0.156E+04	0.190E+03
:	:	:	:	*	0.149E+04	0.171E+03
:	:	:	:	*	0.141E+04	0.152E+03
:	:	:	*	:	0.133E+04	0.133E+03
:	:	:	*	:	0.124E+04	0.114E+03
95	+	-----	+	-----	+	-----
:	:	:	*	:	0.114E+04	0.950E+02
:	:	:	*	:	0.105E+04	0.760E+02
:	:	:	*	:	0.950E+03	0.570E+02
:	:	*	:	:	0.850E+03	0.380E+02
:	:	*	:	:	0.749E+03	0.190E+02
0	+	-----	+	-----	+	-----
:	:	:	:	:	0.647E+03	0.000E+00
0		450	900	1350	1800	
			MOMENT (K IN)			

Appendix C

E-W Column Interaction Diagrams

Maximum Material Properties

C O L U M N I N T E R A C T I O N D I A G

COL A7 1

WIDTH = 14.000 IN HEIGHT = 17.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050

DIST FROM CG AREA

5.560 2.000

.000 2.000

-5.560 2.000

P0 = 0.146E+04 K

PB = 0.537E+03 MB = 0.373E+04 K IN CB = 0.983E+01 IN

LOAD/LINE = 37 K MOMENT/SP = 94 K IN

P (KIPS)						MOMENT	LOAD
1480	+	-----	+	-----	+	-----	+
:	*	:	:	:	:	0.318E+03	0.144E+04
:		*	:	:	:	0.586E+03	0.141E+04
:			*	:	:	0.853E+03	0.137E+04
:				*	:	0.110E+04	0.133E+04
1295	+	-----	+	-----	+	-----	+
:			*	:	:	0.134E+04	0.129E+04
:				*	:	0.156E+04	0.126E+04
:					*	0.177E+04	0.122E+04
:					*	0.196E+04	0.118E+04
:					*	0.214E+04	0.115E+04
1110	+	-----	+	-----	+	-----	+
:				*	:	0.231E+04	0.111E+04
:					*	0.247E+04	0.107E+04
:					*	0.261E+04	0.104E+04
:					*	0.273E+04	0.999E+03
:					*	0.285E+04	0.962E+03
925	+	-----	+	-----	+	-----	+
:					*	0.295E+04	0.925E+03
:					*	0.305E+04	0.888E+03
:					*	0.314E+04	0.851E+03
:					*	0.322E+04	0.814E+03
:					*	0.330E+04	0.777E+03
740	+	-----	+	-----	+	-----	+
:					*	0.337E+04	0.740E+03
:					*	0.345E+04	0.703E+03
:					*	0.351E+04	0.666E+03
:					*	0.358E+04	0.629E+03
:					*	0.364E+04	0.592E+03
555	+	-----	+	-----	+	-----	+
:					*	0.370E+04	0.555E+03
:					*	0.372E+04	0.518E+03
:					*	0.369E+04	0.481E+03
:					*	0.365E+04	0.444E+03
:					*	0.360E+04	0.407E+03
370	+	-----	+	-----	+	-----	+
:					*	0.356E+04	0.370E+03
:					*	0.350E+04	0.333E+03
:					*	0.345E+04	0.296E+03
:					*	0.339E+04	0.259E+03
:					*	0.333E+04	0.222E+03
185	+	-----	+	-----	+	-----	+
:					*	0.327E+04	0.185E+03
:					*	0.315E+04	0.148E+03
:					*	0.298E+04	0.111E+03
:					*	0.277E+04	0.740E+02
:				*	:	0.255E+04	0.370E+02
0	+	-----	+	-----	+	-----	+
0		940	1880	2820	3760	0.235E+04	0.000E+00

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL A7 2

WIDTH = 14.000 IN HEIGHT = 17.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

5.130 1.580
 .000 1.580
 -5.130 1.580

P0 = 0.139E+04 K

PB = 0.508E+03 MB = 0.334E+04 K IN CB = 0.952E+01 IN

LOAD/LINE = 35 K MOMENT/SP = 84 K IN

P (KIPS)						MOMENT	LOAD
1400	+	-----	+	-----	+		
	:	*	:	:	:	0.322E+03	0.136E+04
	:		*	:	:	0.577E+03	0.133E+04
	:		*	:	:	0.826E+03	0.129E+04
	:			*	:	0.106E+04	0.126E+04
1225	+	-----	+	-----	+	0.128E+04	0.123E+04
	:			*	:	0.148E+04	0.119E+04
	:			*	:	0.167E+04	0.116E+04
	:			*	:	0.185E+04	0.112E+04
	:			*	:	0.202E+04	0.108E+04
1050	+	-----	+	-----	+	0.217E+04	0.105E+04
	:			*	:	0.231E+04	0.102E+04
	:			*	:	0.244E+04	0.980E+03
	:			*	:	0.255E+04	0.945E+03
	:			*	:	0.265E+04	0.910E+03
875	+	-----	+	-----	+	0.274E+04	0.875E+03
	:			*	:	0.282E+04	0.840E+03
	:			*	:	0.289E+04	0.805E+03
	:			*	:	0.296E+04	0.770E+03
	:			*	:	0.303E+04	0.735E+03
700	+	-----	+	-----	+	0.308E+04	0.700E+03
	:			*	:	0.314E+04	0.665E+03
	:			*	:	0.319E+04	0.630E+03
	:			*	:	0.323E+04	0.595E+03
	:			*	:	0.328E+04	0.560E+03
525	+	-----	+	-----	+	0.332E+04	0.525E+03
	:			*	:	0.332E+04	0.490E+03
	:			*	:	0.329E+04	0.455E+03
	:			*	:	0.324E+04	0.420E+03
	:			*	:	0.319E+04	0.385E+03
350	+	-----	+	-----	+	0.314E+04	0.350E+03
	:			*	:	0.308E+04	0.315E+03
	:			*	:	0.302E+04	0.280E+03
	:			*	:	0.296E+04	0.245E+03
	:			*	:	0.289E+04	0.210E+03
175	+	-----	+	-----	+	0.276E+04	0.175E+03
	:			*	:	0.259E+04	0.140E+03
	:			*	:	0.242E+04	0.105E+03
	:			*	:	0.224E+04	0.700E+02
	:			*	:	0.205E+04	0.350E+02
0	+	-----	+	-----	+	0.186E+04	0.000E+00
	0	840	1680	2520	3360		

MOMENT (K IN)

COLUMN INTERACTION DIAG

COL A7 3 C7 4

WIDTH = 14.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

5.310 1.200

-5.310 1.200

P0 = 0.119E+04 K

PB = 0.472E+03 MB = 0.287E+04 K IN CB = 0.930E+01 IN

LOAD/LINE = 30 K MOMENT/SP = 72 K IN

P (KIPS)	MOMENT	LOAD
1200 +-----+-----+-----+-----+-----+		
: * : : : : 0.182E+03 0.117E+04		
: * : : : : 0.399E+03 0.114E+04		
: * : : : : 0.609E+03 0.111E+04		
: * : : : : 0.808E+03 0.108E+04		
1050 +-----+-----+-----+-----+-----+	0.995E+03 0.105E+04	
: : * : : : 0.117E+04 0.102E+04		
: : * : : : 0.134E+04 0.990E+03		
: : * : : : 0.149E+04 0.960E+03		
: : * : : : 0.164E+04 0.930E+03		
900 +-----+-----+-----+-----+-----+	0.178E+04 0.900E+03	
: : * : : : 0.190E+04 0.870E+03		
: : * : : : 0.202E+04 0.840E+03		
: : * : : : 0.213E+04 0.810E+03		
: : * : : : 0.223E+04 0.780E+03		
750 +-----+-----+-----+-----+-----+	0.232E+04 0.750E+03	
: : * : : : 0.240E+04 0.720E+03		
: : * : : : 0.248E+04 0.690E+03		
: : * : : : 0.255E+04 0.660E+03		
: : * : : : 0.261E+04 0.630E+03		
600 +-----+-----+-----+-----+-----+	0.267E+04 0.600E+03	
: : * : : : 0.273E+04 0.570E+03		
: : * : : : 0.277E+04 0.540E+03		
: : * : : : 0.282E+04 0.510E+03		
: : * : : : 0.286E+04 0.480E+03		
450 +-----+-----+-----+-----+-----+	0.285E+04 0.450E+03	
: : * : : : 0.281E+04 0.420E+03		
: : * : : : 0.275E+04 0.390E+03		
: : * : : : 0.269E+04 0.360E+03		
: : * : : : 0.260E+04 0.330E+03		
300 +-----+-----+-----+-----+-----+	0.251E+04 0.300E+03	
: : * : : : 0.240E+04 0.270E+03		
: : * : : : 0.228E+04 0.240E+03		
: : * : : : 0.214E+04 0.210E+03		
: : * : : : 0.198E+04 0.180E+03		
150 +-----+-----+-----+-----+-----+	0.182E+04 0.150E+03	
: : * : : : 0.166E+04 0.120E+03		
: : * : : : 0.149E+04 0.900E+02		
: : * : : : 0.132E+04 0.600E+02		
: : * : : : 0.117E+04 0.300E+02		
0 +-----+-----+-----+-----+-----+	0.993E+03 0.000E+00	
0 720 1440 2160 2880		

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL A7 4 A7 5 C7 5

WIDTH = 14.000 IN HEIGHT = 17.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

5.380 .880
 -5.380 .880

P0 = 0.121E+04 K

PB = 0.492E+03 MB = 0.292E+04 K IN CB = 0.970E+01 IN

LOAD/LINE = 31 K MOMENT/SP = 74 K IN

P (KIPS)					MOMENT	LOAD
1240	+	-----	+	-----	+	-----
	:	*	:	:	:	0.923E+02 0.121E+04
	:	:	:	:	:	0.338E+03 0.118E+04
	:	*	:	:	:	0.573E+03 0.115E+04
	:	:	*	:	:	0.796E+03 0.112E+04
1085	+	-----	+	-----	+	-----
	:	:	:	*	:	0.101E+04 0.108E+04
	:	:	:	*	:	0.120E+04 0.105E+04
	:	:	:	*	:	0.139E+04 0.102E+04
	:	:	:	*	:	0.156E+04 0.992E+03
	:	:	:	*	:	0.172E+04 0.961E+03
930	+	-----	+	-----	+	-----
	:	:	:	*	:	0.187E+04 0.930E+03
	:	:	:	*	:	0.201E+04 0.899E+03
	:	:	:	*	:	0.213E+04 0.868E+03
	:	:	:	*	:	0.225E+04 0.837E+03
	:	:	:	*	:	0.235E+04 0.806E+03
775	+	-----	+	-----	+	-----
	:	:	:	*	:	0.245E+04 0.775E+03
	:	:	:	*	:	0.253E+04 0.744E+03
	:	:	:	*	:	0.261E+04 0.713E+03
	:	:	:	*	:	0.268E+04 0.682E+03
	:	:	:	*	:	0.274E+04 0.651E+03
620	+	-----	+	-----	+	-----
	:	:	:	*	:	0.279E+04 0.620E+03
	:	:	:	*	:	0.283E+04 0.589E+03
	:	:	:	*	:	0.287E+04 0.558E+03
	:	:	:	*	:	0.290E+04 0.527E+03
	:	:	:	*	:	0.292E+04 0.496E+03
465	+	-----	+	-----	+	-----
	:	:	:	*	:	0.289E+04 0.465E+03
	:	:	:	*	:	0.284E+04 0.434E+03
	:	:	:	*	:	0.278E+04 0.403E+03
	:	:	:	*	:	0.270E+04 0.372E+03
	:	:	:	*	:	0.260E+04 0.341E+03
310	+	-----	+	-----	+	-----
	:	:	:	*	:	0.249E+04 0.310E+03
	:	:	:	*	:	0.237E+04 0.279E+03
	:	:	:	*	:	0.223E+04 0.248E+03
	:	:	:	*	:	0.207E+04 0.217E+03
	:	:	:	*	:	0.190E+04 0.186E+03
155	+	-----	+	-----	+	-----
	:	:	:	*	:	0.173E+04 0.155E+03
	:	:	:	*	:	0.156E+04 0.124E+03
	:	:	:	*	:	0.137E+04 0.930E+02
	:	:	:	*	:	0.119E+04 0.620E+02
	:	:	:	*	:	0.103E+04 0.310E+02
0	+	-----	+	-----	+	-----
	:	:	:	*	:	0.840E+03 0.000E+00
0		740	1480	2220	2960	
						MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 1

WIDTH = 20.000 IN HEIGHT = 20.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

6.920 4.680
 .000 3.120
 -6.920 4.680

P0 = 0.259E+04 K

PB = 0.927E+03 MB = 0.869E+04 K IN CB = 0.118E+02 IN

LOAD/LINE = 65 K MOMENT/SP = 218 K IN

P (KIPS)						MOMENT	LOAD
2600	+	+	+	+	+		
:	*	:	:	:	:	0.967E+03	0.254E+04
:	*	:	:	:	:	0.149E+04	0.247E+04
:	*	:	:	:	:	0.204E+04	0.240E+04
:	*	:	:	:	:	0.255E+04	0.234E+04
2275	+	+	+	+	+	0.304E+04	0.227E+04
:	:	*	:	:	:	0.349E+04	0.221E+04
:	:	*	:	:	:	0.393E+04	0.215E+04
:	:	*	:	:	:	0.434E+04	0.208E+04
:	:	*	:	:	:	0.472E+04	0.202E+04
1950	+	+	+	+	+	0.509E+04	0.195E+04
:	:	:	*	:	:	0.543E+04	0.189E+04
:	:	:	*	:	:	0.576E+04	0.182E+04
:	:	:	*	:	:	0.606E+04	0.175E+04
:	:	:	*	:	:	0.632E+04	0.169E+04
1625	+	+	+	+	+	0.655E+04	0.163E+04
:	:	:	:	*	:	0.679E+04	0.156E+04
:	:	:	:	*	:	0.701E+04	0.150E+04
:	:	:	:	*	:	0.723E+04	0.143E+04
:	:	:	:	*	:	0.743E+04	0.136E+04
1300	+	+	+	+	+	0.763E+04	0.130E+04
:	:	:	:	*	:	0.783E+04	0.124E+04
:	:	:	:	*	:	0.802E+04	0.117E+04
:	:	:	:	*	:	0.820E+04	0.110E+04
:	:	:	:	*	:	0.839E+04	0.104E+04
975	+	+	+	+	+	0.856E+04	0.975E+03
:	:	:	:	*	:	0.868E+04	0.910E+03
:	:	:	:	*	:	0.863E+04	0.845E+03
:	:	:	:	*	:	0.856E+04	0.780E+03
:	:	:	:	*	:	0.847E+04	0.715E+03
650	+	+	+	+	+	0.837E+04	0.650E+03
:	:	:	:	*	:	0.826E+04	0.585E+03
:	:	:	:	*	:	0.814E+04	0.520E+03
:	:	:	:	*	:	0.802E+04	0.455E+03
:	:	:	:	*	:	0.789E+04	0.390E+03
325	+	+	+	+	+	0.776E+04	0.325E+03
:	:	:	:	*	:	0.751E+04	0.260E+03
:	:	:	:	*	:	0.712E+04	0.195E+03
:	:	:	:	*	:	0.673E+04	0.130E+03
:	:	:	:	*	:	0.628E+04	0.650E+02
0	+	+	+	+	+	0.584E+04	0.000E+00
0		2180	4360	6540	8720		
						MOMENT (K IN)	

C O L U M N I N T E R A C T I O N D I A G

COL B7 2

WIDTH = 16.000 IN

HEIGHT = 20.000 IN

CONC STR = 5.500 KSI

STL YIELD = 62.500 KSI

EU = .0050

DIST FROM CG AREA

6.990 3.810

.000 2.540

-6.990 3.810

P0 = 0.208E+04 K

PB = 0.746E+03 MB = 0.705E+04 K IN CB = 0.119E+02 IN

LOAD/LINE = 53 K MOMENT/SP = 177 K IN

P (KIPS)					MOMENT	LOAD
2120	+	-----	+	-----	+	-----
	:	*	:	:	:	0.521E+03 0.207E+04
	:		*	:	:	0.961E+03 0.201E+04
	:			*	:	0.142E+04 0.196E+04
	:			*	:	0.185E+04 0.191E+04
1855	+	-----	+	-----	+	-----
	:			*	:	0.226E+04 0.185E+04
	:			*	:	0.264E+04 0.180E+04
	:			*	:	0.301E+04 0.175E+04
	:			*	:	0.335E+04 0.170E+04
	:			*	:	0.368E+04 0.164E+04
1590	+	-----	+	-----	+	-----
	:			*	:	0.398E+04 0.159E+04
	:			*	:	0.427E+04 0.154E+04
	:			*	:	0.455E+04 0.148E+04
	:			*	:	0.481E+04 0.143E+04
	:			*	:	0.502E+04 0.138E+04
1325	+	-----	+	-----	+	-----
	:			*	:	0.522E+04 0.132E+04
	:			*	:	0.542E+04 0.127E+04
	:			*	:	0.561E+04 0.122E+04
	:			*	:	0.579E+04 0.117E+04
	:			*	:	0.597E+04 0.111E+04
1060	+	-----	+	-----	+	-----
	:			*	:	0.614E+04 0.106E+04
	:			*	:	0.630E+04 0.101E+04
	:			*	:	0.646E+04 0.954E+03
	:			*	:	0.661E+04 0.901E+03
	:			*	:	0.676E+04 0.848E+03
795	+	-----	+	-----	+	-----
	:			*	:	0.691E+04 0.795E+03
	:			*	:	0.704E+04 0.742E+03
	:			*	:	0.700E+04 0.689E+03
	:			*	:	0.695E+04 0.636E+03
	:			*	:	0.688E+04 0.583E+03
530	+	-----	+	-----	+	-----
	:			*	:	0.680E+04 0.530E+03
	:			*	:	0.672E+04 0.477E+03
	:			*	:	0.662E+04 0.424E+03
	:			*	:	0.652E+04 0.371E+03
	:			*	:	0.642E+04 0.318E+03
265	+	-----	+	-----	+	-----
	:			*	:	0.631E+04 0.265E+03
	:			*	:	0.614E+04 0.212E+03
	:			*	:	0.583E+04 0.159E+03
	:			*	:	0.550E+04 0.106E+03
	:			*	:	0.512E+04 0.530E+02
0	+	-----	+	-----	+	-----
	:			*	:	0.478E+04 0.000E+00
	0	1770	3540	5310	7080	
	MOMENT (K IN)					

C O L U M N I N T E R A C T I O N D I A G

COL B7 3

WIDTH = 16.000 IN HEIGHT = 18.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050

DIST FROM CG AREA

5.990 2.540

.000 2.540

-5.990 2.540

P0 = 0.179E+04 K

PB = 0.659E+03 MB = 0.490E+04 K IN CB = 0.105E+02 IN

LOAD/LINE = 45 K MOMENT/SP = 123 K IN

P (KIPS)					MOMENT	LOAD
1800	+	+	+	+	+	+
:	*	:	:	:	0.516E+03	0.175E+04
:	*	:	:	:	0.855E+03	0.171E+04
:	*	:	:	:	0.120E+04	0.167E+04
:	*	:	:	:	0.151E+04	0.162E+04
1575	+	+	+	+	+	+
:	:	*	:	:	0.181E+04	0.157E+04
:	:	*	:	:	0.209E+04	0.153E+04
:	:	*	:	:	0.236E+04	0.149E+04
:	:	*	:	:	0.261E+04	0.144E+04
:	:	*	:	:	0.284E+04	0.140E+04
1350	+	+	+	+	+	+
:	:	:	*	:	0.305E+04	0.135E+04
:	:	:	*	:	0.326E+04	0.130E+04
:	:	:	*	:	0.345E+04	0.126E+04
:	:	:	*	:	0.360E+04	0.122E+04
:	:	:	*	:	0.374E+04	0.117E+04
1125	+	+	+	+	+	+
:	:	:	:	*	0.388E+04	0.113E+04
:	:	:	:	*	0.400E+04	0.108E+04
:	:	:	:	*	0.412E+04	0.103E+04
:	:	:	:	*	0.423E+04	0.990E+03
:	:	:	:	*	0.433E+04	0.945E+03
900	+	+	+	+	+	+
:	:	:	:	*	0.443E+04	0.900E+03
:	:	:	:	*	0.453E+04	0.855E+03
:	:	:	:	*	0.462E+04	0.810E+03
:	:	:	:	*	0.471E+04	0.765E+03
:	:	:	:	*	0.479E+04	0.720E+03
675	+	+	+	+	+	+
:	:	:	:	*	0.488E+04	0.675E+03
:	:	:	:	*	0.488E+04	0.630E+03
:	:	:	:	*	0.484E+04	0.585E+03
:	:	:	:	*	0.480E+04	0.540E+03
:	:	:	:	*	0.474E+04	0.495E+03
450	+	+	+	+	+	+
:	:	:	:	*	0.468E+04	0.450E+03
:	:	:	:	*	0.462E+04	0.405E+03
:	:	:	:	*	0.455E+04	0.360E+03
:	:	:	:	*	0.447E+04	0.315E+03
:	:	:	:	*	0.440E+04	0.270E+03
225	+	+	+	+	+	+
:	:	:	:	*	0.432E+04	0.225E+03
:	:	:	:	*	0.418E+04	0.180E+03
:	:	:	:	*	0.399E+04	0.135E+03
:	:	:	:	*	0.371E+04	0.900E+02
:	:	:	*	:	0.343E+04	0.450E+02
0	+	+	+	+	+	+
:	:	:	:	*	0.317E+04	0.000E+00
0		1230	2460	3690	4920	

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 4 B7 5

WIDTH = 16.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

5.130 1.580

-5.130 1.580

P0 = 0.138E+04 K

PB = 0.532E+03 MB = 0.338E+04 K IN CB = 0.918E+01 IN

LOAD/LINE = 35 K MOMENT/SP = 85 K IN

P (KIPS)					MOMENT	LOAD
1400	+	+	+	+	+	+
	:	*	:	:	:	0.192E+03 0.136E+04
	:	:	*	:	:	0.442E+03 0.133E+04
	:	:	:	*	:	0.687E+03 0.129E+04
	:	:	:	:	*	0.918E+03 0.126E+04
1225	+	+	+	+	+	+
	:	:	:	*	:	0.114E+04 0.123E+04
	:	:	:	:	*	0.134E+04 0.119E+04
	:	:	:	:	*	0.153E+04 0.116E+04
	:	:	:	*	:	0.172E+04 0.112E+04
	:	:	:	:	*	0.189E+04 0.108E+04
1050	+	+	+	+	+	+
	:	:	:	:	*	0.204E+04 0.105E+04
	:	:	:	*	:	0.219E+04 0.102E+04
	:	:	:	:	*	0.233E+04 0.980E+03
	:	:	:	*	:	0.246E+04 0.945E+03
	:	:	:	:	*	0.257E+04 0.910E+03
875	+	+	+	+	+	+
	:	:	:	:	*	0.268E+04 0.875E+03
	:	:	:	:	*	0.278E+04 0.840E+03
	:	:	:	:	*	0.287E+04 0.805E+03
	:	:	:	:	*	0.296E+04 0.770E+03
	:	:	:	:	*	0.303E+04 0.735E+03
700	+	+	+	+	+	+
	:	:	:	:	*	0.310E+04 0.700E+03
	:	:	:	:	*	0.317E+04 0.665E+03
	:	:	:	:	*	0.323E+04 0.630E+03
	:	:	:	:	*	0.329E+04 0.595E+03
	:	:	:	:	*	0.334E+04 0.560E+03
525	+	+	+	+	+	+
	:	:	:	:	*	0.337E+04 0.525E+03
	:	:	:	:	*	0.333E+04 0.490E+03
	:	:	:	:	*	0.327E+04 0.455E+03
	:	:	:	:	*	0.319E+04 0.420E+03
	:	:	:	:	*	0.310E+04 0.385E+03
350	+	+	+	+	+	+
	:	:	:	:	*	0.299E+04 0.350E+03
	:	:	:	:	*	0.287E+04 0.315E+03
	:	:	:	:	*	0.273E+04 0.280E+03
	:	:	:	:	*	0.256E+04 0.245E+03
	:	:	:	*	:	0.238E+04 0.210E+03
175	+	+	+	+	+	+
	:	:	:	*	:	0.221E+04 0.175E+03
	:	:	:	*	:	0.202E+04 0.140E+03
	:	:	:	*	:	0.184E+04 0.105E+03
	:	:	*	:	:	0.165E+04 0.700E+02
	:	:	*	:	:	0.146E+04 0.350E+02
0	+	+	+	+	+	+
	:	:	:	*	:	0.129E+04 0.000E+00
0		850	1700	2550	3400	
			MOMENT (K IN)			

C O L U M N I N T E R A C T I O N D I A G

COL C7 1

WIDTH = 14.000 IN HEIGHT = 17.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

5.490 2.540
 .000 2.540
 -5.470 2.540

P0 = 0.155E+04 K

PB = 0.543E+03 MB = 0.408E+04 K IN CB = 0.976E+01 IN

LOAD/LINE = 39 K MOMENT/SP = 102 K IN

P (KIPS)						MOMENT	LOAD
1560	+	+	+	+	+		
:	*	:	:	:	:	0.489E+03	0.152E+04
:	*	:	:	:	:	0.757E+03	0.148E+04
:	*	:	:	:	:	0.103E+04	0.144E+04
:	*	:	:	:	:	0.128E+04	0.140E+04
1365	+	+	+	+	+	0.152E+04	0.136E+04
:	*	:	:	:	:	0.174E+04	0.133E+04
:	*	:	:	:	:	0.195E+04	0.129E+04
:	*	:	:	:	:	0.215E+04	0.125E+04
:	*	:	:	:	:	0.234E+04	0.121E+04
1170	+	+	+	+	+	0.251E+04	0.117E+04
:	*	:	:	:	:	0.267E+04	0.113E+04
:	*	:	:	:	:	0.283E+04	0.109E+04
:	*	:	:	:	:	0.294E+04	0.105E+04
:	*	:	:	:	:	0.305E+04	0.101E+04
975	+	+	+	+	+	0.316E+04	0.975E+03
:	*	:	:	:	:	0.326E+04	0.936E+03
:	*	:	:	:	:	0.336E+04	0.897E+03
:	*	:	:	:	:	0.345E+04	0.858E+03
:	*	:	:	:	:	0.354E+04	0.819E+03
780	+	+	+	+	+	0.362E+04	0.780E+03
:	*	:	:	:	:	0.370E+04	0.741E+03
:	*	:	:	:	:	0.378E+04	0.702E+03
:	*	:	:	:	:	0.386E+04	0.663E+03
:	*	:	:	:	:	0.393E+04	0.624E+03
585	+	+	+	+	+	0.400E+04	0.585E+03
:	*	:	:	:	:	0.407E+04	0.546E+03
:	*	:	:	:	:	0.405E+04	0.507E+03
:	*	:	:	:	:	0.402E+04	0.468E+03
:	*	:	:	:	:	0.398E+04	0.429E+03
390	+	+	+	+	+	0.394E+04	0.390E+03
:	*	:	:	:	:	0.389E+04	0.351E+03
:	*	:	:	:	:	0.384E+04	0.312E+03
:	*	:	:	:	:	0.379E+04	0.273E+03
:	*	:	:	:	:	0.373E+04	0.234E+03
195	+	+	+	+	+	0.368E+04	0.195E+03
:	*	:	:	:	:	0.362E+04	0.156E+03
:	*	:	:	:	:	0.350E+04	0.117E+03
:	*	:	:	:	:	0.333E+04	0.780E+02
:	*	:	:	:	:	0.311E+04	0.390E+02
0	+	+	+	+	+	0.289E+04	0.000E+00
0		1020	2040	3060	4080		
						MOMENT (K IN)	

COLUMN INTERACTION DIAG

COL C7 2

WIDTH = 14.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

4.990 2.540
 .000 2.540
 -4.990 2.540

P0 = 0.149E+04 K

PB = 0.504E+03 MB = 0.365E+04 K IN CB = 0.908E+01 IN

LOAD/LINE = 38 K MOMENT/SP = 92 K IN

P (KIPS)					MOMENT	LOAD
1520	+	-----	+	-----	+	-----
	:	*	:	:	:	0.270E+03 0.148E+04
	:		:	:	:	0.520E+03 0.144E+04
	:	*	:	:	:	0.775E+03 0.141E+04
	:		:	:	:	0.101E+04 0.137E+04
	:		:	:	:	0.124E+04 0.133E+04
1330	+	-----	+	-----	+	-----
	:		:	:	:	0.145E+04 0.129E+04
	:		:	:	:	0.164E+04 0.125E+04
	:		:	:	:	0.183E+04 0.122E+04
	:		:	:	:	0.200E+04 0.118E+04
1140	+	-----	+	-----	+	-----
	:		:	:	:	0.216E+04 0.114E+04
	:		:	:	:	0.231E+04 0.110E+04
	:		:	:	:	0.246E+04 0.106E+04
	:		:	:	:	0.257E+04 0.103E+04
	:		:	:	:	0.267E+04 0.988E+03
950	+	-----	+	-----	+	-----
	:		:	:	:	0.277E+04 0.950E+03
	:		:	:	:	0.287E+04 0.912E+03
	:		:	:	:	0.295E+04 0.874E+03
	:		:	:	:	0.304E+04 0.836E+03
	:		:	:	:	0.312E+04 0.798E+03
760	+	-----	+	-----	+	-----
	:		:	:	:	0.319E+04 0.760E+03
	:		:	:	:	0.327E+04 0.722E+03
	:		:	:	:	0.334E+04 0.684E+03
	:		:	:	:	0.341E+04 0.646E+03
	:		:	:	:	0.347E+04 0.608E+03
570	+	-----	+	-----	+	-----
	:		:	:	:	0.354E+04 0.570E+03
	:		:	:	:	0.360E+04 0.532E+03
	:		:	:	:	0.364E+04 0.494E+03
	:		:	:	:	0.362E+04 0.456E+03
	:		:	:	:	0.358E+04 0.418E+03
380	+	-----	+	-----	+	-----
	:		:	:	:	0.355E+04 0.380E+03
	:		:	:	:	0.350E+04 0.342E+03
	:		:	:	:	0.346E+04 0.304E+03
	:		:	:	:	0.342E+04 0.266E+03
	:		:	:	:	0.337E+04 0.228E+03
190	+	-----	+	-----	+	-----
	:		:	:	:	0.332E+04 0.190E+03
	:		:	:	:	0.327E+04 0.152E+03
	:		:	:	:	0.321E+04 0.114E+03
	:		:	:	:	0.303E+04 0.760E+02
	:		:	:	:	0.285E+04 0.380E+02
0	+	-----	+	-----	+	-----
	:		:	:	:	0.266E+04 0.000E+00
0		920	1840	2760	3680	

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL C7 3

WIDTH = 14.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

5.130 2.000

-5.130 2.000

P0 = 0.128E+04 K

PB = 0.465E+03 MB = 0.335E+04 K IN CB = 0.918E+01 IN

LOAD/LINE = 32 K MOMENT/SP = 84 K IN

P (KIPS)					MOMENT	LOAD
1280	+	-	-	-	+	+
:	*	:	:	:	:	0.331E+03 0.125E+04
:	*	:	:	:	:	0.546E+03 0.122E+04
:	*	:	:	:	:	0.761E+03 0.118E+04
:	*	:	:	:	:	0.964E+03 0.115E+04
1120	+	-	-	-	+	+
:	:	*	:	:	:	0.116E+04 0.112E+04
:	:	*	:	:	:	0.134E+04 0.109E+04
:	:	*	:	:	:	0.151E+04 0.106E+04
:	:	*	:	:	:	0.167E+04 0.102E+04
:	:	*	:	:	:	0.182E+04 0.992E+03
960	+	-	-	-	+	+
:	:	*	:	:	:	0.196E+04 0.960E+03
:	:	*	:	:	:	0.210E+04 0.928E+03
:	:	*	:	:	:	0.222E+04 0.896E+03
:	:	*	:	:	:	0.234E+04 0.864E+03
:	:	*	:	:	:	0.245E+04 0.832E+03
800	+	-	-	-	+	+
:	:	*	:	:	:	0.255E+04 0.800E+03
:	:	*	:	:	:	0.265E+04 0.768E+03
:	:	*	:	:	:	0.274E+04 0.736E+03
:	:	*	:	:	:	0.283E+04 0.704E+03
:	:	*	:	:	:	0.291E+04 0.672E+03
640	+	-	-	-	+	+
:	:	*	:	:	:	0.299E+04 0.640E+03
:	:	*	:	:	:	0.306E+04 0.608E+03
:	:	*	:	:	:	0.313E+04 0.576E+03
:	:	*	:	:	:	0.320E+04 0.544E+03
:	:	*	:	:	:	0.326E+04 0.512E+03
480	+	-	-	-	+	+
:	:	*	:	:	:	0.332E+04 0.480E+03
:	:	*	:	:	:	0.334E+04 0.448E+03
:	:	*	:	:	:	0.329E+04 0.416E+03
:	:	*	:	:	:	0.323E+04 0.384E+03
:	:	*	:	:	:	0.315E+04 0.352E+03
320	+	-	-	-	+	+
:	:	*	:	:	:	0.306E+04 0.320E+03
:	:	*	:	:	:	0.295E+04 0.288E+03
:	:	*	:	:	:	0.283E+04 0.256E+03
:	:	*	:	:	:	0.269E+04 0.224E+03
:	:	*	:	:	:	0.253E+04 0.192E+03
160	+	-	-	-	+	+
:	:	*	:	:	:	0.236E+04 0.160E+03
:	:	*	:	:	:	0.220E+04 0.128E+03
:	:	*	:	:	:	0.203E+04 0.960E+02
:	:	*	:	:	:	0.186E+04 0.640E+02
:	:	*	:	:	:	0.169E+04 0.320E+02
0	+	-	-	-	+	+
:	:	*	:	:	:	0.152E+04 0.000E+00
0		840	1680	2520	3360	
			MOMENT (K IN)			

Appendix D

N-S Column Interaction Diagrams

Code Material Properties

C O L U M N I N T E R A C T I O N D I A G

COL A7 1

WIDTH = 17.000 IN HEIGHT = 14.000 IN CONC STR = 3.000 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 28.500 KSI EU = .0030

DIST FROM CG AREA

4.060 3.000

-4.060 3.000

P0 = 0.892E+03 K

PB = 0.390E+03 MB = 0.201E+04 K IN CB = 0.833E+01 IN

LOAD/LINE = 23 K MOMENT/SP = 51 K IN

P (KIPS)

MOMENT

LOAD

C O L U M N I N T E R A C T I O N D I A G

COL A7 2

WIDTH = 17.000 IN HEIGHT = 14.000 IN CONC STR = 5.500 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 32.000 KSI EU = .0030
DIST FROM CG AREA

4.130 2.370

-4.130 2.370

P0 = 0.133E+04 K

PB = 0.544E+03 MB = 0.273E+04 K IN CB = 0.814E+01 IN

LOAD/LINE = 34 K MOMENT/SP = 69 K IN

P (KIPS)					MOMENT	LOAD
1360	+	+	+	+	+	+
	:	*	:	:	:	0.104E+03 0.133E+04
	:	:	:	:	:	0.283E+03 0.129E+04
	:	:	*	:	:	0.500E+03 0.126E+04
	:	:	:	:	:	0.704E+03 0.122E+04
1190	+	+	+	+	+	+
	:	:	:	*	:	0.895E+03 0.119E+04
	:	:	:	:	:	0.108E+04 0.116E+04
	:	:	:	*	:	0.124E+04 0.112E+04
	:	:	:	:	*	0.140E+04 0.109E+04
	:	:	:	:	*	0.155E+04 0.105E+04
1020	+	+	+	+	+	+
	:	:	:	:	*	0.169E+04 0.102E+04
	:	:	:	:	*	0.181E+04 0.986E+03
	:	:	:	:	*	0.193E+04 0.952E+03
	:	:	:	:	*	0.204E+04 0.918E+03
	:	:	:	:	*	0.213E+04 0.884E+03
850	+	+	+	+	+	+
	:	:	:	:	*	0.222E+04 0.850E+03
	:	:	:	:	*	0.231E+04 0.816E+03
	:	:	:	:	*	0.238E+04 0.782E+03
	:	:	:	:	*	0.245E+04 0.748E+03
	:	:	:	:	*	0.251E+04 0.714E+03
680	+	+	+	+	+	+
	:	:	:	:	*	0.256E+04 0.680E+03
	:	:	:	:	*	0.261E+04 0.646E+03
	:	:	:	:	*	0.265E+04 0.612E+03
	:	:	:	:	*	0.269E+04 0.578E+03
	:	:	:	:	*	0.273E+04 0.544E+03
510	+	+	+	+	+	+
	:	:	:	:	*	0.270E+04 0.510E+03
	:	:	:	:	*	0.265E+04 0.476E+03
	:	:	:	:	*	0.259E+04 0.442E+03
	:	:	:	:	*	0.250E+04 0.408E+03
	:	:	:	:	*	0.240E+04 0.374E+03
340	+	+	+	+	+	+
	:	:	:	:	*	0.230E+04 0.340E+03
	:	:	:	:	*	0.218E+04 0.306E+03
	:	:	:	:	*	0.206E+04 0.272E+03
	:	:	:	:	*	0.193E+04 0.238E+03
	:	:	:	:	*	0.179E+04 0.204E+03
170	+	+	+	+	+	+
	:	:	:	:	*	0.164E+04 0.170E+03
	:	:	:	:	*	0.150E+04 0.136E+03
	:	:	:	:	*	0.135E+04 0.102E+03
	:	:	:	:	*	0.119E+04 0.680E+02
	:	:	:	:	*	0.104E+04 0.340E+02
0	+	+	+	+	+	+
	:	:	:	:	*	0.902E+03 0.000E+00
0		690	1380	2070	2760	

MOMENT (K IN)

COL A7 4 A7 5 C7 5

STL YIELD C= 50.000 KSI STL YIELD T= 43.000 KSI EU = .0030

STL YIELD C= 50.000 KSI

STL YIELD T= 43.000 KSI

EU = .0030

DIST FROM CG	AREA
0-100	100
100-200	200
200-300	300
300-400	400
400-500	500
500-600	600
600-700	700
700-800	800
800-900	900
900-1000	1000

4.380 .880

-4.380 .880

P0 = 0.113E+04 K

PB = 0.448E+03 MB = 0.215E+04 K IN CB = 0.762E+01 IN

LOAD/LINE = 29 K MOMENT/SP = 54 K IN

P (KIPS)

MOMENT

LOAD

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 1

WIDTH = 20.000 IN HEIGHT = 20.000 IN CONC STR = 5.500 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 22.800 KSI EU = .0030

DIST FROM CG AREA

6.920 4.680

.000 3.120

-6.920 4.680

P0 = 0.244E+04 K

PB = 0.117E+04 MB = 0.703E+04 K IN CB = 0.134E+02 IN

LOAD/LINE = 61 K MOMENT/SP = 176 K IN

P (KIPS)						MOMENT	LOAD
2440	+	+	+	+	+		
:	*	:	:	:	:	0.871E+03	0.238E+04
:	:	*	:	:	:	0.141E+04	0.232E+04
:	:	:	*	:	:	0.194E+04	0.226E+04
:	:	:	:	*	:	0.244E+04	0.220E+04
2135	+	+	+	+	+	0.290E+04	0.213E+04
:	:	:	*	:	:	0.330E+04	0.207E+04
:	:	:	:	*	:	0.369E+04	0.201E+04
:	:	:	:	*	:	0.406E+04	0.195E+04
:	:	:	:	*	:	0.440E+04	0.189E+04
1830	+	+	+	+	+	0.472E+04	0.183E+04
:	:	:	:	*	:	0.502E+04	0.177E+04
:	:	:	:	*	:	0.529E+04	0.171E+04
:	:	:	:	*	:	0.555E+04	0.165E+04
:	:	:	:	*	:	0.579E+04	0.159E+04
1525	+	+	+	+	+	0.601E+04	0.152E+04
:	:	:	:	*	:	0.622E+04	0.146E+04
:	:	:	:	*	:	0.641E+04	0.140E+04
:	:	:	:	*	:	0.658E+04	0.134E+04
:	:	:	:	*	:	0.675E+04	0.128E+04
1220	+	+	+	+	+	0.690E+04	0.122E+04
:	:	:	:	*	:	0.703E+04	0.116E+04
:	:	:	:	*	:	0.703E+04	0.110E+04
:	:	:	:	*	:	0.701E+04	0.104E+04
:	:	:	:	*	:	0.696E+04	0.976E+03
915	+	+	+	+	+	0.689E+04	0.915E+03
:	:	:	:	*	:	0.680E+04	0.854E+03
:	:	:	:	*	:	0.669E+04	0.793E+03
:	:	:	:	*	:	0.656E+04	0.732E+03
:	:	:	:	*	:	0.643E+04	0.671E+03
610	+	+	+	+	+	0.628E+04	0.610E+03
:	:	:	:	*	:	0.595E+04	0.549E+03
:	:	:	:	*	:	0.563E+04	0.488E+03
:	:	:	:	*	:	0.526E+04	0.427E+03
:	:	:	:	*	:	0.488E+04	0.366E+03
305	+	+	+	+	+	0.448E+04	0.305E+03
:	:	:	:	*	:	0.406E+04	0.244E+03
:	:	:	:	*	:	0.364E+04	0.183E+03
:	:	:	:	*	:	0.325E+04	0.122E+03
:	:	:	:	*	:	0.279E+04	0.610E+02
0	+	+	+	+	+	0.238E+04	0.000E+00
0		1760	3520	5280	7040		

MOMENT (K IN)

COL B7 2

STL YIELD C= 50.000 KSI STL YIELD T= 25.300 KSI EU = .0030

DIST FROM CG	AREA
0	0
1	1
2	4
3	9
4	16
5	25
6	36
7	49
8	64
9	81
10	100
11	121
12	144
13	169
14	196
15	225
16	256
17	289
18	324
19	361
20	400
21	441
22	484
23	529
24	576
25	625
26	676
27	729
28	784
29	841
30	900
31	961
32	1024
33	1089
34	1156
35	1225
36	1296
37	1369
38	1444
39	1521
40	1600
41	1681
42	1764
43	1849
44	1936
45	2025
46	2116
47	2209
48	2304
49	2401
50	2500
51	2601
52	2704
53	2809
54	2916
55	3025
56	3136
57	3249
58	3364
59	3481
60	3600
61	3721
62	3844
63	3969
64	4096
65	4225
66	4356
67	4489
68	4624
69	4761
70	4900
71	5041
72	5184
73	5329
74	5476
75	5625
76	5776
77	5929
78	6084
79	6241
80	6400
81	6561
82	6724
83	6889
84	7056
85	7225
86	7396
87	7569
88	7744
89	7921
90	8100
91	8281
92	8464
93	8649
94	8836
95	9025
96	9216
97	9409
98	9604
99	9801
100	10000

4.990	3.810
-------	-------

.000	2.540
------	-------

-4.990 3.810

PO = 0.196E+04 K

PB = 0.869E+03 MB = 0.442E+04 K IN CB = 0.101E+02 IN

LOAD/LINE = 49 K MOMENT/SP = 111 K IN

P (KIPS)

MOMENT

LOAD

[illegible]

MOMENT (K IN)

COLUMN INTERACTION DIAG

COL B7 3

WIDTH = 18.000 IN HEIGHT = 16.000 IN CONC STR = 5.500 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 25.300 KSI EU = .0030

DIST FROM CG AREA

4.990 3.810

-4.990 3.810

P0 = 0.169E+04 K

PB = 0.750E+03 MB = 0.412E+04 K IN CB = 0.101E+02 IN

LOAD/LINE = 43 K MOMENT/SP = 104 K IN

P (KIPS)					MOMENT	LOAD
1720	+	-----	+	-----	+	-----
:	*	:	:	:	:	0.257E+03 0.168E+04
:	:	*	:	:	:	0.528E+03 0.163E+04
:	:	:	*	:	:	0.840E+03 0.159E+04
:	:	:	:	*	:	0.113E+04 0.155E+04
1505	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.141E+04 0.150E+04
:	:	:	:	*	:	0.166E+04 0.146E+04
:	:	:	:	*	:	0.191E+04 0.142E+04
:	:	:	:	*	:	0.214E+04 0.138E+04
:	:	:	:	*	:	0.235E+04 0.133E+04
1290	+	-----	+	-----	+	-----
:	:	:	:	*	:	0.255E+04 0.129E+04
:	:	:	:	*	:	0.273E+04 0.125E+04
:	:	:	:	*	:	0.291E+04 0.120E+04
:	:	:	:	*	:	0.307E+04 0.116E+04
:	:	:	:	*	:	0.322E+04 0.112E+04
1075	+	-----	+	-----	+	-----
:	:	:	:	*	:	0.335E+04 0.107E+04
:	:	:	:	*	:	0.348E+04 0.103E+04
:	:	:	:	*	:	0.360E+04 0.989E+03
:	:	:	:	*	:	0.371E+04 0.946E+03
:	:	:	:	*	:	0.381E+04 0.903E+03
860	+	-----	+	-----	+	-----
:	:	:	:	*	:	0.391E+04 0.860E+03
:	:	:	:	*	:	0.400E+04 0.817E+03
:	:	:	:	*	:	0.408E+04 0.774E+03
:	:	:	:	*	:	0.412E+04 0.731E+03
:	:	:	:	*	:	0.409E+04 0.688E+03
645	+	-----	+	-----	+	-----
:	:	:	:	*	:	0.404E+04 0.645E+03
:	:	:	:	*	:	0.396E+04 0.602E+03
:	:	:	:	*	:	0.387E+04 0.559E+03
:	:	:	:	*	:	0.373E+04 0.516E+03
:	:	:	:	*	:	0.357E+04 0.473E+03
430	+	-----	+	-----	+	-----
:	:	:	:	*	:	0.340E+04 0.430E+03
:	:	:	:	*	:	0.322E+04 0.387E+03
:	:	:	:	*	:	0.303E+04 0.344E+03
:	:	:	:	*	:	0.282E+04 0.301E+03
:	:	:	:	*	:	0.261E+04 0.258E+03
215	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.240E+04 0.215E+03
:	:	:	*	:	:	0.218E+04 0.172E+03
:	:	:	*	:	:	0.195E+04 0.129E+03
:	:	:	*	:	:	0.173E+04 0.860E+02
:	:	:	*	:	:	0.150E+04 0.430E+02
0	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.130E+04 0.000E+00
0		1040	2080	3120	4160	
			MOMENT (K IN)			

COL B7 4 B7 5

5.500 KSI

DIST FROM CG	AREA
0	0
1	1
2	4
3	9
4	16
5	25
6	36
7	49
8	64
9	81
10	100
11	121
12	144
13	169
14	196
15	225
16	256
17	289
18	324
19	361
20	400
21	441
22	484
23	529
24	576
25	625
26	676
27	729
28	784
29	841
30	900
31	961
32	1024
33	1089
34	1156
35	1225
36	1296
37	1369
38	1444
39	1521
40	1600
41	1681
42	1764
43	1849
44	1936
45	2025
46	2116
47	2209
48	2304
49	2401
50	2500
51	2601
52	2704
53	2809
54	2916
55	3025
56	3136
57	3249
58	3364
59	3481
60	3600
61	3721
62	3844
63	3969
64	4096
65	4225
66	4356
67	4489
68	4624
69	4761
70	4900
71	5041
72	5184
73	5329
74	5476
75	5625
76	5776
77	5929
78	6084
79	6241
80	6400
81	6561
82	6724
83	6889
84	7056
85	7225
86	7396
87	7569
88	7744
89	7921
90	8100
91	8281
92	8464
93	8649
94	8836
95	9025
96	9216
97	9409
98	9604
99	9801
100	10000

-5.130	1.580
--------	-------

P0 = 0.134E+04 K

PB = 0.585E+03 MB = 0.305E+04 K IN CB = 0.960E+01 IN

LOAD/LINE = 34 K MOMENT/SP = 77 K IN

P (KIPS)

MOMENT

LOAD

MOMENT (K IN)

COL C7 1

STL Y

3.990 3.810

-3.990 3.810

P0 = 0.146E+04 K

PB = 0.619E+03 MB = 0.309E+04 K IN CB = 0.851E+01 IN

LOAD/LINE = 37 K MOMENT/SP = 78 K IN

P (KIPS)

MOMENT

LOAD

1480	+	-----+	+	-----+	+	-----+	+	-----+	+	0.203E+03	0.144E+04
:	*	:	:	:	:	:	:	:	:	0.415E+03	0.141E+04
:	*	:	:	:	:	:	:	:	:	0.641E+03	0.137E+04
:	*	:	:	:	:	:	:	:	:	0.856E+03	0.133E+04
1295	+	-----+	+	-----+	+	-----+	+	-----+	+	0.106E+04	0.129E+04
:	:	:	*	:	:	:	:	:	:	0.125E+04	0.126E+04
:	:	:	*	:	:	:	:	:	:	0.142E+04	0.122E+04
:	:	:	*	:	:	:	:	:	:	0.159E+04	0.118E+04
:	:	:	*	:	:	:	:	:	:	0.174E+04	0.115E+04
1110	+	-----+	+	-----+	+	-----+	+	-----+	+	0.189E+04	0.111E+04
:	:	:	*	:	:	:	:	:	:	0.202E+04	0.107E+04
:	:	:	*	:	:	:	:	:	:	0.215E+04	0.104E+04
:	:	:	*	:	:	:	:	:	:	0.227E+04	0.999E+03
:	:	:	*	:	:	:	:	:	:	0.238E+04	0.962E+03
925	+	-----+	+	-----+	+	-----+	+	-----+	+	0.248E+04	0.925E+03
:	:	:	:	:	*	:	:	:	:	0.257E+04	0.888E+03
:	:	:	:	:	*	:	:	:	:	0.266E+04	0.851E+03
:	:	:	:	:	*	:	:	:	:	0.274E+04	0.814E+03
:	:	:	:	:	*	:	:	:	:	0.281E+04	0.777E+03
740	+	-----+	+	-----+	+	-----+	+	-----+	+	0.288E+04	0.740E+03
:	:	:	:	:	*	:	:	:	:	0.295E+04	0.703E+03
:	:	:	:	:	*	:	:	:	:	0.301E+04	0.666E+03
:	:	:	:	:	*	:	:	:	:	0.307E+04	0.629E+03
:	:	:	:	:	*	:	:	:	:	0.307E+04	0.592E+03
555	+	-----+	+	-----+	+	-----+	+	-----+	+	0.303E+04	0.555E+03
:	:	:	:	:	*	:	:	:	:	0.298E+04	0.518E+03
:	:	:	:	:	*	:	:	:	:	0.288E+04	0.481E+03
:	:	:	:	:	*	:	:	:	:	0.278E+04	0.444E+03
:	:	:	:	:	*	:	:	:	:	0.267E+04	0.407E+03
370	+	-----+	+	-----+	+	-----+	+	-----+	+	0.255E+04	0.370E+03
:	:	:	:	:	*	:	:	:	:	0.243E+04	0.333E+03
:	:	:	:	:	*	:	:	:	:	0.229E+04	0.296E+03
:	:	:	:	:	*	:	:	:	:	0.215E+04	0.259E+03
:	:	:	:	:	*	:	:	:	:	0.200E+04	0.222E+03
185	+	-----+	+	-----+	+	-----+	+	-----+	+	0.185E+04	0.185E+03
:	:	:	:	:	*	:	:	:	:	0.170E+04	0.148E+03
:	:	:	:	:	*	:	:	:	:	0.155E+04	0.111E+03
:	:	:	:	:	*	:	:	:	:	0.139E+04	0.740E+02
:	:	:	:	:	*	:	:	:	:	0.123E+04	0.370E+02
0	+	-----+	+	-----+	+	-----+	+	-----+	+	0.107E+04	0.000E+00
0	0	780	1560	2340	3120	MOMENT (K IN)					

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL C7 2

WIDTH = 16.000 IN HEIGHT = 14.000 IN CONC STR = 5.500 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 25.300 KSI EU = .0030
DIST FROM CG AREA

3.990 3.810

-3.990 3.810

P0 = 0.139E+04 K

PB = 0.588E+03 MB = 0.297E+04 K IN CB = 0.851E+01 IN

LOAD/LINE = 35 K MOMENT/SP = 75 K IN

P (KIPS)						MOMENT	LOAD
1400	+	-----	+	-----	+		
:	*	:	:	:	:	0.262E+03	0.136E+04
:	*	:	:	:	:	0.480E+03	0.133E+04
:	*	:	:	:	:	0.690E+03	0.129E+04
:	*	:	:	:	:	0.888E+03	0.126E+04
1225	+	-----	+	-----	+	0.107E+04	0.123E+04
:	*	:	:	:	:	0.125E+04	0.119E+04
:	*	:	:	:	:	0.141E+04	0.116E+04
:	*	:	:	:	:	0.157E+04	0.112E+04
:	*	:	:	:	:	0.171E+04	0.108E+04
1050	+	-----	+	-----	+	0.184E+04	0.105E+04
:	*	:	:	:	:	0.197E+04	0.102E+04
:	*	:	:	:	:	0.209E+04	0.980E+03
:	*	:	:	:	:	0.220E+04	0.945E+03
:	*	:	:	:	:	0.230E+04	0.910E+03
875	+	-----	+	-----	+	0.239E+04	0.875E+03
:	*	:	:	:	:	0.248E+04	0.840E+03
:	*	:	:	:	:	0.256E+04	0.805E+03
:	*	:	:	:	:	0.264E+04	0.770E+03
:	*	:	:	:	:	0.271E+04	0.735E+03
700	+	-----	+	-----	+	0.278E+04	0.700E+03
:	*	:	:	:	:	0.284E+04	0.665E+03
:	*	:	:	:	:	0.290E+04	0.630E+03
:	*	:	:	:	:	0.296E+04	0.595E+03
:	*	:	:	:	:	0.296E+04	0.560E+03
525	+	-----	+	-----	+	0.292E+04	0.525E+03
:	*	:	:	:	:	0.286E+04	0.490E+03
:	*	:	:	:	:	0.277E+04	0.455E+03
:	*	:	:	:	:	0.268E+04	0.420E+03
:	*	:	:	:	:	0.257E+04	0.385E+03
350	+	-----	+	-----	+	0.246E+04	0.350E+03
:	*	:	:	:	:	0.234E+04	0.315E+03
:	*	:	:	:	:	0.221E+04	0.280E+03
:	*	:	:	:	:	0.207E+04	0.245E+03
:	*	:	:	:	:	0.194E+04	0.210E+03
175	+	-----	+	-----	+	0.179E+04	0.175E+03
:	*	:	:	:	:	0.165E+04	0.140E+03
:	*	:	:	:	:	0.150E+04	0.105E+03
:	*	:	:	:	:	0.136E+04	0.700E+02
:	*	:	:	:	:	0.121E+04	0.350E+02
0	+	-----	+	-----	+	0.106E+04	0.000E+00
0		750	1500	2250	3000		

MOMENT (K IN)

COL C7 3

4.130 2.000

-4.130 2.000

P0 = 0.123E+04 K

PB = 0.508E+03 MB = 0.249E+04 K IN CB = 0.814E+01 IN

LOAD/LINE = 31 K MOMENT/SP = 63 K IN

P (KIPS)

MOMENT

LOAD

[illegible]

C O L U M N I N T E R A C T I O N D I A G

COL A7 3 C7 4

WIDTH = 16.000 IN HEIGHT = 14.000 IN CONC STR = 5.500 KSI

STL YIELD C= 50.000 KSI STL YIELD T= 36.900 KSI EU = .0030

DIST FROM CG AREA

4.310 1.200

-4.310 1.200

P0 = 0.116E+04 K

PB = 0.476E+03 MB = 0.226E+04 K IN CB = 0.794E+01 IN

LOAD/LINE = 29 K MOMENT/SP = 57 K IN

P (KIPS)					MOMENT	LOAD
1160	+	-----	+	-----	+	-----
:	*	:	:	:	0.194E+03	0.113E+04
:		*	:	:	0.381E+03	0.110E+04
:			*	:	0.560E+03	0.107E+04
:				*	0.728E+03	0.104E+04
1015	+	-----	+	-----	+	-----
:			*	:	0.886E+03	0.102E+04
:				*	0.103E+04	0.986E+03
:				*	0.117E+04	0.957E+03
:			*	:	0.130E+04	0.928E+03
:				*	0.142E+04	0.899E+03
870	+	-----	+	-----	+	-----
:				*	0.153E+04	0.870E+03
:				*	0.163E+04	0.841E+03
:				*	0.173E+04	0.812E+03
:				*	0.181E+04	0.783E+03
:				*	0.189E+04	0.754E+03
725	+	-----	+	-----	+	-----
:				*	0.196E+04	0.725E+03
:				*	0.202E+04	0.696E+03
:				*	0.207E+04	0.667E+03
:				*	0.211E+04	0.638E+03
:				*	0.215E+04	0.609E+03
580	+	-----	+	-----	+	-----
:				*	0.218E+04	0.580E+03
:				*	0.221E+04	0.551E+03
:				*	0.223E+04	0.522E+03
:				*	0.225E+04	0.493E+03
:				*	0.225E+04	0.464E+03
435	+	-----	+	-----	+	-----
:				*	0.221E+04	0.435E+03
:				*	0.216E+04	0.406E+03
:				*	0.210E+04	0.377E+03
:				*	0.203E+04	0.348E+03
:				*	0.195E+04	0.319E+03
290	+	-----	+	-----	+	-----
:				*	0.185E+04	0.290E+03
:				*	0.175E+04	0.261E+03
:			*	:	0.164E+04	0.232E+03
:			*	:	0.152E+04	0.203E+03
:			*	:	0.140E+04	0.174E+03
145	+	-----	+	-----	+	-----
:			*	:	0.127E+04	0.145E+03
:			*	:	0.113E+04	0.116E+03
:			*	:	0.992E+03	0.870E+02
:		*	:	:	0.851E+03	0.580E+02
:	*	:	:	:	0.726E+03	0.290E+02
0	+	-----	+	-----	+	-----
0		570	1140	1710	2280	
			MOMENT	(K IN)		

Appendix E

N-S Column Interaction Diagrams

Maximum Material Properties

C O L U M N I N T E R A C T I O N D I A G

COL A7 1

WIDTH = 17.000 IN HEIGHT = 14.000 IN
 CONC STR = 3.000 KSI STL YIELD = 50.000 KSI EU = .0050

DIST FROM CG AREA

4.060 3.000

-4.060 3.000

P0 = 0.892E+03 K

PB = 0.321E+03 MB = 0.228E+04 K IN CB = 0.822E+01 IN

LOAD/LINE = 23 K MOMENT/SP = 57 K IN

P (KIPS)					MOMENT	LOAD
920	+	-----	+	-----	+	
:	:	:	:	:	:	:
:	*	:	:	:	0.150E+03	0.874E+03
:	:	:	:	:	0.284E+03	0.851E+03
:	*	:	:	:	0.412E+03	0.828E+03
805	+	-----	+	-----	+	
:	:	*	:	:	0.534E+03	0.805E+03
:	:	:	:	:	0.650E+03	0.782E+03
:	:	*	:	:	0.762E+03	0.759E+03
:	:	:	*	:	0.869E+03	0.736E+03
:	:	:	*	:	0.971E+03	0.713E+03
690	+	-----	+	-----	+	
:	:	:	*	:	0.107E+04	0.690E+03
:	:	:	*	:	0.116E+04	0.667E+03
:	:	:	*	:	0.125E+04	0.644E+03
:	:	:	*	:	0.134E+04	0.621E+03
:	:	:	*	:	0.143E+04	0.598E+03
575	+	-----	+	-----	+	
:	:	:	*	:	0.151E+04	0.575E+03
:	:	:	*	:	0.159E+04	0.552E+03
:	:	:	*	:	0.166E+04	0.529E+03
:	:	:	*	:	0.174E+04	0.506E+03
:	:	:	*	:	0.181E+04	0.483E+03
460	+	-----	+	-----	+	
:	:	:	:	*	0.188E+04	0.460E+03
:	:	:	:	*	0.195E+04	0.437E+03
:	:	:	:	*	0.202E+04	0.414E+03
:	:	:	:	*	0.208E+04	0.391E+03
:	:	:	:	*	0.215E+04	0.368E+03
345	+	-----	+	-----	+	
:	:	:	:	*	0.221E+04	0.345E+03
:	:	:	:	*	0.228E+04	0.322E+03
:	:	:	:	*	0.228E+04	0.299E+03
:	:	:	:	*	0.227E+04	0.276E+03
:	:	:	:	*	0.225E+04	0.253E+03
230	+	-----	+	-----	+	
:	:	:	:	*	0.222E+04	0.230E+03
:	:	:	:	*	0.217E+04	0.207E+03
:	:	:	:	*	0.212E+04	0.184E+03
:	:	:	:	*	0.204E+04	0.161E+03
:	:	:	:	*	0.195E+04	0.138E+03
115	+	-----	+	-----	+	
:	:	:	:	*	0.186E+04	0.115E+03
:	:	:	:	*	0.177E+04	0.920E+02
:	:	:	:	*	0.168E+04	0.690E+02
:	:	:	*	:	0.159E+04	0.460E+02
:	:	:	*	:	0.149E+04	0.230E+02
0	+	-----	+	-----	+	
:	:	:	:	*	0.140E+04	0.000E+00
0		570	1140	1710	2280	
						MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL A7 2

WIDTH = 17.000 IN

HEIGHT = 14.000 IN

CONC STR = 5.500 KSI

STL YIELD = 62.500 KSI

EU = .0050

DIST FROM CG AREA

4.130 2.370

-4.130 2.370

P0 = 0.139E+04 K

PB = 0.479E+03 MB = 0.313E+04 K IN CB = 0.778E+01 IN

LOAD/LINE = 35 K MOMENT/SP = 79 K IN

P (KIPS)					MOMENT	LOAD
1400	+	-----	+	-----	+	-----
:	*	:	:	:	:	0.256E+03 0.136E+04
:	*	:	:	:	:	0.459E+03 0.133E+04
:	*	:	:	:	:	0.664E+03 0.129E+04
:	*	:	:	:	:	0.856E+03 0.126E+04
1225	+	-----	+	-----	+	-----
:	:	*	:	:	:	0.104E+04 0.123E+04
:	:	*	:	:	:	0.121E+04 0.119E+04
:	:	*	:	:	:	0.137E+04 0.116E+04
:	:	*	:	:	:	0.152E+04 0.112E+04
:	:	*	:	:	:	0.166E+04 0.108E+04
1050	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.180E+04 0.105E+04
:	:	:	*	:	:	0.192E+04 0.102E+04
:	:	:	*	:	:	0.204E+04 0.980E+03
:	:	:	*	:	:	0.215E+04 0.945E+03
:	:	:	*	:	:	0.225E+04 0.910E+03
875	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.235E+04 0.875E+03
:	:	:	*	:	:	0.244E+04 0.840E+03
:	:	:	*	:	:	0.252E+04 0.805E+03
:	:	:	*	:	:	0.260E+04 0.770E+03
:	:	:	*	:	:	0.268E+04 0.735E+03
700	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.275E+04 0.700E+03
:	:	:	*	:	:	0.282E+04 0.665E+03
:	:	:	*	:	:	0.288E+04 0.630E+03
:	:	:	*	:	:	0.294E+04 0.595E+03
:	:	:	*	:	:	0.300E+04 0.560E+03
525	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.306E+04 0.525E+03
:	:	:	*	:	:	0.312E+04 0.490E+03
:	:	:	*	:	:	0.311E+04 0.455E+03
:	:	:	*	:	:	0.305E+04 0.420E+03
:	:	:	*	:	:	0.299E+04 0.385E+03
350	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.290E+04 0.350E+03
:	:	:	*	:	:	0.280E+04 0.315E+03
:	:	:	*	:	:	0.269E+04 0.280E+03
:	:	:	*	:	:	0.255E+04 0.245E+03
:	:	:	*	:	:	0.240E+04 0.210E+03
175	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.226E+04 0.175E+03
:	:	:	*	:	:	0.212E+04 0.140E+03
:	:	:	*	:	:	0.197E+04 0.105E+03
:	:	:	*	:	:	0.182E+04 0.700E+02
:	:	:	*	:	:	0.167E+04 0.350E+02
0	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.151E+04 0.000E+00
0		790	1580	2370	3160	
		MOMENT (K IN)				

C O L U M N I N T E R A C T I O N D I A G

COL A7 4 A7 5 C7 5

WIDTH = 16.000 IN

HEIGHT = 14.000 IN

CONC STR = 5.500 KSI

STL YIELD = 62.500 KSI

EU = .0050

DIST FROM CG AREA

4.380 .880

-4.380 .880

P0 = 0.115E+04 K

PB = 0.461E+03 MB = 0.229E+04 K IN CB = 0.795E+01 IN

LOAD/LINE = 29 K MOMENT/SP = 58 K IN

P (KIPS)					MOMENT	LOAD
1160	+	+	+	+	+	+
	:	*	:	:	:	0.160E+03 0.113E+04
	:		*	:	:	0.344E+03 0.110E+04
	:			*	:	0.521E+03 0.107E+04
	:		:	*	:	0.688E+03 0.104E+04
1015	+	+	+	+	+	+
	:	:	:	*	:	0.845E+03 0.102E+04
	:	:	:		*	0.993E+03 0.986E+03
	:	:	:		*	0.113E+04 0.957E+03
	:	:	:		*	0.126E+04 0.928E+03
	:	:	:		*	0.138E+04 0.899E+03
870	+	+	+	+	+	+
	:	:	:	:	*	0.149E+04 0.870E+03
	:	:	:	:	*	0.160E+04 0.841E+03
	:	:	:	:	*	0.169E+04 0.812E+03
	:	:	:	:	*	0.178E+04 0.783E+03
	:	:	:	:	*	0.186E+04 0.754E+03
725	+	+	+	+	+	+
	:	:	:	:	*	0.193E+04 0.725E+03
	:	:	:	:	*	0.199E+04 0.696E+03
	:	:	:	:	*	0.205E+04 0.667E+03
	:	:	:	:	*	0.210E+04 0.638E+03
	:	:	:	:	*	0.214E+04 0.609E+03
580	+	+	+	+	+	+
	:	:	:	:	*	0.218E+04 0.580E+03
	:	:	:	:	*	0.221E+04 0.551E+03
	:	:	:	:	*	0.224E+04 0.522E+03
	:	:	:	:	*	0.227E+04 0.493E+03
	:	:	:	:	*	0.229E+04 0.464E+03
435	+	+	+	+	+	+
	:	:	:	:	*	0.226E+04 0.435E+03
	:	:	:	:	*	0.222E+04 0.406E+03
	:	:	:	:	*	0.217E+04 0.377E+03
	:	:	:	:	*	0.211E+04 0.348E+03
	:	:	:	:	*	0.203E+04 0.319E+03
290	+	+	+	+	+	+
	:	:	:	:	*	0.195E+04 0.290E+03
	:	:	:	:	*	0.185E+04 0.261E+03
	:	:	:	:	*	0.174E+04 0.232E+03
	:	:	:	:	*	0.162E+04 0.203E+03
	:	:	:	:	*	0.149E+04 0.174E+03
145	+	+	+	+	+	+
	:	:	:	:	*	0.136E+04 0.145E+03
	:	:	:	:	*	0.123E+04 0.116E+03
	:	:	:	:	*	0.109E+04 0.870E+02
	:	:	:	:	*	0.952E+03 0.580E+02
	:	:	:	:	*	0.827E+03 0.290E+02
0	+	+	+	+	+	+
	:	:	:	:	*	0.678E+03 0.000E+00
0		580	1160	1740	2320	
			MOMENT (K IN)			

C O L U M N I N T E R A C T I O N D I A G

COL B7 1

WIDTH = 20.000 IN HEIGHT = 20.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

6.920 4.680
 .000 3.120
 -6.920 4.680

P0 = 0.259E+04 K

PB = 0.927E+03 MB = 0.869E+04 K IN CB = 0.118E+02 IN

LOAD/LINE = 65 K MOMENT/SP = 218 K IN

P (KIPS)						MOMENT	LOAD
2600	+	+	+	+	+		
:	*	:	:	:	:	0.967E+03	0.254E+04
:	*	:	:	:	:	0.149E+04	0.247E+04
:	*	:	:	:	:	0.204E+04	0.240E+04
:	*	:	:	:	:	0.255E+04	0.234E+04
2275	+	+	+	+	+	0.304E+04	0.227E+04
:	:	*	:	:	:	0.349E+04	0.221E+04
:	:	*	:	:	:	0.393E+04	0.215E+04
:	:	*	:	:	:	0.434E+04	0.208E+04
:	:	*	:	:	:	0.472E+04	0.202E+04
1950	+	+	+	+	+	0.509E+04	0.195E+04
:	:	:	*	:	:	0.543E+04	0.189E+04
:	:	:	*	:	:	0.576E+04	0.182E+04
:	:	:	*	:	:	0.606E+04	0.175E+04
:	:	:	*	:	:	0.632E+04	0.169E+04
1625	+	+	+	+	+	0.655E+04	0.163E+04
:	:	:	:	*	:	0.679E+04	0.156E+04
:	:	:	:	*	:	0.701E+04	0.150E+04
:	:	:	:	*	:	0.723E+04	0.143E+04
:	:	:	:	*	:	0.743E+04	0.136E+04
1300	+	+	+	+	+	0.763E+04	0.130E+04
:	:	:	:	*	:	0.783E+04	0.124E+04
:	:	:	:	*	:	0.802E+04	0.117E+04
:	:	:	:	*	:	0.820E+04	0.110E+04
:	:	:	:	*	:	0.839E+04	0.104E+04
975	+	+	+	+	+	0.856E+04	0.975E+03
:	:	:	:	*	:	0.868E+04	0.910E+03
:	:	:	:	*	:	0.863E+04	0.845E+03
:	:	:	:	*	:	0.856E+04	0.780E+03
:	:	:	:	*	:	0.847E+04	0.715E+03
650	+	+	+	+	+	0.837E+04	0.650E+03
:	:	:	:	*	:	0.826E+04	0.585E+03
:	:	:	:	*	:	0.814E+04	0.520E+03
:	:	:	:	*	:	0.802E+04	0.455E+03
:	:	:	:	*	:	0.789E+04	0.390E+03
325	+	+	+	+	+	0.776E+04	0.325E+03
:	:	:	:	*	:	0.751E+04	0.260E+03
:	:	:	:	*	:	0.712E+04	0.195E+03
:	:	:	:	*	:	0.673E+04	0.130E+03
:	:	:	:	*	:	0.628E+04	0.650E+02
0	+	+	+	+	+	0.584E+04	0.000E+00
0		2180	4360	6540	8720		

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 2

WIDTH = 20.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

4.990 3.810
 .000 2.540
 -4.990 3.810

P0 = 0.208E+04 K

PB = 0.701E+03 MB = 0.532E+04 K IN CB = 0.908E+01 IN

LOAD/LINE = 53 K MOMENT/SP = 134 K IN

P (KIPS)					MOMENT	LOAD
2120	+	-----	+	-----	+	-----
:	*	:	:	:	0.418E+03	0.207E+04
:	*	:	:	:	0.762E+03	0.201E+04
:	*	:	:	:	0.112E+04	0.196E+04
:	*	:	:	:	0.145E+04	0.191E+04
1855	+	-----	+	-----	+	-----
:	:	*	:	:	0.176E+04	0.185E+04
:	:	*	:	:	0.206E+04	0.180E+04
:	:	*	:	:	0.233E+04	0.175E+04
:	:	*	:	:	0.259E+04	0.170E+04
:	:	*	:	:	0.284E+04	0.164E+04
1590	+	-----	+	-----	+	-----
:	:	:	*	:	0.307E+04	0.159E+04
:	:	:	*	:	0.328E+04	0.154E+04
:	:	:	*	:	0.349E+04	0.148E+04
:	:	:	*	:	0.368E+04	0.143E+04
:	:	:	*	:	0.383E+04	0.138E+04
1325	+	-----	+	-----	+	-----
:	:	:	*	:	0.397E+04	0.132E+04
:	:	:	*	:	0.411E+04	0.127E+04
:	:	:	*	:	0.425E+04	0.122E+04
:	:	:	*	:	0.438E+04	0.117E+04
:	:	:	*	:	0.450E+04	0.111E+04
1060	+	-----	+	-----	+	-----
:	:	:	*	:	0.462E+04	0.106E+04
:	:	:	*	:	0.473E+04	0.101E+04
:	:	:	*	:	0.484E+04	0.954E+03
:	:	:	*	:	0.495E+04	0.901E+03
:	:	:	*	:	0.505E+04	0.848E+03
795	+	-----	+	-----	+	-----
:	:	:	*	:	0.515E+04	0.795E+03
:	:	:	*	:	0.525E+04	0.742E+03
:	:	:	*	:	0.532E+04	0.689E+03
:	:	:	*	:	0.527E+04	0.636E+03
:	:	:	*	:	0.522E+04	0.583E+03
530	+	-----	+	-----	+	-----
:	:	:	*	:	0.516E+04	0.530E+03
:	:	:	*	:	0.509E+04	0.477E+03
:	:	:	*	:	0.501E+04	0.424E+03
:	:	:	*	:	0.493E+04	0.371E+03
:	:	:	*	:	0.485E+04	0.318E+03
265	+	-----	+	-----	+	-----
:	:	:	*	:	0.476E+04	0.265E+03
:	:	:	*	:	0.460E+04	0.212E+03
:	:	:	*	:	0.436E+04	0.159E+03
:	:	:	*	:	0.410E+04	0.106E+03
:	:	:	*	:	0.383E+04	0.530E+02
0	+	-----	+	-----	+	-----
0		1340	2680	4020	5360	
						MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL B7 3

WIDTH = 18.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050

DIST FROM CG AREA

4.990 3.810

-4.990 3.810

P0 = 0.179E+04 K

PB = 0.592E+03 MB = 0.503E+04 K IN CB = 0.908E+01 IN

LOAD/LINE = 45 K MOMENT/SP = 126 K IN

P (KIPS)					MOMENT	LOAD
1800	+	-----	+	-----	+	-----
:	*	:	:	:	:	0.441E+03 0.175E+04
:	*	:	:	:	:	0.727E+03 0.171E+04
:	*	:	:	:	:	0.103E+04 0.167E+04
:	*	:	:	:	:	0.131E+04 0.162E+04
1575	+	-----	+	-----	+	-----
:	:	*	:	:	:	0.157E+04 0.157E+04
:	:	*	:	:	:	0.183E+04 0.153E+04
:	:	*	:	:	:	0.207E+04 0.149E+04
:	:	*	:	:	:	0.229E+04 0.144E+04
:	:	*	:	:	:	0.251E+04 0.140E+04
1350	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.271E+04 0.135E+04
:	:	:	*	:	:	0.290E+04 0.130E+04
:	:	:	*	:	:	0.308E+04 0.126E+04
:	:	:	*	:	:	0.326E+04 0.122E+04
:	:	:	*	:	:	0.342E+04 0.117E+04
1125	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.357E+04 0.113E+04
:	:	:	*	:	:	0.372E+04 0.108E+04
:	:	:	*	:	:	0.386E+04 0.103E+04
:	:	:	*	:	:	0.400E+04 0.990E+03
:	:	:	*	:	:	0.413E+04 0.945E+03
900	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.425E+04 0.900E+03
:	:	:	*	:	:	0.437E+04 0.855E+03
:	:	:	*	:	:	0.449E+04 0.810E+03
:	:	:	*	:	:	0.461E+04 0.765E+03
:	:	:	*	:	:	0.472E+04 0.720E+03
675	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.483E+04 0.675E+03
:	:	:	*	:	:	0.494E+04 0.630E+03
:	:	:	*	:	:	0.503E+04 0.585E+03
:	:	:	*	:	:	0.496E+04 0.540E+03
:	:	:	*	:	:	0.488E+04 0.495E+03
450	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.477E+04 0.450E+03
:	:	:	*	:	:	0.464E+04 0.405E+03
:	:	:	*	:	:	0.449E+04 0.360E+03
:	:	:	*	:	:	0.430E+04 0.315E+03
:	:	:	*	:	:	0.408E+04 0.270E+03
225	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.386E+04 0.225E+03
:	:	:	*	:	:	0.364E+04 0.180E+03
:	:	:	*	:	:	0.341E+04 0.135E+03
:	:	:	*	:	:	0.319E+04 0.900E+02
:	:	:	*	:	:	0.296E+04 0.450E+02
0	+	-----	+	-----	+	-----
:	:	:	*	:	:	0.273E+04 0.000E+00
0		1260	2520	3780	5040	
		MOMENT (K IN)				

COLUMN INTERACTION DIAG

COL B7 4 B7 5

WIDTH = 16.000 IN HEIGHT = 16.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050

DIST FROM CG AREA

5.130 1.580

-5.130 1.580

P0 = 0.138E+04 K

PB = 0.532E+03 MB = 0.338E+04 K IN CB = 0.918E+01 IN

LOAD/LINE = 35 K MOMENT/SP = 85 K IN

P (KIPS)	MOMENT	LOAD
1400 +-----+-----+-----+-----+		
: * : : : : 0.192E+03 0.136E+04		
: * : : : : 0.442E+03 0.133E+04		
: * : : : : 0.687E+03 0.129E+04		
: * : : : : 0.918E+03 0.126E+04		
1225 +-----+-----+-----+-----+	0.114E+04 0.123E+04	
: : * : : : 0.134E+04 0.119E+04		
: : * : : : 0.153E+04 0.116E+04		
: : * : : : 0.172E+04 0.112E+04		
: : * : : : 0.189E+04 0.108E+04		
1050 +-----+-----+-----+-----+	0.204E+04 0.105E+04	
: : * : : : 0.219E+04 0.102E+04		
: : * : : : 0.233E+04 0.980E+03		
: : * : : : 0.246E+04 0.945E+03		
: : * : : : 0.257E+04 0.910E+03		
875 +-----+-----+-----+-----+	0.268E+04 0.875E+03	
: : * : : : 0.278E+04 0.840E+03		
: : * : : : 0.287E+04 0.805E+03		
: : * : : : 0.296E+04 0.770E+03		
: : * : : : 0.303E+04 0.735E+03		
700 +-----+-----+-----+-----+	0.310E+04 0.700E+03	
: : * : : : 0.317E+04 0.665E+03		
: : * : : : 0.323E+04 0.630E+03		
: : * : : : 0.329E+04 0.595E+03		
: : * : : : 0.334E+04 0.560E+03		
525 +-----+-----+-----+-----+	0.337E+04 0.525E+03	
: : * : : : 0.333E+04 0.490E+03		
: : * : : : 0.327E+04 0.455E+03		
: : * : : : 0.319E+04 0.420E+03		
: : * : : : 0.310E+04 0.385E+03		
350 +-----+-----+-----+-----+	0.299E+04 0.350E+03	
: : * : : : 0.287E+04 0.315E+03		
: : * : : : 0.273E+04 0.280E+03		
: : * : : : 0.256E+04 0.245E+03		
: : * : : : 0.238E+04 0.210E+03		
175 +-----+-----+-----+-----+	0.221E+04 0.175E+03	
: : * : : : 0.202E+04 0.140E+03		
: : * : : : 0.184E+04 0.105E+03		
: : * : : : 0.165E+04 0.700E+02		
: : * : : : 0.146E+04 0.350E+02		
0 +-----+-----+-----+-----+	0.129E+04 0.000E+00	
0 850 1700 2550 3400		
MOMENT (K IN)		

COLUMN INTERACTION DIAG

COL C7 1

WIDTH = 17.000 IN HEIGHT = 14.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050

DIST FROM CG AREA

3.990 3.810

-3.990 3.810

P0 = 0.155E+04 K

PB = 0.473E+03 MB = 0.380E+04 K IN CB = 0.768E+01 IN

LOAD/LINE = 39 K MOMENT/SP = 96 K IN

P (KIPS)						MOMENT	LOAD
1560	+	-----	+	-----	+	-----	+
:	*	:	:	:	:	0.391E+03	0.152E+04
:	*	:	:	:	:	0.594E+03	0.148E+04
:	*	:	:	:	:	0.811E+03	0.144E+04
:	*	:	:	:	:	0.101E+04	0.140E+04
1365	+	-----	+	-----	+	-----	+
:	:	*	:	:	:	0.121E+04	0.136E+04
:	:	*	:	:	:	0.139E+04	0.133E+04
:	:	*	:	:	:	0.156E+04	0.129E+04
:	:	*	:	:	:	0.172E+04	0.125E+04
:	:	*	:	:	:	0.188E+04	0.121E+04
1170	+	-----	+	-----	+	-----	+
:	:	:	*	:	:	0.202E+04	0.117E+04
:	:	:	*	:	:	0.216E+04	0.113E+04
:	:	:	*	:	:	0.229E+04	0.109E+04
:	:	:	*	:	:	0.242E+04	0.105E+04
:	:	:	*	:	:	0.253E+04	0.101E+04
975	+	-----	+	-----	+	-----	+
:	:	:	:	*	:	0.265E+04	0.975E+03
:	:	:	:	*	:	0.275E+04	0.936E+03
:	:	:	:	*	:	0.286E+04	0.897E+03
:	:	:	:	*	:	0.295E+04	0.858E+03
:	:	:	:	*	:	0.305E+04	0.819E+03
780	+	-----	+	-----	+	-----	+
:	:	:	:	:	*	0.314E+04	0.780E+03
:	:	:	:	:	*	0.323E+04	0.741E+03
:	:	:	:	:	*	0.332E+04	0.702E+03
:	:	:	:	:	*	0.340E+04	0.663E+03
:	:	:	:	:	*	0.349E+04	0.624E+03
585	+	-----	+	-----	+	-----	+
:	:	:	:	:	*	0.357E+04	0.585E+03
:	:	:	:	:	*	0.365E+04	0.546E+03
:	:	:	:	:	*	0.374E+04	0.507E+03
:	:	:	:	:	*	0.380E+04	0.468E+03
:	:	:	:	:	*	0.375E+04	0.429E+03
390	+	-----	+	-----	+	-----	+
:	:	:	:	:	*	0.367E+04	0.390E+03
:	:	:	:	:	*	0.358E+04	0.351E+03
:	:	:	:	:	*	0.347E+04	0.312E+03
:	:	:	:	:	*	0.332E+04	0.273E+03
:	:	:	:	:	*	0.317E+04	0.234E+03
195	+	-----	+	-----	+	-----	+
:	:	:	:	*	:	0.302E+04	0.195E+03
:	:	:	:	*	:	0.287E+04	0.156E+03
:	:	:	:	*	:	0.272E+04	0.117E+03
:	:	:	:	*	:	0.256E+04	0.780E+02
:	:	:	:	*	:	0.240E+04	0.390E+02
0	+	-----	+	-----	+	-----	+
0		960	1920	2880	3840	0.224E+04	0.000E+00

MOMENT (K IN)

COLUMN INTERACTION DIAG
COL C7 2

WIDTH = 16.000 IN HEIGHT = 14.000 IN
CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
DIST FROM CG AREA

3.990 3.810
-3.990 3.810

P0 = 0.149E+04 K

PB = 0.445E+03 MB = 0.369E+04 K IN CB = 0.768E+01 IN

LOAD/LINE = 38 K MOMENT/SP = 93 K IN

P (KIPS)					MOMENT	LOAD
1520	+	-----	+	-----	+	-----
:	*	:	:	:	0.209E+03	0.148E+04
:	*	:	:	:	0.436E+03	0.144E+04
:	*	:	:	:	0.652E+03	0.141E+04
:	*	:	:	:	0.855E+03	0.137E+04
1330	+	-----	+	-----	+	-----
:	:	*	:	:	0.105E+04	0.133E+04
:	:	*	:	:	0.123E+04	0.129E+04
:	:	*	:	:	0.140E+04	0.125E+04
:	:	*	:	:	0.157E+04	0.122E+04
:	:	*	:	:	0.172E+04	0.118E+04
1140	+	-----	+	-----	+	-----
:	:	*	:	:	0.187E+04	0.114E+04
:	:	*	:	:	0.200E+04	0.110E+04
:	:	*	:	:	0.214E+04	0.106E+04
:	:	*	:	:	0.226E+04	0.103E+04
:	:	*	:	:	0.238E+04	0.988E+03
950	+	-----	+	-----	+	-----
:	:	*	:	:	0.249E+04	0.950E+03
:	:	*	:	:	0.260E+04	0.912E+03
:	:	*	:	:	0.270E+04	0.874E+03
:	:	*	:	:	0.280E+04	0.836E+03
:	:	*	:	:	0.290E+04	0.798E+03
760	+	-----	+	-----	+	-----
:	:	*	:	:	0.299E+04	0.760E+03
:	:	*	:	:	0.308E+04	0.722E+03
:	:	*	:	:	0.317E+04	0.684E+03
:	:	*	:	:	0.325E+04	0.646E+03
:	:	*	:	:	0.334E+04	0.608E+03
570	+	-----	+	-----	+	-----
:	:	*	:	:	0.342E+04	0.570E+03
:	:	*	:	:	0.351E+04	0.532E+03
:	:	*	:	:	0.359E+04	0.494E+03
:	:	*	:	:	0.367E+04	0.456E+03
:	:	*	:	:	0.366E+04	0.418E+03
380	+	-----	+	-----	+	-----
:	:	*	:	:	0.359E+04	0.380E+03
:	:	*	:	:	0.351E+04	0.342E+03
:	:	*	:	:	0.340E+04	0.304E+03
:	:	*	:	:	0.328E+04	0.266E+03
:	:	*	:	:	0.313E+04	0.228E+03
190	+	-----	+	-----	+	-----
:	:	*	:	:	0.298E+04	0.190E+03
:	:	*	:	:	0.283E+04	0.152E+03
:	:	*	:	:	0.268E+04	0.114E+03
:	:	*	:	:	0.253E+04	0.760E+02
:	:	*	:	:	0.238E+04	0.380E+02
0	+	-----	+	-----	+	-----
:	:	*	:	:	0.222E+04	0.000E+00
0		930	1860	2790	3720	
			MOMENT (K IN)			

COL C7 3

4.130	2.000
-4.130	2.000

PO = 0.128E+04 K

PB = 0.451E+03 MB = 0.283E+04 K IN CB = 0.778E+01 IN

LOAD/LINE = 32 K MOMENT/SP = 71 K IN

P (KIPS)

MOMENT LOAD

1280	1120	960	800	640	480	320	160	0
0.292E+03	0.125E+04	0.100E+04	0.116E+04	0.110E+04	0.109E+04	0.106E+04	0.102E+04	0.992E+03
0.478E+03	0.122E+04	0.116E+04	0.130E+04	0.144E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04
0.663E+03	0.118E+04	0.130E+04	0.144E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04	0.200E+04
0.837E+03	0.115E+04	0.130E+04	0.144E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04	0.200E+04
0.100E+04	0.112E+04	0.116E+04	0.130E+04	0.144E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04
0.116E+04	0.109E+04	0.130E+04	0.144E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04	0.200E+04
0.130E+04	0.106E+04	0.144E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04	0.200E+04	0.209E+04
0.144E+04	0.102E+04	0.157E+04	0.169E+04	0.180E+04	0.190E+04	0.200E+04	0.209E+04	0.218E+04
0.157E+04	0.992E+03	0.169E+04	0.180E+04	0.190E+04	0.200E+04	0.209E+04	0.218E+04	0.226E+04
0.169E+04	0.960E+03	0.180E+04	0.190E+04	0.200E+04	0.209E+04	0.218E+04	0.226E+04	0.233E+04
0.180E+04	0.928E+03	0.190E+04	0.200E+04	0.209E+04	0.218E+04	0.226E+04	0.233E+04	0.240E+04
0.190E+04	0.896E+03	0.200E+04	0.209E+04	0.218E+04	0.226E+04	0.233E+04	0.240E+04	0.246E+04
0.200E+04	0.864E+03	0.209E+04	0.218E+04	0.226E+04	0.233E+04	0.240E+04	0.246E+04	0.253E+04
0.209E+04	0.832E+03	0.218E+04	0.226E+04	0.233E+04	0.240E+04	0.246E+04	0.253E+04	0.258E+04
0.218E+04	0.800E+03	0.226E+04	0.233E+04	0.240E+04	0.246E+04	0.253E+04	0.258E+04	0.264E+04
0.226E+04	0.768E+03	0.233E+04	0.240E+04	0.246E+04	0.253E+04	0.258E+04	0.264E+04	0.269E+04
0.233E+04	0.736E+03	0.240E+04	0.246E+04	0.253E+04	0.258E+04	0.264E+04	0.269E+04	0.274E+04
0.240E+04	0.704E+03	0.246E+04	0.253E+04	0.258E+04	0.264E+04	0.269E+04	0.274E+04	0.279E+04
0.246E+04	0.672E+03	0.253E+04	0.258E+04	0.264E+04	0.269E+04	0.274E+04	0.279E+04	0.283E+04
0.253E+04	0.640E+03	0.258E+04	0.264E+04	0.269E+04	0.274E+04	0.279E+04	0.283E+04	0.279E+04
0.258E+04	0.608E+03	0.264E+04	0.269E+04	0.274E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04
0.264E+04	0.576E+03	0.269E+04	0.274E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04
0.269E+04	0.544E+03	0.274E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04
0.274E+04	0.512E+03	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04
0.279E+04	0.480E+03	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04
0.283E+04	0.448E+03	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04
0.279E+04	0.416E+03	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04
0.273E+04	0.384E+03	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04	0.279E+04	0.283E+04
0.267E+04								

MOMENT (K IN)

C O L U M N I N T E R A C T I O N D I A G

COL A7 3 C7 4

WIDTH = 16.000 IN HEIGHT = 14.000 IN
 CONC STR = 5.500 KSI STL YIELD = 62.500 KSI EU = .0050
 DIST FROM CG AREA

4.310 1.200

-4.310 1.200

P0 = 0.119E+04 K

PB = 0.458E+03 MB = 0.245E+04 K IN CB = 0.790E+01 IN

LOAD/LINE = 30 K MOMENT/SP = 62 K IN

P (KIPS)					MOMENT	LOAD
1200	+	+	+	+	+	+
	:	*	:	:	:	0.162E+03 0.117E+04
	:		*	:	:	0.349E+03 0.114E+04
	:			*	:	0.531E+03 0.111E+04
	:				*	0.703E+03 0.108E+04
1050	+	+	+	+	+	+
	:			*	:	0.865E+03 0.105E+04
	:				*	0.102E+04 0.102E+04
	:				*	0.116E+04 0.990E+03
	:				*	0.129E+04 0.960E+03
	:				*	0.142E+04 0.930E+03
900	+	+	+	+	+	+
	:				*	0.153E+04 0.900E+03
	:				*	0.164E+04 0.870E+03
	:				*	0.174E+04 0.840E+03
	:				*	0.183E+04 0.810E+03
	:				*	0.192E+04 0.780E+03
750	+	+	+	+	+	+
	:				*	0.199E+04 0.750E+03
	:				*	0.206E+04 0.720E+03
	:				*	0.213E+04 0.690E+03
	:				*	0.219E+04 0.660E+03
	:				*	0.224E+04 0.630E+03
600	+	+	+	+	+	+
	:				*	0.228E+04 0.600E+03
	:				*	0.233E+04 0.570E+03
	:				*	0.236E+04 0.540E+03
	:				*	0.240E+04 0.510E+03
	:				*	0.243E+04 0.480E+03
450	+	+	+	+	+	+
	:				*	0.245E+04 0.450E+03
	:				*	0.241E+04 0.420E+03
	:				*	0.236E+04 0.390E+03
	:				*	0.230E+04 0.360E+03
	:				*	0.223E+04 0.330E+03
300	+	+	+	+	+	+
	:				*	0.214E+04 0.300E+03
	:				*	0.204E+04 0.270E+03
	:				*	0.194E+04 0.240E+03
	:				*	0.181E+04 0.210E+03
	:				*	0.168E+04 0.180E+03
150	+	+	+	+	+	+
	:				*	0.155E+04 0.150E+03
	:				*	0.142E+04 0.120E+03
	:				*	0.128E+04 0.900E+02
	:				*	0.114E+04 0.600E+02
	:				*	0.100E+04 0.300E+02
0	+	+	+	+	+	+
	:				*	0.876E+03 0.000E+00
0		620	1240	1860	2480	
						MOMENT (K IN)

*** SOIL SAMPLE DATA :

BORING NUMBER = B3
NO. OF SOIL SAMPLES = 10

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	25.0	.055	36.6
2	1	SANDY SILT	2.5	30.0	.275	44.0
3	1	SANDY SILT	5.0	43.0	.550	61.6
4	1	SANDY SILT	10.0	120.0	1.100	144.2
5	1	SANDY SILT	15.0	74.0	1.650	78.9
6	1	SANDY SILT	20.0	72.0	2.200	69.8
7	2	VOLC TUFF	25.0	100.0	2.750	89.5
8	2	VOLC TUFF	30.0	120.0	3.300	100.1
9	2	VOLC TUFF	35.0	100.0	3.850	78.3
10	2	VOLC TUFF	40.0	120.0	4.400	88.6

*** SOIL SAMPLE DATA :

BORING NUMBER = B4
NO. OF SOIL SAMPLES = 8

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	6.0	.055	8.8
2	1	SANDY SILT	2.5	31.0	.275	45.4
3	1	SANDY SILT	5.0	47.0	.550	67.4
4	1	SANDY SILT	10.0	40.0	1.100	48.1
5	1	SANDY SILT	15.0	75.0	1.650	80.0
6	2	VOLC TUFF	20.0	120.0	2.200	116.4
7	2	VOLC TUFF	25.0	120.0	2.750	107.4
8	2	VOLC TUFF	30.0	200.0	3.300	166.9

*** SOIL SAMPLE DATA :

BORING NUMBER = B5
NO. OF SOIL SAMPLES = 7

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	2.5	8.0	.275	11.7
2	1	SANDY SILT	5.0	11.0	.550	15.8
3	1	SANDY SILT	10.0	79.0	1.100	94.9
4	2	VOLC TUFF	15.0	48.0	1.650	51.2
5	2	VOLC TUFF	20.0	200.0	2.200	194.0
6	2	VOLC TUFF	25.0	150.0	2.750	134.3
7	2	VOLC TUFF	30.0	120.0	3.300	100.1

*** SOIL SAMPLE DATA :

BORING NUMBER = B6
NO. OF SOIL SAMPLES = 7

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	13.0	.055	19.0
2	1	SANDY SILT	2.5	10.0	.275	14.7
3	1	SANDY SILT	5.0	22.0	.550	31.5
4	2	VOLC TUFF	10.0	200.0	1.100	240.3
5	2	VOLC TUFF	15.0	120.0	1.650	127.9
6	2	VOLC TUFF	20.0	150.0	2.200	145.5
7	2	VOLC TUFF	25.0	200.0	2.750	179.1

*** SOIL SAMPLE DATA :

BORING NUMBER = B7
NO. OF SOIL SAMPLES = 8

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	17.0	.055	24.9
2	1	SANDY SILT	2.5	26.0	.275	38.1
3	1	SANDY SILT	5.0	56.0	.550	80.3
4	1	SANDY SILT	10.0	35.0	1.100	42.1
5	2	VOLC TUFF	15.0	118.0	1.650	125.8
6	2	VOLC TUFF	20.0	200.0	2.200	194.0
7	2	VOLC TUFF	25.0	200.0	2.750	179.1
8	2	VOLC TUFF	30.0	200.0	3.300	166.9

*** SOIL SAMPLE DATA :

BORING NUMBER = B8
NO. OF SOIL SAMPLES = 10

SAMPLE	LAYER	CLASS	DEPTH (FT)	BPF	PRESS (KSF)	BPFCOR
1	1	SANDY SILT	.5	6.0	.055	8.8
2	1	SANDY SILT	2.5	16.0	.275	23.4
3	1	SANDY SILT	5.0	17.0	.550	24.4
4	1	SANDY SILT	10.0	33.0	1.100	39.7
5	2	VOLC TUFF	15.0	59.0	1.650	62.9
6	2	VOLC TUFF	20.0	55.0	2.200	53.3
7	2	VOLC TUFF	25.0	58.0	2.750	51.9
8	2	VOLC TUFF	30.0	44.0	3.300	36.7
9	2	VOLC TUFF	35.0	45.0	3.850	35.2
10	2	VOLC TUFF	40.0	300.0	4.400	221.4

NSMPS	DEPTH	BPFMX	BPFAV	BPFMN	BPFCMX	BPFCAV	BPFCMN
6	.50	25.00	12.50	6.00	36.63	18.32	8.79
1	.54	31.00	31.00	31.00	45.43	45.43	45.43
8	2.50	52.00	24.38	8.00	76.20	35.72	11.72
8	5.00	56.00	35.38	11.00	80.28	50.71	15.77
8	10.00	200.00	80.75	33.00	240.34	97.04	39.66
8	15.00	120.00	87.75	48.00	127.93	93.55	51.17
8	20.00	200.00	130.88	55.00	193.98	126.94	53.35
8	25.00	200.00	143.50	58.00	179.06	128.48	51.93
7	30.00	300.00	169.14	44.00	250.30	141.12	36.71
2	35.00	100.00	72.50	45.00	78.28	56.75	35.23
2	40.00	300.00	210.00	120.00	221.44	155.01	88.58

I	DEPTH	DRAV	PHIAV	GAMAV	DRCV	PHICAV	GAMCAV
1	.50	48.80	30.86	87.81	59.23	31.74	89.48
2	.54	73.95	32.92	91.83	86.91	34.67	94.86
3	2.50	66.82	32.35	90.69	78.61	33.50	92.87
4	5.00	78.27	33.46	92.78	91.56	35.32	95.97
5	10.00	100.00	36.50	98.00	100.00	36.50	98.00
6	15.00	100.00	41.00	124.00	100.00	41.00	124.00
7	20.00	100.00	41.00	124.00	100.00	41.00	124.00
8	25.00	100.00	41.00	124.00	100.00	41.00	124.00
9	30.00	100.00	41.00	124.00	100.00	41.00	124.00
10	35.00	96.12	40.38	123.07	85.76	38.72	120.58
11	40.00	100.00	41.00	124.00	100.00	41.00	124.00

SOIL DATA FILE NAME = OTOWI.BORING