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SEISMIC PERFORMANCE OF MEDIUM- AND HIGH-RISE REINFORCED CONCRETE BUILDINGS TO TANGSHAN EARTHQUAKE 1976

by

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1. INTRODUCTION

The objective of this paper is to discuss the seismic performance of medium-rise and high-rise R/C buildings subjected to the Tangshan Earthquake in 1976. However, since there were quite few medium- or high-rise R/C buildings in Tangshan city, this paper deals with buildings in Beijing and in Tianjin cities located about 190 km and 100 km far from the epicenter, respectively. In the both cities, the damage to high-rise buildings supported on soft and thick deposit was more serious than those to low and rigid buildings on the same deposit. This phenomena can not be explained satisfactorily by the existing Chinese Seismic Design Code[(1)]. One of the reasons may be a soil-structure interaction has not been fully considered in the codes. This paper tries to explain the causes for the damage of buildings starting with emphasis on damage aspect of high-rise buildings and to present some notable problems on seismic design and seismic diagnosis of high-rise buildings.

2. EARTHQUAKE DAMAGE TO MEDIUM- AND HIGH-RISE R/C BUILDINGS AND TO BRICK BUILDINGS

The seismic response and damage to high-rise buildings, especially to slender high-rise buildings located far from the epicenter and supported on soft and thick deposit were severer than those to rigid brick buildings with medium and low height in Tangshan Earthquake in 1976. For example, the results investigated by Tianjin Architectural Planning Institute to thirty four brick buildings in three seriously damaged regions in Tianjin where the intensity was of VIII in Chinese scale, which is almost same as Modified Mercalli scale, showed: medium damage occured in three buildings of thirty four buildings (9%), i.e., the buildings themselves could still be serviceable with moderate repairing after earthquake, seven buildings (20.5%) were slightly damaged which could still be used without or with minor repairing, and twenty four buildings (70.5%) were undamaged. However, the investigation of damage ratio to more than fourty multi-storey R/C frame buildings in Tianjin showed: 15% of them were severely damaged, 22.5% were damaged medially 7.5% damaged slightly and 55% undamaged.

The damage aspect of brick buildings, of course, were different from those of R/C frame buildings. The damage to brick structures occurred mainly at bearing brick walls, while the damage to R/C frame structures occurred first at in-filled walls of the frame, then at the frame itself. However, from above mentioned examples we know that the damage to flexible R/C frame buildings on soft and thick deposit was severer than that to rigid brick buildings in the same region during the same earthquake.

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3. PRELIMINARY ANALYSIS

Fundamental periods of the high-rise buildings above mentioned are generally greater than 0.6 sec., and some of them greater than 1.0 sec.. When seismic waves produced by strong earthquake propagate from epicenter through soft and thick deposit, their high frequency components are often filtered, while their low frequency components are conserved or amplified, and the duration of the seismic wave elongates. This type of ground motion may give a large response displacement to the structures having long natural periods such as high-rise buildings.

As an example, the records obtained at the apartment building No.10 in Beijing are described. The apartment building No.10 in Beijing is a 19 storey R/C frame structure with shear wall [Photo 1]. According to Chinese Code[(1)], the site soil condition which consists of sandy and clayey soils with gravel layers in between, belongs to Type II. During Tangshan earthquake, the apartment building was subjected to an approximately VI grade shock in Chinese scale. Several in-filled light walls were separated from the columns at 6-8th floors of the frame structure. Fortunately, a 12-channel RDZI-12-66 Type accelerograph made in China was installed in the apartment building. The pick-ups were located on the 5th, 10th and 17th floors, as well as at the basement. Each of them had two horizontal and a vertical components. More than fifty records including the main aftershock of the Tangshan earthquake had been obtained.

Three earhquake waves in respect of their types were selected from the records obtained in the basement as listed in Table 1 and the seismic response displacement to the apartment building was calculated by means of step by step integration.

The main findings by the analysis[(3)] are:

(1) Althogh the maximum values of the acceleration of W02 and W03 recorded at the basement of the apartment were almost equal, the structural response under the effect of earthquake W03 were 2-3 times of those caused by the earthquake W02, because of the different epicentral distances.

(2) The frequency characteristics and the durations of the records of W01 and W02 were different, and the vibrations excited by them were also quite different. In the former the vibration of fundamental mode shape was predominated while in the latter the vibration of 2nd and 3rd mode shapes appeared. If only the maximum accelerations of the two records were adjusted to the same value and the structure still maintained an ideal elastic condition, then the maximum response to the apartment caused by the W01 earthquake would be 3-5 times of that caused by the earthquake W02.

Further analyses were done using several earthquake waves as input ground accelerations. Their maximum acceleration values were all adjusted to 200 gals. The calculated envelopes of storey drift are shown in Fig.1. The input earthquake waves are as follows: a) earthquake W01, b) El Centro earthquake of May 18th, 1940 (NS components), c) Sunpan earthquake of Aug. 16th, 1976 in China (S60°E components), M=7.2, d) artificial stochastic seismic wave conformed to the standard spectrum of Type II site soil (medium soils) according to Chinese Code, and e) artificial stochastic seismic wave conformed to the response spectrum of apartment No.10 site soil of which the thickness is 104 meters.

Fig.1 shows that the response value calculated by observed W01 is about 4-5 times of that obtained from standard spectrum. The response to the waves b) and c) are quite small. Only the value obtained from the waves e) approaches the observed one. The response obtained from standard spectrum appears to be quite small because the criterion in classifying soil condition considers only 10-20 meters thickness of profile of site soil, whereas the predominant periods of site soil increase with increasing the thickness of deposit , and does not take account of the influence of earthquake magnitude and epicentral distance. However, response spectrum of site soil may comprehensively consider above mentioned factors.

The comparison of three response spectra is shown in Fig.2: curve d) is standard spectrum of Type II site soil, and curves e) and a) are response spectra of seismic waves e) and a) mentioned above, respectively. It appears that spectrum d) can not express the site properties of the apartment.

4. INFLUENCE OF STRUCTURAL STIFFNESS ON BUILDING PERFORMANCE

Besides the structural safety, attention must be given to structural stiffness to prevent earthquake damage to non-structural elements such as in-filled walls or architectural ornaments. For example, among ten medium- or high-rise buildings with in-filled walls investigated in Tianjin, all in-filled walls were damaged obviously during Tangshan earthquake. The damage to in-filled walls was widespread phenomena in Beijing although it was not so severe as in Tianjin because the intensity in Beijing was lower than in Tianjin. The expenses for repairing in-filled walls and architectural ornaments are a considerable figure especially for high-rise buildings with modern equipments. By increasing the stiffness of structure, this kind of damage may considerably be mitigated. Tianjin Friendship Hotel [Photo 2] is divided into two parts by an aseismic joint of 15 cm. Tangshan main shock gave damage to all of the in-filled walls of east part of frame structure [Fig.3a)], while the in-filled walls of rigid west part consisting of frames and shear walls were slightly damaged [Fig.3b)]. The in-filled walls of east part were quickly repaired after that shock, but all the repaired walls were damaged again during Tangshan aftershock on Nov. 15th, 1976, because both strength and stiffness were not improved.

Another similar example is Beijing Hotel subjected to the influence of intensity VI in Chinese scale. In the west part of nine storey frame structure the 7-8th floor in-filled walls were broken severely, while the in-filled walls of the east part of eighteen storey frame-shear-wall structure were undamaged.

Thus it can be seen that to incease appropriately structural stiffness is also one of the effective way to prevent damage of non-structural elements for tall buildings.

5. EVALUATION OF DISPLACEMENT OF STRUCTURES

The seismic response displacements of tall buildings on soft soil calculated according to either seismic load or experience formula both stipulated in Chinese Code[(1)] are much smaller than actual response values. The authors consider that one of the main reasons lies in calculation without taking account of soil-structure interaction.

Here, some examples are reviewed [(3),(4)]. The east part of Beijing Hotel of which the height is 75m was subjected to the influence of VI grade during Tangshan main shock. The roof displacement calculated from acceleration records was 8cm. No.10 apartment, 60 m high, was subjected to V grade influence during Tangshan aftershock of magnitude 7.1. The displacement calculated from acceleration records was 3.34 cm which was recorded at the height of 51.3 m. In addition, the aseismic joint of Tianjin Friendship Hotel was designed as 15 cm. Structures at both side of the aseismic joint collided with each other during Tangshan main shock. Photo 3 shows the aseismic joint after being knocked. The railing on the roof was broken. The distance between two ends of broken railing was 30 cm [Photo 4]. It showed that the required width of the aseismic joint of that building for Tangshan earthquake was about 30 cm.

The aseismic joint width (t) for seismic intensity of VIII calculated by Chinese Code is as follows:

$$t = 7+2x(H-15)/3 \text{ (cm)}$$

where, H is the building height of lower part between the two parts in meters.

Assuming,(a) earthquake load as well as earthquake displacement of a building is double when the seismic intensity increases by 1 grade, and (b) the most disadvantageous condition is of that two parts at aseismic joint move contrary each other, then the aseismic joint widths calculated both by Chinese Codes and by the seismic records in case of the intensity of VIII are as shown in Table 2. It shows that those calculated by the Code are smaller than those evaluated according to the seismic records.

In evaluating the response displacement, translation and rotation of foundation should be also considered as well as deflection of structure. For examples, the results by K.S. Wang et al[(4)] indicate that for 10-20 storey R/C shear wall buildings with box foundations on the site soil of Type II and III by Chinese Code, the roof displacements produced by foundation translation U_o and foundation rocking Φ become to 3-10% and 50-70% of their total values $\bar{U}n$, respectively, whereas the corresponding measurement results [Table 3] obtained by means of environmental microtremor and excited vibration test on six shear wall structures of 10-16 storeys in Beijing are about 5% and 60% of the total roof displacement. The calculated values are quite close to those measured. It shows that the roof displacement of shear wall building produced by foundation movement almost equals twice that produced by structural deflection itself.

The followings are calculated displacements of a frame structure of eight storeys and a frame-shear-wall structure wall structure of nine storeys. The fundamental period of above mentioned two structures on several subsoils are shown in Table 4. In order to study the effects of periods of input seismic wave on response to above mentioned stuctures, the predominant periods Tp of EL Centro wave (NS) of May 18th, 1940 were adjusted. Listed in Tables 5 and 6 is the roof displacement ratio, $\bar{U}n/Un$, calculated with or without taking account of soil-structire interaction, respectively. The displacement ratio shown in Tables 5 and 6 depends upon two factors: a) the ratio of building stiffness to subsoil stiffness and b) the predominant period of input earthquake wave. Generally speaking, the roof displacement ratio will increase when the stiffness of building as well as the predominant period of the seismic wave increase , and the effect of subsoil stiffness on the displacement ratio depends to some extent upon the predominant period of seismic wave. In any case, the roof displacement calculated by considering soil-structure interaction will mostly increase and may generally equal to 1.5-2.7 of that calculated without consideration of soil-structure interaction. So, it is necessary to take account of soil-structure interaction in calculating the displacement or the width of aseismic joint of high-rise buildings.

6. AN EXAMPLE OF SEISMIC DIAGNOSIS OF HIGH-RISE BUILDING

It is not always satisfactory to explain the damage phenomena of some high-rise buildings by using the results calculated according to linear elastic analysis. The satisfactory answer may be obtained by means of non-linear response analysis.

For an example, the absorption tower in Tianjin Alkali Factory is shown. The structure was thirteen storey R/C frame structure which was 52 meters high, with raft foundation on soft soil. The 6th-13th floors of the structure collapsed completely [Photo 5], all the broken beams and columns fell in the building and partial damage took place below 5th floor due to Tangshan earthquake. The structural analysis by means of mode superposition according to the existing code shows that the reinforcements of most part of columns above tenth floor were not enough and the reinforcements of some columns were only 30-50 percent of those required. It is clear that partial damage was possible owing to the lack of aseismuc strength, but collapse of 6th-13th floors could not be explained.

Shown in Fig.4 is the result of non-linear member-to-member level analysis of above mentioned frame structure by input of Tianjin earthquake wave of Nov. 15th of 1976, of which the acceleration amplitude was adjusted from 133 gals to 200 gals. From the envelopes of storey drift shown in Fig.4(a), we know that the very great storey drift at 11th floor, 1/40-1/25, might cause remarkable damage concentration. Fig.4(b) shows the mode shape when the 11th storey drift becomes maximum. The Fig.4(b) and the damage distribution shown in Fig.4(c) tells us, the damage of 6th-13th floors is much severer than that of 1st-5th floors and the damage of middle columns is more serious than that of side columns, which corresponds to the real damage of the structure.

7. CONCLUSION

(1) The seismic response of high-rise buildings be determined by using a site-dependent response spectrum accounting for the effects of magnitude, epicentral distance and dynamic characteristics of site soil.

(2) The effect of soft soil on structural displacement of high-rise buildings may not be neglected.

(3) An effectve method for seismic diagnosis of high-rise building is non-linear seismic response analysis, by which we can find out some problems that can not be sought out by common practice.

(4) From the point of view to decrease the structural dispacement and to decrease the repairing cost of architectural ornaments and in-filled walls after earthquake, the entire stiffness of high-rise building should be strengthened.

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REFERENCES

(1) Aseismic Design Code for Industrial and Civil Buildings, TJ 11-78, Chinese Architectural Industry Publishing House, 1979.

(2) Preliminary Summation of Tangshan Eathquake Damage, Institute of Engineering Mechanics, Academia Sinica, Seismological Publishing House, 1978.

(3) Wang, K.S., et al.(1980), "Earthquake Reaponse Analysis of Nineteen Story Prefabrecated R/C Frame Structure", 7th WCEE V.5, pp 73-81.

(4) Wang, K.S., et al.(1982), "Effects of Subsoil on Seismic Response of Shear Wall Structure", Sino-American Symposium on Bridge and Structure, V.1, 4-12.

Table 1 Acceleration Records at Apartment Building No.10, Beijing

Earthquake No.	Time of occurance	Magnitude (MS)	Location of epicenter	Epicentral distance (km)	Duration of records (sec.)	Max. Acc. (gal)
W01	July 28th,1976 (main aftershock)	7.1	118°39'E 39°50'N	190	42.0	26
W02	Dec. 2nd, 1976	5.5	117°32'E 39°35'N	96	9.5	9
W03	May 12th, 1977	6.3	117°49'E 39°18'N	138	18.6	11

Table 2 Aseismic Joint Width (cm)

name of building calculated by	Tianjin Friendship Hotel	Beijing Hotel	Apartment No.10
seismic records	30	64	53.4
Chinese Code	20.4	47	31.2

Table 3 Ratio of Displacement

Building No.	Transverse		Longitudinal	
	$\frac{U_o}{\bar{U}_n}$ (%)	$\frac{H\Phi}{\bar{U}_n}$ (%)	$\frac{U_o}{\bar{U}_n}$ (%)	$\frac{H\Phi}{\bar{U}_n}$ (%)
1	3.0	66.7	7.0	41.4
608	5.0	67.9	6.7	21.3
3	5.0	38.0	8.3	35.0
4	4.5	58.2	7.7	
5	4.5	67.3	20.8	
6	8.3	70.8	20.0	
Average	5.05	61.5	11.75	32.57

Table 4 Soil Condition and Periods of Building

subsoil No.		1	2	3	4	5	6	rigid ground
equivalent shear modulus (t/m)		4342	5800	8200	7737	13940	17880	
equivalent shear wave velocity (m/sec)		150	173	206	200	268	304	
fundamental period of buildings (sec)	frame-shear wall	0.809	0.786	0.683	0.695	0.595	0.575	0.48
	frame	1.19	1.17	1.07	1.08	0.99	0.97	0.89

Table 5 Ratio Un/Un of Frame-shear Wall Building

Subsoil No. / Tp (sec)	1	2	3	4	5	6
0.2	1.15	1.17	1.28	1.26	1.44	1.45
0.3	1.57	1.51	1.27	1.29	1.09	1.05
0.5	1.24	1.25	1.39	1.34	1.56	1.57
0.7	1.22	1.22	1.13	1.15	1.10	1.11
0.9	2.48	2.46	2.50	2.51	2.10	1.93
1.2	2.78	2.69	2.14	2.18	1.24	1.30

Table 6 Ratio Un/Un of Frame Building

Subsoil No. / Tp (sec)	1	2	3	4	5	6
0.2	0.97	0.97	1.16	1.14	1.23	1.22
0.3	1.51	1.45	1.30	1.31	1.30	1.29
0.5	1.04	1.03	1.22	1.20	1.20	1.19
0.7	1.20	1.20	1.07	1.04	1.02	1.02
0.9	1.40	1.41	1.41	1.42	1.28	1.30
1.2	1.74	1.67	1.47	1.56	0.98	0.94

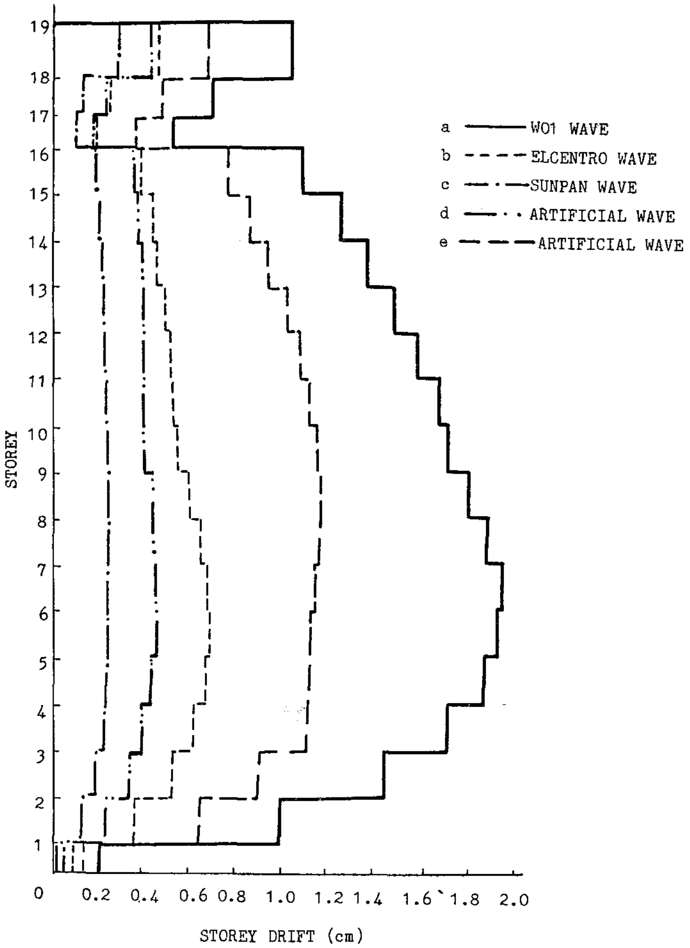


Fig.1 Response Displacements of Apartment Building #10

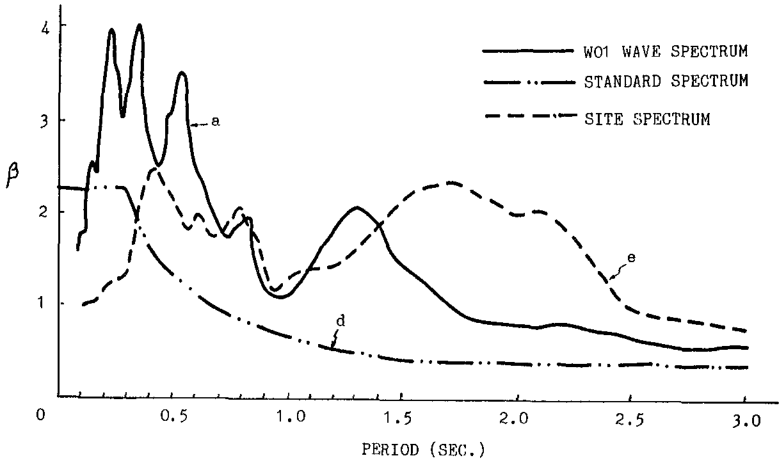


Fig.2 Response Spectra

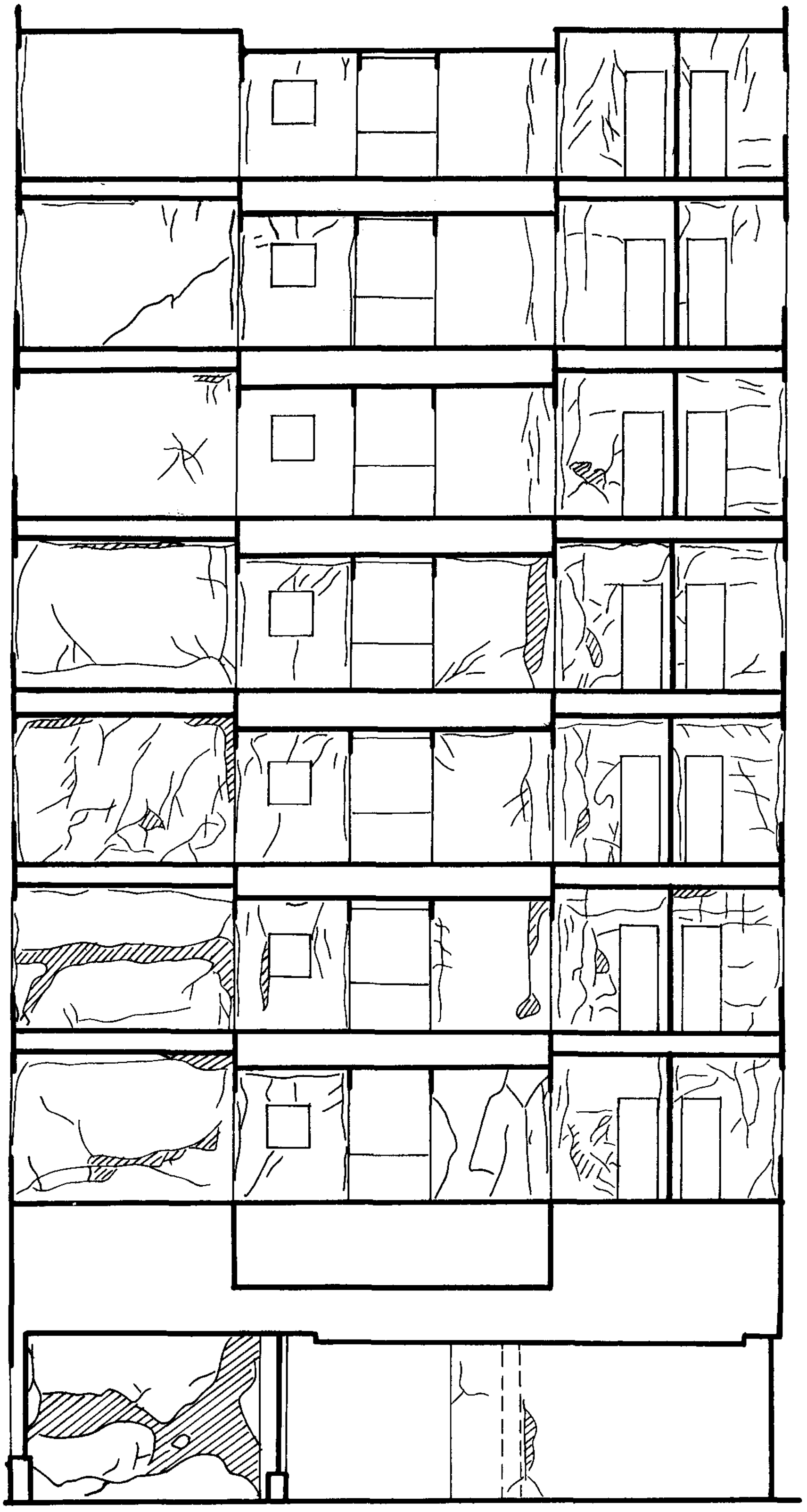


Fig.3 a Cracks of In-filled Wall of Tianjin Friendship Hotel [East part]

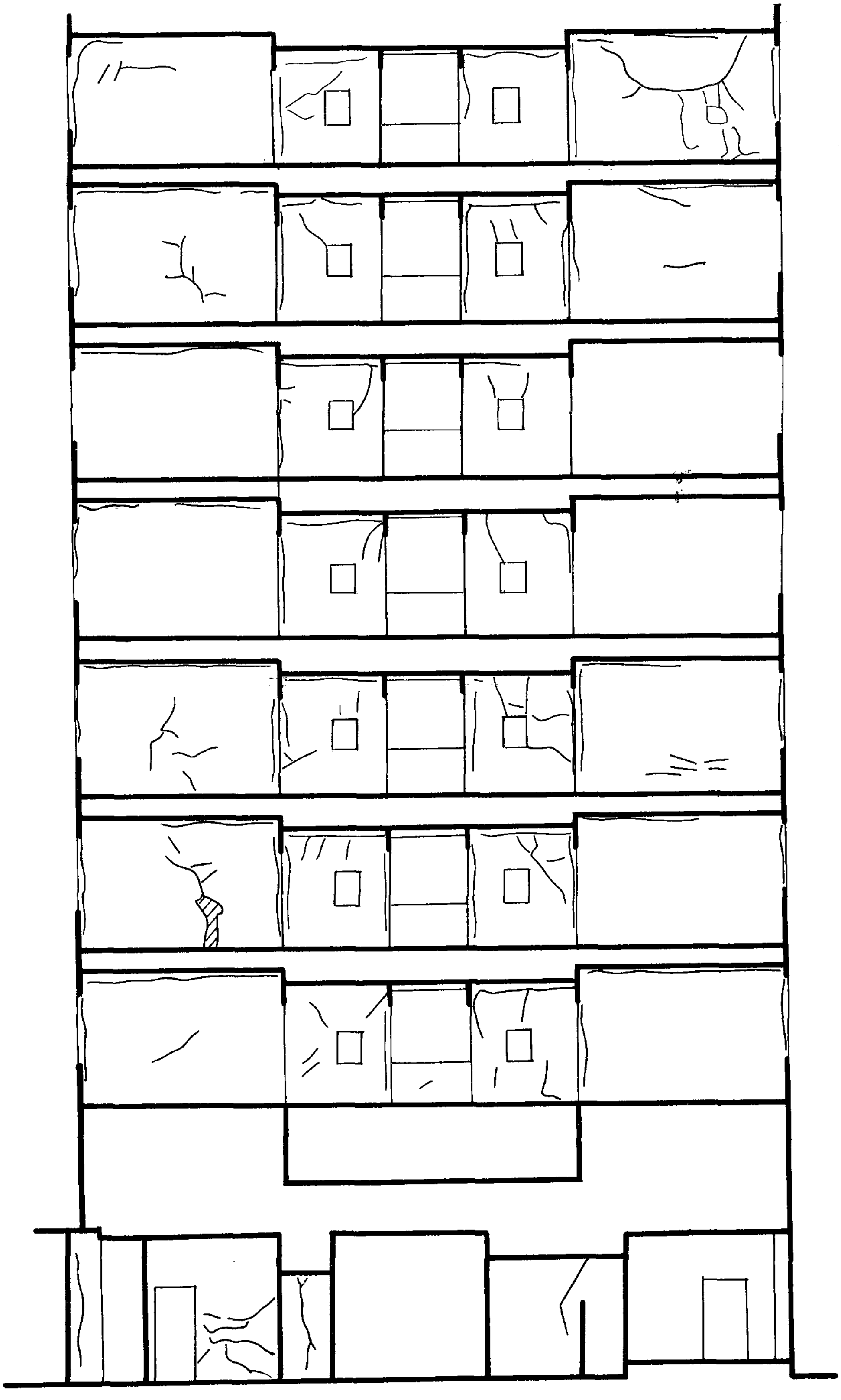
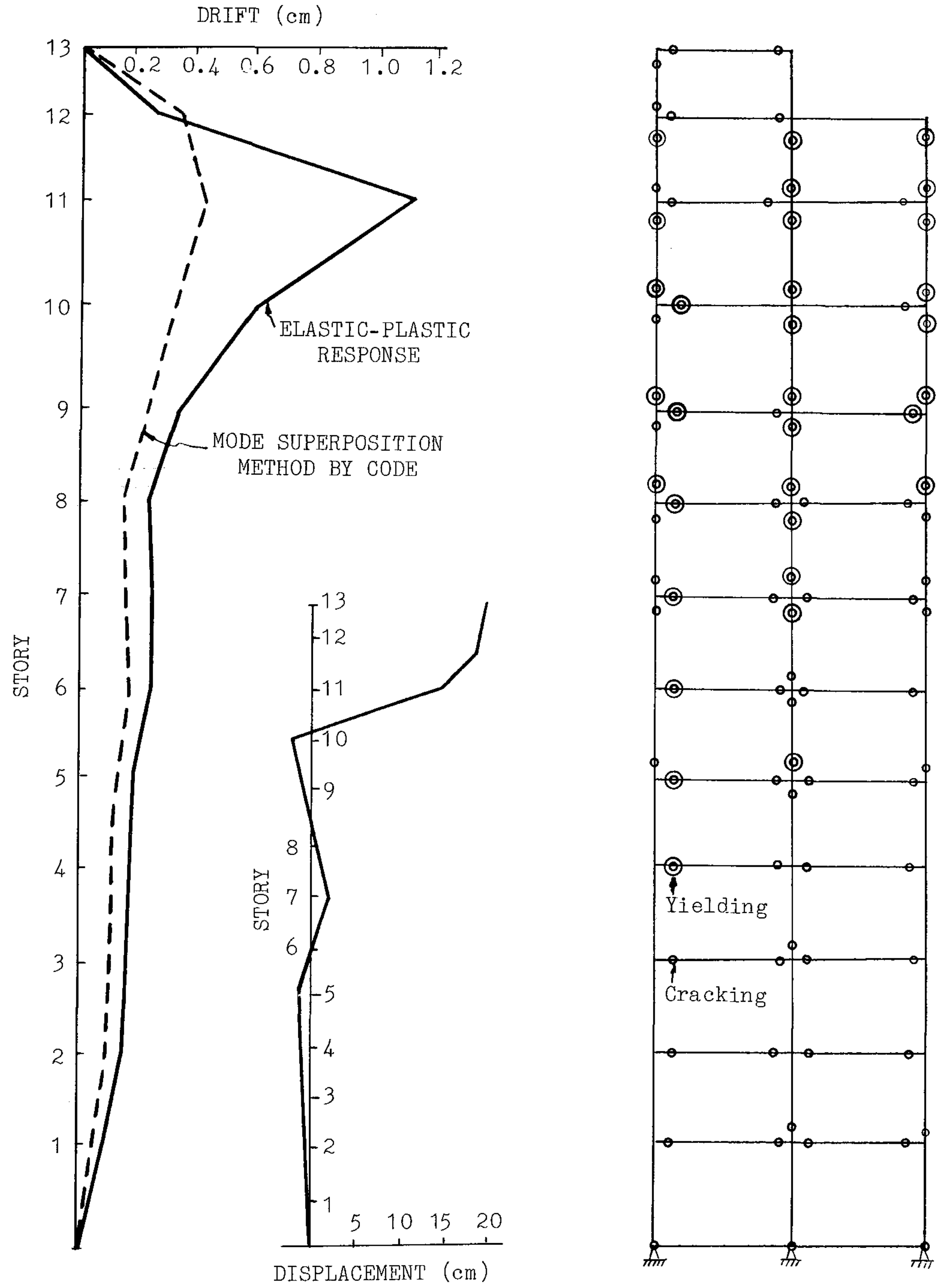


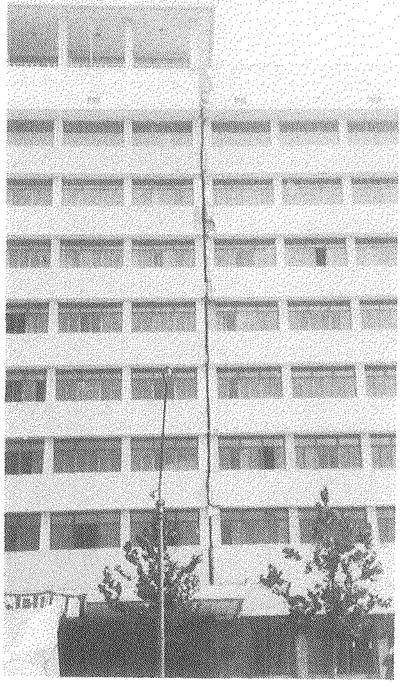
Fig.3b Cracks of In-filled Wall of Tianjin Friendship Hotel [West part]

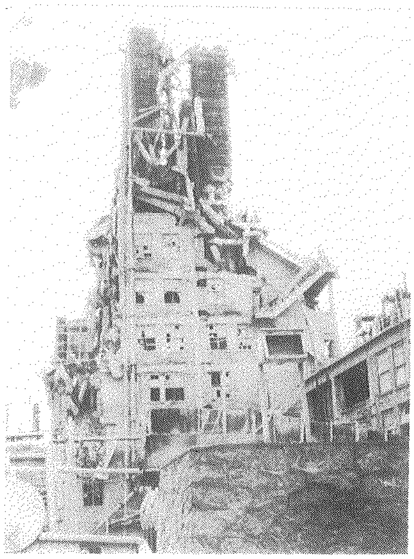
FIG. 4B

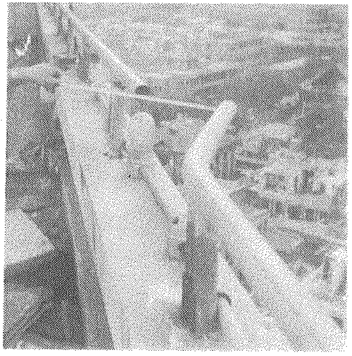


(a) (b) (c)

Fig.4 Response Displacementand Failure Mechanism of Tianjin Alkali Factory







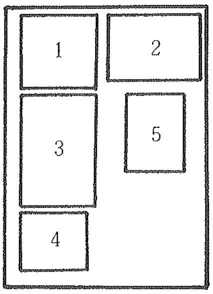


Photo 1 Apartment Building #10, Beijing
Photo 2 Tianjin Friendship Hotel
Photo 3 Expansion Joint of
Tianjin Friendship Hotel
Photo 4 Railing of Tianjin Friendship Hotel
Photo 5 Tianjin Alkali Factory