A Case Study of Retrofit Design of Damaged High-rise RC Building in 921 Earthquake

Yi-Ping Tseng[1] and Teu-Chiang Wu[2]

1、Introduction

A strong earthquake (Richard magnitude up to 7.3) occurs in Nantour at 1:47 AM in September 21, 1999. This earthquake is named as 921 earthquake. The ground acceleration of 921 earthquake is as large as .981 g. Strong vertical earthquakes are also observed. It is known as the strongest earthquake in Taiwan during the last century.

2、General Descriptions of Damaged Building

The damaged building in the present case study is an 11-story reinforced concrete building with one story basement. It will be called the present building. Especially, the soil liquefaction is found in the neighborhood area in Nantour. Around the site, two 3-story buildings without basement settle 50～60cm。

The damages of the present building can be summarized as below：

1.some cracks of .5 mm width occur in beams and columns of lower stories.

2.some brick walls and RC walls of lower stories fall down, and some are seriously damaged.

3.large pieces of concrete in the stairway down to basement, and some reinforced bars

4. most of basement walls leak, especially in the corner.

The damages of the present building which occurred in 921 earthquake is not very serious, however, some disadvantages for the seismic resistance of the present building are observed：

1. L-shaped irregular building

2.open space in the first story

3、Repair Design of Damages

Based on engineering experience and those related earthquake damaged records, the repair design is addressed. The main work consists of ：

1.inspects the damage of the structure.

2.records the length, width, depth, and etc. of damage.

3.address the repair design.

4.summarize the damages.

5.estimates the repair expenditure.

4、Soil Liquefaction

One characteristics of 921 earthquake is that soil liquefaction is common. Except coastal area, many inner land area also occur the soil liquefaction. It is then necessary for this case to implement the soil drilling。

Two holes were drilled with 20m depth. According to the 86 building code and the Seed

[1] Professional Civil Engineer, President of Do & Find Consultants Co. Ltd.
[2] Professional Structural Engineer, Do & Find Consultants Co. Ltd.

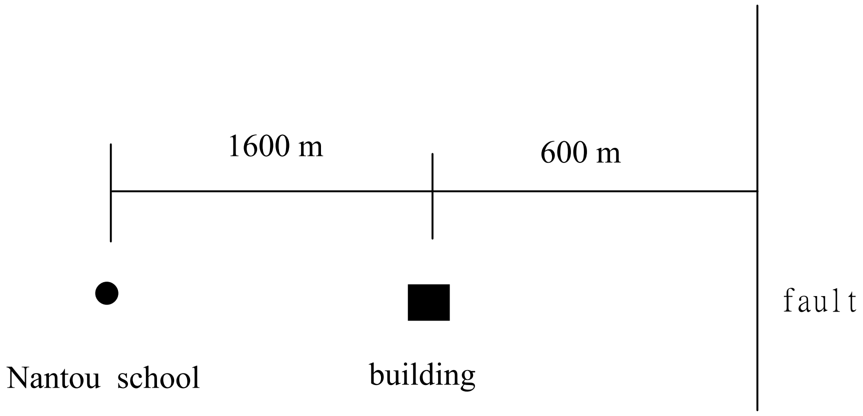
method, JRA method, NJRA method and T-Y method in academic literature, the soil liquefaction potential is evaluated. The evaluated results of hole #BH-1 can be summarized as：liquefaction potential is found only at 9.0m depth, but none of the other spots (Seed method and T-Y method). It is thereby concluded that only small soil liquefaction would occur in our research neighborhood area.

Method	Seed	NJRA	T-Y	Code*
D(m)	Liquefaction potential			
1.5	safe	safe	Safe	Safe
3.0	Safe	Safe	Safe	Safe
4.5	Safe	Safe	Safe	Safe
6.0	Safe	Safe	Safe	Safe
7.5	Safe	Safe	Safe	Safe
9.0	Liquid	Safe	Liquid	Safe
10.5	Liquid	Safe	Safe	Safe
12.0	Safe	Safe	Safe	Safe
13.5	Liquid	Safe	Safe	Safe
15.0	safe	safe	safe	safe

* code is the 1997 building code, wherein the soil liquefaction is included firstly.

5、Reason of Damages

The nearest earthquake monitor station of Central Weather Bureau from the present building is the Nantour elementary school. The distance is 1600 m from the site to Nantour elementary school.



The ground acceleration observed in Nantour elementary school is:

South and North direction: 420.02 gal
East and West direction: 340.10 gal
Vertical direction: 275.38 gal

The related location is drawn as the following picture. Which indicates that earthquake occurred around the present building is very strong. Since the distance from site to the Che-Long-Puu fault is 600 m, which is much less than the distance 2200 m from Nantour elementary school to Che-Long-Puu fault, it is concluded that the earthquake is much stronger than in Nantour elementary school. In other word, the earthquake resistance demand is more than the 1989 code requirement, 1997 code requirement and even the 88.12.29 current code requirement. In brief, excess magnitude earthquake is the main reason for building damage.

6、Difference of Building Codes

The present building is designed in 1993, and built in 1995. It is designed according to Building Code (1989).

After 1989, a new building code is published in 1997. The current building code is published in 2000, whereas the seismic zones are defined based on the 921 strong ground motion record. The different seismic design base shears are investigated herein.

6.1 1989 Code

According to 1989 code, the lateral seismic loading is：

V=Z K C I W

Where the seismic zone coefficient Z=.8 (medium zone), the structural system coefficient K=.8 (partially ductile RC building with nonstructural wall). The height of building is $h_n = 34.50\,\text{m}$, therefore the fundamental period is $T = 0.06 h_n^{3/4}$=.854 sec. The coefficient of seismic base shear is $C = \frac{1}{8\sqrt{T}} \leqq 0.15$，therefore C = 0.146. For apartment, I=1. W is the total weight of the building.

6.2 1997 Code

According to 1997 code, the lateral seismic loading is：

$$V = \frac{ZI}{1.4\alpha_y}\left(\frac{C}{F_u}\right)_m W ,$$

or

$$V = \frac{ZI}{3.5\alpha_y F_u}\left(\frac{C}{F_u}\right)_m W$$

with

$$\left(\frac{C}{F_u}\right)_m \leqq 1.0$$

where the coefficient of horizontal ground acceleration seismic zone Z=0.23 (Nantour is zone 2). The fundamental period is different; $T = 0.07 h_n^{3/4}$ =.997 sec. The spectrum coefficient of horizontal ground acceleration is $C = \frac{1.5}{T^{2/3}}$ = 1.503，for the type II site. The coefficient for the initial yielding is α_y=1.5(RC building). The ductility capacity of structural system R=3.2 (moment-resistant frame system with nonstructural wall). Allowable ductile capacity is $R_a = 1 + \frac{1}{2}(R-1) = 2.2$. The reduction coefficient for structural system is the calculated： $F_u = R_a = 2.2$. Modified coefficient for spectrum is $\left(\frac{C}{F_u}\right)_m$ =.6. The lateral seismic loading is then calculated:

$$V = \frac{ZI}{1.4\alpha_y}\left(\frac{C}{F_u}\right)_m W = \frac{0.23\times1}{1.4\times1.5}\times0.6012\ W=0.06585\ W$$

6.3 Current Code

Nantour locates in medium zone for 1989 code. It is zone 2 for 1997 code, and the building is expected to resist .23g ground acceleration. However, Nantour becomes zone I in 2000 code, the ground acceleration increases to .33g. The seismic design base shear becomes

$$V=\frac{0.33\times1}{1.4\times1.5}\times0.6012\times W = 0.095W$$

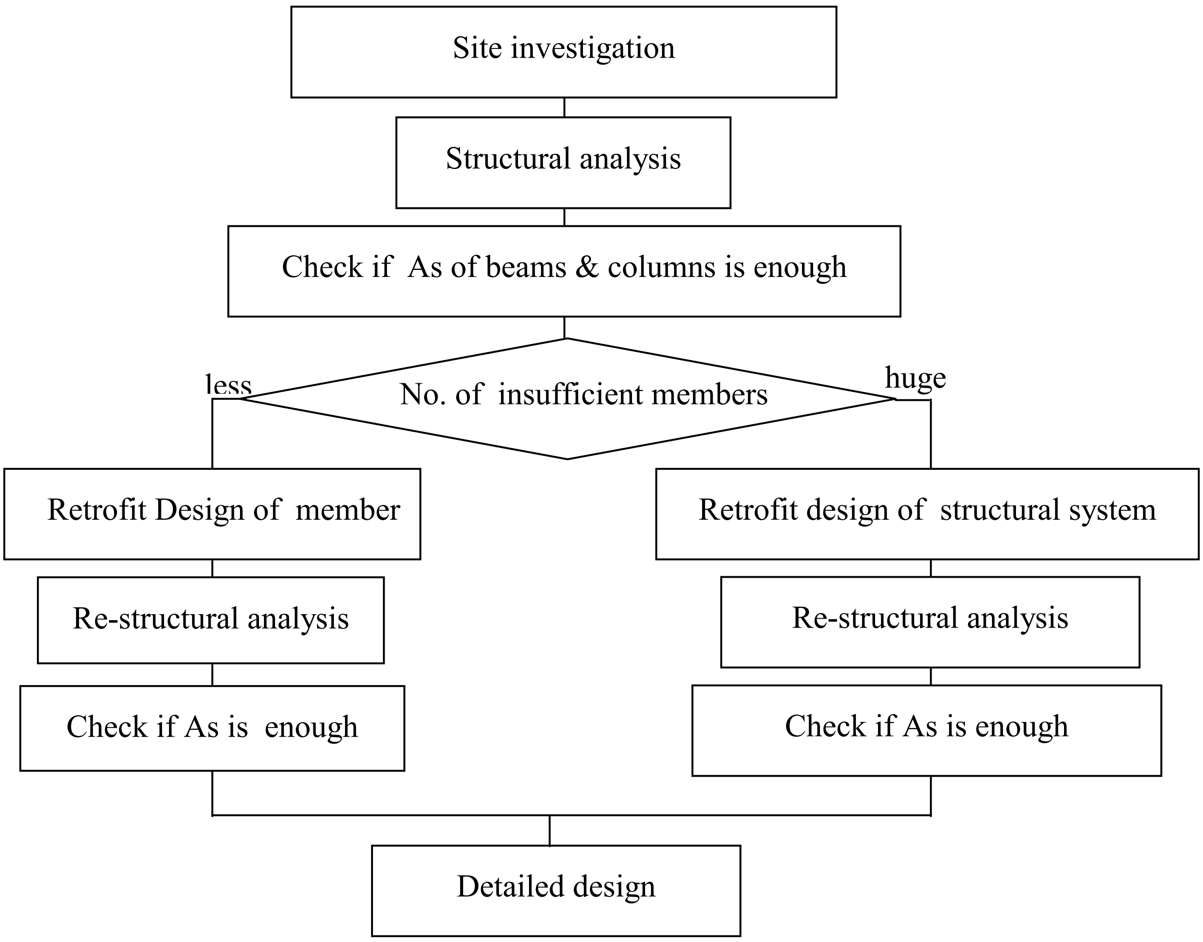
6.4 Comparison

The lateral seismic loading based on different codes is compared in the following：

1989 code	1997 code	Current code
0.09344 W	0.06585 W	0.095 W

The base shear of current code is close to the 1989 code, this is the reason why the damage of the present building is not serious.

7、Flow Chart of Retrofit Design



The check of As is executed for every beam and column members based on the 1989 code or current code. The retrofit planning for four cases are presented and are chosen by the committee of residents.

In this case, the more strict retrofit design is the one according to the current code. The rules for the check of reinforced As are:

1. when the deviation of necessary As and provided As for columns is less than 5 %, the retrofit is not required.
2. when the deviation of necessary As and provided As for beams is less than 10 %, the retrofit is not required.

8、Structural Data

8.1 General Data

The height of basement is 4.7m, the first floor 3.5m, second to eleventh floor 3m. The width of exterior RC wall is 15cm, and the interior wall is brick. Mat foundation is used, and the slurry wall is 20 cm thick.

8.2 Structural System

According to current code, the building belongs to moment-resistant frame. The reduction coefficient of structural system can be R=4. However, R=3.2 is adopted since they are nonstructural walls.

8.3 Material Properties

The designed concrete compressive strength is 210 kg/cm^2. Three concrete cores of diameter 5.5 cm (2”) are taken per every story. The average of concrete strength is 229.88kg/cm^2 by compressive test data. However, the strength is adopted in re-structural analysis of retrofit design.

8.4 Weight per Story

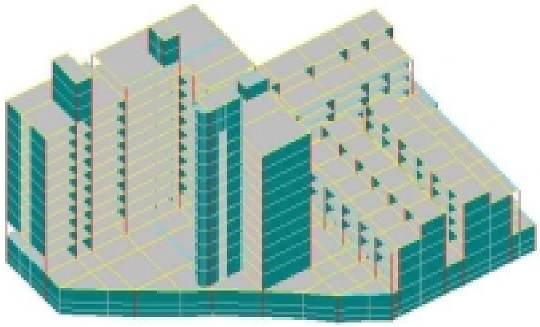
story	Beam (ton)	column (ton)	slab (ton)	wall (ton)	weight (ton)
PF	48.31	88.7	92	240.93*	469.94
RF	75.31	17.84	121.03	79.44	293.62
11F	166.02	60.14	275.28	138.76	640.2
10F	166.02	84.6	275.28	236.51	762.41
9F	166.02	84.6	275.28	236.51	762.41
8F	166.02	84.6	275.28	236.51	762.41
7F	166.02	84.6	275.28	236.51	762.41
6F	166.02	84.6	275.28	236.51	762.41
5F	177.47	84.6	275.28	233.89	771.24
4F	177.47	84.6	275.28	233.89	771.24
3F	177.47	84.6	275.28	232.31	769.66
2F	165.41	91.65	230.75	186.37	674.18
Σ	1817.56	935.13	2921.3	2528.14	8202.13

8.5 Soil type

The boring data is used to calculate the fundamental vibration period is T_G=.213 sec, according to current code. The soil type is then Type II, medium soil.

8.6 Model

The structural analysis is by ETABS 6.2, which modeling is



8.7 Fundamental Period

The fundamental period for both x and y directions calculated according to codes are :

T_c=.854 sec by 1989 code,

T_c=.997 sec by current code.

The fundamental period calculated by ETABS dynamic analysis is

T_{dx}=.99 sec, T_{dy}=.88 sec

According the code, the adopted fundamental period adopted in calculation of lateral seismic loading is

$T=Min(1.4T_c, T_d)$

i.e.

T_x=.99 sec, T_y=.88 sec

9、ETABS Structural Analysis

According to the building code, for irregular building (21m or higher than 7 story) or building higher than 50m the dynamic spectrum analysis is required. The present building is 34.5m high, however, is L-shaped. Therefore, static and dynamic analyses are both executed.

9.1 Modeling Assumption

1.The cross sections of beam and column members are input according to the original design.
2.Rigid diaphragm is assumed.
3.The rigid zone is 100% for RC structures.
4.The basement is modeled, and the underground soil spring is also modeled.
5.The brick wall is not modeled. But its weight is counted.
6.20cm thick RC wall on stairway is modeled.
7.The modeling of 15 cm exterior RC wall is:
(1)at first, the original thickness is 15 cm.
(2)If the induced shears of the walls in seismic loading are larger than the shear strength provided by the walls, the thickness of wall is reduced to 5 cm.

9.2 Loading

1.Dead load is the weight of story calculated in section 8.4, the total weight is 8202 ton.
2.Live load is: 200 kg/m2 for 1F-11F (resident), 500 kg/m2 for basement (parking lot).
3.Lateral seismic base shear is calculated by 1989 code or current code.
4.In static analysis, the seismic loading is distributed in the floors, proportional to the weight and the height of story.

9.3 Additional Eccentricity

The lateral seismic loading should be applied in the location of 5% eccentricity on both sides of gravity center. Furthermore, additional eccentricity is required for the irregular buildings. For the present building:

story	Additional eccentricity A_x			
	EQx-5%	EQx-5%	EQy+5%	EQy-5%
RF	1.0109	1.04313	0.7609	0.76643
11F	0.80276	0.95929	0.99452	0.80401
10F	0.79767	0.95356	1.01112	0.81817
9F	0.79475	0.95002	1.02278	0.8282
8F	0.7926	0.94651	1.03094	0.8355
7F	0.79128	0.94395	1.03558	0.83979
6F	0.79068	0.9421	1.03434	0.84024
5F	0.79109	0.94011	1.027	0.83578
4F	0.79387	0.94024	1.00812	0.82188
3F	0.80236	0.94624	0.96373	0.78847
2F	0.83039	0.96801	0.84688	0.69973

9.4 Vertical Seismic loading

The vertical seismic loading is not considered in 1989 code. In 2000 code, it should be considered in seismic zone of strong ground motion only. In current code, the vertical seismic loading must be considered in every seismic zone.

The vertical seismic loading is calculated for slab, beam, and columns independently. The weight of beam and slab is 6.07 times of the weight of column. In the sense of average, the vertical seismic loading is calculated as

$$Vz = \frac{0.105 \times 6.07 + 0.147 \times 1}{(6.07 + 1)} = 0.111W$$

i.e. Vz = 0.111D

9.5 Loading Combination

According to the code, 65 sets of combination is as follows:

type	D	L	$E_{s,x}$	$E_{s,y}$	$E_{d,x}$	$E_{d,y}$	EQ_v
1	1.4	1.7					
2	0.75×1.4	0.75×1.7	± 0.75×1.87				±0.75× 0.3×1.87
3	0.75×1.4	0.75×1.7		±0.75× 1.87			±0.75× 0.3×1.87
4	0.75×1.4	0.75×1.7			± 0.75×1.87		±0.75× 0.3×1.87
5	0.75×1.4	0.75×1.7				±0.75× 1.87	±0.75× 0.3×1.87

6	0.75×1.4	0.75×1.7	± 0.75× 0.3×1.87				±0.75×1.87
7	0.75×1.4	0.75×1.7		±0.75× 0.3× 1.87			±0.75×1.87
8	0.75×1.4	0.75×1.7			± 0.75× 0.3×1.87		±0.75×1.87
9	0.75×1.4	0.75×1.7				±0.75× 0.3× 1.87	±0.75×1.87
10	0.9		± 1.43				± 0.3×1.43
11	0.9			± 1.43			± 0.3×1.43
12	0.9				± 1.43		± 0.3×1.43
13	0.9					± 1.43	± 0.3×1.43
14	0.9		±0.3×1.43				±1.43
15	0.9			±0.3× 1.43			±1.43
16	0.9				±0.3×1.43		±1.43
17	0.9					±0.3× 1.43	±1.43

Where D is dead load, L is live load, $E_{s,x}$ and $E_{s,y}$ are X、Y-direction pseudo horizontal lateral loading, $E_{d,x}$ and $E_{d,y}$ are X、Y-direction dynamic horizontal lateral loading, and EQ_v is vertical seismic loading.

10、Evaluation of Structural Safety

Four cases are investigated to check whether the reinforced bar is enough for every beam and column.

10.1 Four Cases

According to static or dynamic analyses, 1989 code or current code, four cases are studied:

case 1: 1989 code, static analysis
case 2: 1989 code, dynamic analysis
case 3: current code, static analysis
case 4: current code, dynamic analysis

It is noted that

1. The RC walls are modeled in retrofit design. But, the 15 cm RC walls are not modeled for new building.
2. Different concrete compressive strengths are adopted for different stories.
3. The tensile strength of reinforced bar is assumed to be the same as original design.
4. The longitudinal bars and the stirrups of beams and columns are all checked.

10.2 Longitudinal Bars of Columns

After comparison of the provided As and the original As, the number of columns with insufficient reinforced bars is:

1989 code : 1
current code : 6

10.3 Stirrup of Columns

After comparison of the provided As and the original As, the present columns satisfy the requirement of "seismic design", but not "ductile design".

10.4 Longitudinal Bars of Beams

After comparison of provided As and the original As, the percentage of beams with insufficient reinforced bars is:

For the top bars:

story	1989 code		Current code	
	case 1	case 2	case 3	case 4
4F	2.1	4.3	4.3	8.5
3F	2.1	4.3	4.3	5.3
2F	2.1	1.1	2.1	2.1
1F	2.1	1.1	2.1	3.2

For the bottom bars:

story	1989 code		Current code	
	case 1	case 2	case 3	case 4
4F	4.3	4.3	7.4	9.6
3F	4.3	3.2	6.4	5.3
2F	3.2	2.1	4.3	4.3
1F	2.1	2.1	3.2	2.1

10.5 Stirrups of Beams

After comparison of the provided As and the original As, the percentage of beams with insufficient reinforced bars is:

story	1989 code		Current code	
	case 1	case 2	case 3	case 4
4F	12.8	10.6	13.8	14.9
3F	10.6	10.6	13.8	11.7
2F	9.6	14.9	12.8	16
1F	9.6	13.8	14.9	18.1

11、Retrofit Design

After the evaluation of structural safety, the preliminary retrofit designs and associated budgets are presented to the committee. The consultants company meets with the committee, explains the retrofit methods, budgets, expected effects, and influences. After the preliminary design is chosen, the detail design can proceed.

11.1 Preliminary Design

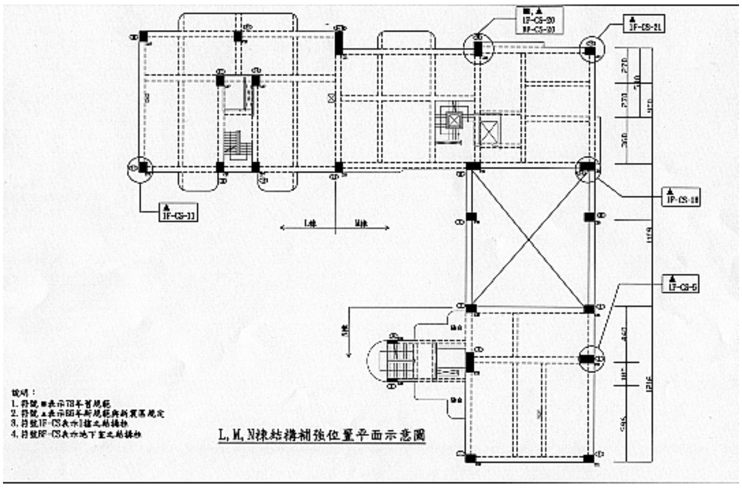
Since the number of beams and columns with insufficient reinforced bars and stirrups for the present building are small, the structural system retrofit method is not necessary for the present building. The following member retrofit methods are adopted:

1. The columns are strengthened by 6 mm steel plates surrounded along the whole height.
2. The bottom bars of beams are strengthened by 6 mm steel under the three under faces.
3. The upper bars of beams are neglected, since the slab is expected to provide enough reinforcement, and the insufficient steel is not much.

11.2 Detail Design

Out of four cases in the preliminary design, the retrofit case 4 according the current code is approved. The consultants company investigates the damaged building again. The main purpose is to record what will be destroyed and how they can be repaired. In some cases, the position of retrofit is impossible. Then, the retrofit planning should be changed, and the structural analysis should be executed again.

In the present building, the location of retrofit is shown in the following figure:



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11.3 Re-analysis

Since the retrofit of column with insufficient reinforced bar need be strengthened by the steel plate, the stiffness of column is changed. Therefore, the structure after retrofit should be re-analyzed. The columns strengthened by steel plates are modeled by equivalent cross sections.

12、Conclusions

In this paper, a retrofit design of damaged building in 921 earthquake is introduced. After several case study, it is then concluded that:

1.There is no retrofit code in Taiwan. Then, the present building code for new buildings is partially adopted.
2.There are no rules for retrofit design and construction nowadays in Taiwan. Therefore, the retrofit design flowchart is important. The presented retrofit design procedure is thought to be suitable.
3.The preliminary design is especially necessary for retrofit, since the budget and strengthening methods vary hugely. After the preliminary design is approved, the detail design proceeds.
4.The retrofit design should consider the different building codes. And, the retrofit budget varies for the adopted codes.
5.It is suggested that 20 cm RC walls are modeled in the structural analysis modeling. The 15 cm RC walls are modeled first, but should be adjusted in the way in this paper.
6.The whole consideration should be taken into account, since the decoration is usually destroyed during retrofit. Therefore, the investigation and the associated design for the decoration and nonstructural equipments are very important.
7.The building after retrofit should be re-analyzed, if the stiffness is changed.
8.The bores drilling is necessary if soil liquefaction occurs.