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CORRELATION OF CODE DESIGN RECOMMENDATIONS FOR BUILDING WITH STRUCTURAL WALLS BASED ON OBSERVATION OF ACTUAL EARTHQUAKE RESPONSE RECORDS

R. L. Boroschek and F. V. Yañez

Civil Engineering Department, University of Chile
Blanco Encalada 2120, 4 Piso.
Santiago Chile

ABSTRACT

The observed response of medium highrise buildings during the last earthquakes in USA, Chile, Mexico and Japan have indicated that buildings with structural walls or dual systems of frame and walls behave considerably well during strong shaking. To study the actual behavior of buildings with shear walls, the response records from two USA buildings were studied. The basic behavioral characteristics of these buildings were identified along with various engineering design parameters. Response parameters identified included the dynamic properties, base rocking of walls, interstory drift and torsion. Comparison with the behavior of a USA space frame structure is also made. The characteristics of Chilean structural wall buildings are presented and related with observed response. Comparison with basic design recommendation are presented.

KEYWORD

reinforced concrete structural walls; shear walls; seismic records; dynamic properties, dynamic response; seismic design.

INTRODUCTION

There are several design recommendations for analyzing and designing buildings with structural walls. These recommendations were developed based on damage survey of existing structures after severe earthquakes and experimental and analytical work. But it is now possible to study directly the behavior of these structures using actual response records.

The extensive instrumentation of building structures in California and recent seismic events created a large amount of data. Its analysis permits the evaluation of the actual behavior of the structures. In this manner it is possible to correlate structural and nonstructural damage and behavior with typical design and analytical assumptions.

This essay presents the instrumentation of two buildings structured with walls, their dynamic characteristics, and their response to three events. The response of a third building structured with a space frame is presented as a comparison.

The buildings are located in the city of San Jose, California 19 to 23 kilometers from the epicenters. Building 1 and 2 are 0.7 km apart. Building 3 is located 2 km from Building 1 and 2.

Additionally the response is related with the typical Chilean buildings that have survive severe earthquakes.

BASIC STRUCTURAL CHARACTERISTICS OF THE BUILDINGS

USA-Building 1

This is a 10 storey residential building, designed and constructed between 1971 and 1972 (CSMIP Station 57356), Fig. 1. The earthquake and gravity load resisting system consist of postentioned lightweight concrete flat slabs connected to structural walls. In the transverse direction (EW) the walls are regularly spaced. In the longitudinal direction (NS) the walls are located along the center line. One of the walls ends at the sixth level and there are several irregularities in the first floor. The walls are supported on a continuous footing resting over piles. Detailed description of Building 1 and Building 2 can be found in Mahin et. al (1989) and Boroschek et. al (1990).

The Building was instrumented under the California Strong Motion Program (CSMIP). The instruments consist of 13 Force Balance Accelerometers connected to an analog central recording station. Sensor location is presented in Fig. 1. As shown later these arrangement permits the identification of the dynamic characteristics, the interstory drifts, torsion, slab deformation and wall base rotation.

USA-Building 2

This a 10 storey commercial building with one underground level, Fig 2. It was designed in 1964 and constructed in 1967. The gravity load resisting system consist of lightweight concrete slabs supported on a frame system. The lateral load resisting system consist of structural walls and frame system in the transverse direction (EW) and frames in the longitudinal direction (NS). The building is supported in a 1.5 meter depth foundation slab. The instrumentation is similar to that of Building 1.

USA-Building 3

This is a 13 story office building, designed in 1972 and finished in 1976, Fig. 3. The vertical and lateral resisting system consist of steel space frames. Detailed description of the structure can be found in Boroschek and Mahin (1991).

The building is instrumented with 22 sensors as shown in Fig. 3.

Response records

The systems have recorded a series of events, the behavior is described for the event of Morgan Hill, 1984 (Ml=6.2 (BRK)), Mt. Lewis 1986 (Ml=5.8) and the Loma Prieta, 1989 (Ml=7.0 (BRK)). The records obtained were process by CSMIP. A band pass filter with different corner frequencies were used. The upper corner was set between 3.3 and 5.6 seconds according to the level of noise. According to CSMIP (1985) this implies a nominal error of 1.5 gal in accelerations and 0.5 cm in displacement (3.3 sec corner). In this study these nominal values are considered and responses lower than these values are disregarded. Additional care is taken when a signal presents periods close to the corner frequency of the filters.

Dynamic Properties of the Building

Parametric and non parametric system identification techniques were used to identify the basic dynamic properties. The parametric techniques were based on the development of Beck (1978) and McVerry (1979). A detailed description of these procedures is presented and results are in Soto (1994). The fundamental

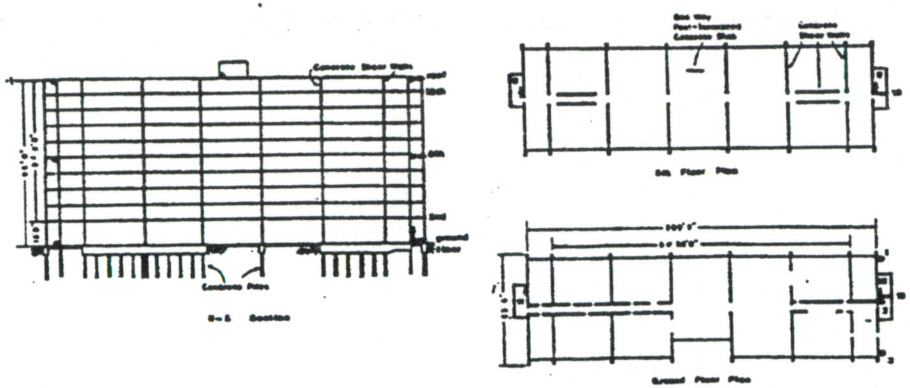

Fig. 1. Building 1. Basic characteristics and sensor location. (CSMIP,1985).

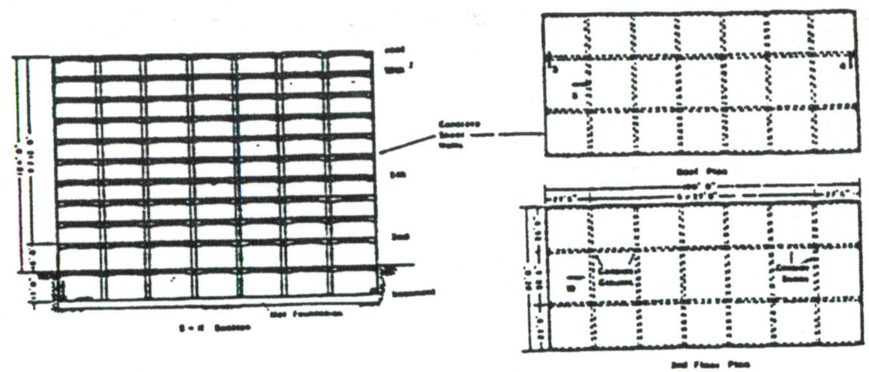

Fig. 2. Building 2. Basic characteristics and sensor location. (CSMIP,1985).

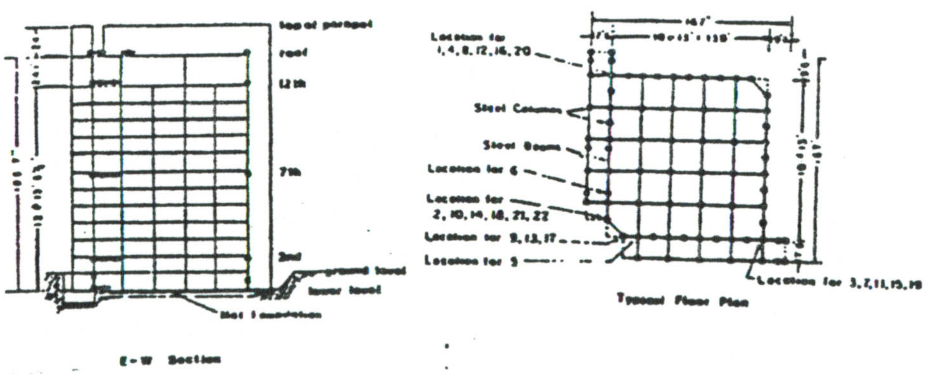

Fig. 3. Building 3. Basic characteristics and sensor location. (CSMIP,1985).

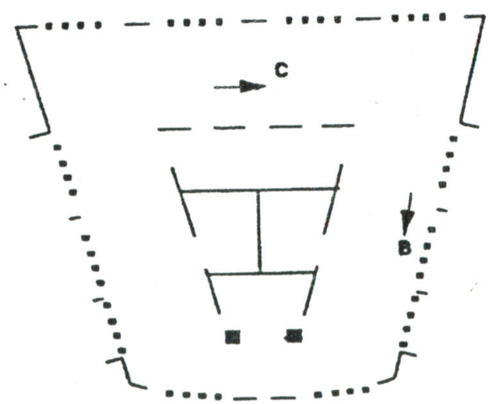

Fig. 5. Typical floor plan of a Chilean office Building.

periods found are shown in Table 1.

Table 1: Fundamental Periods (SEC)

DIRECTION	BUILDING 1	BUILDING 2	BUILDING 3
EW	0.45 (0.33)	0.62 (0.36/0.73)	2.2 (1.7)
NS	0.70 (0.50)	0.95 (1.1)	2.1 (1.7)

Response Envelopes

Table 2,3,4 present response envelopes for the events studied.

Response characteristics were obtained directly from time records. For the interstory drift index (IDX) a linear relation was considered between recording points at each time step. A spline could also be used but is was not considered necessary for this case. IDX was obtained by subtracting consecutive records, and indexes with and without rocking motion were considered. To evaluate the rocking motion the response of vertical base records are analyzed in time and frequency domain. Also the importance of base rocking in the overall roof displacement is evaluated assuming a rigid body motion of the structure. Rocking is only considered if values are over the noise level of the system.

Torsion is evaluated subtracting records of the same level taking into account the ground input torsion. Also these records are used to evaluate the slab deformation.

The base coefficient is calculated from the estimated inertia forces. These forces are obtained by interpolating the time history records for the intermediate floors and considering the floor story weight as derived from structural plans. In the case where more than one record exist in a given direction the average values are considered.

There were no free field instrument and therefore limited information on soil structure interaction was obtained.

COMMENTS ON THE RESPONSE

1. The structural systems show a strong difference in the response. The story displacements and the interstory drift index as it can be expect are considerably lower in the bearing wall structure and the dual system. For Building 1 the measured drift coefficient corresponds to 36% (EW) and 19% (NS) of the recommended Uniform Building Code limit (UBC, 1991). If the IDX are scaled to have the same base as required by the UBC the values obtained are 44% (EW) and 22%(NS). For Building 2 the IDX value are 30% (EW) and 39% (NS) for non scale values. Scale values are 20% in the dual system direction and 16% for the frame system, assuming a special moment resisting frame. For Building 3 the scale value is 20% never the less when the actual base shear is used the IDX for this building is 340% of the limit established by the UBC.

The large drift in Building 3 cause considerable functional and nonstructural damage. The cause for these behavior are explained in Boroschek and Mahin (1991). This difference is clearly observed in Fig. 4 that show the relative displacement of the highest representative recording level for the three structures divided by the record position (average IDX). Also this can be observed considering that the maximum relative displacement at the roof level for the wall system is around 40% of the maximum absolute displacement, in contrast for building 3 therefore the relative maximum drift is 85% of the maximum total displacements.

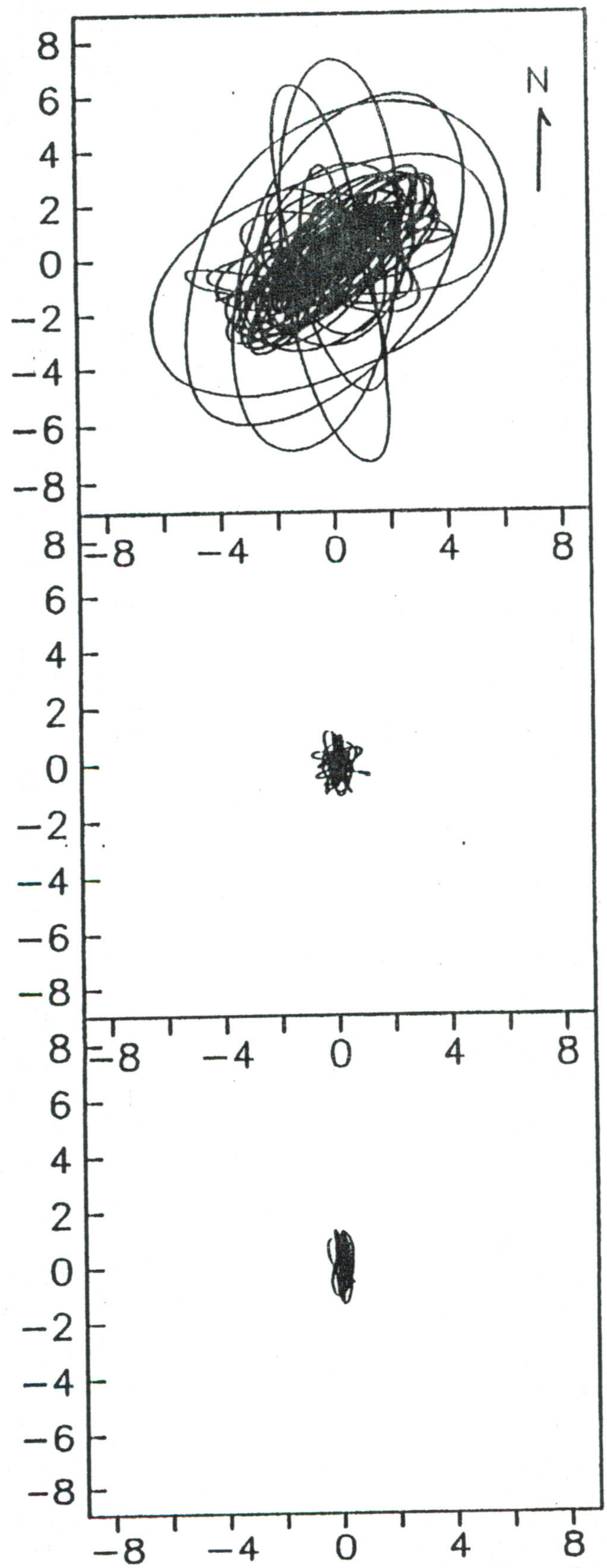

Fig. 4. Average IDX for the tree buildings during Loma Prieta earthquake. Upper Fig. corresponds to the steel moment resisting frame (3), middle one corresponds to the dual system (2), and bottom one corresponds to the bearing wall building (1).

Table 2: Building 1 Response Envelope

	MORGAN HILL	MT LEWIS	LOMA PRIETA
MAX. GROUND VERTICAL ACCEL.(g)	0.04	0.03	0.09
MAX. GROUND HORIZONTAL ACCEL.(g)	0.06	0.04	0.12
MAX. STRUCTURAL ACCEL.(g)	0.22	0.12	0.37
MAX. RELATIVE DISPL. EW (cm)	0.76	--	1.57
MAX. RELATIVE DISPL. NS (cm)	2.06	--	4.06
MAX. IDX. EW (1/1000)	0.2	0.5	0.78
MAX. IDX. NS (1/1000)	1.0	0.4	1.8
MAX. TORSION (cm)	0.53	< 0.50	0.94
BASE SHEAR COEFFICIENT EW (V/W)	0.096	0.05	0.15
BASE SHEAR COEFFICIENT NS (V/W)	0.100	0.04	0.16
MAX BASE ROTATION (cm) (DIF VERT REC)	< 0.50	< 0.50	0.53
FLOOR SLAB DEFORMATION (cm)	< 0.50	< 0.50	< 0.50

Table 3: Building 2 Response Envelope

	MORGAN HILL	MT LEWIS	LOMA PRIETA
MAX. GROUND VERTICAL ACCEL.(g)	0.03	0.02	0.06
MAX. GROUND HORIZONTAL ACCEL.(g)	0.06	0.03	0.10
MAX. STRUCTURAL ACCEL. (g)	0.22	0.09	0.37
MAX. RELATIVE DISP. EW (cm)	2.43	0.68	6.43
MAX. RELATIVE DISP. NS (cm)	3.25	1.49	5.00
MAX. IDX. EW (1/1000)	0.6	0.20	1.5
MAX. IDX. NS (1/1000)	1.2	0.55	1.3
MAX. TORSION (cm) (DIFF OPPOSITE REC)	< 0.50	< 0.50	1.71
BASE SHEAR COEFFICIENT EW (V/W)	0.13	0.047	0.21
BASE SHEAR COEFFICIENT NS (V/W)	0.11	0.044	0.15
MAX. WALL ROTATION (cm)	< 0.50	< 0.50	< 0.50
MAX. SLAB DEFORMATION (cm)	< 0.50	< 0.50	< 0.50

2. Rotation of walls at base level was observed but information is limited due to its low level that makes interpretation of the records uncertain. For Building 2 the derived rigid body motion at the highest recording position corresponds to 36% of the relative roof displacement and 18% of the absolute roof displacements.

3. The fundamental period obtained from records are compared with UBC 1991 recommend formulas. Larger differences are found for wall systems. For building 3 the difference indicates the considerable flexibility of this kind of structures.

The excellent behavior of wall buildings 1 and 2, especially building 1 in EW direction, indicates the positive effect of a large amount of walls, limiting the interstory drift and therefore limiting the structural and nonstructural damage. These facts are consistent with the excellent performance of similar chilean buildings during severe earthquakes. Although the american seismic movements may be very different from the chilean ones, the resulting response in both cases have similar characteristics.

It is interesting to note that the amount of wall area to floor area of building 1 in EW direction is about 40% of that of a similar chilean buildings.

CHILEAN PRACTICE OF REINFORCED CONCRETE BUILDING DESIGN

The 1985 Llo-Lleo earthquake showed that buildings designed according to the chilean practice suffered limited damage mainly due to structural walls which are almost exclusively used in highrise buildings for gravity and lateral loads. Some features of this practice are as follows.

Table 4 : Building 3 Response Envelope

	MORGAN HILL	MT LEWIS	LOMA PRIETA
MAX. GROUND VERTICAL ACCEL. (g)	0.04	0.04	0.10
MAX. GROUND HORIZONTAL ACCEL.(g)	0.02	0.02	0.11
MAX. STRUCTURAL ACCEL. (g	0.17	0.32	0.34
MAX. RELATIVE DISPL. EW (cm)	18.64	15.63	34.72
MAX. RELATIVE DISPL. NS (cm)	17.94	33.19	38.17
MAX. IDX. EW (1/1000)	4.1	3.1	8.2
MAX. IDX. NS (1/1000)	4.0	7.2	8.5
MAX. TORSION (cm)	7.28	12.22	12.32
BASE SHEAR COEFFICIENT EW (V/W)	0.08	0.06	0.18
BASE SHEAR COEFFICIENT NS (V/W)	0.09	0.15	0.17

Materials

All buildings use concrete with specified strength of 20 to 25 MPa, typically.

Reinforcing bars up to 32 mm diameter with 420 MPa yield strength are currently used. Although 280 MPa yield strength was mainly used in the past.

Structural system

Most reinforced concrete buildings are in the range of 5 - 20 stories high with one or two basement stories, with regular floor plan, usually symmetric with respect of both axes. The usual story height is 2.4 m. for residential buildings.

The main seismic resistance system are structural walls (walls with irregular openings are very common) with a wall area to floor area ratio of about 6% in both directions. The nominal shear stress in the walls is in the range of 0.2 to 0.8 MPa. It is not rare to find wall thickness of 300-400 mm for 6 to 8 story buildings constructed before 1980.

The periods for chilean buildings is typically close to 0.05 times the number of representative stories over ground.

These facts come from an old code provision (NCh 429, 1957) requiring (a) that the diagonal tension in any element should not be larger than the values of column 2, Table 5 and (b), that the total diagonal tension should be taken by shear reinforcement if the diagonal tension were larger than the values of column 3, Table 5. These limits applied when the working stress method of design is used.

Table 5 Limits for diagonal tension

1 Concrete Grade	2 Diagonal Tension (MPa)	3 Diagonal Tension (MPa)
120	1.4	0.4
160	1.6	0.6
180	1.7	0.65
225	1.8	0.7
300	2.0	0.8

Clearly the chilean designer tried not to overcome the limits of column 3 in order to use minimum shear reinforcement ratio of 0.2% distributed in both faces.