PARAMETRIC STUDY OF ROOF DIAPHRAGM STIFFNESS REQUIREMENTS

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ABSTRACT

A common assumption made in performing a dynamic seismic analysis for a building is that the roof/floor system is "rigid". This assumption would appear to be reasonable for many of the structures found in nuclear power plants, since many of these structures are constructed of heavily reinforced concrete having floor/roof slabs at least two feet in thickness, and meet the code requirements for structural detailing for seismic design.

The roofs of many Department of Energy (DOE) buildings at the Oak Ridge Y-12 Plant* in Oak Ridge, Tennessee, have roofs constructed of either metal, precast concrete or gypsum plank deck overlaid with rigid insulation, tar and gravel. In performing natural phenomena hazard assessments for one such facility, it was assumed that the existing roof performed first as a flexible diaphragm (zero stiffness) and then, rigid (infinitely stiff). For the flexible diaphragm model it was determined that the building began to experience significant damage around 0.09 g's. For the rigid diaphragm model it was determined that no significant damage was observed below 0.20 g's.

A Conceptual Design Report has been prepared for upgrading/replacing the roof of this building. The question that needed to be answered here was, "How stiff should the new roof diaphragm be in order to satisfy the rigid diaphragm assumption and, yet, be cost effective?". This paper presents a parametric study of a very simple structural system to show that the design of roof diaphragms needs to consider both strength and stiffness (frequency) requirements.

This paper shows how the stiffness of a roof system affects the seismically induced loads in the lateral, vertical load resisting elements of a building and provides guidance in determining how "rigid" a roof system should be in order to accomplish a cost effective design.

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INTRODUCTION/BACKGROUND

A Conceptual Design Report (CDR) has been prepared for Building 9206 at the Y-12 Plant. The CDR addresses the need to bring this facility into closer compliance to the DOE General Design Criteria, 6430.1A, for natural phenomena hazards reduction. Part of this CDR involves replacing the existing roof system. The existing roof consists of concrete roof panels 2'-0" wide by 8'-9" long that are supported by purlins running 8'-9" on centers normal to the direction of the concrete roof panels. The concrete panels, which are not shear connected to the purlins and to each other, are 2 3/4" deep, and overlain with a built-up tar and gravel roof. The only lateral force resisting elements of Building 9206 consist of steel columns and beams infilled with unreinforced masonry hollow clay tile walls (HCTW). The beam to column connections are AISC Type 2, non-moment resisting; hence, the HCTWs act as shear walls to resist lateral forces.

Building 9206 is approximately 260 feet long north-south by 165 feet wide east-west. It has a two story center section, approximately 120 feet long by 165 feet wide, bounded on the north and south by one story sections that are approximately 60 feet long by 165 feet wide and 80 feet long by 165 feet wide, respectively. A schematic of Building 9206 is shown in Figure 1. With the exception of some minor modifications and the rearrangement of some internal walls, there have been no major changes to Building 9206 since it was originally constructed in 1944. The roofs of the three sections are as described above. The second floor of the center section is a reinforced five (5) inch thick concrete slab. Column lines are spaced 20 feet on centers north-south and 26 feet on centers east-west.

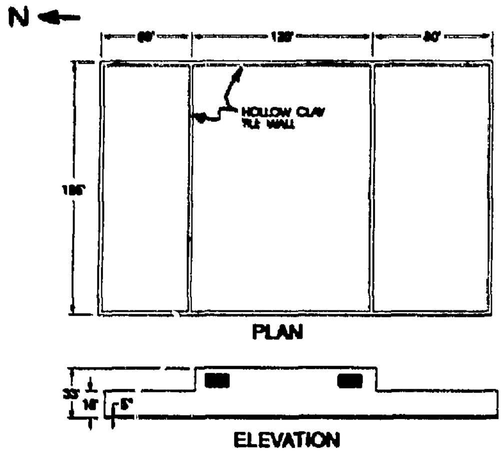

Figure 1: Schematic of Building 9206

Prior to the CDR work several analyses had been completed to support the Y-12 Plant Safety Analysis Program for this facility. The first analytical study was to determine damage scenarios as a function of peak ground acceleration and wind velocity up to predicted collapse of the facility. Because the existing roof system, which was not shear connected to the purlins, was considered to be very flexible, only planar, two dimensional models representative of each column line were used to determine the building's response. The unreinforced HCTWs were modeled using membrane plate elements. The analysis results indicated that the fundamental frequencies of these models (column lines) ranged from 2.9 to 21.0 Hz. The results also indicated that the low frequency (weak) column lines would start to collapse at about 0.08 g's peak ground acceleration with failure progressing to the higher frequency (stronger) column lines such that total damage (>80% and not reusable) to the building's structure was expected to occur at about 0.16 g's. The wind analysis indicated that severe damage to the roof structure would occur at about 90 mph. The roof damage was due to uplift because the concrete roof panels were not connected to the purlins.

At the time the CDR work was initiated, a feasibility study was performed to basically reinforce the more flexible (weak) column lines and to tie the roof down to prevent uplift. Due to the congestion of pipes and equipment it was considered too difficult to get to the underside of the roof to add tie downs and/or horizontal diagonal braces, and because of the effect construction would have on operations, it was decided to extend the feasibility study to include replacing the entire roof.

At this point it became clear that the building response would have to be recalculated based on a more rigid diaphragm design. For this analysis a simple "stick" model was used where the properties of the model were computed based on the geometry of the unreinforced HCTWs and assuming the roof as an infinitely stiff diaphragm. The results of this study indicated that none of the HCTWs were now overstressed due to the redistribution of forces. The fundamental frequency of the building, for this case, in each of the two horizontal directions, was 16.3 hz N-S and 19.2 hz E-W. From this study it was clear that replacing the roof would not only fix the tie down problem but, by adding diaphragm stiffness, might alleviate the necessity of retrofitting (reinforcing) the HCTWs. However, to satisfy the need to finish the CDR the roof was designed based on a recommendation contained in Ref. [1]. A four (4) inch thick lightweight slab was selected, and cost estimates made for inclusion into the CDR. For this purpose it was assumed that the slab would satisfy the "rigid" diaphragm assumption; thus, no retrofit costs for the HCTWs were included in the CDR.

With the CDR finished it was decided to do a simplified parametric study to provide guidelines for selecting a trial design for a diaphragm or to determine if existing roof or floor systems qualify as a rigid diaphragm. Finally, the guidelines were used to assess the capability of the selected 4 inch roof slab to provide the necessary stiffness and strength to distribute the forces as assumed by the "rigid" diaphragm solution; and to determine if a more cost effective solution could be obtained. This paper presents the results of the study.

COMPUTER MODEL

The geometry for the single story computer model is shown in Figure 2. The model has three degrees of freedom, Δ_z, θ_x, and θ_y with ground motion input along the Z-axis. The column lines and roof diaphragm were modeled with simple steel beams. Four models, A-D, were selected as shown in Table 1. For model A the properties for the two exterior columns, which were assumed fixed at Nodes 10 and 30, and the interior column (fixed at Node 20), were selected to represent a typical exterior and interior column line frequency of Building 9206. Again, for these calculations, the existing roof was assumed to be a flexible diaphragm. Model A also represents a building with stiff (15.3 hz.) exterior column lines and flexible (1.88 hz.) interior column lines. Model B was chosen to represent flexible exterior and stiff interior column lines. Model C represents all stiff column lines and Model D all flexible column lines. For all models the column properties were held constant while the properties of the diaphragm beam were varied to create a simple beam frequency which ranged from 0.25 Hz. to infinity. Considerable of thought was given on how to compute the properties of the diaphragm beam. Since Building 9206 had stiff exterior and flexible interior column lines, it was decided to use a simple pinned-pinned beam spanning between exterior columns (50 feet) and compute (by hand) its properties assuming a uniform load of 300 lbs per foot (i.e., the interior column was assumed to have zero lateral stiffness).

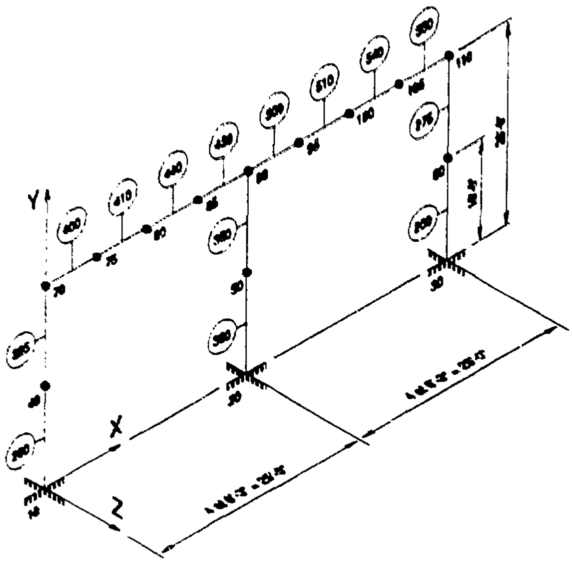

Figure 2: Computer Model for Parametric Study

The frequencies in Table 1, computed for the columns for the flexible diaphragm case, assumed a tributary length of the diaphragm of 12.5 feet for the exterior columns and 25 feet for the interior columns for determining the column masses. For the rigid diaphragm case the system frequency was computed assuming equal displacements for all columns. For both cases the mass was lumped at the top of the columns.

TABLE 1
BASIC MODEL PARAMETERS

MODEL	COLUMN INFORMATION		FREQUENCIES		
	EXTERNAL COL.	INTERNAL COL.	FLEXIBLE DIAPHRAGM		RIGID
			EXTR	INTR	SYS
	SIZE	SIZE	HZ	HZ	HZ
A	W14X730	W14X43	15.3	1.88	10.9
B	W14X43	W14X730	2.65	18.8	7.89
C	W14X730	W14X730	15.3	18.8	13.3
D	W14X43	W14X43	2.65	1.88	2.30

NUMERICAL ANALYSIS

The analyses used GTSTRUDL on a DEC micro-vax. The response spectrum approach was used to calculate deflections, forces, and reactions. A median response spectrum per NUREG/CR-0098 for 7% damping and a peak horizontal ground acceleration of 1.0-g was used in the analysis. The following tabulation gives the acceleration and frequency at the transition points on the curve. Figure 3 is a plot of this response spectrum.

Accel. G's	Freq. Hertz
0.048	0.100
0.293	0.248
1.887	1.599
1.887	8.000
1.000	33.000
1.000	100.000

RESULTS OF PARAMETRIC STUDY

The results are presented in Table 2. For each Model A-D,

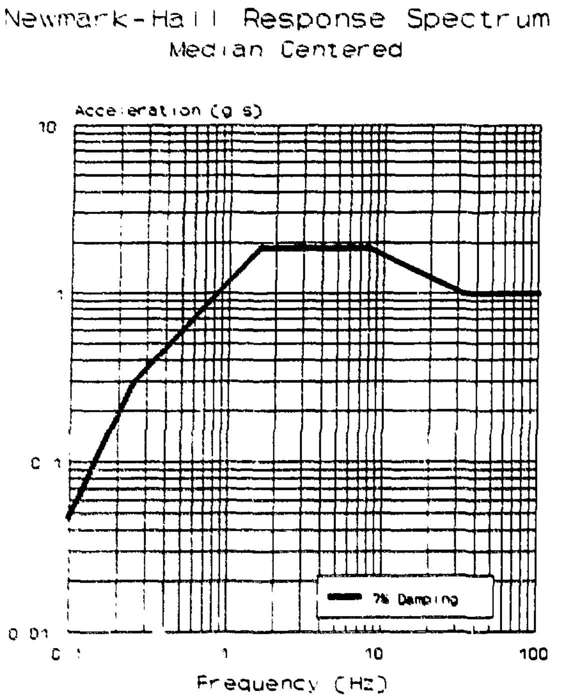

Figure 3: Response Spectrum used in Parametric Study

results are presented for twelve diaphragm beam frequencies from 0.25 Hz to infinity. The results for the infinite case, which were hand computed and represents the ideal goal for this study, are also presented in Table 1 as previously noted. The frequencies vs maximum columns moments are plotted for each model in Figure 4. For Models A, C, and D, the exterior column moments were plotted, and for Model B, the interior column moments. For Model A, which has stiff exterior columns and a flexible interior column, column moments closely approach the infinitely stiff diaphragm case when the pinned-pinned diaphragm beam frequency equals the frequency of the stiff column line frequency of 15.3 Hz, which was computed assuming a flexible diaphragm (Table 2). A similar observation is also true for Model B, which has flexible exterior columns and a stiff interior column, at a diaphragm beam frequency of 10.8 Hz which is the stiff column line frequency. For Model C, which has all stiff columns, column moments reached approximately 90% of the rigid diaphragm goal at a diaphragm beam frequency of 15.3 Hz. Model D, which has all flexible column lines, indicates that approximately 85% of the rigid diaphragm goal was reached at a diaphragm beam frequency of 2.65 Hz. To better illustrate this, the data in Table 2 has been replotted in Figures 5, 6 and 7 in a normalized, non-dimensional form. Here the ratios of the maximum column moment computed for each diaphragm beam frequency to the maximum column moment computed for the infinite diaphragm beam frequency are plotted against the ratios of the diaphragm beam frequency to maximum column frequency for each model. The ordinates represent the percentage of obtaining the maximum column moment for the rigid diaphragm affect as the diaphragm frequency varies with respect to the maximum column frequency.

To recap, Model A simulates Building 9206, i.e., very stiff exterior column lines (all bays infilled with HCTWs) and flexible interior column lines (few or no infilled bays). For the very flexible diaphragm, Model A01, the maximum moments and shears are carried by the most flexible column. For the rigid diaphragm, Model A12, the stiffer exterior columns have the largest moments and shears. This observation agrees with the earlier planar, two dimensional model studies to determine damage scenarios.

TABLE 2
RESULTS OF PARAMETRIC STUDY

MODEL ID	Diaph. Beam Freq. First Mode, Hertz	System Freq. Hertz	Maximum Displ. Inches	Column Moments Kip-Ft. Extr Joint 10/30	Column Moments Kip-Ft. Intr Joint 20	Column Bending stress KSI Extr Joint 10/30	Column Bending stress KSI Intr Joint 20	Max. Beam Moments Kip-Ft. Mom Y
A01	0.25	1.30	11.022	65.7	258.4	0.62	49.45	32.13
A02	0.50	1.75	6.756	75.2	333.4	0.71	63.81	30.45
A03	1.01	2.07	5.200	94.9	276.4	0.89	52.90	45.45
A04	3.02	3.45	1.942	182.7	103.2	1.71	19.75	123.2
A05	5.04	4.96	0.941	218.2	50.0	2.05	9.57	157.2
A06	10.07	7.72	0.376	256.0	20.0	2.40	3.83	179.1
A07	15.11	9.08	0.243	255.2	12.9	2.40	2.47	170.3
A08	20.15	9.73	0.196	251.8	10.4	2.36	1.99	163.4
A09	33.24	10.36	0.160	247.6	8.5	2.32	1.63	155.8
A10	50.37	10.59	0.149	245.8	7.88	2.30	1.51	152.8
A11	100.73	10.72	0.142	244.6	7.52	2.29	1.44	151.0
A12	Infinite	10.92	———	242.5	7.26	———	———	———
B01	0.25	1.50	9.216	82.7	234.7	15.83	2.20	37.65
B02	0.50	2.62	3.253	104.0	250.2	19.90	2.35	42.47
B03	1.01	3.55	1.988	105.7	238.0	20.23	2.23	48.72
B04	3.02	5.04	1.100	59.0	341.6	11.29	3.20	108.8
B05	5.04	6.17	0.705	37.5	434.7	7.18	4.08	144.6
B06	10.07	7.27	0.418	22.2	514.5	4.25	4.82	159.3
B07	15.11	7.55	0.355	18.9	527.1	3.62	4.94	158.8
B08	20.15	7.65	0.333	17.7	530.6	3.39	4.97	158.1
B09	33.24	7.73	0.315	16.8	533.0	3.22	5.00	157.3
B10	50.37	7.76	0.309	16.4	533.7	3.14	5.00	157.0
B11	100.73	7.78	0.306	16.3	534.0	3.12	5.01	156.9
B12	Infinite	7.89	———	16.0	534.1	———	———	———
C01	0.25	1.56	8.904	76.7	242.1	0.72	2.27	37.63
C02	0.50	3.06	2.404	79.9	257.2	0.75	2.41	39.26
C03	1.01	5.65	0.725	83.2	290.4	0.78	2.72	40.95
C04	3.02	9.79	0.196	80.3	315.8	0.75	2.96	28.01
C05	5.04	10.75	0.161	89.9	282.7	0.84	2.65	33.59
C06	10.07	11.86	0.126	119.7	221.4	1.12	2.08	47.54
C07	15.11	12.39	0.109	135.0	190.6	1.27	1.79	53.61
C08	20.15	12.66	0.100	141.9	175.5	1.33	1.65	57.31
C09	33.24	12.92	0.092	147.9	161.0	1.39	1.51	60.38
C10	50.37	13.01	0.089	149.9	155.7	1.41	1.46	61.37
C11	100.73	13.07	0.087	151.1	152.5	1.42	1.43	61.93
C12	Infinite	13.27	———	150.4	150.4	———	———	———
D01	0.25	1.27	11.35	72.6	254.3	13.89	48.67	32.65
D02	0.50	1.69	7.238	88.9	346.7	17.01	66.35	31.49
D03	1.01	1.91	5.883	110.6	312.7	21.17	59.85	41.62
D04	3.02	2.18	4.201	169.6	223.3	32.46	42.74	68.16
D05	5.04	2.24	3.814	181.5	202.8	34.74	38.81	74.02
D06	10.07	2.27	3.621	187.0	192.5	35.79	36.84	76.60
D07	15.11	2.28	3.583	188.0	190.5	35.98	36.46	77.08
D08	20.15	2.28	3.570	188.4	189.8	36.06	36.33	77.24
D09	33.24	2.28	3.559	188.7	189.2	36.11	36.21	77.38
D10	50.37	2.28	3.555	188.8	189.0	36.13	36.17	77.42
D11	100.73	2.28	3.553	188.8	188.9	36.13	36.15	77.45
D12	Infinite	2.30	———	188.7	188.7	———	———	———

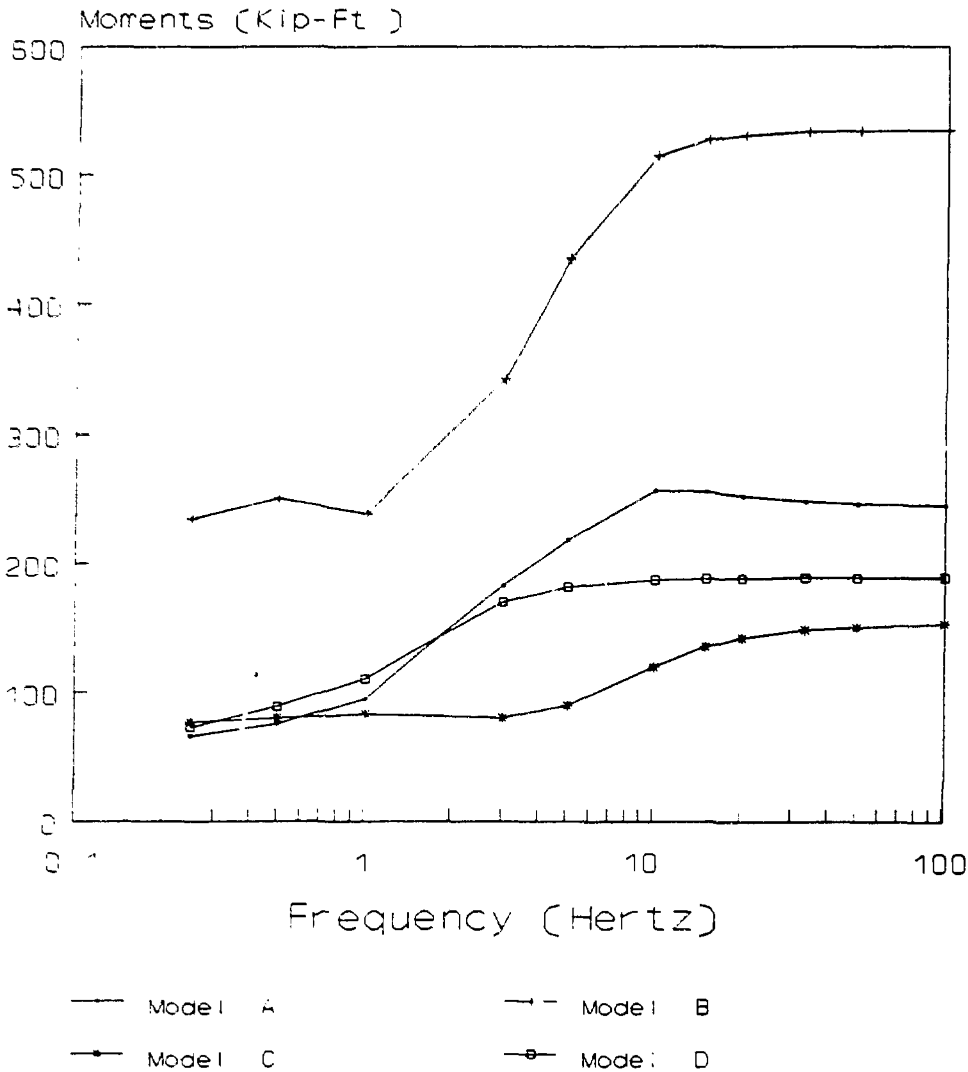

Figure 4. Plot of column moments versus diaphragm frequencies

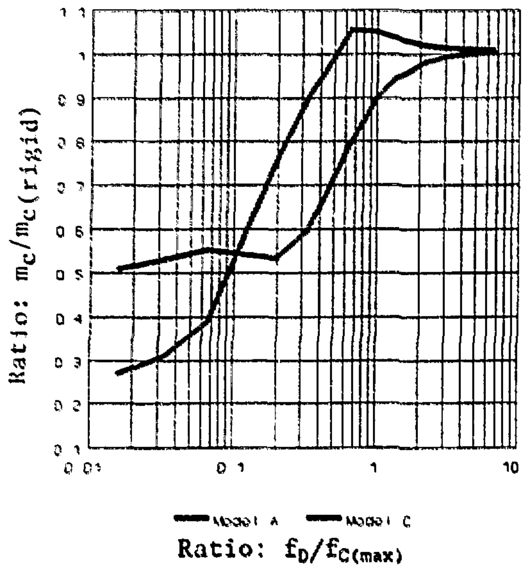

Figure 5: Non-dimensional plot of Models A and C results

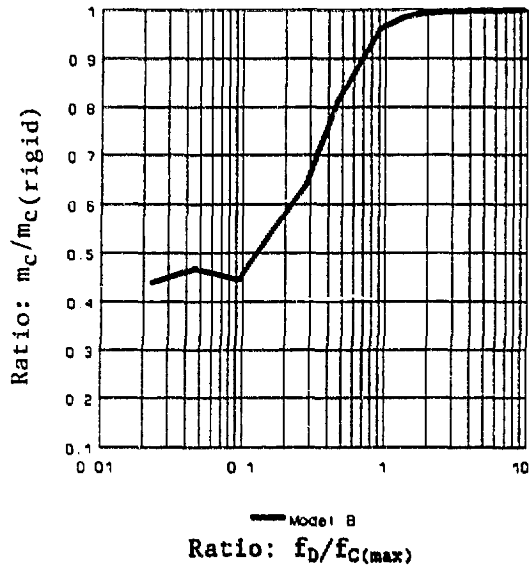

Figure 6: Non-dimensional plot of Model B results

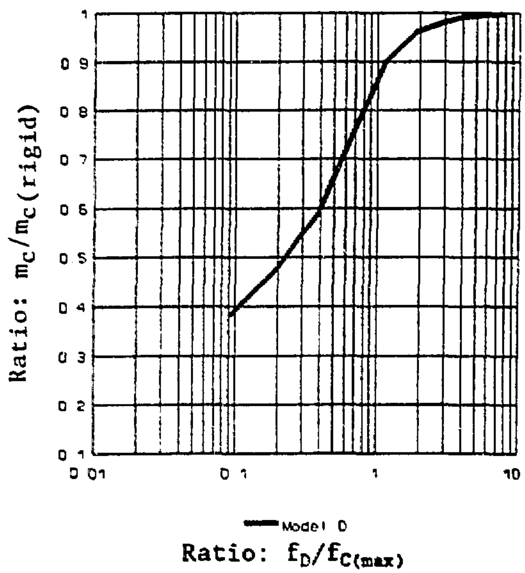

Figure 7: Non-dimensional plot of Model D results

ROOF DIAPHRAGM DESIGN STUDY

This design study is based on results consistent with Models A-D. The first step was to compile, from the numerical analyses, the displacements of the roof diaphragm beam for the nodes shown in Figure 2 for all models. The displacements relative to the ends of the diaphragm (at the exterior columns) were then determined. The largest relative displacement was typically at the center, Node 90. Next, to obtain a static equivalent load for design, the average diaphragm acceleration, which was also obtained from the numerical analyses, was multiplied by the assumed mass (300 lbs. per ft.). The moments and shears computed for this uniform load closely matched the same forces as obtained by the numerical analyses. Finally, from Reference [1], a diaphragm shear stiffness G' was computed to be consistent with the static equivalent loading and the maximum relative diaphragm beam displacement (Node 90). For these calculations, the diaphragm dimensions were assumed to be 50 by 34 feet.

The diaphragm shear stiffness G', for Models A11, B11, C11, and D11, which represent the nearly infinitely stiff diaphragm case is listed in Table 3. Also, listed is the stiffness for Models A07, B06, C07, and D06, which represent the stiffness required to essentially duplicate the rigid diaphragm case as previously discussed (Figure 3). For these models the actual roof system required to give these stiffnesses is listed. The "D" in the table refers to metal deck and the number (in inches) to the depth of concrete required to achieve the stiffness. It is clear that some other structural component in addition to standard metal deck is required to obtain a 'rigid' diaphragm. From this preliminary study it appears that acceptable results may be obtained by proper design of a metal deck by itself.

TABLE 3
DIAPHRAGM SHEAR STIFFNESS

MODEL ID	G' K/in.	ROOF SYSTEM
A11	2254	D+4
A07	51	D
B11	4291	D+6
B06	43	D
C11	2276	D+4
C07	51	D
D11	2291	D+4
D06	23	D

CONCLUSIONS AND RECOMMENDATIONS

The use of a rigid horizontal diaphragm to distribute seismically induced lateral loads to the stiffer (stronger) vertical load resisting elements is desirable in a structure in order to achieve an economical design. The fundamental frequency (stiffness) of the diaphragm needed to accomplish this is related to the fundamental frequencies of the elements being joined. Every building structure is unique, but its roof or floor system can achieve 80 to near 100% of the results expected from a rigid diaphragm with only a fraction of the stiffness required for the rigid diaphragm. This study has shown that the fundamental frequency of the diaphragm needs only to be equal to or slightly greater than the highest fundamental frequency of the elements being joined. As shown by this study, a special metal decking properly designed may suffice. A diaphragm design requiring only a metal deck is not only cost effective, but it also reduces the dead load and, hence, inertial loads that the rest of the structure must support. This will lead to a further reduction in cost of the overall structure.

It is recommended that the modeling procedures in this paper be followed to select a trial design for a diaphragm or to determine if an additional more complex model of the facility's building structure needs to be made.

REFERENCES

[1] Luttrell, Larry, Diaphragm Design Manual, Steel Deck Institute, Inc., Second Edition, 1990.