BUILDING DAMAGE IN 2010 CHILE OFFSHORE MAULE EARTHQUAKE
NO.4 FOURTEEN STORY RESIDENTIAL BUILDING IN VINA DEL MAR

Chile Earthquake　RC building　Viña del Mar
Seismic assessment　Lateral capacity　Pushover analysis

Member ○Niwade LUMLERDLUCKSANACHAI 1*
Member Susumu KONO 2**
Member Taiki SAITO 3***

1. INTRODUCTION

The epicenter of the earthquake was just offshore from the Maule Region, approximately 400 km southwest of Viña del Mar. A residential building in Viña del Mar (Called Building F hereafter) was damaged in this earthquake. The damage was observed at site two months after the earthquake and its causes were investigated by numerically analyzing the building.

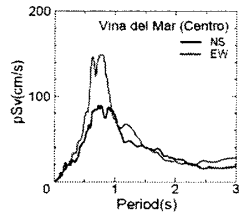

Fig. 1 Pseudo Velocity Response Spectra [1]

The pseudo velocity response spectra for 5% damping in Vina del Mar is shown in Fig. 1 [1]. A relatively large peak value of 150 cm/s can be seen at periods of 0.5s and 0.7s.

2. GENERAL INFORMATION OF BUILDING

The information of Building F is taken from Reference [2] and [3]. Building F is a 14 story reinforced concrete building which was constructed in 1978. Its plan and elevation are shown in Fig. 2 and Fig.3, respectively. The structural system consists of structural walls with flat slabs in both longitudinal and transverse directions. The structural walls are arranged almost symmetrically in plan with the exception of some wall openings that are staggered from floor to floor. The walls are effectively continuous over height. The foundation is a mat founded seven meter below grade. Structural walls are coupled by relatively deep beams at several locations over the height.

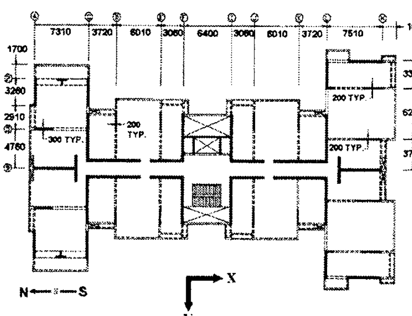
Fig.2 First floor plan of Building F

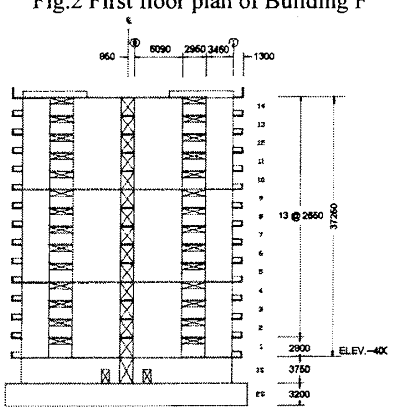
Fig. 3 Elevation of Building F

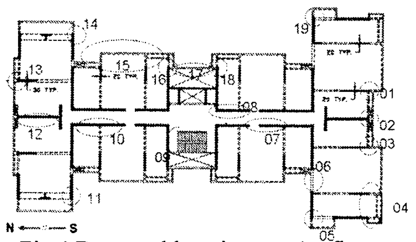
Fig.4 Damaged locations on 1st floor

Fig.4 shows the damaged location on 1st floor. The damage descriptions correspond to the damaged location numbers in Fig.4 are shown in Table 1

Table 1 Damage description

Location	Damage description
1, 2, 3	Wall edge crushed in the vertical direction. Vertical reinforcement at edge buckled.
4	Cover spalling due to compression in the vertical direction. Minor damage.
5	Cover is about to spall due to compression in the vertical direction. Minor damage.
6	Shear cracks next to window opening
7, 8, 10	Walls failed in shear. Supplemental walls on two faces constructed after 1985 EQ came out due to insufficient anchorage.
9	A plastic hinge formed at the north side of an external staircase at each floor.
11	Wall edge crushed in the vertical direction. Too many vertical reinforcement was lap spliced to make congestion and concrete may not have been consolidated. Windowpanes were broken.
12	Wall has horizontal cracks.
13, 14	Finishing mortal spalled.
15	Windowpanes were completely taken out probably due to heavy damage.
16	Finishing mortal spalled. Added column crushed in compression near the ground.
17	The wall behind the elevator has multiple diagonal shear cracks.
18	Wall edge crushed in the vertical direction near the ground. Vertical reinforcement at edge buckled.
19	Finishing mortal closed to the ground spalled.

Shear wall thickness was 300 mm (1st-4th Floor), 250 mm (5th-9th Floor), 200 mm (10th-Roof). Longitudinal and shear reinforcement ratio of the walls is 1% and 0.2% respectively. Slab thickness is 130 mm through the height.

Table 2 Wall ratio

	Wall length, SL (m)	Wall length ratio, SL/A_{floor} (1/m)	Wall area, SA (m^2)	Wall area ratio, SA/A_{floor}
X direction	89.90	0.0967	25.51	0.0274
Y direction	93.20	0.1002	26.28	0.0283

Reinforcing steel type 'A63-42H' has the yield strength of 412 N/mm^2. Nominal compressive strength of concrete is 25.0 N/mm^2 (converted for ∅100X200 cylinder).

Total weight of the building is 138 MN, the total area of shear walls of first floor is 54.69 m^2 and the axial force ratio, N/Af_c' is 0.102. Typical floor area is

Table 3 Period and mode shape of first three modes of the building with beam (based on Sec. 3.1)

Mode	Period (second)	Mode Shape
1st	0.731	X-direction translation
2nd	0.697	Torsion
3rd	0.562	Y-direction translation

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930 m^2 and wall ratio in X and Y direction is shown in Table 2. The period and mode shape of first three modes of the building with beam are shown in Table 3

3. NUMERICAL ANALYSIS

3.1 Modeling

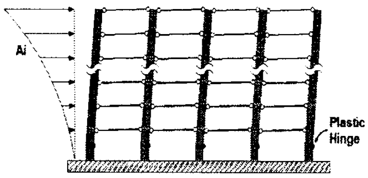

Fig. 5 Model assumption

The building was modeled with frame analysis program STERA3D [4]. The slab was assumed to be rigid in axial direction and have no flexural stiffness. Ai Distribution was employed for the vertical distribution of seismic loading.

Vertical load assumptions for live load is 3000 N/m^2 (Typical), 1000 N/m^2 (Roof), 1000 N/m^2 (Partition wall) and 1000 N/m^2 (Finishing)

STERA3D is used to evaluate lateral capacity of the building, The stress-strain relation of concrete is a bilinear model with slope 0.001E$_c$ beyond f'$_c$ and that of steel is an elasto-plastic model.

3.2 Analytical results

Most shear walls in the longitudinal direction (X-direction hereafter) failed in shear at the basement or first floor, therefore the numerical results in X-direction is presented.

The capacity of building in X-direction by STERA3D is compared to the base shear based on the virtual work theory assuming the identical failure mechanism. The capacities are close to each other for models without beams. However, beams were connected shear walls to shear walls in reality, hence the model with beams is considered close to the real behavior. Fig.6 shows the base shear capacity of 0.24

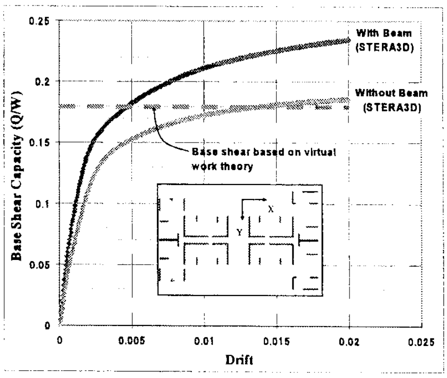

Fig. 6 Building capacity (North bound loading)

3.3 Shear capacity by AIJ equation

The lateral load capacity based on shear failure mode of walls was computed with shear capacity formula by AIJ. The total base shear capacity is 0.634 and 0.437 for Eqs. (1) and (2), respectively.

$$V_u = t_w l_w p_s \sigma_{sy} \cot\phi + \frac{1}{2}\tan\theta(1-\beta)t_w l_{wa} v \sigma_B \tag{1}$$

$$Q_{su} = \left\{ \frac{0.068 p_{te}^{0.23}(18+F_C)}{\sqrt{M/(Q\cdot l)+0.12}} + 0.85\sqrt{p_{se}\cdot\sigma_{wy}} + 0.1\sigma_{0e} \right\} \cdot b_e \cdot j_e \tag{2}$$

As shown in Fig.6 the capacity based on flexure failure mode (Wall base) of building is 24% of the building weight while capacity based on shear failure mode (All walls) is at least 43% of building weight. Therefore, the flexure failure mode should be dominant but the shear failure of walls occurred in reality. The difference is considered due to dynamic effects.

From Table 3 the period of first and second modes are close to the period at peak value of pseudo velocity response spectra which the first mode is X-translation and second mode is torsion. The failure mechanism may occur in the combination of the first two modes. However the real failure modes cannot be simulated

4. CONCLUSIONS

Pushover Analysis seems to be unable to describe the causes of damage to the building. Moreover, the period of 1st and 2nd mode are close to the period at peak value of pseudo velocity response spectra. Hence, the first and second modes may be activated to cause the shear failure of walls. Therefore, it is necessary to have further studies in the future considering dynamic effects.

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*1 Master student, Dept. of Architecture & Architectural Engineering, Kyoto University

*2 Associate Prof., Dept. of Architecture & Architectural Engineering, Kyoto University, Ph.D., JCI Member

*3 Chief Research Engineer, International Institute of Seismology and Earthquake Engineering, Ph.D., JCI Member