ty of columns. Finally, 260 buildings constructed before 1970 remained as present objective buildings.

Next, the committee applied the first, second and third screening method of the guideline compiled by JBDPA (1977)to several buildings and made the following items clear.

a. Almost of all objective school buildings are structurally designed following to the standardized simple rectangular-shaped plan and the typical plan as shown in Fig.23. Accordingly seismic performance of school buildings are generally considered excellent in the transverse direction because of many shear walls but not enough in the longitudinal direction because of few walls and shear force concentration to short columns resulting from neglected non-structural walls.
b. Although elastic response shear forces to the ground floor of buildings with different floor numbers are nearly proportional to the numbers of floors. However, cross sectional areas of the ground floor columns of the buildings with diffent floor members are not proportionally large. Accordingly, the highest four-story buildings would be most vulnerable.
c. The results by the first screening method will be underestimated in this case and the results by the second method will be easy to fail in overestimation. Accordingly, the third method are preferable to be adopted.
d. However, it was made clear the expense and man power necessary to the third screening investigation are around 5 time the first method and around two times the second method Accordingly, simplification of the higher screening method would be achieved to diagnose many school buildings.

Based on above-mentioned consideration, the simplified second and third methods were both developed by the committee and the applicability of th[illegible]e simplified method was verificated. In the simplified third method, total structure with regu[illegible] arranged structural members was assumed to be represented by typical three or four vertical m[illegible]mbers per one floor.

The latter simplified third method was later adopted as an appendix of the revised edition of the guideline compiled by JBDPA(Hirosawa 1988, JBDPA 1991).

Thus, about 120 four-story school buildings constructed before 1970 were investigated by the simplified second method and it was made clear that many of the four-story municipal school buildings constructed before 1970 were poorly aseismic especially in the first and second floors of longitudinal direction. Later, in order to decide the priority of execution of the actual retrofit construction work and to decide principal plan for retrofit design for each building, the simplified third method were applied to "need retrofit buildings".

As mentioned above, in case of seismic diagnostic investigation on many objective buildings with structural characteristics similar to each other, it is concluded effective and rational to develop simplified screening method of rigorous solution considering the common structural characteristics.

[illegible] apartment houses in Tokyo Metropolis (Hirosawa 1981)

Tokyo Metropolis had constructed dozens of public apartment houses with open space at the [illegible]ement shortage of parking area. As understood from Fig.24, theses [illegible]er stories for dwelling use but almost no walls in the both directions of the first [illegible] Hospital in U.S.A. is well known as it was severely damaged by the 1971 San Fernando earthquake but also in Japan, several buildings with the soft first [illegible] damaged by the 1983 Miyagiken-oki Earthquake Soon after this experience, Tokyo Metropolis decided to investigate seismic safety of this kind of apartment houses with soft first story and asked Building Research Institute, Ministry of Construction, Japan to investigate them with practical support of a private investigation company. However in this time, dwellers agreement for evacuation from the objective buildings could not be obtained, so actual strengthening, even if it is necessary for the upper portion, had to be limited only to the open ground story.

With this restriction, diagnostic investigation on 48 buildings in Table 15 was carried out between 1979 and 1981 basing on the guideline mode by JBDPA(1977) and 46 buildings out of them were judged unsafe.

In the guideline, influence of unpreferable vertical distribution of horizontal rigidity is considered as one factor of structural configuration index(S_D) by a certain coefficient to decrease calculated horizontal bearing capacity and the minimum of the coefficient is empirically decided as 0.8. Considering this, especially to evaluate the effect by strengthening, the following dynamic response analyses etc. were also applied to estimate all buildings.

Adopted four methods and check points in each method are as follows.

a. Is values by the third screening method(equivalent ground acceleration : 200 – 250 gal

b Response story shear(Qres) by elastic response analysis with α=225 gal(Hachinohe 1968 EW and EL Centro 1940 ES)

c. Necessary horizontal bearing capacity Qu by the current Building Standard Law(equivalent ground acceleration · 300 – 400 gal)

d. Response ductility factor μ res by elasto-plastic response analysis with α=450 gal(Hachinohe 1968 EW and EL Centro 1940 ES) In this investigation, it was concluded that the case of the story where two or more results out of the four fail in poor safety would be judged as unsafe.

As mentioned above, conclusive items on this investigation are as follows

a. There may be some cases, where sufficient strengthening is difficult to be done, because of some restrictions to strengthening.

b. Adoption of several additional analyses besides the static method will be needed in case of buildings with special dynamic characteristics.

Table 11 Strength Reduction Factors η for Damaged Members (for the Application of 2nd Level Screening)

Damage degree of structural members	Kinds of structural members				
	COL-M	COL-S	COL-SW	WALL-M	WALL-S
I	1.0	1.0	1.0	1.0	1 0
II	1.0	0 8	0.8	0.9	1 0
III	0.4(1+F/3.2)	0.4	0.4	0.6	0.4(1+F/2)
IV	0.3	0	0	0.3	0 3
V	0	0	0	0	0

Note : the damage degrees of structural members should comply with the Table 4 given in the GUIDELINES FOR POST-EARTHQUAKE DAMAGE INSPECTION AND EVALUATION

(Notation)

COL-M · Columns of flexural failure mode

COL-S · Columns of shear failure mode

COL-SW Columns with side walls

WALL-M Walls of flexural failure mode

WALL-S Walls of shear failure mode

F F-index in the evaluation standard

Table 12 Outline of the Seismic Investigations Excecuted by Shizuoka Prefecture, Yokohama City and Tokyo Metropolis

Municipal	Investigated Buildings			Investigation		Strengthening		
	Number of bldgs	Use	Number of Stories	Duration	Investigated by	Number of bldgs	Duration	Main Method
Shizuoka Prefecture	1896	School etc.	~ 4 ~	'77 ~ '86	Qualified Architectures Offices	465	'82 ~ '87	Infilling Wall, Steel Braces
Yokohama City	870	School etc.	mainly 3 ~ 4	82 ~ '84	Technical Committee	30~40% of the total	'87 ~	Steel Braces Column Jacketing
Tokyo Metropolis	48	Apartments	4 ~ 6	'79 ~ '81	Building Research Institute	46	'81 ~ '83	Infilling Wall, Additional Side Wall Column Jacketing

Table 13 Is and ET Indices of the Shizuoka Prefectural Buildings

Levels	Judgement	Factors (E_T-Es)	Number of Buildings	Percentages
A	enough resistance	Is ≧ E_T	266	14 %
B	need a check-up in detail	Is ≧ 0.7 E_T	379	20 %
C	need reinforcement	0.3E_T < Is < 0 7 E_T	758	40 %
D	need urgent reinforcement		398	21 %
E	need rebuilding	0 3E_T ≧ Is	95	5 %
Total	———		1,896	100 %

Table 14 Classification by Number of Stories versus construction Year of the Objective School Buildings Built in Yokohama City

Construcion Year / Number of Stories	Before 1970	From 1971 to 1980	After the New Seismic Code in 1981	Total
4	130	270	45	445
3	110	110	45	265
2	15	45	40	100
1	5	40	15	60
Total	260	465	145	870

Table 15 List of Investigated Apartment Houses with Soft First Story in Tokyo Metropolis (Number of Buildings)

Investigated Year	Diagnosis and Result		Retrofit Design & Construction	
	Number of Investigated Buildings (Number of Stories, Construction Year)	Result (Need Retrofit)	Design	Construction
1979	10 (3 ~6, 1967~1973)	8	—	—
1980	1 (5 , 1970)	1	9	—
1981	37 (4 ~6, 1967~1974)	37	37	9
1982	—	—	—	19
1983	—	—	—	18
Total	48	46	46	46

5 POST EARTHQUAKE COUNTERMEASURES FOR DAMAGED BUILDINGS

5.1 Outline

As described in Sec. 1, the following technical guidelines regarding post-earthquake countermeasures for damaged buildings were edited basing on the results of the national project and published in 1991 for general use(JBDPA, 1991)

a) Guidelines on damage inspection and evaluation

i. Emergency inspection and evaluation; This is used to check safety of buildings suffered from an earthquake against aftershock.

ii. Damage classification and judgment for restoration; This is used to classify the damaged buildings by the possibility of restoration

b) Guidelines for restoration techniques

i. Emergency restoration techniques; This is used to prevent or mitigate progressive damage of a building due to aftershock.

ii. Permanent restoration techniques, This is used to provide enough seismic capability to the damaged building for the future use.

Objective structures of these technical guidelines are not only building structures but civil structures including bridges, roads and dams etc.. But here, outlines, characteristics and examples of application will be briefly explained mainly about reinforced concrete buildings. For reference. English edition of the guidelines on civil structures and reinforced concrete buildings were already published in U.S.A(National Center for Earthquake Engineering Research 1986, Ohkubo 1990)

5.2 Guidelines on damage inspection and evaluation (JBDPA 1991, Ohkubo 1990)

5.2.1 Guideline for emergency inspection and evaluation Procedure

Persons in charge of "emergency inspection and evaluation", here after called as inspectors, inspect the given items, evaluate the results according to the given techniques and recommend the after-treatment to the building owners or residents

(Inspection Items) a. maximum settlement and maximum inclination as a whole building, b. damage to structures, c. possible falling objects, d. possible overturning objects. The investigation may be done only from outside of a building. However in case of buildings of public use, it must be done also from inside of the building.

(Judgment Techniques).

a. Inspectors evaluate the damage state of each inspection item in the given sheets and classify the DAMAGE DEGREE into there categories(A,B or C)

b. The RISK LEVEL for building structures is judged as follows, based on the number of DAMAGE DEGREE evaluated(DANGER) A building, that has more than one-C-RANK(DAMAGE DEGREE) or more than two B-RANK items, shall be judged as "DANGER". (CAUTION), (SAFETY)<omitted>

c. The RISK LEVEL for possible falling or overturning object is also judged as "CAUTION", or "SAFETY".(Emergency Treatment) Inspectors should recommend an emergency treatment for the inspected building to its owner or residents as the given ways such as "Entrance Prohibited", etc , based on the RISK LEVEL. (Change of Judgment) The first judgment for a building may be changed after emergency restoration or another emergency inspection.

5.2.2 Guideline for damage classification and judgment for restoration procedure

Structural engineers in charge of classification and judgment for restoration inspect the given items, classify the DAMAGE DEGREE of a building according to the given techniques [illegible] should be strengthened or not, according to the given requirements. Inspectors may [illegible] prescribed sheet for the inspection

(Inspection Items) a. maximum settlement and maximum inclination as a whole building, b damage degree of structural members. The investigation may be done at the most damaged story of the building

(Classification Techniques) DAMAGE DEGREE of a building on structural members shall be classified into the following five categories according to the DAMAGE RATIO, D which is defined as the total sum of D, shown as a function of the i-th DAMAGE RANK and the damage ratio(B/A) of the members classified into the i-th RANK.

(SLIGHT) D < 5, (SMALL) 5 < D < 10, (MODERATE) 10 < D < 50

(SEVERE) 50 < D, (COLLAPSE) D = 50

(Judgment of Damage Degree Classification) The final classification for DAMAGE DEGREE of a building may take the severest of the classified results on settlement, inclination and structural members, where classification techniques on settlement and inclination are both omitted here.

(Judgment for Necessity of Strengthening) Judgment on the necessity of strengthening of the damaged building for future use is recommended to comply with the guidelines shown in the matrix of the seismic intensity of the suffered earthquake and the decided DAMAGE DEGREE of each building as shown in Table 9. When the result fell in "advanced investigation", it is recommended to apply the 2nd or 3rd level procedure to the building as described in 2.6 2.

5.3 Guidelines on restoration techniques of damaged buildings

5.3.1 Guideline for emergency restoration techniques

Emergency restoration techniques are for the buildings judged as "DANGER" or "CAUTION" by the emergency evaluation. Several examples of actual emergency restoration method such as H-shaped steel as urgent support to severely damaged column, wirerope winding to cracked column and steel brace infilling to damaged frame to increase horizontal capacity are described in the guideline. However, there is no description on the objective capacity for emergency restoration and method to evaluate effect of the applied emergency restoration work mainly because of uncertain possibility and uncertain strength of aftershock.

5.3.2 Guidelines for permanent restoration techniques

(Scope) The permanent restoration techniques are expected to be applied to the building judged as "REPAIRING" or "STRENGTHENING" by the damage classification.

(Judgment on Necessity for strengthening) Refer to 2.6.2

(Investigation) Detailed investigation before restoration shall be necessary for the building judged as "should be STRENGTHENED".

(Strengthening Design) The aspects on the building function, facade of the building, workmanship restoration works, construction period, economic problem as well as structural requirements by the current codes shall be considered for the strengthening. Seismic performance of structures after strengthening may be evaluated by the standard as written in 2.6 3.

(Requirements for Member Strengthening) Several actual and effective methods to repair or strengthen the damaged columns, walls, beams etc. are described with experimental back data(Ministry of Construction 1986).

5.4 Examples of application of the guidelines

Fortunately, Japan never experiences any severe earthquake after the guidelines completed. However, during carrying out the research projects and making a draft for the guidelines, two moderate earthquakes hunted Japan in 1983 and the big one in Mexico in 1988 and the draft was experimentally applied to the several damaged buildings damaged by these earthquakes. They are Namioka Town hospital building, of 5-story R/C, severely damaged by the 1983 Nihonkai-chubu Earthquake(Hirosawa 1985), Kurayoshi-higashi city office building, of 3-story R/C, moderately damaged by the 1983 Tottori Earthquake(Hirosawa 1984) and a private 9-story office building(Hirosawa 1987) moderately damaged by the 1985 Mexico Earthquake. The permanent restoration techniques were mainly applied to them. The outline of characteristics of their damage, main restoration method adopted, improvement of seismic performance and so on are listed in Table16.

Further, a draft for damage classification was also applied to the 12 buildings, of 3-21 story R/C, damaged by the 1985 Mexico Earthquake.

Results on classified damage degree on them were recognized almost equal to the results by the Mexico investigation team(Okada 1985).

Table 16 Outline of the Investigated and Restored R/C Building, Ovserved Damage Characteristics and Urgent & Permanent Restoration Work

Building	Number of Stories (Above/Below)	Year of Constr-ction	Suffered Earthquake	Damage Condition: General (DAMAGE DEGREE)	Damage Condition: Characteristics
Namioka Town Hospital[13]	5 / 1	1970	1983 Nihonkai-chubu	Third and fourth stories were severely collapsed in the longitudinal (SEVERE, D=53.8)	Concentration of damage to the intermediate stories due to special profile of the building
Kurayoshi-higashi City Office[14]	3 / 0	1958, 1970 (Addition)	1983 Tottori	Two corner columns out of the 16 columns at the second floor were widely cracked (MODERATE, D=27.4)	Damage by torsional vibration caused by additional entrance floor slabs connecting to the road
M-private office building[15]	9 / 1	1982	1988 Mexico	Bending and shear cracks appeared in the columns, walls and beams. Exterior and Interior finish were also moderately damaged (MODERATE, D=20.9)	• Moderately collapsed medium-rise building • Damage to finish

Building	Restoration: Urgent	Restoration: Permanent	Is-Value: Before Erq.	Is-Value: After Erq.	Cost Ratio(%) [Restoration / Renewal]
Namioka Town Hospital[13]	• Wire rope winding and H-steel support to heavily damaged columns at third and fourth floor	• Infilled shear walls • Column Jacketing by welded wire fabrics or steel plates	0.19~0.34 (Longitudinal) 0.31~0.79 (Transverse)	0.54~0.89 (L) 0.54~1.19 (T)	30 ~ 38
Kurayoshi-higashi City Office[14]	• H-steel support to the corner columns • K-shaped steel braces at each floor of the outside frame	• K-shaped steel braces • Column Jaketing by welded by welded hoops • Cut-off of waist high R/C wall	0.24~0.56 (L) 0.34~0.51 (T)	0.92~1.67 (L) 0.87~1.47 (T)	15
M-private office building[15]	• Only partial "Off-Limit"	• K-shaped steel braces • Increase of thickness of existing wall • Column Jacketing by welded hoops	0.19~0.43 (L) 0.16~0.29 (T)	0.74~1.60 (L) 0.93~1.05 (T)	20 ~ 30

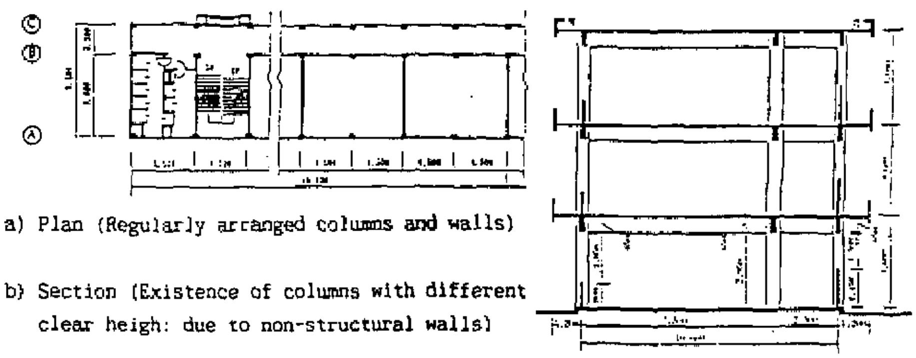


Fig.23 Plan and Section of a Typical Standardized School Building

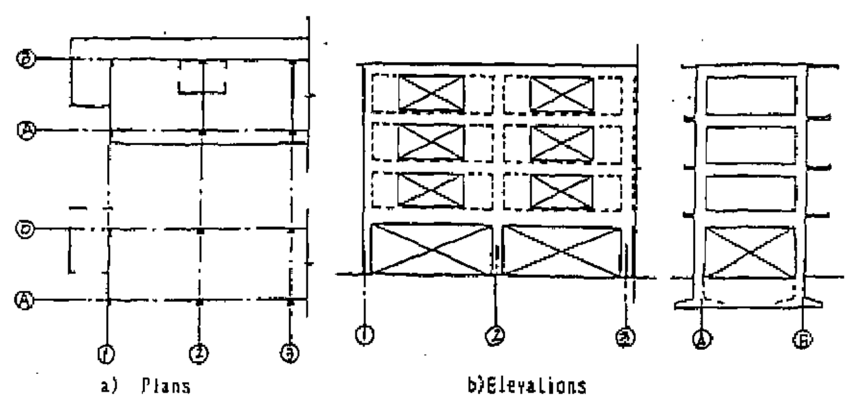


Fig.24 Plan and Elevation of a Typical Apartment House with Soft First Story

6 CONCLUSION

1) Due to insufficient data, experience and understanding of seismic phenomena, great deal of work is yet to be done in the area of seismic retrofit. Some of the problems to be solved are summarized as follows

a. Both analytical and experimental approaches should be used to assess the effect of retrofit on the overall behavior of buildings. Experimental verification of subassemblages is required Workmanship and detailing of connections which greatly affect the response of overall structures should be investigated using large or full scale specimens

b Existing test data must be evaluated systematically to obtain global information on not only the increased strength or ductility but also energy dissipating capacity and stiffness deterioration.

c Additional information is required with respect to the use of precast concrete, steel systems, and devices. The studies of other scheme of seismic retrofit, for example, to use base isolation system , is also required.

2) Through the above mentioned experiences, following items on existing or damaged buildings are pointed out as important.

a Also in Japan, there are many seismically vulnerable buildings which should be investigated Compilation of technical guidelines for seismic diagnosis and retrofit under supervision of the administrative authority would be necessary and effective.

b During recent 10 years, many experimental and analytical studies related to seismic evaluation and retrofitting of existing or damaged buildings, were carried out in Japan. Technical guidelines for not only existing but damaged buildings, by which quantitative estimation of their seismic performance can be done, were successfully compiled for reinforced concrete, steel and wooden structures. These will be effectively applicable to the same kind of buildings in the world.

c. In order to compile this kind of technical guidelines for general use and to extend execution of seismic investigation and retrofit of seismically vulnerable existing buildings and damaged buildings, national countermeasures. local governmental d.. ..ion and budgetary are essentially important.

ACKNOWLEDGEMENTS

A great many of the references, published in Japanese, have been omitted in this paper. This paper is, however, relied strongly on the work of others, and therefore the author wishes to acknowledge the invaluable contribution of other Japanese researchers.

REFERENCES

Akiyama T., Hirosawa M. 1988. "Seismic Inspection and Strengthening of Public Reinforced Concrete Buildings, Mainly in Case of School Buildings, 421-426, Vol.7, Proc. 9WCEE, Tokyo

Aoki H., et al. 1990 "Frame Test of Seismic Strengthening of Existing RC Building by Using Steel Element," Proc., Annual Meeting of Architectural Institute of Japan (in Japanese)

Aoyama H , et al 1984. "Strength and Behavior of Postcast Shear Walls for Strengthening of Existing RC Buildings," Proc., 8WCEE, San Francisco

Aoyama H., and Ichinose T. 1984. "Experimental Study on Seismic Behavior of RC Subassemblages with Slitted Spandrel Walls," Proc., 8WCEE, San Francisco

Arakawa T. 1980. "Effects of Welded Band Plates on Aseismic Characteristics of Reinforced Concrete Columns," Proc., 7WCEE, Istanbul

Endo T., et al. 1984. "Practices of Seismic Retrofit of Existing Concrete Structures in Japan," Proc., 8WCEE, San Francisco

Higashi Y , and Kokusho S 1975. "The Strengthening Method of Existing RC Buildings", Proc., Review Meeting of US-Japan Cooperative Program in Earthquake Engineering, Honolulu

Higashi Y., Okubo M., and Fujimata K. 1977 "Behavior of RC Columns and Frames Strengthened by Adding Precast Concrete Wall." Proc., 6WCEE, New Delhi

Higashi Y., et al. 1980. "Experimental Study of Strengthening Reinforced Concrete Structures by Adding Shear Wall," Proc., 7WCEE, Istanbul

Higashi Y., Endo T., and Shimizu Y. 1984. "Experimental Studies on Retrofitting of Reinforced Concrete Building Frames," Proc., 8WCEE, San Francisco

Hirosawa M., Kitagawa Y., Yamazaki Y. 1981. "Retrofitting on Medium-Rise Reinforced Concrete/Housing Structures" Proceedings of the Second Seminar on Repair and Retrofit of Structures-US/Japan Cooperative Earthquake Engineering Research Program Sponsored by NSF, Department of Civil Engineering, the University of Michigan, Ann Anbor, Michigan

Hirosawa M., Kitagawa Y., Yamazaki Y. and Teshigawara M. 1985. "Analysis on Damage of the Namioka Town Hospital Building during the 1983 Nihonkai-chubu Earthquake and Retrofit Design of the Building", BRI Research Paper No.117, August. 1985, BRI. MOC., Japan

Hirosawa M., Ohkubo M. Akiyama T., Shimizu Y. 1988. "A Proposal of Simplified Third Screening Seismic Diagnosis Method for Reinforce Concrete School Buildings with Recutting lar-Shaped Plan", pp. 10-24, '88.7, Building Disaster Prevention(in Japanese)

Hirosawa M., Yamazaki Y., Thukagoshi H. 1984. "Analysis on Damage of the Kurayoshi-higashi City office Building During the Tottori Earthquake of 1983", BRI Research Paper No.109, Dec. 1984, Building Research Institute, Ministry of Construc1tion, Japan

Hirosawa M., Yamanaka H , Akiyama T. 1987. "Seismic Inspection of Nine-Story RC Building Damaged by the Mexico Earthquake, 1985", Report No.12, Proceedings of the Seminar on Repair and Retrofit of Existing Structures, US-Japan Panel on Wind and Seismic Effects, UJNR, Thukuba, Ibaraki, Japan, May 8-9 1987

Hrosawa M 1992. "Current Conditions of Seismic Investigation and Retrofitting of Existing and Damaged Buildings in Japan," Proc., Intel Symposium on Earthquake Disaster Prevention, CENAPRED, Mexico

Japan Building Disaster Prevention Association, 1977, revised 1990. "Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings, " (in Japanese)

Japan Building Disaster Prevention Association, 1977, revised 1990. "Guidelines for Seismic Retrofitting Design of Existing Reinforced Concrete Buildings," (in Japanese)

Japan Building Disaster Prevention Association 1991 "Standard for Inspection of Damage Degree and Guideline for Restoration Technique -Rainforced Concrete Edition, Steel Structure Edition and Wooden Structure Edition-", (in Japanese)

Japan Concrete Institute, 1984. "Handbook for Seismic Strengthening of Existing Reinforced Concrete Structures," Gihodo Shuppan Co. Ltd. (in Japanese)

Kaminosono T. 1992. "Evaluation Method for Seismic Capacity of Existing Reinforced Concrete Buildings in Japan," Proc., International Symposium on Earthquake Disaster Prevention, CENAPRED, Mexico

Kaminosono T., Okada T., Hirosawa M. 1990. "Aseismic Performance and Strengthening of Existing Reinforced Concrete Building Structures", Workshop on Evaluation, Repair, and Retrofit of Structures, UJNR Task Committees C and D, May 1990, Caithersburg, Maryland,. S A.

Katsumata H., and Takeda T. 1988. "Study on Strengthening with Carbon Fiber for Earthquake Resistant Capacity of Existing Reinforced Concrete Columns." Proc., 9WCEE, Tokyo, Kyoto

Katsumata H., Seki M., Kumazawa F., and Okada T. 1989 "Retrofit Method of Existing Reinforced Concrete Structures by Ductile Steel Braces, "Proc , Japan Concrete Institute, Vol. 11, No. 2(in Japanese)

Kawabata S., Kamiya Y.et al. 1984. "A Case Study of Seismic Strengthening of Existing Reinfoced Concrete Building in Sizuoka Prefecture, Japan", Preeedings of the 8th WCEE, San-frencisco

Ministry of Construction 1986. "Manuals for Seismic Restoration Techniqus for Buildings(Draft)", 1986(in Japanese)

National Center for Earthquake Engineering Research 1986. "Technical Note of the Pubric Works Research Institute Vol.45, Manual for Repair Methods of Civil Engineering Structues Damaged by Earthquake"

Okada T., Murakami M. et al 1988. "Seismic Capacity of Reinforced Concrete Buildings which suffered 1985. 9. 19-20 Mexico Earthquake", pp.291-296, Vol.7, 9WCEE, Tokyo

Ohkubo M. 1991 "Current Japanese System on Seismic Capacity and Retrofit Techniques for Existing Reinforced Concrete Buildings and Post-earthquake Damage Inspection and Restoration Techniques, SSRP-91/02, UC San Diego

Sasaki T., et al. 1975. "An Experimental Study on Earthquake Resistant Strengthening Work for Exisiting RC Buildings," Proc., Review Meeting of US-Japan Cooperative Program in Earthquake Engineering, Honolulu

Shimizu Y., et al. 1992 "Investigation of an Old Brick Masonry Constructed in the Middle of the Meizi Era," Proc., Annual Meeting of Architectural Institute of Japan (in Japanese)

Sugano S., and Fujimura M. 1980. "Aseismic Strengthening of Existing Reinforced Concrete Buildings," Proc., 7WCEE, Istanbul

Sugano S. 1981. "Seismic Strengthening of Existing RC Buildings in Japan," Bull., New Zealand National Society for Earthquake Engineering, Vol 14, No 4

Sugano S., and Endo T. 1983. "Seismic Strengthening of RC Buildings in Japan," IABSE Symposium, Venice

Sugano S., and Fujimura M. 1985. "Seismic Strengthening of an Exisiting Reinforced Concrete Building by Steel Braces and Panels," Trans., Japan Concrete Institute. (inJapanese)

Sugano S. 1989. "Study of the Seismic Behavior of Retrofitted Reinforced Concrete Buildings," Proc., Structures Congress '89, ASCE, San Francisco

Sugano S. 1992. "Research and Design for Seismic Retrofit of Existing Buildings in Japan", Proc., International Symposium on Earthquake Disaster Prevention, CENAPRED, Mexico

Tasai T. 1991. "Repair and Retrofit of Damaged Buildings", Lecture Note for the 8-th Earthquake Engineering Seminar of IISEE. BRI. Japan, December 11

Usami H., Azuchi T., et al 1988. "Seismic Strengthening of Existing Reinforced Concrete Buildings in Shizuoka Prefecture, Japan", 415-420, Vol.7, Proc. 9WCEE

Yamamoto Y., and Kiyota S. 1983. "Experimental Study on Strengthening of RC Buildings. Part 2 Strengthening by Steel Systems," Proc., 29th Structural Engineering Symposium, Architectural Institute of Japan and Japan Society of Civil Engineers (in Japanese)

Yoshimura K., Kikuchi K., and Kuroki M. 1991. "Seismic Shear Strengthening Method for Existing R/C Short Columns, "Proc., 3rd Intel Conference on Steel-Concrete Composite Structures

Yamamoto Y., Hirosawa M. 1991. "Ultimate Seismic Strength and Ductility Index of Reinforced Concrete Building Strengthened with steel Brace and Steel Panel", Proceedings of the Fifth seminar on Repair and Retrofit of Structures-US/Japan Cooperative Earthquake Engineering Research Program Sponsored by NSF.

[III] EXAMPLE - I --- NAMIOKA TOWN HOSPITAL

1. INTRODUCTION

The Namioka Town Hospital, a reinforced concrete five stories building, suffered earthquake damage by the 1983 Nihonkai chubu Earthquake. The application results of the GUIDELINES FOR POST-EARTHQUAKE DAMAGE INSPECTION AND EVALUATION and the GUIDELINES FOR RESTORATION TECHNIQUES FOR DAMAGED BUILDINGS are presented as an example.

2. OUTLINES OF THE EARTHQUAKE

According to the report by the Japanese Meteorological Agency, the earthquake (named as the 1983 NIHONKAI-CHUBU EARTHQUAKE) has the following features :

Date and time	May 26, 1983.	12:00 pm (Japan time)
Epicenter	: 138 54' 40 24'	
Depth of epicenter	: 0 -10 km	
Magnitude	7.7	
Seismic Intensity	V (JMA)	

(The maximum horizontal acceleration on the surface of the ground is assumed as approximately 200 gal at the site of the building, according to another research report about the damage of Namioka Town Hospital)

The features of the maximum after-shock are :

Date and time	: June 21, 1983	15:25 pm (Japan time)
Epicenter	139 09' 41 16'	
Magnitude	: 7.0	

[COMMENTARY]

Fig. C-1 shows the location of epicenter and the distribution of ground acceleration recorded by Strong Motion Observation System. Fig. C-2 shows the ground acceleration records that was recorded at the Namioka Dam site (7 km from the Hospital) The maximum acceleration was approximately 130 gal in the North-South component.

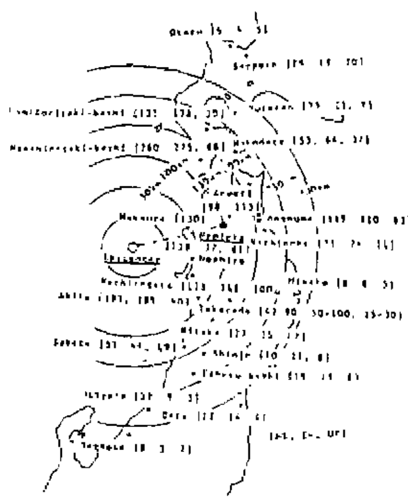


Fig C-1 EPICENTER AND DISTRIBUTION OF GROUND ACCELERATIONS

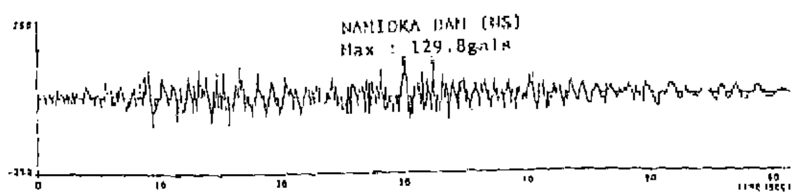


Fig C-2 GROUND ACCELERATION RECORDS AT THE NAMIOKA DAM SITE

3. OUTLINES OF BUILDING AND DAMAGE

3.1 Buildings

The Namioka Town Hospital is located at about 190 km east of the epicenter. The construction of buildings began in 1968, and the buildings completed in 1970. The hospital building has a little complicated floor planning with a court as shown in Fig 3.1 through Fig 3.3. The north building (ward for inpatients) consists of the six storied reinforced concrete frame structures with including one story's basement floor. A part of east block (for outpatients) is two stories, and the others (for examinations) are single story building.

Table-3.1 shows the size and rebars arrangement of typical columns, beams and walls.

The specified concrete strength in the design f_c was 210 kgf/cm^2. The rebars used was SD35 (nominal yield strength is required as 3500 kg/cm^2) for the 13 mm diameter bar (D13) through the 25 mm diameter bar (D25).

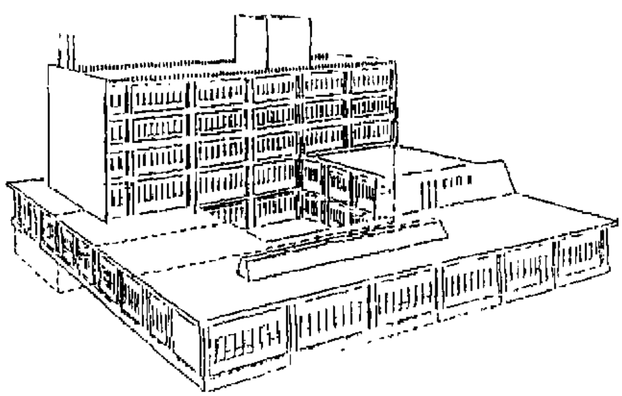


Fig 3.1 : PERSPECTIVE VIEW OF THE NAMIOKA TOWN HOSPITAL

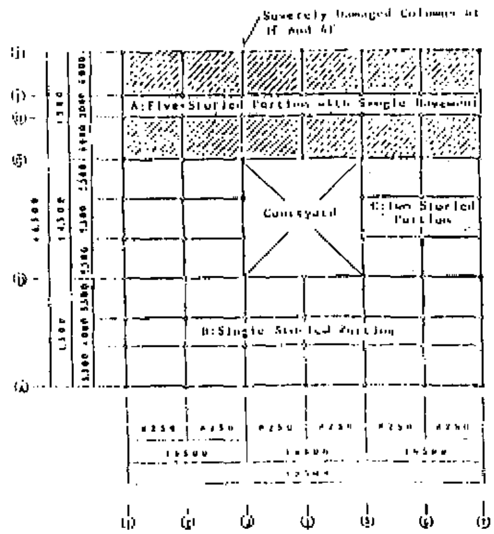


Fig. 3.2 : FIRST LEVEL FLOOR PLAN OF THE HOSPITAL

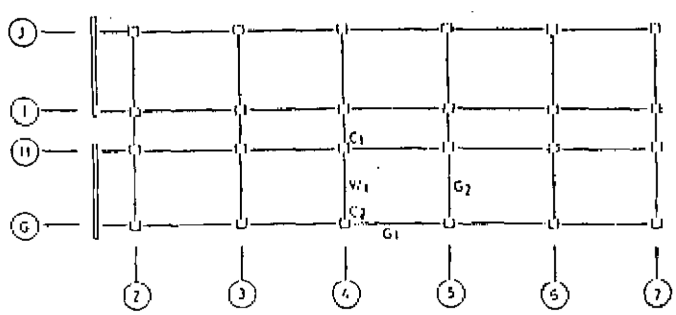


Fig. 3.3 : THIRD LEVEL FLOOR PLAN OF SICKROOM BUILDING

Table-3.1 : SECTION LISTS OF TYPICAL COLUMNS, BEAMS AND WALLS

Columns

Story	C1	C2
5	500 × 500	500 × 500
Main Bars	10 - D22	8 - D22
3	600 × 600	600 × 600
Main Bars	16 - D22	12 - D22
1	700 × 700	700 × 700
Main Bars	12 - D22	12 - D22

Note : Hoop 9φ @ 200 Top & Base 9φ @ 100
Diagonal Hoop 9φ @ 600

Walls

	W1
	t = 120
Section	
Vertical	9φ @ 300
Horizontal	9φ @ 300
Corner	2 - 13φ
	t = 120
Section	
Vertical	9φ @ 300
Horizontal	9φ @ 300
Corner	2 - 13φ
	t = 200
Section	
Vertical	9φ @ 250
Horizontal	9φ @ 250
Corner	2 - D19

Beams

Story	G1		G2		
	End	Center	GJ End	Center	HI End
R					
B × D	300 × 800		300 × 800		
Top	4 - D22	2 - D22	3 - D22	2 - D22	4 - D22
Bottom	2 - D22	3 - D22	2 - D22	6 - D22	3 - D22
Stirrup	End	Center	End	Center	
	End	Center	GJ End	Center	HI End
4					
B × D	350 × 800		350 × 800		
Top	6 - D25	3 - D25	6 - D25	3 - D25	5 - D25
Bottom	3 - D25	2 - D25	3 - D25	4 - D25	2 - D25
Stirrup					
	End	Center	GJ End	Center	HI End
2					
B × D	400 × 900		400 × 900		
Top	7 - D25	3 - D25	6 - D25	3 - D25	6 - D25
Bottom	[illegible] - D25	3 - D25	3 - D25	3 - D25	3 - D25
Stirrup					

Note Stirrup 9φ @ 300 Center
9φ @ 200 End

3.2 Damage of Structures

(1) Overall building

The north building (six storied frame structures including a basement floor) suffered the severest damage by the earthquake. The lower three stories including a basement floor of the north building did not suffer severe damage than the upper three stories. It might be the effect due to the other east, west, and south block of the building. The severest damage was observed at the third floor (the fourth level including a basement floor) of the north building. The damage degree of the fourth and fifth floor of north building followed the third floor's damage, respectively. The damage of X-direction (west - east) was severer than the Y-direction.

A lot of window glass were damaged and fallen at the upper floor than the third floor. The reinforced concrete parapets at the west and east end of roof inclined. The reinforced concrete chimney at the west end of roof also inclined. The finishing tiles or mortar on the exterior walls of north building spalled off.

(2) X-direction of North BUilding

The crack pattern of J-frame in X-direction is shown in Fig. 3. 4. Almost all the columns between the third line and the sixth line upper than third floor had clear shear cracks. In particular, the shear cracks on the columns at third floor were rather big with bond splitting failure cracks or concrete spalling. Fig. 3. 5 shows the typical shear failure with bond splitting failure cracks. The reinforced concrete walls upper than third floor, which were not designed as structural shear wall, suffered severe damage as shown in Fig. 3. 4 and Fig. 3. 6. The non-structural spandrel walls which were on the floor beams did not suffered any damage. However, floor beams had several narrow flexural cracks, which were observed after ceilings were removed.

(3) Y-direction of North Building

Many narrow shear cracks were observed on each shear wall between the second and seventh line in Y-direction upper than second floor. Also some columns with non-structural spandrel walls on the second floor had the narrow shear cracks. The big shear cracks were observed on the roof beam that supported the wall of penthouse.

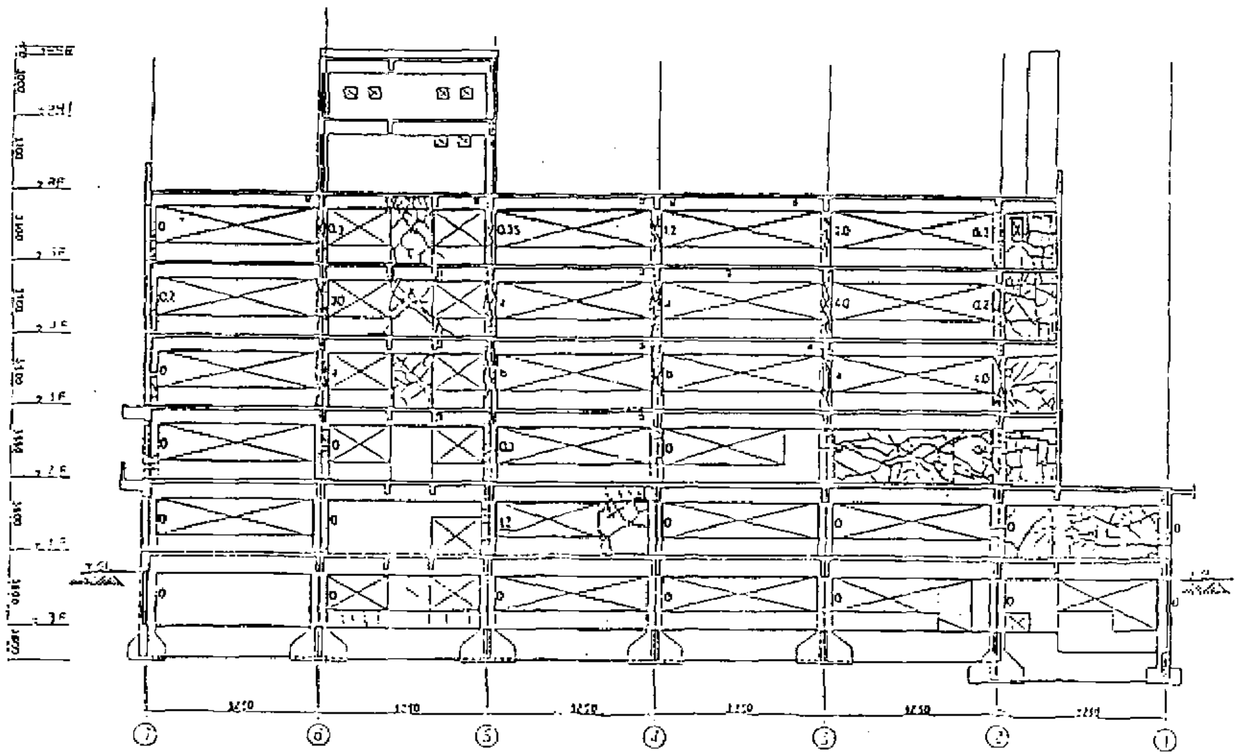
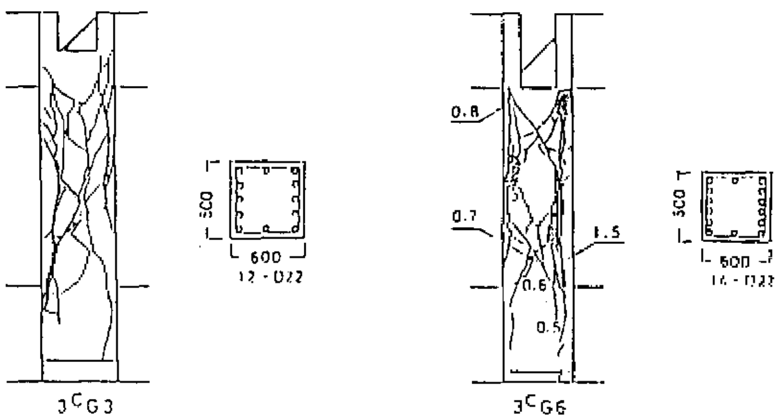


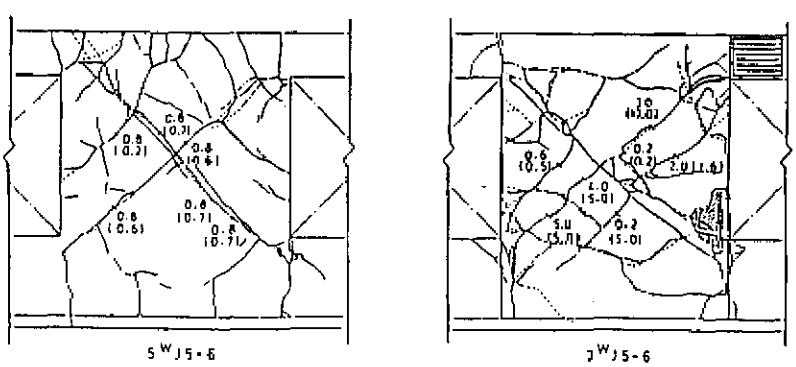
Fig. 3.4 : CRACK PATTERN OF EXTERIOR J-FRAME IN X-DIRECTION



(a) EXAMPLE OF SHEAR FAILURE COLUMN (C_{G3} on 3rd floor)

(b) EXAMPLE OF BOND SPLITTING FAILURE (C_{G6} on 3rd floor)

FIG. 3. 5 : CRACK PATTERN OF COLUMNS WITH SEVERE DAMAGE



(a) Wall W_{J5-6} on 5th floor

(b) Wall W_{J5-6} on 3rd floor

Fig. 3. 6 : CRACK PATTERN OF WALLS IN X-DIRECTION

4. RESULTS OF EMERGENCY INSPECTION AND EVALUATION

4.1 Settlement and Inclination of Building, and Damage of Members

(1) Inspection from Outside of The Building

The DAMAGE RANK concerning overall settlement and overall inclination of a building was judged as "RANK-A", because any visible deformation on the ground surrounding the building and any visible inclination of a building were not observed.

The DAMAGE RANK concerning damage of the structures was judged as "RANK-C" without a computation by Eq. (3.2) given in the "GUIDELINES FOR DAMAGE INSPECTION AND EVALUATION", because the severe damage corresponding to "RANK-IV" (concrete spalled) or "RANK-V (core concrete crushed) were observed on the several columns at the third floor.

Some items concerning POSSIBLE FALLING OBJECTS were judged as "RANK-C", because : (i) many tiles on the east and west exterior walls spalled, (ii) the reinforced concrete parapets on the east and west side of roof inclined inward outside of the building, and (iii) the roof chimney inclined remarkably.

(2) Inspection of Inside of The Building

Damage corresponding to "RANK-III" (crack width : 1 - 2 mm) was observed on the several interior columns. The finishing mortar of interior walls spalled off. The cover concrete at the landing of interior stairs was spalling, and possible falling was observed on it. Some medical equipments and an elevator suffered small damage.

4.2 Evaluation Results and Emergency Treatment

(1) Result of Evaluation

The RISK LEVEL of the building was judged as "DANGER" according to the provision given in Section 2.3 in the GUIDELINES FOR DAMAGE INSPECTION AND EVALUATION, because there was the "RANK-C" item concerning structural damage.

The RISK LEVEL concerning possible falling objects was judged as "DANGER", because there were more than one "RANK-C" items concerning damage of exterior walls, parapets, or roof chimney.

(2) Emergency Treatment

Entrance into the building was prohibited according to the provision given in Section 2.4 in the GUIDELINES FOR DAMAGE INSPECTION AND EVALUATION, because the RISK LEVEL of a building was evaluated as "DANGER". Also the approaching to the east and west outside of the building was prohibited according to "DANGER" of possible falling objects. The results of EMERGENCY INSPECTION are summarized in Table-4.1.

Table-4 1 RESULTS OF EMERGENCY INSPECTION

FORM FOR POST-EARTHQUAKE EMERGENCY INSPECTION
<FOR REINFORCED CONCRETE BUILDINGS>

TIME AND DATE OF INSPECTION
TIME ☐ a.m. ☐ p.m. ______ DAY ______ MON ______ YEAR ______
NAME OF INSPECTOR ______
1 DESCRIPTION OF BUILDING INSPECTED
1.1 NAME NAKAOKA TOWN HOSPITAL
1.2 LOCATION ______
1.3 OWNER ______ TEL ______
1.4 CONTACT PERSON ______ TEL ______
1.5 USE
☐ PRIVATE USE ☐ RESIDENCE ☐ APARTMENT ☐ OFFICE ☐ STORE
☐ WAREHOUSE ☐ FACTORY
☐ OTHER ______
☒ PUBLIC USE : ☐ SCHOOL ☐ NURSERY ☐ CITY HALL ☐ PUBLIC HALL
☐ POLICE ST ☐ FIRE ST ☒ HOSPITAL ☐ GYMNASIUM
☐ ASSEMBLY HALL ☐ BROADCAST ST
☐ OTHERS ______
1.6 NUMBER OF FLOORS ABOVE THE GROUND 5
BASEMENT 1 PENT HOUSE 2
1.7 STRUCTURAL SYSTEM : ☒ MOMENT RESISTING FRAME ☐ FLAT SLAB
☐ WALL (BOX) TYPE STRUCTURE
1.8 CLADDING ☒ MORTAR ☒ TILE ☐ CURTAIN WALL ☐ BRICK
☐ SHEET METAL , ☐ NONE ☐ OTHERS ______

2 INSPECTION
2.1 INSPECTION FOR STRUCTURES

(EXTERIOR DAMAGE)	DAMAGE RANK (A)	(B)	(C)
· OVERALL INCLINATION (Deg)	☒ < 1	☐ 1 - 2	☐ > 2
· OVERALL SETTLEMENT (m)	☒ < 0.2	☐ 0.2 - 1	☐ > 1.0

· DAMAGE TO STRUCTURAL MEMBERS AT THE FLOOR NO. 3
(a) IN CASE OF FRAME STRUCTURE / FLAT SLAB
TOTAL NUMBER OF EXTERIOR COLUMNS 12
NUMBER / RATIO OF EXTERIOR COLUMNS TO DAMAGE RANK
RANK-IV 4 4/12 ☐ < 10 % ☐ 10 - 20 % ☒ > 20 %
RANK-V 2 2/12 ☐ < 1 % ☐ 1 - 10 % ☒ > 10 %
(b) IN CASE OF WALL (BOX) TYPE STRUCTURES
TOTAL LENGTH OF EXTERIOR WALLS ______ (m)
LENGTH / RATIO OF EXTERIOR WALLS TO DAMAGE RANK
RANK-IV ______ m / ☐ < 10 % ☐ 10 - 20 % ☐ > 20 %
RANK-V ______ m / ☐ < 1 % ☐ 1 - 10 % ☐ > 10 %

(INTERIOR DAMAGE)
(a) IN CASE OF FRAME STRUCTURE / FLAT SLAB
TOTAL NUMBER OF INTERIOR COLUMNS 12
NUMBER / RATIO OF INTERIOR COLUMNS TO DAMAGE RANK
RANK-IV 0 0/12 ☒ < 10 % ☐ 10 - 20 % ☐ > 20 %
RANK-V 0 0/12 ☒ < 1 % ☐ 1 - 10 % ☐ > 10 %
(b) IN CASE OF WALL (BOX) TYPE STRUCTURES
TOTAL LENGTH OF INTERIOR WALLS ______ (m)
LENGTH / RATIO OF INTERIOR WALLS TO DAMAGE RANK
RANK-IV ______ m / ☐ < 10 % ☐ 10 - 20 % ☐ > 20 %
RANK-V ______ m / ☐ < 1 % ☐ 1 - 10 % ☐ > 10 %

2.2 POSSIBLE RISK OF OVERTURNING OR FALLING OBJECTS

	RISK LEVEL (a) SAFETY	(b) CAUTION	(c) DANGER
(EXTERIOR)			
· WINDOW GLASS	☐	☐	☒
· SIGN-TOWER	☐	☐	☐
· SIGNBOARD	☐	☐	☐
· STAIRCASE	☐	☐	☐
· CLADDING	☐	☐	☒
· EAVES	☐	☐	☐
· BALCONY	☐	☐	☐
· PARAPET	☐	☐	☒
· ELEVATED TANK	☐	☐	☐
· COOLING TOWER	☐	☐	☐
· CHIMNEY	☐	☐	☒
· PENTHOUSE	☐	☐	☐
· OTHERS ______	☐	☐	☐
(INTERIOR)			
· CEILING	☐	☐	☐
· LIGHTING APPARATUS	☐	☐	☐
· OTHERS ______	☐	☐	☐

3 RESULTS OF INSPECTION
· TOTAL NUMBER OF RISK LEVEL-(c) ______
· TOTAL NUMBER OF RISK LEVEL-(b) ______

4 DECISION AND SUGGESTION

	BUILDING	BUILDING-SURROUNDINGS
· SAFETY	☐	☐
· CAUTION (ENTRY LIMITED)	☐	☐
· DANGER (OFF LIMITS)	☒	☒

5. EMERGENCY RESTORATION

5.1 Emergency Treatment Immediately After The Earthquake

Some columns which were judged as "RANK-V" were restored temporarily, because use of a part of the building (examination rooms for outpatients at the first floor of the north building) in succession after the earthquake was required by the director of the Hospital.

(1) Basic Policy for Emergency Treatment

(a) Some technique should be applied to mitigate the degradation of capacity to sustain a vertical load for some columns damaged severely at the third floor.
(b) Strengthening, such as adding steel bracing system, should be applied to recover the lateral capacity in X-direction at the third or fourth floor.
(c) The inclined parapets or chimney should be demolished.

(2) Emergency Treatment

The above (b) was not carried out as an emergency treatment, because there were some problems concerning the construction and time. The technique winding a column by steel wire as shown in Fig.5.1 was applied for some columns as the policy (a) above. A research was available for applying the technique, because the effectiveness had been investigated before the earthquake by some tests which focussed on emergency restoration techniques.

The work winding steel wire was begun after three days of the earthquake, and it continued for two days.

The inclined parapets were tied to a penthouse with steel wire to prevent falling down after three days of the earthquake. After several days they were demolished as well as an inclined chimney.

5.2 The Second Emergency Restoration

Many after-shocks occurred for one month after the main earthquake. The maximum after-shock was on June 21th and the magnitude was 7.0 by the modified Richter Scale. The damage of some columns progressed by the after-shocks. The director of the Hospital desired to use a part of the second floor as wards after the end of June. The second emergency strengthening for some columns was carried out to make the director's demand possible.

Three columns on the third floor and two columns on the fourth floor were strengthened by using steel ties and the H-shape steel supports. Fig.5.2 and Fig.5.3 show the pictures after emergency strengthening.

The work for strengthening was carried out during July 5th through 20 th. The strengthening cost per a column was a little expensive than the first emergency strengthening.

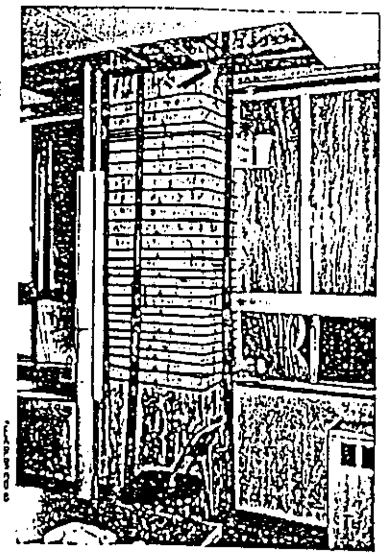


Fig. 5. 1 : EMERGENCY RESTORATION BY STEEL WIRE

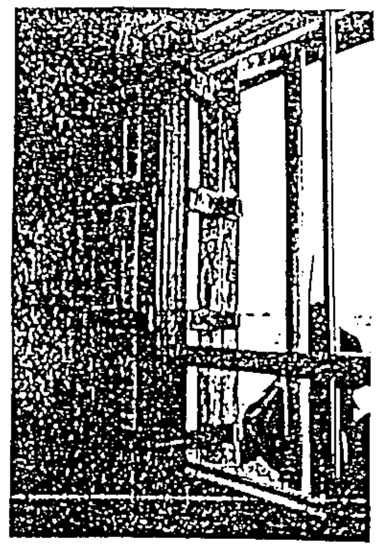


Fig. 5. 2 : EMERGENCY SUPPORT BY H-SHAPE HOT ROLL STEEL

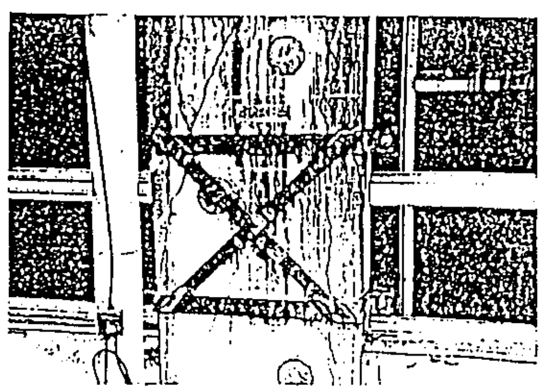


Fig. 5. 3 : EMERGENCY RESTORATION BY STEEL TIES

6. RESULTS OF DAMAGE CLASSIFICATION EVALUATION

6.1 Damage Rank of Members

The damage inspection for structural members was done at the third floor, where was the severest damage in the north building. The Damage Rank of Members, which were classified according to Table-3.1 given in the Section 3 of GUIDELINES FOR DAMAGE INSPECTION AND EVALUATION, is presented in Fig. 6.1.

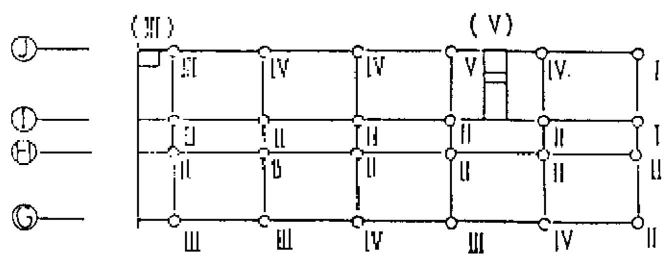


Fig. 6.1 : DAMAGE RANK OF COLUMNS AT THE THIRD FLOOR

6.2 Evaluation Results and Permanent Treatment

The Damage Ratio D for X-direction at the third floor was evaluated by Eq. (3.2) which was given in the Section 3.3 of GUIDELINES FOR DAMAGE INSPECTION AND EVALUATION, using the Damage Rank of Columns given in Fig. 6.1.

Damage Rank I : $D_1 = 10B_1/A = 10 \times 2/24 = 0.8$
Damage Rank II : $D_2 = 26B_2/A = 26 \times 11/24 = 11.9$
Damage Rank III : $D_3 = 60B_3/A = 60 \times 5/24 = 12.5$
Damage Rank IV : $D_4 = 100B_4/A = 100 \times 4/24 = 16.7$
Damage Rank V : $D_5 = 1000B_5/7/A = 1000 \times 2/7/24 = 11.9$

<u>Sum of D_i = $D_1 + D_2 + D_3 + D_4 + D_5$ = 53.8 ≥ 50</u>

where, B_i : number of columns which were identified to Damage Rank i.

A : total number of columns at the floor.

The DAMAGE DEGREE of a building was judged as "SEVERE" according to the Section 3.3 of GUIDELINES FOR DAMAGE INSPECTION AND EVALUATION, because the sum of Damage Ratio ΣD_i was more than 50 as above

IGTHENING" was also required for this building to use con sly, according to Table 3.2 of the GUIDELINES FOR DAMAGI TION AND EVALUATION

6.1 shows the form that was used for Damage Degree Classi on Inspection

Table-6.1 : RESULTS OF DAMAGE DEGREE CLASSIFICATION

TIME AND DATE OF INSPECTION

TIME [ ]A - [ ]P = ______ MONTH ______ DAY ____ YEAR ______

1 DESCRIPTION ON BUILDING INSPECTED

1.1 BUILDING NAME _NAHIOKA TOWN HOSPITAL_

1.2 LOCATION ______

1.3 OWNER ______ TEL ______

1.4 CONTACT PERSON ______ TEL ______

1.5 USE

[ ]PRIVATE USE : [ ]RESIDENCE [ ]APARTMENT [ ]OFFICE [ ]STORE
[ ]WAREHOUSE [ ]FACTORY

[X]PUBLIC USE : [ ]SCHOOL [ ]NURSERY [ ]CITY HALL [ ]PUBLIC HALL
[ ]POLICE ST [ ]FIRE ST [X]HOSPITAL [ ]GYMNASIUM
[ ]ASSEMBLY HALL [ ]BROADCAST ST
[ ]OTHERS ______

1.6 NUMBER OF FLOORS ABOVE THE GROUND _5_
BASEMENT _1_ PENTHOUSE _2_

1.7 STRUCTURAL SYSTEM : [X]MOMENT RESISTING FRAME [ ]FLAT SLAB
[ ]WALL (BOX) TYPE STRUCTURE

1.8 FOUNDATION SYSTEM [X]WITH PILES [ ]WITHOUT PILES

1.9 CIRCUMSTANCE AT SITE [X]FLAT [ ]SLOPE [ ]HEIGHTS [ ]DEPRESSION
[ ]TOP OF PRECIPICE [ ]BOTTOM OF PRECIPICE
[ ]SEASIDE [ ]LAKESIDE

1.10 CLADDING , MORTAR [X]TILE [ ]CURTAIN WALL [ ]BRICK [ ]SHEET METAL
[ ]NONE [ ]OTHERS ______

1.11 DOCUMENTS ON DESIGN OR CONSTRUCTION , [X]PRESERVED [ ]NONE

2 EVALUATION OF OVERALL SETTLEMENT, S : MAXIMUM SETTLEMENT (METER)
[X]NONE (S=0) [ ]SMALL (S ≤ 0.2) [ ]MEDIUM (0.2 < S ≤ 1.0) [ ]SEVERE (S > 1.0)

3 EVALUATION OF OVERALL INCLINATION, θ : MAXIMUM INCLINATION (RADIAN)
[X]NONE (θ=0) [ ]SMALL (θ ≤ 1/100) [ ]MEDIUM (1/100 < θ ≤ 3/100)
[ ]SEVERE (3/100 < θ ≤ 6/100) [ ]OVERTURNED (θ > 6/100)

4 EVALUATION OF STRUCTURAL DAMAGE , the result on the story at where the most severe damage was observed shall be represented here

4.1 THE STORY EVALUATED _3_ DIRECTION OF FRAME [X]X [ ]Y

4.2 TOTAL NUMBER (LENGTH) OF COLUMNS (WALLS), A_0 _24_

4.3 TOTAL NUMBER (LENGTH) OF COLUMNS (WALLS) INSPECTED A _24_

4.4 INSPECTED RATIO A/A_0 _24/24_

4.5 THE NUMBER (LENGTH) OF COLUMNS (WALLS) WITH EACH DAMAGE DEGREE

DAMAGE DEGREE	0	I	II	III	IV	V
NUMBER (LENGTH)	0	2	11	5	4	2

4.6 CALCULATION FOR DAMAGE RATIOS OF STRUCTURE, D_i AND THE SUM

DAMAGE LEVEL I : $D_1 = 10n_1/A$ = _0.8_ (not greater than 5)
DAMAGE LEVEL II : $D_2 = 26n_2/A$ = _11.9_ (not greater than 13)
DAMAGE LEVEL III : $D_3 = 60n_3/A$ = _12.5_ (not greater than 30)
DAMAGE LEVEL IV : $D_4 = 100n_4/A$ = _16.7_ (not greater than 50)
DAMAGE LEVEL V : $D_5 = 1000n_5/7A$ = _11.9_ (not greater than 50)

THE SUM OF D_i, $\Sigma D = \Sigma D_1 \sim D_5$ = _53.8 > 50_

5 IDENTIFICATION OF DAMAGE DEGREE FOR THE ENTIRE BUILDING
[ ]NONE [ ]SLIGHT [ ]SMALL [ ]MEDIUM [X]SEVERE [ ]COLLAPSE

6 DAMAGE OF SUB-STRUCTURES :

PENTHOUSE	[ ]SLIGHT	[ ]SMALL	[ ]MEDIUM	[ ]SEVERE	[ ]COLLAPSE
EXTERIOR STAIRCASE :	[ ]SLIGHT	[ ]SMALL	[ ]MEDIUM	[ ]SEVERE	[ ]COLLAPSE
CHIMNEY	[ ]SLIGHT	[ ]SMALL	[ ]MEDIUM	[X]SEVERE	[ ]COLLAPSE
EXTERIOR PASSAGE :	[ ]SLIGHT	[ ]SMALL	[ ]MEDIUM	[ ]SEVERE	[ ]COLLAPSE
EXPANSION JOINTS :	[ ]SLIGHT	[ ]SMALL	[ ]MEDIUM	[ ]SEVERE	[ ]COLLAPSE

THE OTHERS _ROOF PARAPET : SEVERE_

7 DAMAGE OF FOUNDATION ,

PILES : [ ]DAMAGED [X]NOT DAMAGED [ ]UNCERTAIN
LIQUEFACTION [ ]OCCURRED [X]NOT OCCURRED [ ]UNCERTAIN

8 REMARKS OR MEMO : _SHOULD BE STRENGTHENED_
FOR FUTURE USE

7. PERMANENT RESTORATION

7.1 Outlines

The followings were carried out to recognize the damage more clearly and to make the restoration planning

INVESTIGATION IN SITE : detailed crack observation for structural members, tests of structural materials (core concrete and rebars), damage investigation for non-structural members and building equipments, bearing test for piles.

ANALYSIS : damage degree analysis by the advanced technique which was given in the Section 3.3 of GUIDELINES FOR RESTORATION TECHNIQUES FOR SUFFERED BUILDINGS, seismic screening for the condition before damage, and non-linear dynamic response analysis.

In this section, the outlines about the damage degree analysis by the advanced technique and the restoration construction are presented.

7.2 Judgment for Necessary Strengthening

The necessary strengthening for the structures of north building was required by the DAMAGE DEGREE CLASSIFICATION. In addition to that, an analysis by the advanced technique that is given in the GUIDELINES FOR RESTORATION TECHNIQUES OF SUFFERED BUILDINGS was carried out to get an information about the remaining seismic performance of structures.

7.2.1 Seismic Performance Before Damage

(1) Structural Members Analyzed

The seismic performance before earthquake damage in X-direction at the third floor was estimated as followings by using the second level procedure of seismic screenings. Fig 7.1 shows the layout plan of columns or walls at the third floor

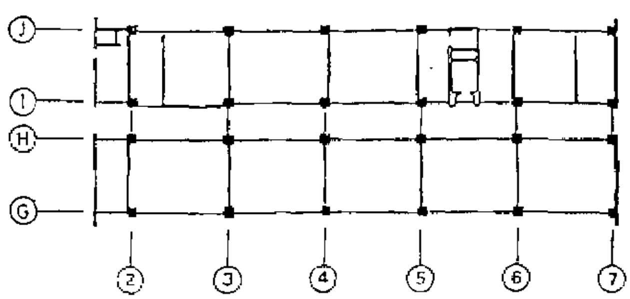


Fig. 7.1 : LAYOUT PLAN OF COLUMNS OR WALLS AT THE 3RD FLOOR

(2) Calculation of Index E_o

The lateral strength (shear strength or shear force corresponding to flexural failure), value of Index F or failure patterns of columns or walls at the third floor for the state before damage are summarized in the Table-7.1.

Table-7.1 : STRENGTH, INDEX-F OR FAILURE PATTERN OF MEMBERS

MEMBER NOTATION	STRENGTH V_u (ton)	INDEX-F	FAILURE PATTERN	NUMBER OF MEMBERS	$n \times V_u$ (ton)
C_{G2}	56.0	1.0	shear	1	56.0
C_{G3-4}	56.8	1.0	shear	2	113.6
C_{G5}	58.6	1.0	shear	1	58.6
C_{G6}	49.9	1.0	shear	1	49.9
C_{G7}	52.3	1.3	flexure	1	52.3
C_{H2}	43.6	1.3	flexure	1	43.6
C_{H3-5}	50.8	1.0	shear	3	152.4
C_{H6}	40.6	1.0	shear	1	40.6
C_{H7}	37.2	1.3	flexure	1	37.2
C_{I2}	43.8	1.3	flexure	1	43.8
C_{I3}*	133.6	1.0	shear	1	133.6
C_{I4}	47.1	1.0	shear	1	47.1
C_{I5-6}	52.7	1.0	shear	2	105.4
C_{I7}	37.2	1.3	flexure	1	37.2
C_{J2}*	188.1	1.0	shear	1	188.1
C_{J3}	48.3	1.0	shear	1	48.3
C_{J4}	58.2	1.0	shear	1	58.2
C_{J5}	49.9	1.0	shear	1	49.9
C_{J6}	62.0	1.0	shear	1	62.0
C_{J7}	52.4	1.3	flexure	1	52.4
W_{I2}	9.6	2.0	flexure	1	9.5
W_{J5}	31.4	1.0	flexure	1	31.4

$\Sigma V_u = \underline{1,471.2}$

note. columns with * represent the column with side-wall)

The value of Index E_o was calculated as follows :

- Building weight upper than third floor $\Sigma W = 2{,}970$ (ton)
- Number of building story : $n = 5$ (except basement floor)
- Floor level analyzed : $i = 3$ (the third floor)
- Coefficient by floor level : $(n+1)/(n+i) = 6/8 = 0.75$
- Value of Strength Index C $\Sigma V_u / \Sigma W = 1,471.2/2,970 = 0.495$
- Value of Ductility Index $F = 1.0$ (The values of index F for some flexure failure pattern columns were larger than 1.0 However, all the structural members were grouped to one to get a conservative result.)
- Value of Index $E_o = (n+1)/(n+i)\ \sqrt{(C \times F)^2}$

$$= 0.75\ \sqrt{(0.495 \times 1.0)^2}$$
$$= \underline{0.371}$$

(3) Calculation of Index S_D

The following factors which affected to the calculation of Index S_D were estimated according to the instruction written in the STANDARD FOR EVALUATION OF SEISMIC CAPACITY OF EXISTING REINFORCED CONCRETE BUILDINGS (see Table-9 of the STANDARD). The value of other factors which were not estimated here was assumed as 1.0.

a:<u>Regularity</u> --- $a = 16.5^2/(44.35 \times 15.0 + 16.5 \times 16.5)$
$= 0.29 < 0.3$
$G_a = 0.9$ (by Table-9 of STANDARD)
$R_a = 0.5$ (ditto)
$q_a = \{1 - (1 - G_a)R_a\} = \{1 - (1 - 0.9)0.5\}$
$= \underline{0.95}$

h:<u>Basement</u> ----- Area of basement floor $= 775.5\ m^2$
Area of 1st floor $= 2,029.5\ m^2$
$h = 775.5/2,029.5 = 0.38$
$G_h = 0.8$ (by Table-9 of STANDARD)
$R_h = 1.0$ (ditto)
$q_h = \{1.2 - (1 - G_h)R_h\} = \{1.2 - (1 - 0.8)1.0\}$
$= \underline{1.0}$

l:<u>Eccentricity ratio</u>
Location of centroid of lateral stiffness : S
S = 8.97 m (from column line G)
Location of centroid of gravitational load : G
G = 7.70 m (from column line G)
Eccentricity distance between S and G
$= 8.97 - 7.70 = 1.27$ m
Length of frame (X-direction) : L = 44.35 m
Length of frame (Y-direction) : B = 15.0 m

$l = 1.27/\sqrt{44.35^2 + 15.0^2} = 0.027$
$G_l = 1.0$ (by Table-9 of STANDARD)
$R_l = 1.0$ (ditto)
$q_l = \{1 - (1 - G_l)R_l\} = \{1 - (1 - 1.0)1.0\}$
$= \underline{1.0}$

n:<u>Mass stiffness ratio</u>
4th floor's lateral stiffness = 267.8
3rd floor's lateral stiffness = 339.4
Building weight upper than 4th floor = 2096 ton
Building weight upper than 3rd floor = 2970 ton
Mass stiffness ratio at 4th floor : r_4
$= 267.8/2096 = 0.127$
Mass stiffness ratio at 3rd floor : r_3
$r_3 = 339.4/2970 = 0.114$
Coefficient $b = (N - 1)/N = (3 - 1)/3 = 0.67$
(N: number of stories upper than 3rd floor)
$n = (r_4/r_3)b = (0.127/0.114)0.67 = 0.75 < 0.8$
$G_n = 0.9$ (by Table-9 of STANDARD)
$R_n = 1.0$ (ditto)
$q_n = \{1 - (1 - G_n)R_n\} = \{1 - (1 - 0.9)1.0\} = \underline{0.9}$

$$\begin{aligned} \text{Index } S_D &= q_a \times q_b \times \cdots\cdots\cdots \times q_o \\ &= 0.95 \times 1.0 \times 1.0 \times 0.9 \\ &= \underline{0.855} \end{aligned}$$

(4) Calculation of Index T

The value 1.0 was assumed for the Index T, because the special deterioration on the building structures was not reported by the questionnaire to the director of the Hospital and others.

(5) Index I_s Before Damage

$$I_s = E_o \times S_D \times T = 0.371 \times 0.855 \times 1.0 = \underline{0.317}$$

7.2.2 Seismic Performance After Damage

(1) Capacity Reduction Factor η Assumed

Fig. 7.2 shows the capacity reduction factors of columns at the third floor after the earthquake. The capacity reduction factors are given in the table of GUIDELINES FOR RESTORATION TECHNIQUES OF SUFFERED BUILDINGS.

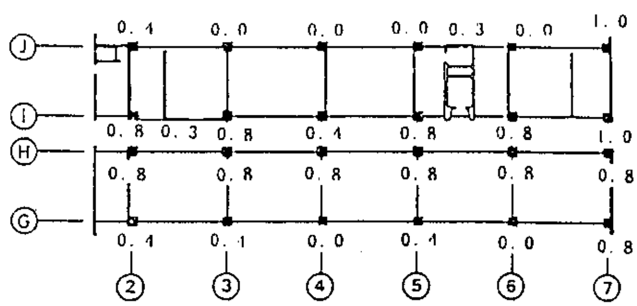


Fig. 7.2 : CAPACITY REDUCTION FACTOR η BY EARTHQUAKE DAMAGE

(2) Remaining Lateral Seismic Capacity

Fig. 7.3 shows the remaining lateral strength of columns which were obtained by multiplying the lateral strength before damage by η-factor shown in Fig. 7.2.

(3) Index E_o After Damage

- Sum of lateral strength after damage, ΣV_u = 644.9 ton
- Building weight upper than 3rd floor, ΣW = 2,970 ton

· Index C, $C = \Sigma V_u / \Sigma W = 644.9 / 2,970 = \underline{0.217}$
· Index F assumed, $F = \underline{1.0}$
· Index E_0 after damage, $E_0 = (n+1)/(n+i)\sqrt{(C \times F)^2}$
$= (5+1)/(5+3)\sqrt{(0.217 \times 1.0)^2}$
$= 0.75 \times 0.217 = \underline{0.163}$

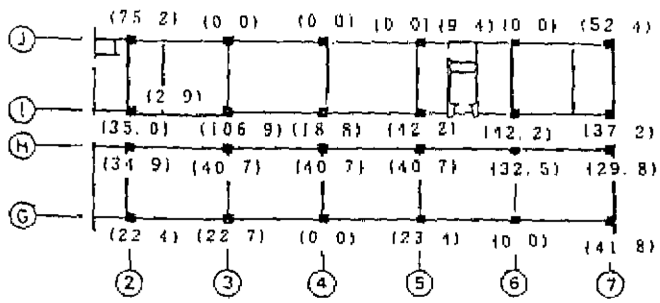


Fig.7.3 REMAINING LATERAL STRENGTH OF COLUMNS OR WALLS

(4) Index S_D After Damage

The factors concerning ECCENTRICITY RATIO q_1 and MASS STIFFNESS RATIO q_n were modified as follows considering stiffness degradation due to damage

i: Eccentricity ratio

Location of centroid of lateral stiffness : S
$S = 8.31$ m (from column line G)
Location of centroid of gravitational load : G
$G = 7.70$ m (from column line G)
Eccentricity distance between S and G
$= 8.31 - 7.70 = 0.61$ m
Length of frame (X-direction) : $L = 44.35$ m
Length of frame (Y-direction) : $H = 15.0$ m

$l = 0.61/\sqrt{44.35^2 + 15.0^2} = 0.013$
$G_1 = 1.0$ (by Table-9 of STANDARD)
$R_1 = 1.0$ (ditto)
$q_1 = \{1 - (1 - G_1)R_1\} = \{1 - (1 - 1.0)1.0\}$
$= \underline{1.0}$

ii: Mass stiffness ratio

4th floor's lateral stiffness = 184.0
3rd floor's lateral stiffness = 163.7
Building weight upper than 4th floor = 2096 ton
Building weight upper than 3rd floor = 2970 ton
Mass stiffness ratio at 4th floor : r_4
$r_4 = 184.0/2096 = 0.088$
Mass stiffness ratio at 3rd floor : r_3
$r_3 = 163.7/2970 = 0.055$
coefficient $b = (N - 1)/N = (3 - 1)/3 = 0.67$

(N number of stories upper than 3rd floor)
$\alpha = (r_4/r_3)b = (0.088/0.055)0.67 = 1.07 < 1.2$
$G_n = 1.0$ (by Table-9 of STANDARD)
$R_n = 1.0$ (ditto)
$q_n = [1 - (1 - G_n)R_n] = [1 - (1 - 1.0)1.0]$
$= \underline{1.0}$

Index $S_D = q_a \times q_b \times \cdots\cdots\cdots \times q_n$
$= 0.95 \times 1.0 \times 1.0 \times 1.0$
$= \underline{0.95}$

(5) Index T After Damage

The value 1.0 was assumed for Index T, because cracks or any other deterioration were reflected into the capacity reduction factors η.

(6) INdex I_s After Damage

$I_s = E_o \times S_D \times T = 0.163 \times 0.95 \times 1.0 = \underline{0.155}$

7.2.3 Damage Degree and Judgment for Necessary Strengthening

Using the values of I_s before and after damage the damage degree of Namioka Town Hospital was estimated .

$\Phi = (1 - I'_s/I_s)\times100 = (1 - 0.155/0.317)100 = 51.1$ (%)

This building was constructed in 1968. The seismic intensity by the Japanese Meteorological Agency was assumed more than V at the site. Therefore, necessary strengthening was required for future use of this building, according to Table-3.2 given in the Section 3.3 of the GUIDELINES FOR RESTORATION TECHNIQUES FOR SUFFERED BUILDINGS

7.3 Strengthening Planning

7.3.1 Basic Policy

The followings were discussed ;
- Strengthening should be required for not only third floor but also each floor, considering the seismic performance as an entire building after strengthening
- The medical function should not be disturbed as possible by the arrangement of strengthening elements
- Any economical techniques should not be applied basically

7.3.2 Target I_s Value for Strengthening

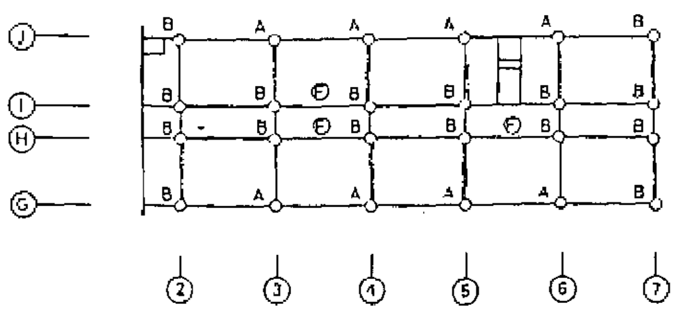
The judgment value I_{so} is usually assumed as 0.6 in seismic screening for common existing buildings. The restoration guidelines recommends a little higher I_s value for strengthening of a damaged building However, the I_s value 0.6 was determined as a target for strengthening of this building, considering that this building is located in the region whose zoning factor Z for seismicity is 0.9.

7.3.3 Policy for Strengthening

Followings were emphasized in the strengthening planning .

- The frames in X-direction should keep the flexural failure pattern as possible after strengthening.
- The columns with damage rank IV or V should be reconstructed after demolishing damaged concrete and rearranging rebars.
- The columns with damage rank III should be strengthened by jacketing by steel plates or concrete with welded wire fabrics.
- The columns with damage rank less than II may be repaired by epoxy resin injection.
- Some shear walls should be in-filled into the X-direction's frame upper than the 3rd floor in order to increase the lateral strength.
- The boundary beams adjacent to an in-filling shear wall also should be strengthened to prevent the shear failure.

Fig.7.4 shows the layout plan of in-filling shear walls or strengthening and repairing techniques proposed for the 3rd floor.



(note) A : recast concrete in site after demolishing damage concrete
B : jacketing by concrete with welded wire fabrics
C : jacketing by steel plates
D : strengthening by steel ties
E : repairing by epoxy resin injection
(F) : jacketing by concrete with welded wire fabrics for beams
: in-filling reinforced concrete shear wall
: increasing thickness of existing shear wall

Fig.7.4 : LAYOUT PLAN OF IN FILLING SHEAR WALLS OR STRENGTHENING AND REPAIRING TECHNIQUES PROPOSED (3RD FLOOR)

Fig. 7.5 through Fig. 7.9 show the details for the strengthening techniques proposed.

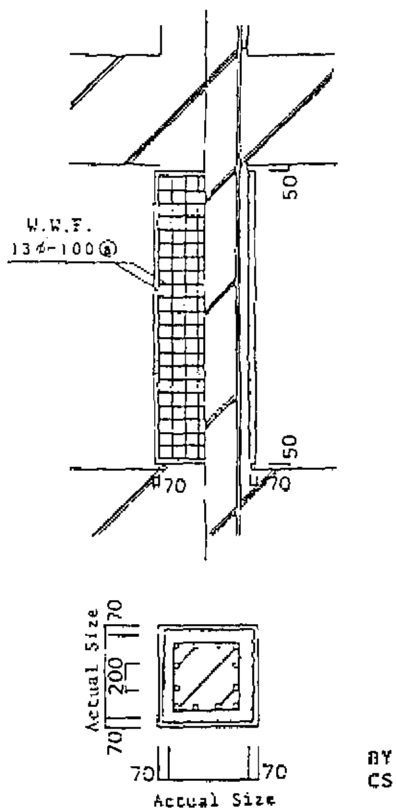


Fig. 7.5 : COLUMN JACKETED BY WELDED WIRE FABRICS

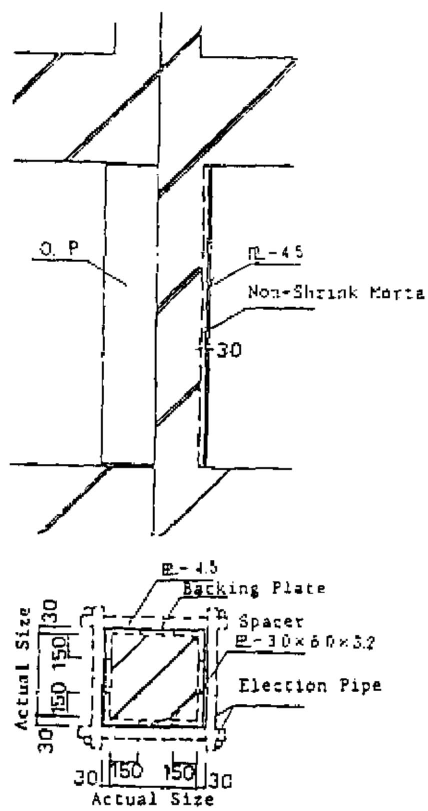


Fig. 7.6 : COLUMN JACKETED BY STEEL PLATES

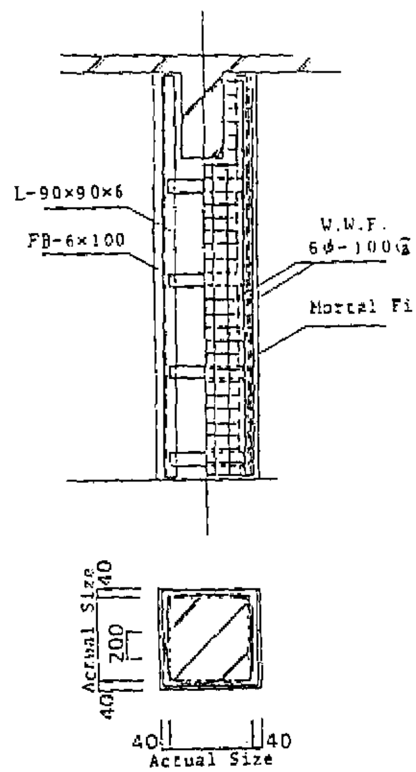


Fig. 7.7 : COLUMN RETROFITTED BY TIE PLATES

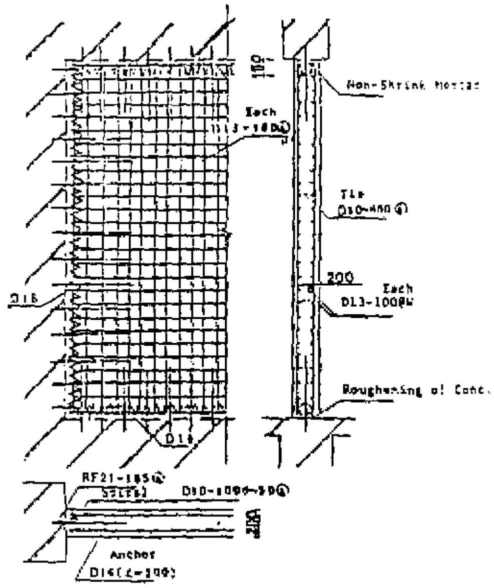


Fig. 7. 8 : REBARS ARRANGEMENT OF IN-FILLED SHEAR WALL

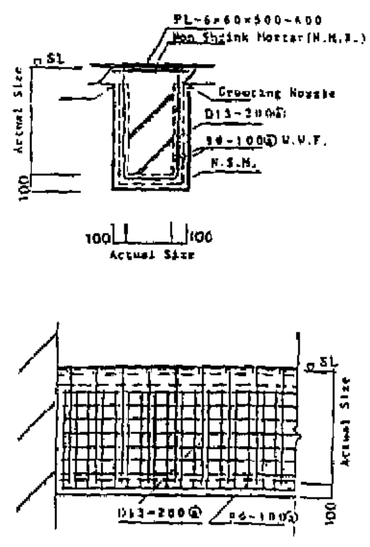


Fig. 7. 9 : DETAILS OF BEAM RETROFITTED