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16. Abstract (Limit: 200 words)

This report summarizes some of the results obtained from the analysis of three different types of buildings: 13-story steel frame (pilot building); 11-story concrete moment resisting frame; and 17-story concrete shear wall building. Details about the design data, the underlying assumptions of the analysis and the computer program used can be found in earlier reports. This report covers a general discussion of the results obtained and predictions of the damage states of these buildings from the various intensities of the earthquake motion used. Interstory displacement data is given in tabular form. Observations on the performance of different design types are made.

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List of Internal Study Reports

1. R.V. Whitman, "Preliminary Work Plans and Schedules," August, 1971.

2. E.H. Vanmarcke and R.V. Whitman, "Background for Preliminary Expected Future Loss Computations," October, 1971.

3. P.J. Trudeau, "Identification of Typical Soil Profiles in the Boston Basin Area," November, 1971.

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1. INTRODUCTION

The purpose of this report is to summarize some of the results obtained from the analysis of three different types of buildings:

a) 13-story steel frame (pilot building)

b) 11-story concrete moment resisting frame

c) 17-story concrete shear wall building

Details about the design data, the underlying assumptions of the analysis and the computer program used, can be found in references (1), (2) and (4). Here we will limit ourselves to a rather general discussion of the results obtained and at the same time we will try to predict the damage states of these buildings from the various intensities of the earthquake motion used.

2. 13-STORY STEEL FRAME BUILDING (PILOT BUILDING)

This building was designed and built according to the Boston Code which had no provisions for earthquake loads. It was found however that the original design that accounted only for gravity and wind, was sufficient for earthquake loads corresponding to U.B.C. seismic zones 1 and 2. Then the building was redesigned for zones 3 and 4 and dynamic analysis of the three different designs were performed. Block masonry walls which form the elevator area were included in the analysis, as elements that break in shear when their interstory deflection exceeds a limiting value. This limiting value was estimated from the properties of the materials used, the total gross area of the walls and from assumptions about workmanship. The numbers used are:

Block walls in the x direction: $\delta_{br} = .02''$

Block walls in the y direction: $\delta_{br} = .033''$

It should be emphasized again that there is a lot of guessing in these two numbers. However, it is believed that their accuracy is consistent with other phases of the project, namely, the seismic risk estimate and the evaluation of damage from both the empirical data and the theoretical analysis.

In the following pages interstory displacements and damage states will be summarized for the various levels of the earthquake intensities used. Also the number of block walls that break in each direction will be listed, so that a better distinction of the estimated damage states can be made.

2.1 Design for Zones 0 - 1 - 2

a) Natural periods (sec.)

$T_x = 1.53$ $\quad T_y = 1.95$ $\quad$ (block walls included)

$T_x = 5.27$ $\quad T_y = 4.5$ $\quad$ (block walls not included)

b) Yield displacements* (in.)

i) x direction

$\delta_{max} = 1.30"$

$\delta_{min} = .70"$

$\delta_{aver} = .85"$ $\quad$ ($\ddot{u}_g$(1^{st} yield) = .061g)

ii) y direction

$\delta_{max} = 1.25"$

$\delta_{min} = .50"$

$\delta_{aver} = .70"$ $\quad$ ($\ddot{u}_g$(1^{st} yield) = .085g)

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* Yield displacements correspond to the 1^{st} breaking point of a trilinear force-deflection diagram, for which total shear for the frame is half the ultimate.

c) Interstory displacements (in.)

		X			Y			No. of broken walls		
$\ddot{u}_{gmax}$	M.M.I.	Max.	Min.	Average	Max.	Min.	Average	X	Y	Damage State
.007	IV	.22"	.009"	.017÷.22	.032	.008	.026	1	0	0÷1
.015	V	.28"	.008"	.017÷.28	.23	.015	.032÷.18	1	5	0÷1
.03	VI	.26"	.012	.018÷.18	.23	.022	.032÷.18	8	10	1÷2
.05	VI.5	.40"	.13	.28"	.33	.12	.25"	12	12	2÷3 **----
.07	VII	.48"	.15"	.30	.45	.17"	.30"	12	13	3.
.15	VIII	1.16 (μ_{max}=1.42)	.31	.50	1.17 (μ_{max}=1.65)	.35	.60"	12	13	4.
.27	1.75*VIII	2.95 (μ_{max}=3.6	.44	.75"	2.43 (μ_{max}=3.12)	.46	.85"	13	13	5÷6

**The dotted line in the column with the damage states indicates the level at which first yielding will take place.

2.2 Design for Zone 3

a) Natural periods (sec)

$T_x = 1.5$ $T_y = 1.82$ (Block walls included)

$T_x = 4.2$ $T_y = 3.31$ (Block walls not included)

b) Yield displacements (in)

i) x direction

$\delta_{max} = 1.03"$

$\delta_{min} = .65"$

$\delta_{aver} \simeq .90$ ($\ddot{u}_g$(1st yield) = .106g)

ii) y direction

$\delta_{max} = 1.1$

$\delta_{min} = .54$

$\delta_{aver} \simeq .75$ ($\ddot{u}_g$(1st yield) = .064g)

c) Interstory displacements (in) (see chart on following page)

2.3 Design for Zone 4

a) Natural periods (sec)

$T_x = 1.42$ $T_y = 1.75$ (Block walls included)

$T_x = 3.08$ $T_y = 2.95$ (Block walls not included)

b) Yield displacements (in)

i) x direction

$\delta_{max} = 1.08"$

$\delta_{min} = .52"$

$\delta_{aver} \simeq .70"$ ($\ddot{u}_g$(1st yield) = .078g)

ii) y direction

$\delta_{max} = 1.04"$

$\delta_{min} = .65"$

$\delta_{aver} \simeq .82"$ ($\ddot{u}_g$(1st yield) = .093g)

c) Interstory Displacements - Zone 3

		X			Y			No. of Broken Walls		Damage State
$\ddot{u}_{gmax}$	M.M.I.	Max	Min	Average	Max	Min	Average	X	Y	
.007	IV	.22	.008	.017÷.22	.094	.013	.020	1	1	0 ÷ 1
.015	V	.25	.01	.018÷.15	.17	.05	.030÷.07	5	7	1 ÷ 2
.03	VI	.22	.015	.018÷.17	.28	.12	.18	9	13	1 ÷ 2
.05	VI.5	.33	.018	.22	.50	.22	.28	11	12	2 ÷ 3
.07	VII	.43	.22	.30	.65	.26	.38	12	13	3
.15	VIII	.83	.45	.58	1.45 (μ_{max}=2)	.45	.65	13	13	4 ÷ 5
.27	1.75*VIII	1.95 (μ_{max}=2)	.60	.90	2.59 (μ_{max}=3.6)	.54	.85	13	13	5 ÷ 6

c) Interstory displacements (Zone 4)

$\ddot{u}_{gmax}$	M.I.	X Max	X Min	X Average	Y Max	Y Min	Y Average	No. of Broken Walls X	No. of Broken Walls Y	Damage State
.007	IV	.12	.008	.016÷.12	.028	.007	.019	1	0	0 ÷ 1
.015		.20	.012	.018÷.08	.14	.019	.025÷.10	7	8	1 ÷ 2
.03	VI	.28	.11	.14	.18	.09	.15	12	12	2.
.05	VI.5	.52	.21	.25	.33	.17	.25	12	13	2 ÷ 3
.07	VII	.73	.29	.35	.47	.25	.35	12	13	3.
.15	VIII	1.72 (μ_{max}=1.6)	.50	.70	1.20 (μ_{max}=1.6)	.44	.72	13	13	4 ÷ 5
.27	1.75*VIII	2.53 (μ_{max}=2.34)	.65	.90	2.41 (μ_{max}=3.2)	.60	1.00	13	13	5 ÷ 6

Certain conclusions can be reached by looking at the results for the three different designs of the pilot building. First, the interstory displacements depend very heavily on the state of the non-structural block walls. For any particular story in which the block walls have not cracked, the corresponding interstory drift is very small and hence the damage (structural and non-structural) should be negligible. On the other hand if the intensity of the shaking is such that it breaks the block walls, then the interstory drift is a function only of the frame stiffness (which is several times smaller than that of the walls). In this case the increase of the interstory drift and subsequently of the damage goes in proportion to the intensity, at least in the elastic range. When the frame starts yielding, this increase is less rapid, but at that point structural damage starts occurring, in which case, the estimated maximum ductility factors must be used in conjunction to the maximum interstory drifts for assessing the damage states. It should be noted however that the way the computer program deals with these block walls is highly ideal. In reality even after considerable cracking has taken place, unless the walls collapse, they still have considerable amounts of resistance left, which could influence the interstory drift rather significantly.

The most important observation, however, is that increase of the earthquake design level does not seem to reduce the damage potential of the original design. The damage states seem to be almost the same for all three designs and it is only for the last case (design for Zone 4) that the intensity to cause first yield has slightly increased. The explanation for this rather odd behavior can be found in references (1) and (3) and the obvious conclusion is that the optimum design for this particular (very flexible) building is that with no earthquake provisions.

3. 17-STORY CONCRETE SHEAR WALL BUILDING

This building was again designed for 5 earthquake zones 0, 1, 2, 3 and 4. Concrete moment resisting frames were used in the longitudinal direction and shear walls in the transverse. The interior columns, designed only for gravity loads, were included in the analysis as parts of frames, whose girders were portions of the slab. This was done in an attempt to reproduce the response better, because although these columns do not contribute significantly to the ultimate strength of the building, they add considerable stiffness, shifting it to a different spectral region. In addition to that, the calculated maximum ductility factors for these frames are a good measure of the damage at the points where the interior columns support the slabs.

Shear walls are treated as far-coupled systems with bilinear moment-curvature diagrams. Stiffnesses and strengths are estimated as described in (1) from the dimensions and the reinforcement of the walls. For each story level the maximum shear capacity of the wall was estimated, and at each time step it was compared with the shear force carried by the wall at the same level. If at any instant this shear capacity was exceeded, a brittle shear failure was assumed by the program in that story and the part of the wall above was treated as an infill. In estimating structural damage in the shear walls, it is good to keep in mind that the definition of the ductility factor here is different than that of a frame (1). Shear walls are not designed as ductile structures (although they could be) and small increases of the applied moment above the value that determines the elastic limit can cause excessive amounts of yielding which the wall is not designed to take, resulting in a catastrophic failure. It is for this reason that it has been recommended for shear walls to be designed so that they remain elastic under the strongest earthquakes. It is the author's feeling that small increase of the wall reinforcement would be sufficient for elastic behavior for many of the cases analyzed.

At the time this report is written, complete results are available only for two designs: Zones 0 and 3. They are tabulated below:

3.1 Design for Zone 0

a) Natural periods (sec)

$T_x = 4.38$

$T_y = 3.14$

b) Yield displacements (ft) (for frames only)

i) x direction

δ_{max} $.04 \div .08$ $\quad \delta_{aver} \simeq .06 \quad \ddot{u}_g$(1st yield) *CMRF: .18g

δ_{min} $.035 \div .07$ $\quad$ *N.S.FR: $.03 \div .06$g

ii) y direction

$\delta_{max} \simeq .01 \div .018$ $\quad \delta_{aver} \simeq .015 \quad \ddot{u}_g$(1st yield) N.S.FR: $.03 \div .042$g

$\delta_{min} \simeq .05 \div .01$ $\quad$ *SH.WALL .08g

c) Interstory displacements (ft) (see following page)

3.2 Design for Zone 3

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*CMRF = Concrete moment resisting frame
N.S.FR = Non-structural frame (formed from the interior columns)
SH.WALL = Shear wall

c) Interstory Displacements (Zone 0) (ft)

		X			Y			Damage State
$\ddot{u}_{gmax}$	M.I.	Max	Min	Aver.	Max	Min	Aver.	
.007	IV	.002	.0007	.0018	.003	.0004	.0025	0
.015	V	.004	.0014	.0038	.006	.0009	.0053	0
.03	VI	.009	.003	.0076	.013	.0017	.011	0 ÷ 1
.05	VI.5	.015	.005	.012	.021 (μ_{max}=1.2)	.003	.015	1 ÷ 2 (3)
.07	VII	.022	.007	.016	.030 (μ_{max}=1.70)	.004	.026	2 ÷ 4
.15	VIII	.050	.012	.030	.049 (μ_{max}=4.5)	.007 (SH.W)	.038	5 ÷ 6
.27	1.75 VIII	C O L L A P S E						8

a) Natural periods (sec)

$T_x = 3.13$

$T_y = 2.32$

b) Yield displacements (ft) (for frames only)

i) x direction

$\delta_{max} \simeq .04 \div .08$ $\quad \delta_{aver} \simeq .06 \quad \ddot{u}_g$(1st yield) C.M.R.F. $\simeq .07 \div .14g$

$\delta_{min} \simeq .02 \div .04$ $\quad$ N.S.FR $\simeq .02 \div .04g$

ii) y direction

$\delta_{max} \simeq .01 \div .018$ $\quad \delta_{aver} \simeq .015 \quad \ddot{u}_g$(1st yield) N.S.FR $\simeq .04 \div .065g$

$\delta_{min} \simeq .05 \div .01$ $\quad$ SH.WALL $\simeq .14g$

c) Interstory displacements (ft) (see following page)

Due to the relatively large stiffness of the shear walls, the interstory displacements for this type of building are small, at least before the walls start yielding or cracking. So very little damage (at least of the type associated with interstory drift) is to be expected even for intensities up to VII. After cracking starts occuring however, because of the brittle nature of the walls, structural damage is probably inevitable and transition from a lower damage state to a very high one, is possible even if the Mercalli intensity of the earthquake changes only by one scale.

Another interesting observation is that the distribution of damage with height is different than that of a building with frames, because of the difference in the deflected shapes of the two types of structures. Framed structures deforming like a shear beam will have most of the damage at the lower floors, while shear wall buildings that deform as canti-

c) Interstory Displacements (Zone 3) (ft)

		X			Y			Damage State
$\ddot{u}_{gmax}$	M.I.	Max	Min	Aver	Max	Min	Aver	
.007	IV	.0032	.0008	.0026	.0020	.0002	.0014	0
.015	V	.007	.001	.0055	.0043	.004	.003	0
.03	VI	.014	.0035	.011	.0086	.0008	.006	0 ÷ 1
.05	VI.5				.0145	.0016	.010	0 ÷ 1
.07	VII				.020 (μ_{max}(fr)≃1.1)	.0022	.015	1 ÷ 2
.15	VIII				.045 (μ_{max}(fr)=2.4, μ_{max}wall=3.9)	.0048	.030	4 ÷ 5
.27	1.75*VIII	CMRF: μ_{max} = 2.7			SH.W1: μ_{max} = 10.5			~ 7 ÷ 8
		N.S.FR: μ_{max} = 10			SH.W2,3: μ_{max} = 9			

levers should be expected to suffer more at the top. Structural damage of the walls, however, should be expected at the lower floors or in case of interaction with frames around the middle.

As a final comment in this section, it should be mentioned that absolute accelerations for the shear wall buildings are relatively higher than those in framed structures, so damage associated with acceleration should be expected higher.

4. 11-STORY CONCRETE FRAME BUILDINGS

This building has the same floor plan with the 17-story shear wall building, but uses moment resisting concrete frames in the transverse direction instead of shear walls. As of this moment we have complete results for the zone-3 design, while for zones 2 and 4 we have results only for the transverse direction and the strongest earthquake.

4.1 Design for Zone 3

a) Natural periods (sec)

$T_x = 2.35$

$T_y = 2.04$

b) Earthquake to first yield

i) x direction: $\ddot{u}_g$ (1st yield) CMRF $\simeq .07 \div .14g$

N.S.FR $\simeq .13 \div .26g$

ii) y direction: $\ddot{u}_g$(1st yield) CMRF $\simeq .06 \div .12g$

c) Interstory displacements (ft) (see following page)

c) Interstory Displacements (Zone 3) (ft)

		X			Y			Damage State
$\ddot{u}_{gmax}$	M.I.	Max	Min	Aver	Max	Min	Aver	
.007	IV	.0028	.0009	.0020	.0027	.0007	.0019	0
.015	V	.006	.002	.0042	.0058	.0015	.004	0
.03	VI	.012	.004	.0086	.011	.003	.008	0 ÷ 1
.05	VI.5	.02	.0064	.0143	.019	.005	.014	1 ÷ 2 (3)
.07	VII	.028	.009	.02	.027	.007	.019	2 ÷ 3
.15	VIII	.060	.020	.045	.055	.013	.035	3 ÷ 4
.27	1.75*VIII	.092	.024	.08	.126	.02	.065	5 ÷ 7
		$\mu_{max} = 2$			$\mu_{max} = 4.8$			

4.2 Design for Zone 2

a) Natural period

$T_y = 2.65$ sec

b) Interstory displacements (ft)for the strongest earthquake (1.75 * VIII)

$\delta_{max} \simeq .14'$

$\delta_{min} \simeq .027'$ $\quad \delta_{aver} \simeq .080$

c) Maximum ductility factor: $\mu_{max} = 4.6$

d) Damage state: 5 ÷ 7

4.3 Design for Zone 4

a) Natural period

$T_y = 1.79$ sec

b) Interstory displacements (ft) for the strongest earthquake (1.75 * VIII)

$\delta_{max} = .16'$ $\quad \delta_{aver} \simeq .065$

$\delta_{min} = .016$

c) Maximum ductility factor: $\mu_{max} = 6.17$

d) Damage state: 5 ÷ 7

The main observation here (which is also true for both the pilot and the shear wall buildings) is that increase of the design forces even up to superzone-4, does not alter the damage state caused by the very strong earthquake (1.75 * VIII). If this is set as an objective when designing for Zone 4, then the doubling of the base shear coefficient is not sufficient. Other factors and strategies should probably be sought.

As a final summary we have tabulated the damage states of all the cases analyzed, in the next page. Blanks are to be filled when more results from the analysis become available.

DAMAGE STATES OF BUILDINGS ANALYZED

		PILOT BUILDING			17-STORY CONC. SH. WALL					11-STORY CONCRETE FRAME				
$\ddot{u}_{gmax}$	M.M.I	0-1-2	3	4	0	1	2	3	4	0	1	2	3	4
.007	IV	0÷1	0÷1	0÷1	0			0					0	
.015	V	0÷1	1÷2	1÷2	0			0					0	
.03	VI	1÷2	1÷2	2	0÷1			0÷1					0÷1	
.05	VI.5	2÷3	2÷3	2÷3	1÷2			1÷2					1÷2	
.07	VII	3	3	3	2÷4			1÷2					2÷3	
.15	VIII	4	4÷5	4÷5	5÷6			4÷5					3÷4	
.27	1.75*VIII	5÷6	5÷6	5÷6	8			7÷8				5÷7	5÷7	5÷7

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