Building damage by the 2011 off the Pacific coast of Tohoku earthquake and coping activities by NILIM and BRI collaborated with the administration

by

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ABSTRACT

This paper presents the outlines of the strong motions observed mainly by the Building Research Institute (BRI) Strong Motion Network, the motion induced building damage and the tsunami induced building damage by the 2011 off the Pacific coast of Tohoku earthquake (the Great East Japan earthquake) (hereinafter referred to as the Tohoku earthquake). Coping activities in order to establish necessary technical standards by the National Institute for Land & Infrastructure Management (NILIM) and BRI collaborated with the administration are also outlined.

As for the strong motions, it presents observed motions at in and out of the buildings to show the earthquake input and the response of the buildings. It also presents observations of the super high-rise buildings and the seismically isolated buildings under long-period earthquake ground motions. Ministry of Land, Infrastructure Transport and Tourism (MLIT) and NILIM with the assistance of BRI released the Tentative New Proposal [1.1] for countermeasure against long-period earthquake ground motion on super high-rise buildings and seismically isolated buildings, whose general outline is also introduced.

As for the motion induced building damage, it presents the general outline of the field survey on wood houses, structural steel buildings, reinforced concrete buildings, seismically isolated buildings and so on with the classified typical damage patterns for each structural systems based on the Quick Report [1.2], which does not give complete survey because of the widely damaged area affected by the Tohoku earthquake.

As for the tsunami induced damage of buildings, the general outline of damage of wood houses, structural steel buildings and reinforced concrete buildings is presented with typical damage patterns observed also from the Quick Report. The database on the damaged buildings subject to tsunami which is compiled in the Quick Report is utilized to verify the tsunami load defined in the Guidelines [1.3] for tsunami evacuation building made by the Cabinet Office.

In consideration of the building damage and so on by the Tohoku earthquake, “tsunami”, “non-structural elements”, “long-period earthquake ground motion” and “liquefaction” are important technical issues to be countermeasured by the administration. On these issues, the general outline of the coping activities by NILIM and BRI collaborated with the administration utilizing the Grant-in-aid [1.4] for maintenance and promotion of building codes / standards is introduced.

KEYWORDS: Building Damage, Falling Down of Ceilings, Liquefaction, Long-Period Ground Motion, Tsunami, 2011 off the Pacific Coast of Tohoku Earthquake

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1. INTRODUCTION

The Tohoku earthquake of moment magnitude (Mw) 9.0 occurred at 14:46 JST on March 11, 2011 and generated gigantic tsunami in the Tohoku and Kanto Areas of the northeastern part of Japan. This was a thrust earthquake occurring at the boundary between the North American and Pacific plates. This earthquake is the greatest in Japanese recorded history and the fourth largest in the world since 1900 according to U.S. Geological Survey [1.5]. An earthquake of Mw 7.5 foreshock preceded the main shock on March 9 and many large aftershocks followed including three Mw 7-class aftershocks on the same day of the main shock.

As the epicentral distribution of the aftershocks of the Tohoku earthquake (hypocentral region) is widely located off the coast of the prefectures of Iwate, Miyagi, Fukushima and Ibaraki with approximately 450km in length in North-South direction and 150km in width in East-West direction. The distance from these prefectures to the fault plane is almost the same, thus the places with the seismic intensity of about 6 (6+ or 6-) according to the Japan Meteorological Agency (JMA) widely spread in these prefectures. The maximum JMA seismic intensity of 7 was recorded by the strong motion recording network (K-NET) [1.6] of the National Research Institute for Earth Science and Disaster Prevention (NIED) at Kurihara City (K-NET Tsukidate) shown by the purple color in Fig. 1.1, Miyagi Prefecture, where the instrumental seismic intensity was 6.6.

Field survey by NILIM and BRI was started from Kurihara City and was followed by the locations shown in Fig. 1.2. In this paper, the results of the field survey at these locations are reported. In the coastal area from Aomori Prefecture to Miyagi Prefecture, the tsunami induced building damage was mainly surveyed. The area facing to the Pacific Ocean in Fukushima Prefecture was excluded from the survey in the cause of the accident in Fukushima Daiichi Nuclear Power Station. At the catchment basin area of Tone River in the border between Ibaraki and Chiba Prefectures and Urayasu City on the Tokyo Bay, damage of residential land associated with liquefaction was surveyed.

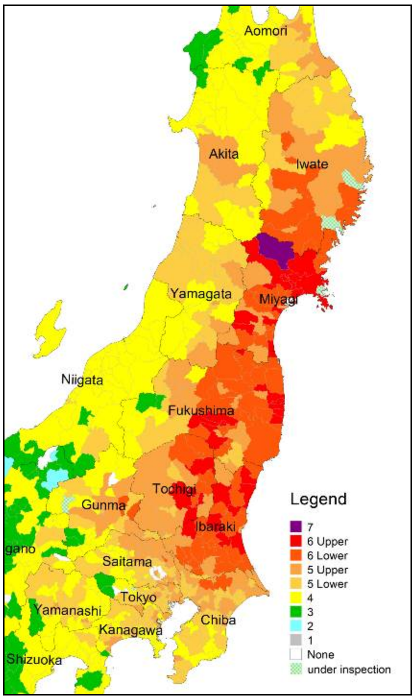


Fig. 1.1 JMA Seismic Intensity Map

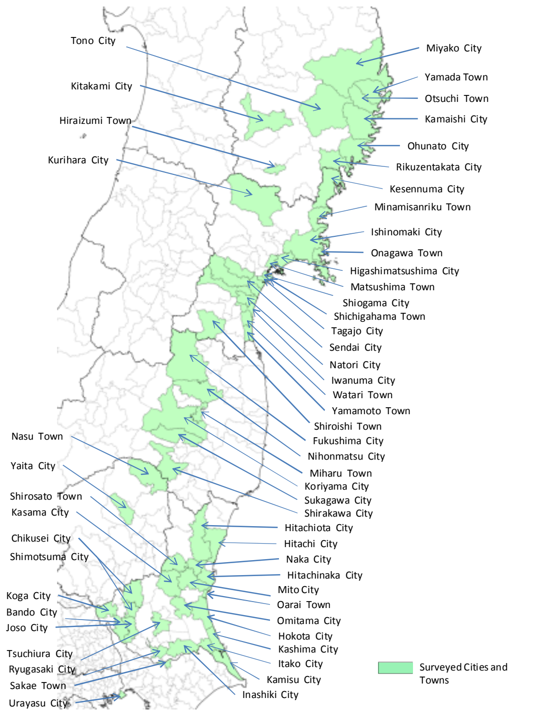
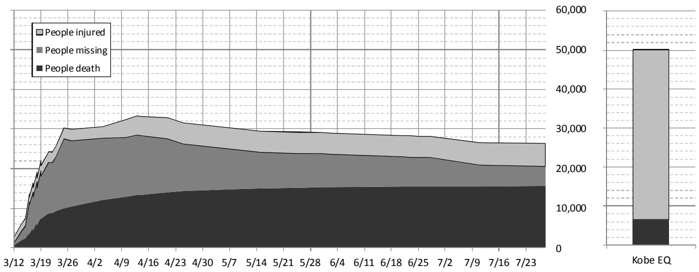


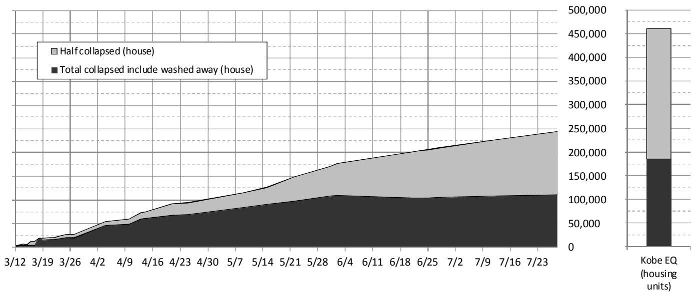
Fig. 1.2 Locations of Surveyed Cities and Towns

These surveys were made from the viewpoint of finding whether there is something to reflect to current building structural codes or not. Namely, are there new damage pattern in the motion induced building damage? In case for tsunami induced building damage, the technical information on the collapsed, overturned, washed away buildings are collected to verify the load effect by tsunami. As will be explained in the following chapters, the motion induced building damage is generally small and especially those buildings which satisfy current seismic code after 1981 performed quite well with almost no severe damage such as collapse reported.

Strong motions are recorded in good quality during many large scale aftershocks in addition to main shock and foreshock at in and out of the buildings by the BRI Strong Motion Network and so on. Especially, they are valuable to see the characteristic of long-period earthquake ground motion, which is thought to be strongly generated and amplified in the occasion of huge earthquake, and to see the corresponding building responses.



(a) People death, missing and injured



(b) Total collapse and half collapse of houses

Fig. 1.3 Progress of People Death (top) and Building Collapse (bottom) by the Tohoku Earthquake compared with those by Kobe Earthquake

The statistical data on the people death and the building collapse are announced from the National Policy Agency [1.7] (August 15, 2011). Fig. 1.3(a) shows the progress of people death which is compared to that in the 1995 Hyogoken Nambu earthquake (Kobe earthquake), which shows that the number of people death in the Tohoku earthquake is almost 2.5 times as that in the Kobe earthquake. Fig. 1.3(b) shows the progress of building collapse which is again compared to that in Kobe earthquake. The unit of the statistical data is different, “house” for the Tohoku earthquake and “housing units=household” for the Kobe earthquake, but it can be said that “total collapse” and “half collapse” are both almost half in the Tohoku earthquake. The Tohoku earthquake is much larger than the Kobe earthquake and the affected area is far larger as a result, and also the washed away houses are included in the total collapsed buildings. Thus, the motion induced building damage is far below in damage ratio than that in the Kobe earthquake. The Press Release [1.8] by the Urban Bureau of MLIT showed that the total collapsed houses in the tsunami inundation area reached about 120,000, which is almost the same number of total collapse by the National Policy Agency. This fact agrees well with the result by the field survey.

As the lessons from the Tohoku earthquake, the following administrative issues are listed up: load effect on building by tsunami, countermeasure for falling down of non-structural elements such as ceiling, countermeasure for long-period earthquake ground motion on super high-rise buildings etc., and countermeasure for liquefaction of residential land. NILIM and BRI are cooperatively working on coping activities collaborated with the administration, which is also introduced in this paper.

2. EARTHQUAKE AND GROUND MOTIONS

2.1 Earthquake Mechanism

The Tohoku earthquake occurred with wide seismic source extending from off Sanriku to the coast of Ibaraki Prefecture as shown in Fig. 2.1 [2.1]. The figure also shows the slip distribution on the fault place estimated using the strong motion records from the near recording stations shown with green triangles. The green star shows the epicenter, the rupture initiation location for the main shock.

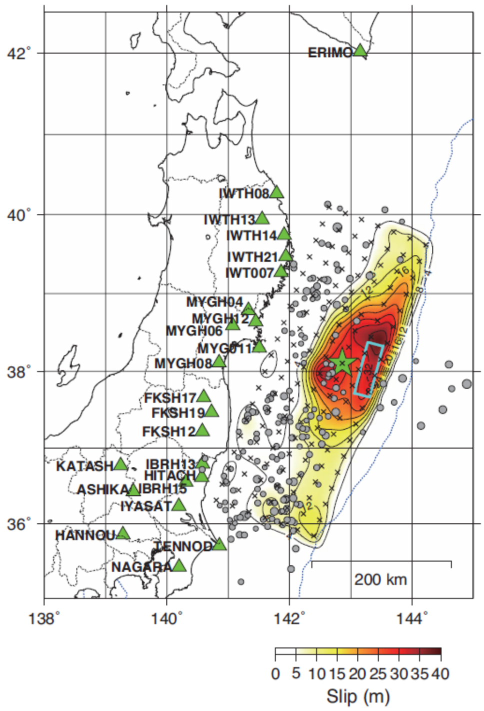


Fig.2.1 Slip Distribution on Earthquake Source analyzed using Near Site Strong Motion Records (Yoshida et al., 2011 [2.1])

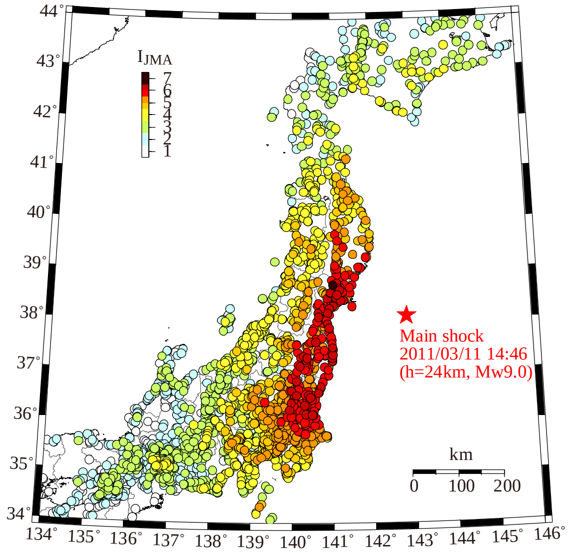


Fig.2.2 JMA Seismic Intensity Distribution for the Main Shock

It was also clarified that the Tohoku earthquake occurred on the subduction zone between the Pacific and the North American plates and was of thrust type. The Tohoku earthquake generated large tsunami and the devastation caused the unprecedented enormous loss of lives and substantial damage to the broader society in the eastern Japan.

The magnitude of the event was determined Mw 9.0 by JMA, the largest in the earthquake instrumentation history of Japan.

2.2 Distribution of Seismic Intensities

Fig. 2.2 shows the distribution of JMA seismic intensities observed during the main shock. An asterisk(★) represents the location of the epicenter. The seismic intensity 7 was observed in Kurihara City, Miyagi Prefecture, and JMA seismic intensity 6+ was observed in wide area of Miyagi, Fukushima, Ibaraki, and Tochigi Prefectures. The area with JMA seismic intensity 6- extends to Iwate, Gunma, Saitama, and Chiba Prefectures.

2.3 Characteristics of Earthquake Motions

During the Tohoku earthquake, severe ground motions were observed in wide area, and massive amounts of strong motion records were accumulated. This section describes the characteristics of strong motion records at recording stations that suffered high seismic intensities, based on K-NET of NIED [1.6]. Fig. 2.3 shows acceleration waveforms and pseudo velocity response spectra with the damping ratio of 5% of strong motion records at K-NET Tsukidate station that recorded JMA seismic intensity 7, and K-NET Sendai and K-NET Hitachi stations among seismic intensity 6+ stations.

Among strong motion recording stations, only K-NET Tsukidate, which is located in Kurihara City, Miyagi Prefecture, recorded the seismic intensity 7 during the main shock of the Tohoku earthquake. From the acceleration records in the upper-row in Fig. 2.3, a maximum acceleration in the N-S direction is understood to have reached almost 3700 cm/s^2, representing that the main shock caused excessively severe earthquake motions. As seen from the pseudo velocity responses on the right diagram, a response in the N-S direction with a period of about 0.2 seconds becomes particularly large. This indicates earthquake ground motions that are dominated by short periods.

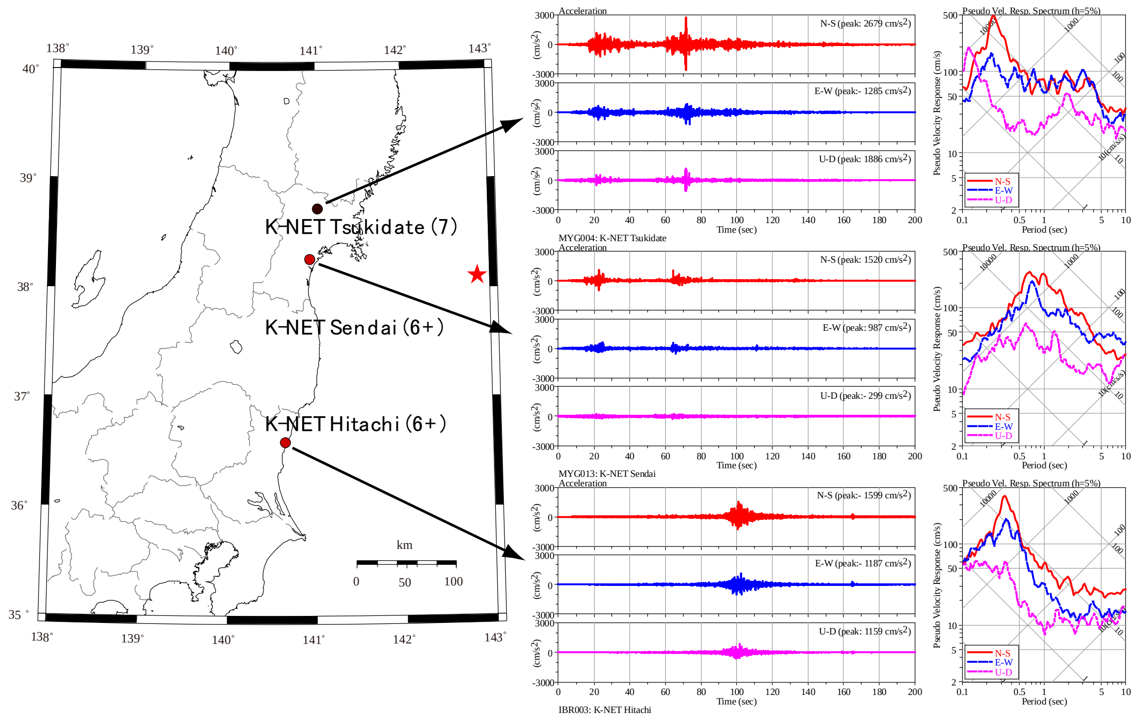


Fig.2.3 Acceleration Waveforms and Pseudo Velocity Response Spectra recorded at K-NET Stations

K-NET Sendai, which is located about 4 km east from the Sendai Station, recorded Intensity 6+ during the main shock. A maximum acceleration in strong motion records (mid-row in Fig. 2.3) obtained from the network exceeds 1500 cm/s^2 in the N-S direction, indicating a higher level of the main shock. In contrast to K-NET Tsukidate, earthquake motions that were recorded in K-NET Sendai are dominated by a period range of 0.5 to about 1 second, and a maximum response velocity exceeds 200 cm/s. This result is considered to reflect ground conditions in the area of K-NET Sendai that is covered with a thick alluvium.

The lower-row in Fig. 2.3 shows strong motion records that were obtained from K-NET Hitachi in Hitachi City, Ibaraki Prefecture. A seismic intensity of the main shock that was measured in this network represented seismic intensity 6+. A maximum acceleration in the N-S direction reached a higher level, or about 1600 cm/s^2, while the pseudo velocity response spectra had a peak at about 0.3 seconds. On the other hand, the response was sharply reduced at a period longer than 0.5 seconds. This indicates that the earthquake motions were dominated in short period range. Both records of K-NET Tsukidate and K-NET Sendai show two wave groups at about 20 seconds and 70 seconds on the time axis, but the strong motion record obtained at much southern station such as K-NET Hitachi, Ibaraki Prefecture in Kanto Area shows one large wave group. This phenomenon may have occurred associated with the focal rupture process and the wave propagation to recording stations.

2.4 Results of BRI Strong Motion Network

The BRI conducts strong motion observation that covers buildings in major cities across Japan [2.2]. When the Tohoku earthquake occurred, 58 strong motion instruments placed in Hokkaido to Kansai Areas started up. Peak accelerations of the strong motion records are listed in Table 2.1. Locations of the strong motion stations are plotted in Fig. 2.4 and Fig. 2.5. Among them, about 30 buildings suffered a shaking with seismic intensity 5- or more. This section presents some characteristic strong motion records.

Table 2.1 Strong motion records obtained by BRI strong motion network (1/4)

Code	Station name	Δ (km)	I_{JMA}	Azi-muth	Loc.	Max. Acc. (cm/s^2) H1	H2	V
SND	Sendai Government Office Bldg. #2	175	5.2	074°	B2F*	163	259	147
					15F	361	346	543
THU	Tohoku University	177	5.6	192°	01F*	333	330	257
					09F	908	728	640
MYK	Miyako City Hall	188	4.8	167°	01F	138	122	277
					07F	246	197	359
					GL*	174	174	240
IWK	Iwaki City Hall	210	5.3	180°	B1F*	175	176	147
					09F	579	449	260
TRO	Tsuruoka Government Office Bldg.	275	3.9	182°	01F*	34	36	14
					04F	37	39	15
HCN2	Annex, Hachinohe City Hall	292	5.2	164°	GL*	286	210	61
					G30	86	89	49
					G105	36	46	32
					10F	120	123	206
					01F	91	122	73
					B1F	100	104	58
HCN	Main bldg., Hachinohe City Hall	292	4.6	164°	B1F*	97	110	55
					06F	348	335	78
AKT	Akita Prefectural Office	299	4.3	087°	08F	175	192	44
					B1F*	50	47	24
ANX	Building Research Institute	330	5.3	180°	A01*	279	227	248
					A89	142	153	102
					BFE	194	191	136
					8FE	597	506	344
					MBC	203	206	152
					M8C	682	585	311
BRI	Training Lab., BRI	330	5.4	180°	01F*	281	273	165
TKC	Tsukuba City Hall (Base-isolation)	334	5.2	004°	B1F*	327	233	122
					01F	92	76	198
					06F	126	91	243
NIG	Niigata City Hall	335	3.9	061°	B1F*	28	40	14
					07F	39	55	14
HRH	Hirosaki Legal Affairs Office	346	3.4	195°	01F*	28	25	15
TUS	Noda Campus, Tokyo Univ. Of Science	357	5.1	000°	01F*	269	263	151
YCY	Yachiyo City Hall	361	5.3	302°	B1F	140	135	92
					GL*	312	306	171
					07F	486	359	145
NIT	Nippon Institute of Technology	362	5.1	288°	GL*	230	197	79
					01F	150	119	63
					06F	283	322	131
MST	Misato City Hall	367	4.9	258°	01F	72	104	71
					GL*	130	127	73
					07F	219	190	106

Note) Δ: epicentral distance, I_{JMA} : JMA instrumental seismic intensity (using an asterisked sensor), Azimuth: clockwise direction from North, H1, H2, V : maximum accelerations in horizontal #1 (Azimuth), horizontal #2 (Azimuth+90°) and vertical directions

Table 2.1 Strong motion records obtained by BRI strong motion network (2/4)

Code	Station name	Δ (km)	I_{JMA}	Azimuth	Loc.	Max. Acc. (cm/s^2) H1	H2	V
FNB	Educational Center, Funabashi City	368	4.7	357°	01F	144	147	63
					GL*	133	145	105
					08F	359	339	141
CHB	Chiba Government Office Bldg. #2	369	4.9	346°	B1F	152	122	51
					08F	375	283	117
					GL*	168	175	100
ICK	Gyotoku Library, Ichikawa City	375	5.2	321°	01F*	164	163	71
					02F	178	186	80
					05F	240	300	104
EDG	Edogawa Ward Office	377	4.8	003°	01F*	112	112	69
					05F	256	299	77
ADC	Adachi Government Office Bldg.	377	4.8	012°	01F*	118	103	71
					04F	266	146	95
SIT2	Saitama Shintoshin Government Office Building #2	378	4.4	340°	B3F*	74	63	42
					10FS	119	138	62
					27FS	248	503	107
SITA	Arena, Saitama Shintoshin Government Office Building	378	4.5	313°	01F*	90	105	47
TDS	Toda City Hall	380	5.0	354°	GL*	203	206	53
					B1F	140	173	65
					08F	425	531	160
AKB	Akabane Hall, Kita Ward	380	4.6	354°	B1F*	85	139	59
					06F	180	250	86
SMD	Sumida Ward Office	380	4.3	000°	20F	385	290	81
					08F	263	197	46
					B1F*	69	66	34
NMW	National Museum of Western Art (Base-isolation)	382	4.8	218°	GL*	265	194	150
					B1FW	100	79	84
					01FW	76	89	87
					04F	100	77	90
UTK	Bldg. #11, University of Tokyo	383	4.7	348°	7FN	181	212	58
					7FS	201	360	160
					01F	73	151	49
					GL*	197	218	79
TKD	Kosha Tower Tsukuda	385	4.4	180°	01F*	87	98	41
					18F	118	141	64
					37F	162	198	108
CGC	Central Government Office Bldg. #6	386	4.4	208°	01F*	90	86	45
					20B	208	148	173
					19C	179	133	130
CG2	Central Government Office Bldg. #2	386	4.2	208°	B4F*	75	71	49
					13F	137	113	72
					21F	121	131	104
CG3	Central Government Office Bldg. #3 (Base-isolation)	386	4.5	208°	B2F*	104	91	58
					B1F	55	41	62
					12F	94	82	104

Note) Δ: epicentral distance, I_{JMA} : JMA instrumental seismic intensity (using an asterisked sensor), Azimuth: clockwise direction from North, H1, H2, V : maximum accelerations in horizontal #1 (Azimuth), horizontal #2 (Azimuth+90°) and vertical directions

Table 2.1 Strong motion records obtained by BRI strong motion network (3/4)

Code	Station name	Δ (km)	I_{JMA}	Azi-muth	Loc.	Max. Acc. (cm/s²) H1	H2	V
NDLA	Annex, National Diet Library	387	4.5	354°	B8F	61	88	53
					B4F	68	101	56
					01F*	76	104	84
					04F	125	192	94
NDLG	Ground, National Diet Library	387	5.0	354°	G35	72	71	51
					G24	95	116	54
					GL*	224	201	93
NDLM	Main Bldg., National Diet Library	387	4.5	354°	01S*	70	94	60
					17S	458	489	111
NKN	Nakano Branch, Tokyo Legal Affairs Bureau	390	4.8	359°	06F	172	375	56
					01F*	126	158	54
TUF	Tokyo University of Marine Science and Technology	390	5.0	000°	01F	174	169	60
					GL*	181	189	71
					07F	316	223	66
KDI	College of Land, Infrastructure and Transport	401	4.6	090°	03F	129	329	55
					01F	110	136	53
					GL*	167	143	50
KWS	Kawasaki-minami Office, Labour Standards Bureau	401	4.7	045°	01F*	107	77	30
					02F	133	123	49
					07F	366	304	76
NGN	Nagano Prefectural Office	444	2.7	157°	B1F*	8	7	8
					11F	35	27	9
HKD	Hakodate Development and Construction Department	447	3.5	180°	GL*	25	28	13
HRO	Hiroo Town Office	466	2.7	140°	01F*	17	20	8
YMN	Yamanashi Prefectural Office (Base-isolation)	468	3.9	006°	B1F	47	39	18
					GL*	51	44	20
					01F	37	52	20
					08F	41	51	25
SMS	Shimoda Office, Shizuoka Prefecture	517	2.9	225°	GL*	12	19	10
SMZ	Shimizu Government Office Bldg.	520	4.2	165°	01F*	28	40	15
					11F	81	56	18
KSO	Kiso Office, Nagano Prefecture	524	2.6	292°	B1F*	9	10	8
					6F	32	31	10
KGC	Kushiro Government Office Bldg. (Base-isolation)	558	2.6	167°	GL*	12	14	6
					G10	10	10	4
					G34	5	5	3
					B1F	8	12	4
					01F	10	16	6
					09F	16	19	12
HKU	Hokkaido University	567	2.7	172°	GL*	10	9	5
NGY	Nagoya Government Office Bldg. #1	623	3.1#	174°	GL*	8	15	-
					B2F	9	14	7
					12F	25	46	7
MTS	Matsusaka Office, Mie Prefecture	688	2.3	216°	07F	16	8	4
					01F*	6	5	3

Note) Δ: epicentral distance, I_{JMA} : JMA instrumental seismic intensity (using an asterisked sensor), Azimuth: clockwise direction from North, H1, H2, V : maximum accelerations in horizontal #1 (Azimuth), horizontal #2 (Azimuth+90°) and vertical directions, # : calculated from two horizontal accelerations because of trouble on the vertical sensor

Table 2.1 Strong motion records obtained by BRI strong motion network (4/4)

Code	Station name	Δ (km)	I_{JMA}	Azi-muth	Loc.	Max. Acc. (cm/s^2)		
						H1	H2	V
MIZ	Maizuru City Hall	726	0.9	085°	01F	1	2	2
					05F*	1	1	2
OSK	Osaka Government Office Bldg. #3	759	2.9	189°	18F	65	38	7
					B3F*	11	9	5
SKS	Sakishima Office, Osaka Prefecture	770	3.0	229°	01F*	35	33	80
					18F	41	38	61
					38F	85	57	18
					52FN	127	88	13
					52FS	129	85	12

Note) Δ: epicentral distance, I_{JMA} : JMA instrumental seismic intensity (using an asterisked sensor), Azimuth: clockwise direction from North, H1, H2, V : maximum accelerations in horizontal #1 (Azimuth), horizontal #2 (Azimuth+90°) and vertical directions

2.4.1 Strong motion records of damaged buildings

Among buildings in the BRI Strong Motion Network, at least 4 buildings suffered severe earthquake motions and then some damage. One example of the damaged buildings is the building of the Human Environmental Course, Tohoku University. This is the 9-story steel reinforced concrete (SRC) school building that is located in the Aobayama Campus. This building has a long history of strong motion recordings. Among them, strong motion records on the ninth floor of the building that were obtained during the 1978 Miyagi-Ken-Oki earthquake are well known to have represented a maximum acceleration of more than 1000 cm/s^2.

During the Tohoku earthquake, multi-story shear walls suffered flexural failure and other damage. The appearance of the building is shown in Photo 2.1, and strong motion records obtained during the main shock in Fig. 2.6. This figure shows (a) acceleration waveforms in the transverse direction, (b) acceleration waveforms in the longitudinal direction, (c) building displacement in the transverse direction (relative displacement to first floor of the 9-story building), (d) building displacement in the longitudinal direction, and (e) fundamental natural periods of the building that were calculated every 10 seconds [2.3].

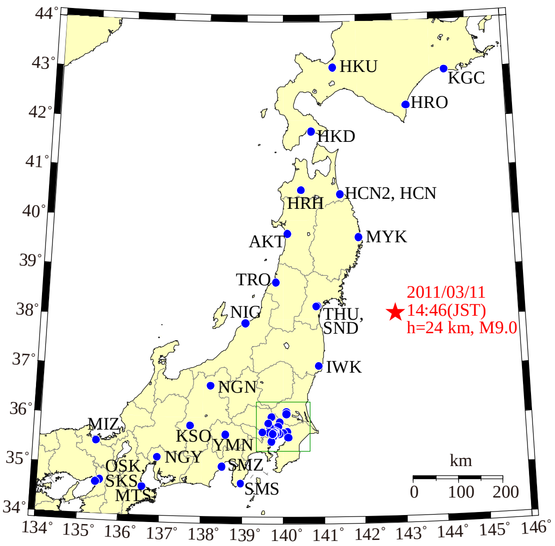


Fig.2.4 Locations of Epicenter (★) and Strong Motion Network (●)

Thick and thin lines in Fig. 2.6 (a) and (b) represent acceleration waveforms on the first and the ninth floors, respectively. Maximum accelerations on the first floor exceeded 330 cm/s^2 in both of the directions. A maximum acceleration on the ninth floor was 2 to 3 times larger than on the first floor, and exceeded 900 cm/s^2 in the transverse direction. The fundamental natural periods in Fig. 2.6 (e) represented about 0.7 seconds at the initial stage of the earthquake motion in both of the directions, but increased to about 1 second in the first wave group at the time of 40 to 50 seconds, and increased from 1.2 seconds to about 1.5 seconds in the second wave group at the time of 80 to 100 seconds. Due to the seismic damage, the fundamental natural period finally became twice longer than that at the initial stage, and was reduced to 1/4 on a stiffness basis.

An additional comparison was made in Fig.2.7. The 1978 record at the 1st floor was compared with the 2011 record in velocity. The difference is clear that the earthquake size was different and the duration time of the 2011 Tohoku earthquake was much longer.

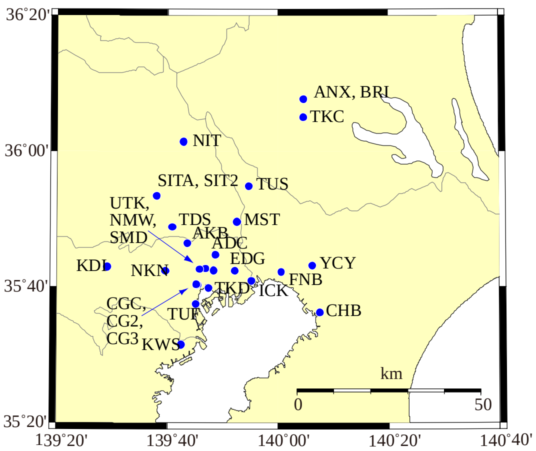


Fig. 2.5 Strong Motion Network in Kanto Area (corresponds to green rectangle in Fig. 2.4)

Photo 2.1 Appearance of the Building of the Human Environmental Course, Tohoku University

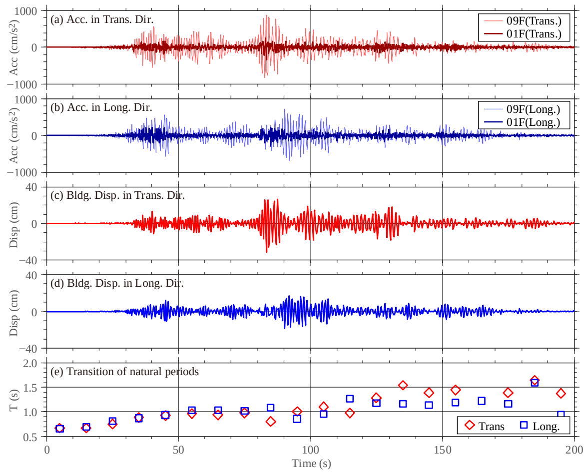


Fig. 2.6 Strong Motion Records of the Human Environmental Course, Tohoku University and Transition of Natural Periods with Time

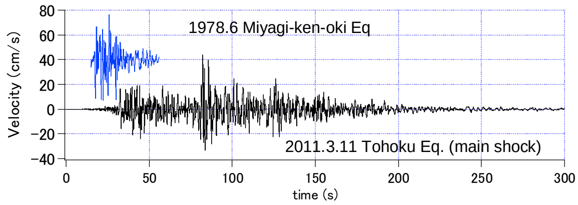


Fig.2.7 Comparison of the Recorded Motions at the 1st floor of Tohoku University Building between 1978 Miyagi-Ken-Oki and 2011 Tohoku earthquakes

2.4.2 Long-period earthquake motions in Tokyo and Osaka Bays

Long-period earthquake motions and responses of super high-rise buildings that are shaken under the motions have been socially concerned in recent years. When the Tohoku earthquake occurred, long-period earthquake motions were observed in Tokyo, Osaka and other large cities that are away from its hypocenter. This section presents two cases in Tokyo and Osaka from the BRI strong motion network.

First, a 37-story reinforced concrete (RC) super high-rise building (TKD in Table 2.1) on the coast of Tokyo Bay is introduced. Fig. 2.8 shows time histories of displacement (in two horizontal directions of S-N and W-E) that were calculated from the integration of acceleration records on the 1st and 37th floors in this building, and building displacements that were calculated by subtracting the displacements on the 1st floor from those on the 37th in the two horizontal directions. A maximum value of ground motion displacement was about 20 cm. It is understood that the ground in itself was greatly shaken. A displacement of the building in itself that was caused by its deformation reached 15 to 17 cm.

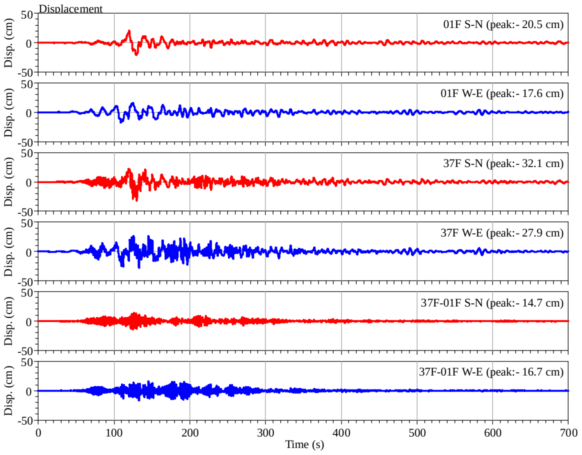


Fig. 2.8 Displacement Waveforms observed at a 37-story Residential Building in Tokyo Bay Area

Fig. 2.9 shows strong motion records that were obtained from the 55-story steel office building on the coast of Osaka Bay that is 770 km away from the hypocenter. This figure shows absolute displacements in the SW-NE and in the NW-SE directions on the 1st floor, absolute displacements in both of the directions on the 52nd floor, and building displacements (relative displacements of 52th floor to 1st floor) in both of the directions, from the top to the bottom. A ground motion displacement was not large, or less than 10 cm, but the 52nd floor in the building suffered a large motion with a zero-to-peak amplitude of more than 130 cm.

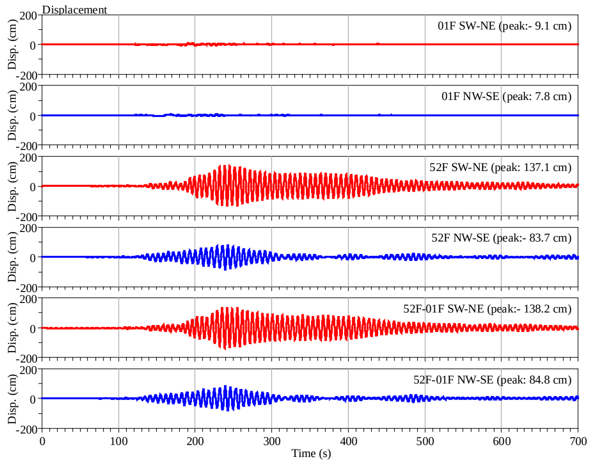


Fig. 2.9 Displacement Waveforms observed at a 55-story Office Building in Osaka Bay Area

In order to examine the properties of earthquake motions on both of the coasts of Tokyo Bay and Osaka Bay, pseudo velocity response spectra with a damping ratio of 5% of strong motion records that were obtained from the 1st floors in the buildings at the two locations are shown in Fig. 2.10. The response spectrum (left) in the records on the coast of Tokyo Bay had a peak at a period of 1 to 1.2 seconds, at 3 seconds and at 7 seconds, but a relatively flat shape in general.

On the other hand, the response spectrum (right) in the records on the coast of Osaka Bay had a large peak at 7 seconds, and amplitude of the response was not much different from on the coast of Osaka Bay. The coincidence of the fundamental natural period (6.5 to 7 seconds) in the steel office building with a predominant period of the earthquake motion is considered to have caused a resonance phenomenon and then large earthquake responses were observed at the top.

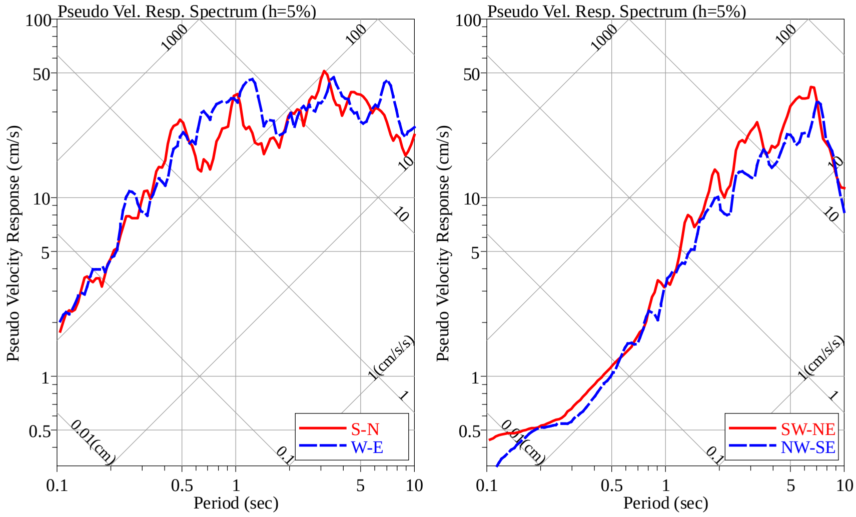


Fig. 2.10 Pseudo Velocity Response Spectra with Damping Ratio of 5% of Records in Tokyo Bay Area (left) and Osaka Bay Area (right)

3. DAMAGE OF BUILDINGS DUE TO EARTHQUAKE MOTIONS

3.1 Policy for Earthquake Damage Investigation for Buildings

The Tohoku Earthquake brought about building damage in a wide area of various prefectures on the Pacific coast in eastern Japan such as Iwate, Miyagi, Fukushima, Ibaraki and Chiba.

An epicentral region of this earthquake has a length of about 450 km and a width of about 150 km, almost in parallel with the Pacific coast in eastern Japan. A distance from the fault plane to the above prefectures is almost same. Observed earthquake motions in Sendai City close to the epicenter are not much different from those in a city far away from Sendai, for instance, Tsukuba City.

Based on these circumstances, we decided to widely survey damaged wood houses as a primary damage investigation in the northern part of Miyagi Prefecture (Kurihara City) where JMA Seismic Intensity 7 was observed, and in a wide area of Miyagi to Ibaraki Prefectures including inland Tochigi Prefecture that suffered larger damage than coastal prefectures. In addition, as a secondary investigation, we planned to select affected areas from those subject to the primary investigation to conduct a more detailed survey on houses collecting house plans and layout of wood-shear-walls.

In order to conduct a damage investigation of steel buildings, it was decided that mainly a primary appearance investigation would be done in Sendai City with a large stock of steel buildings, and in Fukushima and Ibaraki Prefectures. As mentioned later, significant damage of structural elements was visually limited, while there were so many types of damage of nonstructural elements such as falling of exterior cladding. Consequently, focusing not on private buildings that are difficult to investigate but on school gymnasiums in Ibaraki Prefecture where many damage cases were reported that enable interior investigations, we decided to continue the primary investigation. For reference, the school gymnasiums can be seen to be similar to factories and warehouses. If the structural damage in interior building is clarified in future, more detailed investigation as a secondary investigation on buildings other than the gymnasiums will be considered.

For a damage investigation for reinforced concrete buildings, in addition to an investigation of reportedly collapsed buildings, a primary investigation will be conducted on city halls and other public buildings that located in a wide area of the north to the south as done in the damage investigation for wood houses, and damage patterns whether they are similar or different from previously grasped patterns are examined. If there are characteristic damage patterns that should be incorporated into technical standards, the secondary investigation will be considered.

A primary investigation for damage of building lands and foundations will be conducted in Itako City, Ibaraki Prefecture, and in Urayasu City, Chiba Prefecture and its peripheral areas that was subject to severe liquefaction in the region of Kanto Area. The areas that had been affected by the 1978 Miyagi-Ken-Oki earthquake were damaged again. In these areas, also a primary damage investigation that focuses on developed housing lands will be conducted in some areas of Miyagi, Fukushima and Tochigi Prefectures.

In order to survey the damage of nonstructural elements, a primary investigation will be performed, altogether with a damage investigation for steel and reinforced concrete buildings including a requested investigation of ceiling falls in the Ibaraki Airport Building as an administrative support.

3.2 Damage of Wood Houses

3.2.1 Introduction

The survey area and the reasons of the choice are as follows;

- Kurihara City in Miyagi Prefecture: The seismic intensity 7 was recorded.
- Osaki City in Miyagi Prefecture: As a result of damage survey by others [3.1], heavy damage was reported,
- Sukagawa City in Fukushima Prefecture: RC buildings were heavily damaged.
- Nasu Town and Yaita City in Tochigi Prefecture, and Hitachiota City and Naka City in Ibaraki Prefecture: As a result of damage survey by others [3.2], damage information has never been reported, at the time of our survey.
- Ishinomaki City in Miyagi Prefecture: Although it was almost included in the belt of seismic center, the area of the city was not flooded by the tsunami.
- Joso City and Ryugasaki City in Ibaraki Prefecture: We had damage information in the neighborhood of NILIM and BRI.

The positions of the surveyed cities and towns are shown in Fig. 3.1.

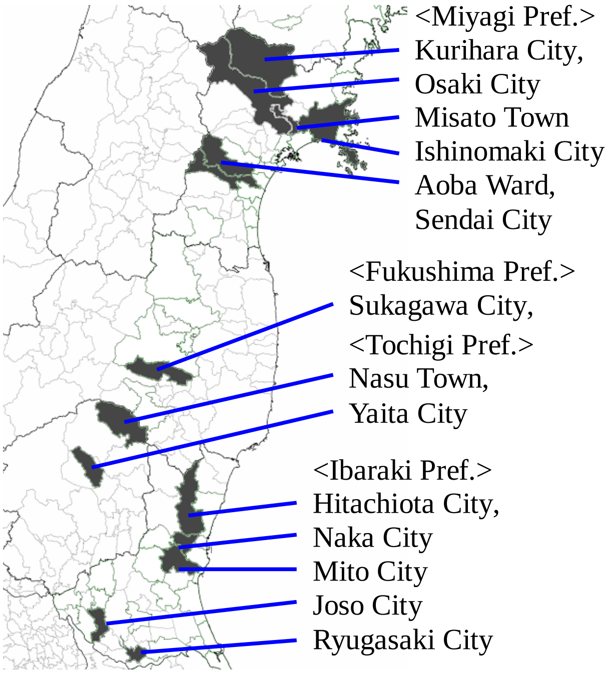


Fig. 3.1 Positions of Surveyed Cities and Towns

3.2.2 Results of the survey

(1) Kurihara City, Miyagi Prefecture

K-NET Tsukidate (MYG004 : Instrumental seismic intensity 6.6) is set up on the hillock about 3m higher than southern parking lot of Kurihara lyceum. There was a possibility of the amplification of earthquake motions. In Wakayanagi, the ground was bad and Sand eruption by liquefaction was observed. Damage of houses caused by ground transformation (Photo 3.1) and houses with store were also observed (Photo 3.2).

Great residual deformation was observed in the longitudinal direction in the large-scale wood building used to be a movie theater then renovated to a factory (Photo 3.3)

Photo 3.1 Damage of Houses caused by Ground Transformation

Photo 3.2 Damage of Houses with Store

Photo 3.3 Large-scale Wood Building renovated to a Factory

(2) Osaki City, Miyagi Prefecture

Every warehouse with the mud walls renovated as store or gallery (Photo 3.4) were damaged heavily or slightly. There was the one whose roof system with roof tiles collapsed and fell down, as shown in Photo 3.5. There was the house for combined residential and commercial use (=store combined house) (Photo 3.6) with story shear deformation whose exterior mortar came off and wood lath under the mortar near the opening of the ventilation fan were deteriorated and attacked by termites. Such damage was confirmed in the other buildings. Most of these damage occurred along the small river, except for a few case, and it was considered that the soft ground near the river might amplify the earthquake ground motion.

The rare damage example (Photo 3.7) that only the 2nd story collapsed was observed.

Photo 3.4 Warehouse with Mud Walls damaged Heavily or Slightly

Photo 3.5 Warehouse with Mud Walls whose Roof System with Roof Tiles fell down

Photo 3.6 Store Combined House with Story Deformation

Photo 3.7 School Building whose2nd Story Collapsed

(3) Sendai City, Miyagi Prefecture

From the results of investigations on Oritate and Seikaen where the damage of the houses were serious, it was found that almost all of the damage of houses were caused by ground transformation. Moreover, ground transformation caused retailing wall collapse (Photos 3.8, 3.9), landslide and damage of houses (Photo 3.10). In Komatsujima, Aoba ward, drop off the mortar wall and damage of columns and by bio-deterioration and termite were observed in the house with shop (Photo 3.11).

Photo 3.8 Damage of Retaining Wall and Houses (Oritate, Aoba Ward)

Photo 3.9 Damage of House

Photo 3.10 Damage of House caused by the Ground Transformation (Oritate)

Photo 3.11 Drop Off the Mortar Wall (Komatsujima)

(4) Sukagawa City, Fukushima Prefecture

A lot of damaged wood houses were found around the collapsed reinforced concrete building. For instance, they were the fallen mortar wall of the second floor of dwelling with shop (Photo 3.12), several decay and Japanese subterranean termite's damage on frame and wood sheathing of mortar wall (Photo 3.13).

The Dozo (Japanese traditional wood storehouse coated with clay and plaster finish) was greatly damaged around the hotel that the window glass was broken, and it had residual deformation (Photo 3.14). The damage of roof's collapsing of the Dozo (Photo 3.15) was also found.

The sand eruptions from liquefied ground and the damage of the roof tile were seen here and there in some places at Minami-machi, Sukagawa City.


Photo 3.12 Fallen Mortar of Wall (Wood House with Shop)


Photo 3.13 Bio-deterioration of Column and Based Mortar


Photo 3.14 Dozo with Residual Deformation


Photo 3.15 Fallen Roof Tile of Dozo

(5) Hitachiohta City, Ibaraki Prefecture

There were a lot of damaged fence made by the Ohyaishi stone. The collapsed farm type house was observed (Photos 3.16, 3.17).


Photo 3.16 Collapsed Wood House


Photo 3.17 Breakage of Entrance Part

(6) Naka City, Ibaraki Prefecture

There were a lot of collapsed (Photo 3.18) or heavily damaged barns. The damaged house with shop was observed (Photo 3.19). The two story wood house with mortar finish collapsed at the urban area (Photo 3.20).


Photo 3.18 Collapsed Barn


Photo 3.19 Damaged House with Shop


Photo 3.20 Collapsed House

3.2.3 Concluding remarks

As a result of damage survey on the wood houses due to ground motion in Kurihara City, Osaki City, Misato Town, Ishinomaki City, Sendai City in Miyagi Prefecture, Sukagawa City in Fukushima Prefecture, Nasu Town, Yaita City in Tochigi Prefecture, and Hitachiota City, Naka City, Mito City, Joso City, Ryugasaki City in Ibaraki Prefecture, the followings were provided.

1) The damage on the many wood houses due to ground motion was confirmed in Osaki City in Miyagi Prefecture, Sukagawa City in Fukushima Prefecture, Nasu Town in Tochigi Prefecture, and Hitachiota City and Naka City in Ibaraki Prefecture.
2) Though the seismic intensity 7 was recorded in Kurihara City, Miyagi Prefecture, it was felt that the damage on wood houses was not so much.
3) The damage on the wood houses caused by the failures of residential land was confirmed in Sendai City, Miyagi Prefecture, and Yaita City, Tochigi Prefecture. There were many number of the damage as such, too.
4) The damage of the roof tile in Fukushima and Ibaraki Prefectures was felt much larger than Miyagi Prefecture where an earthquake occurred frequently.
5) The possibility that the ground motion was amplified on the land filled up from meadow or rice field, even if the residential land did not fail, was suggested in Kurihara City, Osaki City in Miyagi Prefecture, Nasu Town in Tochigi Prefecture, Hitachiota City, Naka City, Joso City, Ryugasaki City in Ibaraki Prefecture, and so on.
6) In Osaki City, Miyagi Prefecture, the plural rare damage examples that residual story deformation of 2nd floor was larger than that of 1st floor were confirmed.

The selected individual buildings will be surveyed in detail and each of the damage causes will be discussed in future, based on the results of the damage summary of the above-mentioned wood houses.

3.3 Damage of Reinforced Concrete Buildings

3.3.1 Introduction

The Tohoku earthquake widely caused a lot of damage to buildings in Tohoku and Kanto Areas of Japan. NILIM and BRI investigated the damage of reinforced concrete (RC) buildings and reinforced concrete buildings with embedded steel frames (referred to as steel reinforced concrete, or SRC) in the affected areas where the earthquake intensity was classified as JMA seismic intensities 6+ and 6- in Iwate, Miyagi, Fukushima and Ibaraki Prefectures. The objective of the field investigation was to see the picture of the outline of the overall damage on the buildings and to classify the damage pattern of them. The survey was conducted several times from March 14 to the middle of May in the areas as shown in Fig. 3.2. The outline of the investigation is described below.

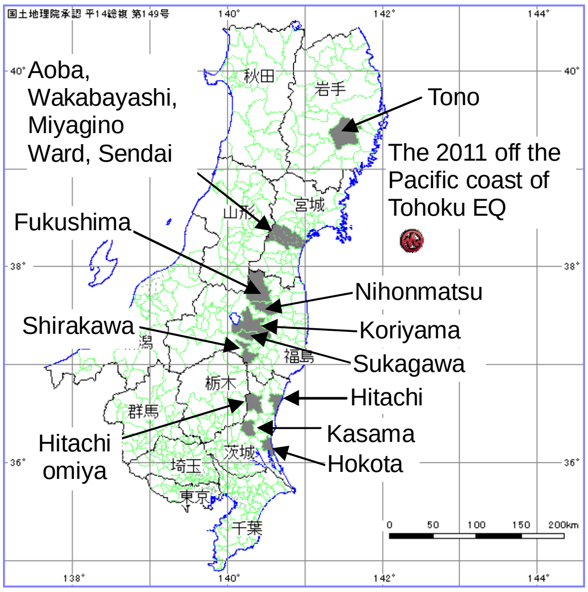


Fig. 3.2 Investigated Area (add a postscript on KenMap」)

3.3.2 Characteristics of damage on RC buildings

In the earthquake, strong earthquake motions were observed in various locations of the Tohoku and the Kanto Areas and caused various types of damage in a wide area. At the same time, the damage concentrated on specific areas was not seen generally. In general, we had the impression that structural damage was not particularly great in comparison with JMA seismic intensities measured in the locations. Consequently, there is not a significant difference in damage situations among the locations. However, the damage to structural members was somewhat concentrated on limited areas, such as Wakabayashi Ward in Sendai City and Sukagawa City. It is known that these areas formed paddy fields or moats. Therefore, it can be well estimated that ground conditions in the areas possibly contributed to the damage.

The types of structural damage on RC buildings identified by the field surveys are those that had been observed in past earthquake damage investigations. Some serious types of damage were observed, such as story collapse of low-rise buildings, collapse of soft-first story (pilotis), and the loss of vertical load carrying capacity of columns due to shear failure. Most of severely damaged buildings were designed with the previous seismic design code that was governed before June 1981. Some SRC buildings designed under the current seismic design code, which has been coming into force after June 1981, caused damage of buckling of their longitudinal reinforcements near base plates at the bottom of column. The same damage is known to have occurred also in the Kobe earthquake. In addition, buildings designed under the current seismic design code were confirmed to have no collapse but some damage like shear cracks at their beam-column joints or horizontal cracks at their concrete placing joints.

The types of the damage of RC and SRC buildings that were observed through the site investigation are classified into those for structural and nonstructural elements in the following.

A) Damage of structural elements
- A-1) Collapse of first story
- A-2) Mid-story collapse
- A-3) Shear failure of columns
- A-4) Flexural failure at the bottom of column and base of boundary columns on multi-story shear walls
- A-5) Pullout of anchor bolts and buckling of longitudinal reinforcements at exposed column base of steel reinforced concrete (SRC) buildings
- A-6) Shear failure or bond splitting failure of link beams of multi-story coupled shear walls
- A-7) Building tilting
- A-8) Destruction, failure or tilting of penthouses
- A-9) Damage of seismic retrofitted buildings

B) Damage of nonstructural elements
- B-1) Flexural failure at the bottom of column with wing wall
- B-2) Damage of nonstructural wall in residential building

B-3) Damage and falling of external finishing
B-4) Tilting or dropout of components projecting above the roof
B-5) Collapse of concrete block wall and stone masonry wall

3.3.3 Damage of structural elements

A-1) Collapse of first story

The 3-story RC building which was at the intersection in Sukagawa City, having a few walls on the facade on the first story and many walls on the back of the first story and the second story and higher, was severely damaged on the first story. The corner columns faced the intersection were significantly destroyed. The loss of axial load carrying capacity of the first-story columns caused the drop of the second and higher stories (Photos 3.21 and 3.22).

Photo 3.21 First-story Collapsed Building

Photo 3.22 Close-up View of the Fallen Story

A-2) Mid-story collapse

The 3-story office building in Wakabayashi Ward, Sendai City partially collapsed on the second story and tilted (Photo 3.23). Only the second story has openings on the wall at the gable side, as shown on the left wall of Photo 3.23. For this reason, it was assumed that the openings were intensively deformed and resulted in shear failure of the short columns formed by hanging and spandrel walls. Shear failure of the long columns on the third story was observed possibly due to the effect of the collapse of the second story. Damage of the columns and beams on the first story was not seen, while shear cracks were observed on the nonstructural walls.

Photo 3.23 Mid-story Collapsed Building

A-3) Shear failure of columns

The shear failure occurred on the first-story columns in the two-story RC building in Aoba Ward, Sendai CIty (Photos 3.24 and 3.25). Some columns of the building were intact after the main shock on March 11, but aftershocks caused shear failure to some of them, as shown on the right of Photo 3.25. It was confirmed that the aftershocks accelerated the damage level of this building.

Photo 3.24 Appearance of Damaged Building

Photo 3.25 Shear Cracks on First-story Columns

The 3-story RC building constructed in 1964 on a hill in Kasama City suffered also damage. Cracks in the ground were observed around the building. As seen in the photo, the RC structure on the first story was greatly damaged. Shear failure occurred on many exterior columns, which were made shorter in clear height by the hanging and spandrel walls without structural slit, as shown in Photo 3.26. In addition, the failures of the shear walls with openings were observed (Photo 3.27)


Photo 3.26 Shear Failure of Column


Photo 3.27 Shear Failure of Wall with Opening

A-4) Flexural failure at the bottom of column and base of boundary columns on multi-story shear walls

The building that consists of 9-story SRC and 2-story RC structures in Aoba Ward, Sendai City suffered from the earthquake (Photo 3.28). In the high-rise building, the multi-story shear wall on the gable side was subject to flexural failure at the third floor. Crushing of concrete and buckling of the longitudinal reinforcements were observed at the bottom of the boundary column of shear wall, as shown in Photo 3.29. This building was also damaged by the Miyagi-Ken-Oki earthquake in 1978 and had been retrofitted.


Photo 3.28 Appearance of Damaged Building


Photo 3.29 Crushing at the Bottom of Column on the Multi-story Shear Wall

A-5) Pullout of anchor bolts and buckling of longitudinal reinforcements at exposed column base of steel reinforced concrete (SRC) buildings

The damage at the bottom of SRC column and shear wall was observed too on the building in Shirakawa City (Photo 3.30), which was composed of RC and SRC structures. Pullout of anchor bolts of the exposed-type column base was detected. In consequence, the reinforcing bars were forced to stretch large and the buckling of them occurred around the base plate, as shown in Photo 3.31.

This type of damage was observed not only in buildings designed under the previous seismic design code but also in some buildings done under the current seismic design code.

Photo 3.30 Damage on the Bottom of SRC Column and Shear Wall

Photo 3.31 Close-up View of the Bottom of SRC Column

A-6) Shear failure or bond splitting failure of link beam of multi-story coupled shear walls

The shear failure or bond splitting failure occurred on the link beam connecting coupled shear walls from low-rise to high-rise stories on the 8-story RC building in Aoba Ward, Sendai City, as shown in Photo 3.32. The link beams has two openings at the center of them, and were damaged around these parts (Photo 3.33).

Photo 3.32 Appearance of Damaged Building

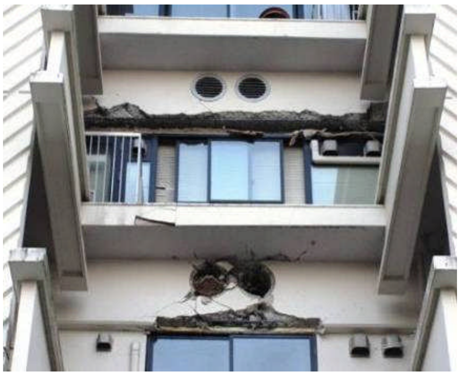


Photo 3.33 Damage on Boundary Beam with Opening

A-7) Building tilting

Photo 3.34 shows a residential building that sank and leaned in the longitudinal direction in Shirakawa City. The balcony, of which height above ground level was about 77cm, went down to ground surface in the gable side, as shown in Photo 3.35. Significant settlement was observed too on foot walk in surrounding area.

Photo 3.34 Appearance of Sank and Leaned Building

Photo 3.35 Sank Balcony

A-8) Destruction, failure or tilting of penthouses

The damage on penthouses was observed everywhere, like tilting of it in Aoba Ward, Sendai City, as shown in Photo 3.36.

Photo 36 Damaged Penthouse

A-9) Damage of seismic retrofitted buildings

Photo 3.37 shows the 2-story RC office building constructed in 1969 in Hitachiomiya City. The building was retrofitted with framed steel braces in the longitudinal direction in 2003, because the seismic index of structure, I_S on the first story of the building was below the seismic demand index of structure, I_{S0} by the seismic evaluation method [3.3]. Meanwhile, the building in the span direction was not retrofitted, as a consequence of that the seismic index of structure in the direction satisfied the seismic demand index of structure. The steel braces are eccentrically installed to the center axis of the beams and columns.

Shear cracks occurred on the columns with the framed steel braces at the earthquake, as shown in Photo 3.38, although the remarkable damage such as yield of steel were not seen on the braces.

Photo 3.37 Appearance of Damaged Building

Photo 3.38 Shear Crack on Column with Framed Steel Braces

There were many seismic retrofitted buildings including school buildings in the affected areas where the great earthquake motions were observed. Based on the results of our investigation, these retrofitted buildings were hardly damaged or slightly harmed, it means that the seismic strengthening on existing buildings worked effectively against the earthquake.

3.3.4 Damage of nonstructural elements

B-1) Flexural failure at the bottom of column with wing wall

The separation of cover concrete at the bottom of wing wall was observed on the 5-story RC building constructed in 2007 in Sukagawa City (Photo 3.39). In here, we classified this as the damage of nonstructural elements, because the wing wall is generally designed as the nonstructural element, which is not expected to resist the external force.

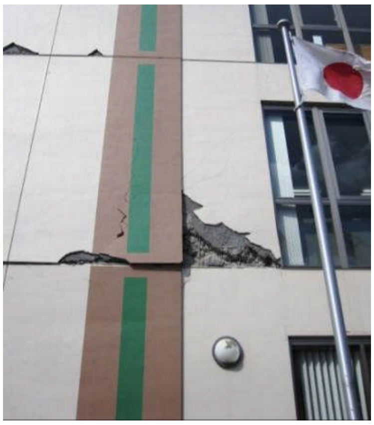


Photo 3.39 Separation of Concrete of Wing Wall

B-2) Damage of nonstructural wall in residential building

The nonstructural walls around the front doors from low-rise to top floors were subject to shear failure, while the doors were deformed on the 10-story SRC residential building constructed in Aoba Ward, Sendai City in 1996, as shown in Photos 3.40 and 3.41. In addition, shear cracks were observed on the mullion walls on balconies in some of the low-rise floors.

The shear cracks on the mullion walls also occurred in the 8-story RC hotel in Sukagawa City (Photo 3.42).

The cases where shear cracks occurred on the nonstructural walls around the front door or on the mullion wall on the balcony were relatively often observed in urban residential buildings, regardless of the seismic design codes applied to.

Photo 3.40 Appearance of Damaged Building

Photo 3.41 Shear Failure of Nonstructural Wall

Photo 3.42 Shear Crack on Nonstructural Wall

B-3) Damage and falling of external finishing

Photos 3.43 show the case where the autoclaved lightweight aerated concrete (ALC) panel on the upper floor in the 8-story building fell away, and Photo 3.44 is the case where the tile on exterior wall was dropped, in Aoba Ward, Sendai City.

These kinds of damage relatively often occurred in buildings without structural damage, not limited to specific areas. Despite of the construction period and the seismic design codes applied to, these damage are often seen in many buildings.

Photo 3.43 Dropped ALC Panels from 8 Story

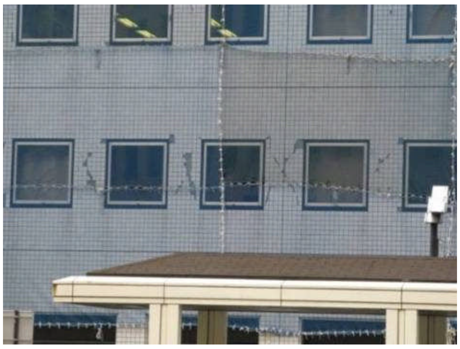


Photo 3.44 Damage of Tile on Exterior Wall

B-5) Collapse of concrete block wall and stone masonry wall

The collapse of concrete block wall and stone masonry wall are well known as earthquake damage caused by strong seismic motion. The damage of those was often observed in the field investigation, as shown in Photos 3.45 and 3.46.

Photo 3.45 Collapse of Concrete Block Wall

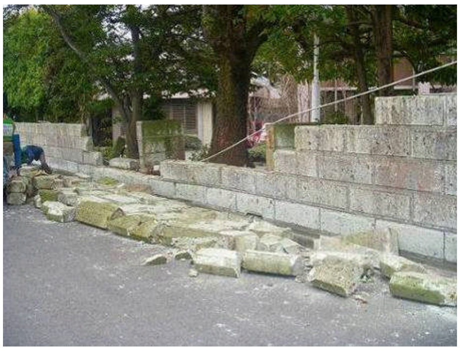


Photo 3.46 Collapse of Stone Masonry Wall

3.3.5 Concluding remarks

We classified the types of damage of reinforced concrete (RC) and steel reinforced concrete (SRC) buildings that were caused by earthquake motions under the Tohoku earthquake, and described the damage on structural and nonstructural elements. As mentioned early, almost all of the types of damage were observed in past destructive earthquakes such as the Kobe earthquake in 1995 and the Mid Niigata Prefecture earthquake in 2004. However, the following types of structural damage that were observed in the Kobe earthquake have not been confirmed within the scope of the investigation conducted so far.

- Story collapse of soft-first story building designed under the current seismic design code
- Mid-story collapse in mid-rise and high-rise buildings
- Overturning of buildings
- Failure of beam-column joint in building designed under the current seismic design code
- Fracture of pressure welding of reinforcements
- Falling of pre-cast roof in gymnasium

In general, there are only a few cases of serious structural damage that was caused by earthquake motions. On the contrary, it was the remarkable cases caused by the earthquake that public buildings like city hall under the past seismic design code suffered from severe damage and could not be continuously used. The main cause of the damage on these buildings was the loss of the vertical load carrying capacity due to shear failure of short columns. The fact makes us reconfirm that seismic retrofit on these buildings is particularly important, which must be operated as the disaster-prevention facilities.

3.4 Damage of Steel Gymnasiums

3.4.1 Introduction

The damage of general steel buildings such as offices and shops caused by the Tohoku earthquake in the areas of Ibaraki, Fukushima and Miyagi Prefectures with JMA seismic intensity around 6 was investigated for some two weeks after the earthquake. The structures of steel buildings are generally covered with exterior cladding and interior finishing. For this reason, the real situations of the damage to columns, beams and braces may not be correctly determined under the exterior damage investigation. Therefore, the damage investigation on steel gymnasiums whose structural members are generally exposed was considered and conducted. The damage investigation for such steel gymnasiums was carried out in the areas of Ibaraki Prefecture with JMA seismic intensity around 6. This section describes the outline of the damage investigation for the steel gymnasiums.

3.4.2 Outline of damage investigation for steel gymnasiums

i) Outline of damage investigation for high school gymnasiums in Ibaraki Prefecture

Gymnasiums designed under the previous seismic code were greatly damaged in the Mid Niigata Prefecture earthquake in 2004, but most of them under the current seismic code were not damaged [3.4, 3.5, 3.6]. Consequently, as the subject of the damage investigation, steel gymnasiums constructed under the previous seismic code were mainly chosen. The investigation covered a wide range of areas in Ibaraki Prefecture where JMA seismic intensity 5+ to 6+ was recorded (Ooarai Town, Shirosato Town, Hitachi City, Mito City, Naka City, Hitachinaka City, Chikusei City, Kasama City, Hokota City, Tsuchiura City, Bando City, Koga City, Shimotsuma City and Joso City). The main purpose of the investigation is to determine what damage pattern was often distributed in these areas and in which area the pattern was often distributed. A total of 44 gymnasiums in high schools were chosen and investigated.

ii) Outline of damage investigation for elementary and junior high school gymnasiums in Mito City

In general, building size (total floor area) of high school gymnasiums seems to be larger than elementary and junior high school gymnasiums. In order to know an effect of building size on earthquake damage situation, damage investigation of elementary and junior high school gymnasiums was considered and conducted. The result of the damage investigation for the high school gymnasiums in Ibaraki Prefecture showed that the areas around Mito City suffered relatively larger structural damage than other areas. Then, Mito City was chosen for the survey area of the damage investigation for gymnasiums in elementary and junior high school. A total of 22 gymnasiums in elementary and junior high school constructed under the previous seismic code in Mito City were investigated.

3.4.3 Classification and characteristics of damage of steel gymnasiums

For this earthquake damage investigation, a total of 66 gymnasiums in the high schools within Ibaraki Prefecture and in the elementary and junior high schools within Mito City were surveyed. The damage of the gymnasiums was classified into the types of (1) to (7). The types of (1) to (6) and the type of (7) refer to structural damage and to nonstructural one, respectively.

(1) Buckling and fracture of brace member and fracture of its joint
(2) Buckling of diagonal member of latticed column
(3) Damage of connection (bearing support part) between RC column and steel roof frame
(4) Deflection, buckling and fracture of roof horizontal brace
(5) Cracking of column base concrete
(6) Other (Overturning of floor strut, etc.)

(7) Nonstructural damage such as dropping of ceilings and exterior walls and breakage of windows

Each damage photograph is shown for each damage type in the following.

(1) Buckling and fracture of brace member and fracture of its joint

Buckling of brace member (Photo 3.47) and fracture of brace joint (Photos 3.48-3.50) were observed. Angle section was often used for many brace members, but circular hollow section steel (Photo 3.49) was also used for brace members. Fractured sections include steel plate inserted into steel pipe, end of bracing member and section loss part by bolt hole. These types of the damage are classified into the severe damage category based on the damage evaluation standard [3.7]. The gymnasiums constructed under the previous seismic code that were severely damaged by the Mid Niigata Prefecture earthquake in 2004 had accounted for about 30% of the total [3.4, 3.5, 3.6]. It is impressed that a rate of the gymnasiums severely damaged by the Tohoku earthquake was lower than by the Mid Niigata Prefecture earthquake in 2004.

Photo 3.47 Buckling of Brace

Photo 3.48 Net Section Fracture at Bolt Hole

a) Fracture at column top b) Fracture at brace crossing

Photo 3.49 Fracture of Brace Welded Connection

Photo 3.50 Fracture of Bolts

(2) Buckling of diagonal member of latticed column

In one of the investigated gymnasiums, buckling of diagonal members in some latticed columns was observed (Photo 3.51). Damage of column buckling caused in steel frames for span direction had not been observed under the damage investigations of the Mid Niigata Prefecture earthquake in 2004 [3.4, 3.5, 3.6].

a) Latticed column b) Buckling of diagonal member

Photo 3.51 Buckling of Diagonal Member of Latticed Column

(3) Damage of connection (bearing support part) between RC column and steel roof frame

In the investigated gymnasiums, exposure of anchor bolts due to spalling of the concrete at connection (bearing support part) between the RC column and steel roof frame (Photo 3.52), spalling of finish mortars on the RC column at the roof bearing support part, pullout of hole-in anchors were observed. This type of the damage was observed in some gymnasiums in this damage investigation.

Photo 3.52 Spalling of Concrete

(4) Deflection, buckling and fracture of roof horizontal brace

Roof horizontal braces were damaged in 2 high school gymnasiums and 5 elementary and junior high school gymnasiums. Such damage mainly occurred at horizontal braces with turnbuckles; obvious deflection of the horizontal brace, fracture at thread and fracture of bolt connections were observed (Photo 3.53).

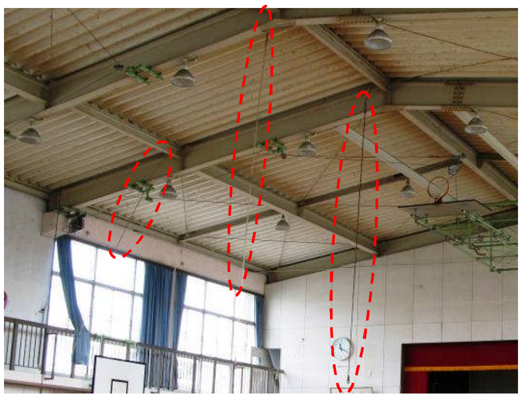


Photo 3.53 Fracture of Horizontal Brace

(5) Cracking of column base concrete

Damage of cracking of the column base concrete and mortar in the gallery of some gymnasiums was observed (Photo 3.54). Concrete and mortar of steel column base at a ground level was also cracked. However, almost all of these cracking are classified into minor or slight damage.

Photo 3.54 Cracking of Column Base Concrete

(6) Other (Overturning of floor strut, etc.)

As the other types of the structural damage, the following damage was observed; (a) overturning of floor strut (Photo 3.55), (b) tilting of concrete block self-standing wall and (c) peeling of paints of beam member which was observed near the top of the V-shaped roof beam or arch beam (Photo 3.56). In terms of the peeling of paints, it is undetermined whether a yielding occurred to the beam member or not.

Photo 3.55 Overturning of Floor Strut

Photo 3.56 Peeling of Paints of Beam

(7) Nonstructural damage such as dropping of ceilings and exterior walls and breakage of windows

The types of nonstructural damage of gymnasiums include dropping of ceilings and lighting equipment (Photos 3.57 and 3.58), breakage of windows (Photo 3.59), dropping of exterior walls (Photo 3.60), dropping of interior walls and eave soffit. In particular, the severe damage such as dropping of extensive ceiling in the high school gymnasiums was observed than in the elementary and junior high school gymnasiums.

Photo 3.57 Dropping of Ceiling Materials

Photo 3.58 Dropping of Ceiling Materials

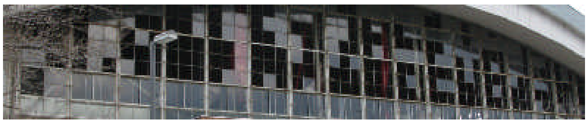


Photo 3.59 Breakage of Windows

Photo 3.60 Falling of Exterior Finish Materials

3.4.4 Concluding remarks

Damage of the steel gymnasiums constructed under the previous seismic code in the areas with JMA seismic intensity around 6 in Ibaraki Prefecture was investigated, and the outline of the investigation was described in this section. The results of the damage investigation of the steel gymnasiums are summarized as follows.

a) Structural damage of the steel gymnasiums

1) The types of observed structural damage of the gymnasiums are classified into the following six categories. (1) Buckling and fracture of brace member and fracture of its joint, (2) Buckling of diagonal member of latticed column, (3) Damage of connection (bearing support part) between RC column and steel roof frame, (4) Deflection, buckling and fracture of roof horizontal brace,

(5) Cracking of column base concrete, and (6) Other (overturning of floor strut, etc.).

2) In 3 of the investigated 66 gymnasiums, severe structural damage such as "fracture of brace member and joint" occurred. This rate of the damage seems to be smaller than that in the Mid Niigata Prefecture earthquake in 2004.

3) Severe structural damage was observed in Mito City, Hokota City and Naka City than in other areas.

b) Nonstructural damage of the steel gymnasiums

1) The types of observed nonstructural damage include dropping of ceilings, dropping of exterior and interior walls, falling of eave soffit and breakage of windows.

2) In 4 of the investigated gymnasiums, ceiling materials were extensively dropped, which is classified into the severe damage category. In some of the gymnasiums, many windows were broken.

3) Severe nonstructural damage was observed in Mito City, Hokota City and Hitachi City than in other areas.

4) Severe structural and nonstructural damage seems to occur in the high school gymnasiums rather than in the elementary and junior high school gymnasiums.

3.5 Damages due to Failures of Residential Land

3.5.1 Introduction

This section reports the outline of damage situations associated with liquefaction in the catchment basin area of Tone River on the border between Ibaraki and Chiba Prefectures and in Urayasu City, Chiba Prefecture, and the outline of damage situations of developed housing area in Miyagi and Fukushima Prefectures.

3.5.2 Liquefaction damage of catchment basin area of Tone River

Damage associated with liquefaction has occurred in the same areas as in the areas where liquefaction was reported in past earthquakes, such as old river channel, reclaimed lagoon, reclaimed pond and reclaimed paddy field. This section describes the damage situations in Nishishiro, Inashiki City, in Hinode, Itako City and in Kamisu City within Ibaraki Prefecture. For reference, liquefaction damage in Nishishiro, Inashiki City and in Hinode, Itako City was reported after the 1987 East off Chiba Prefecture earthquake.

(1) Nishishiro, Inashiki City in Ibaraki Prefecture

Large-scale and extensive damage occurred within the zone of about 500 m four directions that encloses Route 51 of National Highway and Yokotone River on the east of the road. Route 11 of Prefectural Road was closed to vehicles, and sand boiling, great road upheaval or severe fissure that is associated with liquefaction was seen mainly along the road. As a ground transformation, the ground subsided up to about 40 cm, and transversely moved up to about 1 m. Private automobile were buried in boiled sand to the extent of a half of height of their tires. This indicates a great amount of sand boil.

Finishes of the sidewalks around a large-scale commercial establishment along Route 11 of Prefectural Road were scattered. The subsidence of the surrounding ground was about 40 cm, and the settlement of the facility in itself was slight. The commercial building was tilted about 0.7/100 in the longitudinal direction. We visually observed a foundation type of the building from an opening between surrounding fissures. The type was confirmed to be a pile foundation (Photo 3.61).

Photo 3.61 Situations around the Commercial Building and State of Pile Head

Sand boiling was seen everywhere on the roads or sites also in surrounding buildings lots. A house constructed on an embankment was tilted to an adjacent warehouse with sand boiling. An angle of tilting was 5.0/100 (Photo 3.62). It can be assumed that liquefaction occurred at the concentration of the loads of two adjacent buildings and the house was tilted to the direction.

Photo 3.62 House Tilted 5.0/100

(2) Hinode, Itako City in Ibaraki Prefecture

In Hinode, large-scale damage occurred in on corner within the zone of about 200 m four directions near Hitachi-tone River. Sand boiling, lift of buried structures, and subsidence or tilting of power poles, which were caused by liquefaction, were seen everywhere on the road and sites. Many buildings facing the road subsided 20 to 30 cm from the front sidewalk (Photo 3.63). For reference, foundation cracks or gaps were not observed in the investigated range.

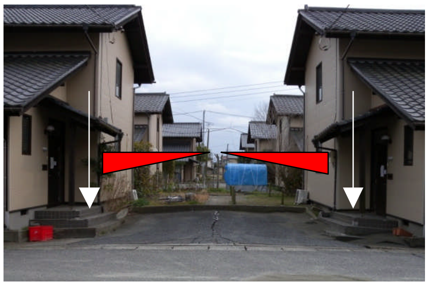


Photo 3.63 Subsidence of Two Houses enclosing Vacant Land

3.5.3 Liquefaction damage in Urayasu City, Chiba Prefecture

A reclaimed ground accounts for 3/4 of a gross area in Urayasu City at present. The southern part of this city is an area that was developed under a reclamation project using sea sand. In the result, the area consists of a weak layer up to GL-40m. For reference, liquefaction damage was reported after the 1987 East off Chiba Prefecture earthquake. The damage situations are given below.

(1) Mihama

In Mihama, subsidence and tilting that was caused by liquefaction were observed in a house that has a dry area in a basement (Photo 3.64). An angle of tilting of the house was about 3 degrees. It is considered that the basement was lifted and another remaining parts of the house subsided. Around this house, a house's site was totally covered with boiled sand, and a fence's foundation was deformed. In addition, a carport in one building was ruptured by liquefaction and moved (Photo 3.65). A unit of the carport and the building was separated and moved about 50 cm probably due to the movement of the ground associated with liquefaction.

Photo 3.64 Tilted House

Photo 3.65 Moved Carport

(2) Irifune

In Irifune, a difference in settlement between adjacent buildings on spread and pile foundations was observed. The building on spread foundation subsided about 35 cm from the front sidewalk, while the building on pile foundation was elevated about 30 cm (Photo 3.66). Other settled and tilted buildings were dotted

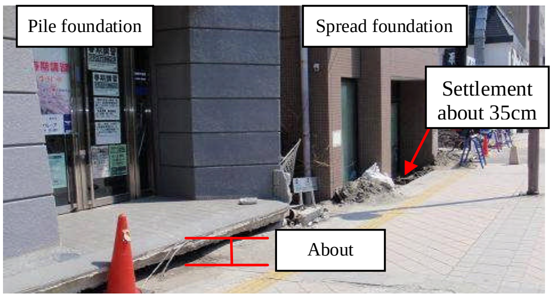


Photo 3.66 Difference in Damage between Support Mechanisms

6.5.4 Damage of developed housing area

The damage investigation for developed housing area was conducted in some areas of Miyagi, Fukushima and Tochigi Prefectures, but the damage in Miyagi and Fukushima Prefectures is reported in this section.

(1) Near 5-chome, Oritate, Aoba-ku, Sendai City, Miyagi Prefecture

In one corner of a large-scale housing area, where a slope in the N-NE direction had been developed, ground transformation by sliding of the housing site embankment to the slope direction, and damage to the retaining walls by ground transformation were often observed (Photo 3.67). Houses on the site were recognized to have different damage patters, such as movement, subsidence and tilting without structural damage, structural great deformation and fractured foundation.

Photo 3.67 Damage of Retaining Wall and House Movement and Tilting by Sliding and Ground Transformation

(2) Near Aoyama 2-chome and Midorigaoka 4-chome, Taihaku-ku, Sendai City, Miyagi Pref.

This area is located at one corner of a large-scale housing area where a contoured hill was developed. Ground transformation by sliding of the housing site embankment to the slope direction, and damage to the retaining walls by ground transformation, were often observed. The damaged area at 4-chome, Midorigaoka under this earthquake was almost same as under the 1978 Miyagi-Ken-Oki earthquake. The land at 2-chome, Aoyama is wavier than at 4-chome, Midorigaoka. Near the zone of 2-chome, Aoyama, large-scale sliding of the embankment occurred (Photo 3.68). In this zone, large deformation and damage were seen on both of upper structures and foundations of houses on the housing area. In other places with embankment sliding, deformation and damage of upper structures of houses were observed, but it seemed that there was limited significant damage to foundations. Near 2-chome, Aoyama, a retaining wall for the housing area with a height of over 5 m was damaged.

Photo 3.68 Group of Houses damaged by Sliding and Ground Transformation

(3) Near Numanoue, Fushiogami, Fukushim City, Fukushima Prefecture

This area is located at one corner of a large-scale housing area where a hill was developed. The result of visual inspection showed ground transformation by land sliding on the slope of the hill. This ground transformation caused serious damage to houses. In the result, some houses was in a state of sliding on the slope of the hill (Photo 3.69). On the other hand, houses near the top of the hill suffered only damage associated with slight transformation of housing area embankment.

Photo 3.69 Land Sliding on Slope on the Southwest of the Hill and Sliding House

3.5.5 Concluding remarks

The outline of the damage situations in the investigate scope is as follows.

1) Damage caused by liquefaction:
In the catchment area of Tone River and the coastal zone of Tokyo Bay, extensive damage such as sand boiling or ground transformation associated with liquefaction was confirmed. Highly tilted buildings were seen, but visual cracks or fissures on the foundations investigated were not observed.

2) Damage to housing area:
Large damage with transformations such as ground sliding was observed mainly in the elevated and developed housing area (particularly marginal part). In some areas, transformations occurred again in the developed lots that had been affected by the past earthquakes.

3.6 Response of Seismically Isolated Buildings

3.6.1 Introduction

Miyagi Prefecture and nearby areas have experienced disastrous earthquakes frequently, therefore, reflecting high consciousness of earthquake risks, there are many seismically isolated buildings (SI buildings) constructed in those areas. Investigation team was dispatched on July 1st and 2nd in 2011 to observe performance of SI buildings during the Tohoku earthquake and ask persons in charge of the buildings about the damage. In total, 16 SI buildings in Miyagi Prefecture and 1 building in Yamagata Prefecture were investigated.

3.6.2 Behavior of SI buildings

(1) SI building A

i) Building information
The SI building A is a reinforced concrete office building with 9-story super-structure and 2-story basement, located in Miyagino in Sendai City (Photo 3.70). The building was retrofitted by using base isolation technique putting isolation devices on the top of columns in B1F. The floor plan has the 26.4 m × 54 m rectangular shape and 40 high-damping rubber bearings (HRBs) are installed.

ii) Building performance during earthquake
Observation results are summarized as follows:

a) According to the person in charge of the building, no furniture was turned over and no structural damage was observed.
b) However, some damage was observed at the cover-panels of fire protection and the expansion joints near the boundary between isolated and non-isolated floors (Photo 3.71). It seems that parts of expansion joints were not well operated due to the large displacement of SI building floor during earthquake.
c) The ground surrounding the building partially subsided around 10 cm.

iii) Earthquake motion records
This building has accelerometers at B2F, 1F and

9F (top floor). Also, there is a scratch board in B1F to record the displacement of isolation floor. Furthermore, there is an accelerometer installed by JMA in the basement of an adjacent building. The maximum acceleration values of these accelerometers at main shock are listed in Table 3.1.

From the trace on the scratch board installed on the SI building floor, the maximum displacement was estimated as around 18 cm at the main shock (on March 11, 2011) and around 10 cm at the aftershock (on April 7, 2011).

Photo 3.70 Overview of SI Building A

(a) Damage to the panel

(b) Damage to the expansion joint

Photo 3.71 Damage near the Boundary between Isolated and Non-isolated Floors

Table 3.1 Maximum Acceleration Values

Location	Direction		
	NS [gal]	EW [gal]	Vertical Z [gal]
Basement of adjacent bldg.	409.9	317.9	251.4
B2F (below SI)	289.0	250.8	234.9
1F (above SI)	120.5	143.7	373.7
9F	141.7	169.9	523.9

(2) SI building B

i) Building information

The SI building B is a 14-story reinforced concrete building used for condominium, located in Miyagino in Sendai City (Photo 3.72). The building has the U-shape plan and the corners of the building are separated by expansion joints. The NRBs, Lead dampers, U-shape steel dampers are installed in the SI floor.

ii) Building performance during earthquake

Observation results are summarized as follows:

a) According to the person in charge of the building, no furniture was turned over and no structural damage was observed inside of rooms. However, the damage to the expansion joint was observed.
b) Drop of the ceramic tiles on outer wall (Photo 3.73) and shear crack on the wall in the first floor parking space (Photo 3.74) were observed. The subsidence of ground around 10 cm was observed near the building.
c) No damage was found to NRBs by visual inspection (Photo 3.75), however, paint of U-shape dampers was peeled off (Photo 3.76) and many cracks were found on Lead dampers (Photo 3.77).

Photo 3.72 Overview of SI building B

Photo 3.73 Drop of Ceramic Tiles

Photo 3.74 Shear Crack on the Wall

Photo 3.75 NRB

Photo 3.76 U-shape Steel Damper

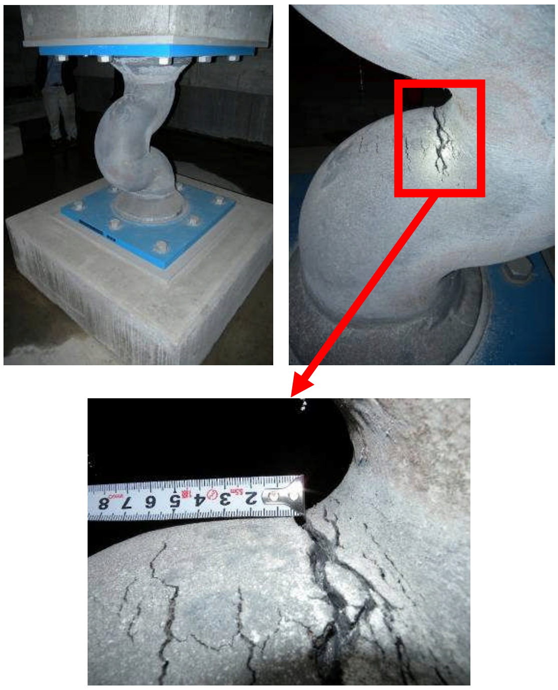


Photo 3.77 Lead Damper and Crack on the Surface

3.6.3 Concluding remarks

Investigation results of SI buildings in Miyagi Prefecture and one SI building in Yamagata Prefecture is summarized as follows:

a) Super-structures of SI buildings suffered almost no damage even under strong shaking with JMA intensity 6 upper. It verifies the excellent performance of SI buildings.
b) There are 8 buildings with scratch boards to measure displacement of the SI building floor. In most cases, the maximum displacement has been estimated as around 20 cm. There is one case with the maximum displacement estimated over 40 cm.
c) In some buildings, damage was observed at the expansion joints. It seems that parts of expansion joints were not well operated due to the large displacement of SI building floor during earthquake.
d) Subsidence of ground around the building was observed in some buildings.
e) Many cracks were found in lead dampers. These cracks might be increased by the aftershocks.
f) Peeling off of paint was observed widely for U-shape steel dampers. In some cases,

residual deformation of steel was remained.

3.7 Conclusion

This section summarizes results of damage of buildings due to earthquake motion through surveys on wood houses, steel buildings, reinforced concrete buildings, residential land, foundation, non-structural elements and seismically isolated buildings. The results can be summarized as follows even though it is in the stage of the Quick Report.

1) Wood houses: The damages of upper structure were confirmed in several areas however the damage of wood houses seems not so heavy as an impression in Kurihara City where seismic intensity 7 was recorded. Many damages of structure were observed due to deformation of developed residential land in Sendai City, Miyagi Prefecture and Yaita City, Tochigi Prefecture. The damage of roof tiles could be more observed in both Fukushima and Ibaraki Prefectures than in Miyagi Prefecture where earthquakes were frequently occurred since the 1978 Miyagi-Ken-Oki earthquake. The damage types are almost similar to those of the past earthquakes.

2) Steel frame structures: There was almost no damage of main steel structure members such as columns and beams. Damages of vertical braces' rupture etc. were observed in the school gymnasium that was constructed in the years of old seismic code (before 1981) however the damage ratio is smaller than the case of the Niigata-Ken Chuetsu earthquake in 2004. On the other hand, damages of non-structural elements including falling of ceilings were observed comparatively more than the past cases.

3) Reinforced concrete structures: Most of structural damages in reinforced concrete structure were observed in the buildings designed based on the old seismic codes. Though the number of damaged buildings is not large compared to the seismic intensity, damage types were mostly similar to the past seismic damages that include severe damage such as loss of axial force bearing capacity due to shear failure of columns.

4) Residential land, Foundation: One of the observed characteristics is liquefaction in wide areas that could not be occurred in the past earthquakes in Japan. Researches on the mechanism and considerations of counter-measures will be necessary not only for individual buildings but also for infrastructure like roads and water supply and sewage systems. In a part of residential land, heavy damage such as collapse of ground was observed equal to the past damaged earthquakes.

5) Damages of non-structural elements of comparatively old construction types were confirmed in many cases. In addition, break and falling of non-structural elements at rather higher parts were also confirmed.

6) Seismically isolated buildings: Super-structures of SI buildings suffered almost no damage. However, damage was observed at the expansion joints in some buildings. Also, many cracks in lead dampers and peeling off of paint of U-shape steel dampers were found.

4. DAMAGE TO BUILDINGS IN INUNDATION AREAS DUE TO TSUNAMI

4.1 Purpose of Investigation

The purpose of this investigation is to understand an overview of buildings damaged by tsunami, to obtain basic data and information required to evaluate mechanisms for causing damage to the buildings and to contribute to tsunami load and tsunami-resistant designs for buildings such as tsunami evacuation buildings, by means of collecting building damage cases by tsunami, classifying the damage patterns for different structural categories, and making a comparison between the calculated tsunami force acting on buildings and the strength of the buildings.

The NILIM and BRI jointly created a tsunami damage investigation team[4] that consists of 27 members. The joint team collected national and international standards and codes concerning tsunami evacuation buildings and tsunami loads and surveyed about 100 buildings and structures in three site investigations.

4.2 Summary of Damage in Inundation Area Due to Tsunami

Table 4.1 gathered up the damage statistics of main cities, towns and villages, 49 local governments in six prefectures, in the tsunami inundation area based on a survey by Fire and Disaster Management Agency (FDMA) on August 11, 2011 [4.1]. As for the dwelling house damage, about 106,000 houses were completely destructed or missing, about 100,000 houses were partially damaged, about 106,000 houses were below partially damaged, and totally about 370,000 houses were damaged and 160 fires occurred in this area. Each number of complete destruction or missing of houses of Ishinomaki or Sendai City in Miyagi Prefecture is more than 19,000. That of Kesen-numa City in Miyagi Prefecture is more than 8,500. Those of Higashimatsushima City in Miyagi Prefecture and Minamisoma City in Fukushima Prefecture are more than 4,500. Those of Miyako City, Kamaishi City, Rikuzentakata City in Iwate Prefecture and Minamisanriku Town in Miyagi Prefecture are more than 3,000. There are areas which suffered serious damage in Iwate, Miyagi and Fukushima Prefectures.

4.3 Classification of Damage Patterns

4.3.1 Reinforced concrete buildings

(1) Collapse of first floor

A case where column capitals and bases on the first floor in a building were subject to flexural failure and subsequently to story collapse was seen in two-story buildings (Photo 4.1).

These buildings have a column-to-beam frame. The first floor has a relatively small number of walls, but many concrete block walls are placed on the second floor. The first and second floors of the building in Photo 4.1 are used as shop and dwelling, respectively. The relevant buildings are estimated to have structural characteristics of low strength and stiffness on the first floor. As an opening on the second floor is not large, it is assumed that the second floor suffered a large tsunami wave pressure and the shear force acting on the first floor exceeded the lateral load-bearing capacity, resulting in the collapse of the building. Story collapse of the first floor has not been observed in 3-story or higher buildings in the past investigations. In 3-story buildings, in general, reinforced concrete walls are often used for the first floor. For this reason, the strength of the first floor is considered to have been larger.

Photo 4.1 Story Collapse of 2-story Reinforced Concrete Building

[4] Damage Investigation Team (The members' positions as of April 20, 2011) - National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure and Transport (8 members): Isao Nishiyama, Akiyoshi Mukai, Ichiro Minato, Atsuo Fukai, Shuichi Takeya, Hitomitsu Kikitsu, Hiroshi Arai, and Tomohiko Sakata; Building Research Institute (19 members): Juntaro Tsuru, Nobuo Furukawa, Masanori Iiba, Shoichi Ando, Wataru Gojo, Hiroshi Fukuyama, Yasuo Okuda, Taiki Saito, Bun-ichiro Shibazaki, Koichi Morita, Hiroto Kato, Tsutomu Hirade, Takashi Hasegawa, Tadashi Ishihara, Norimitsu Ishii, Yushiro Fujii, Haruhiko Suwada, Yasuhiro Araki, and Toshikazu Kabeyasawa

Table 4.1 Damage Statistics in Inundation Areas due to Tsunami *

Prefecture	City, Town, Village	Human Damage			House Damage			
		Dead	Missing	Injury	Complete Destruction or Missing	Partial Damage	Below Partial Damage	Fire
Aomori	Hachinohe	1	1	17	250	769		2
	Hashikami	0	0	0	12	8	1	
	Total	1	1	17	262	777	1	2
Iwate	Hirono	0	0	0	10	16	5	
	Kuji	2	2	8	65	210		
	Noda	38		17	309	169		1
	Hudai	0	1	1				
	Tanohata	14	19	8	225	45	4	
	Miyako	420	124	33	3,669	1,006	176	6
	Yamada	597	256	Unknown	2,789	395	120	2
	Otsuchi	796	653	Unknown	3,677			2
	Kamaishi	881	299	Unknown	3,188	535	120	
	Ofunato	331	118	Unknown	3,629		Unknown	2
	Rikuzentakata	1,546	569	Unknown	3,159	182	27	
	Total	4,625	2,041	67	20,720	2,558	452	13
Miyagi	Kesen-numa	1,004	410	Unknown	8,533	2,313	3,248	8
	Minamisanriku	550	437	Unknown	3,167	144	Unknown	5
	Onagawa	535	414	2	2,939	337	640	5
	Isinomaki	3,153	890	Unknown	19,065	3,354	10,199	23
	Higashimatsushima	1,044	104	Unknown	4,589	4,672	2,471	1
	Matsushima	2	0	37	213	1,321	1,184	2
	Rifu	1	1	1	59	508	1,732	
	Shiogama	20	1	10	682	2,784	3,973	8
	Shichigahama	66	6	Unknown	729	460	1,067	
	Tagajo	188	3	Unknown	1,662	2,993	5,097	15
	Sendai	704	33	2,276	19,922	41,344	56,347	39
	Natori	911	82	Unknown	2,786	922	8,060	12
	Iwanuma	183	1	293	720	1,545	2,403	1
	Watari	256	5	44	2,459	1,032	1,985	3
	Yamamoto	670	23	90	2,200	1,042	1,086	2
	Total	9,287	2,410	2,753	69,725	64,771	99,492	124
Fukushima	Shinchi	107	3	3	548	Unknown		
	Soma	454	5	71	1,049	643	3,092	
	Minamisoma	633	38	59	4,682	975		
	Namie	141	43					
	Futaba	29	6	1	58	5		
	Okuma	73	1		30			
	Tomioka	19	6					
	Naraha	11	2	5	50			
	Hirono	2	1		Unknouwn	Unknown		
	Iwaki	308	39	4	6,585	18,931	21,800	3
	Total	1,777	144	143	13,002	20,554	24,892	3
Ibaraki	Kitaibaraki	5	1	188	339	1,569	5,745	3
	Takahagi	1		19	131	728	3,213	
	Hitachi			166	403	3,016	11,229	4
	Tokai	4		5	56	104	3,150	2
	Hitachinaka	2		27	79	720	5,863	1
	Oarai	1		6	10	268	1,087	
	Hokota			15	96	524	4,863	3
	Kajima	1			368	1,726	2,567	4
	Kamisu			6	139	1,660	3,011	3
	Total	14	1	432	1,621	10,315	40,728	20
Chiba	Choshi			19	23	105	1,938	
	Asahi	13	2	12	336	931	2,358	
	Total	13	2	31	359	1,036	4,296	0
Sum Total		15,717	4,599	3,443	105,689	100,011	169,861	162

* Fire and Disaster Management Agency (FDMA) on August 11, 2011 [4.1]

(2) Overturning

Overturning was observed in 4-story or lower buildings. In all overturned buildings, the maximum inundation depth exceeded their height. Overturning types include building that felt sidelong (Photo 4.2) and buildings that turned upside down. Most of the overturned buildings are of mat foundation. In some overturned buildings on pile foundation, piles were pulled out.

Photo 4.2 Overturning of 3-story Reinforced Concrete Building

An overturning case was often seen in 4-story or lower buildings with relatively small size of openings. However, there were many cases where 4-story or lower buildings with large size of openings were not overturned. Consequently, a size of an opening on an exterior wall is considered to have greatly affected overturning.

In some cases, there were tsunami traces at the heights of the upper end of openings on the top floor inside the buildings whose heights were exceeded by maximum inundation depths. It is considered that air has accumulated in the space between the ceiling and the upper end. Overturning is considered to occur when an overturning moment by tsunami wave force exceeds an overturning strength by a dead load of a building (considering the effect of buoyancy as required). A building, in which a distance from the upper end of an opening on each floor to a ceiling is long, may be overturned even by a slight horizontal tsunami force when buoyancy significantly acts on the building.

(3) Movement and washed away

Most of the overturned buildings were moved from their original positions. It is estimated that large buoyancy acted on the buildings. Moved and overturned buildings left no dragged traces on the ground. One of the buildings climbed over a concrete block fence on an adjoining land (about 2 m) without destroying the fence (Photo 4.3). The building seems to have floated up by buoyancy. Some of the 2-story apartment houses with the same shape that were overturned were washed away and missing. A buoyancy and large horizontal force seem to have acted on these buildings.

Photo 4.3 2-story Reinforced Concrete Building that Climbed over the Fence and Overturned

(4) Tilting by scouring

When tsunami acted on a building, a strong stream was generated around the corner of the building, resulting in many large holes on the ground that were bored by scouring. In one case, a building on mat foundation fell into a hole bored by scouring (Photo 4.4).

Photo 4.4 2-story Reinforced Concrete Building that was Tilted by Scouring

(5) Fracture of wall (fracture of opening)

When tsunami acts on openings in a building and opposite openings are smaller than the affected openings, a stream flowing from the affected openings concentrate on the opposite small openings. In one observed case related to this event, a stream generated by tsunami provided a large pressure to a reinforced concrete non-structural wall around small opposite openings. The pressure enlarged the concrete wall to the outside and fractured the wall reinforcement. A tsunami wave force that acts on a building will be reduced if the size of opening affected by the force becomes larger. The same trend is considered to apply to an outlet surface of the stream.

A case where such wall reinforcement was fractured is often seen in wall members with single layer bar arrangement. In one damaged building (Photo 4.5), a 300 mm-thick shear wall with double layer bar arrangement and a support span of more than 10 m and without no 2-story floor was bent inside by a tsunami wave pressure. However, a shear wall in an area (Photo 4.5 Back of the building), where there is a floor on the second story and a support span is not long in the same building, was not bent.

Photo 4.5 Out-of-plane Fracture of Reinforced Concrete Shear Wall without Floor

(6) Debris impact

Debris impact was seen in most of the non-structural members such as window and ceiling materials. The number of cases of clear damage to skeletons was not large, but in one observed case, a multi-story wall in an apartment house was probably bored by debris impact (Photo 4.6).

Photo 4.6 Wall Opening Generated by Debris Impact

4.3.2 Steel buildings

(1) Movement and washed away by fracture of exposed column base

A typical case of building movement and washed away is that a building moved and flew due to the fracture of anchor bolts and/or base plates at steel exposed column bases and the fracture of a weld between the column and the base plate (Photo 4.7). In most cases, a foundation and some column bases were left in a site, but the body of a building was moved beyond the site and missing.

Photo 4.7 Steel Building Overturned by Fracture of Column Base Anchor Bolts

(2) Movement and washed away by fracture of capital connection

In damage cases relatively often seen a column top connection on the first or second floor in a building was fractured and the building was moved and washed away. When a column base has a large strength like concrete encases type or embedded type, this type of fracture is considered to occur. In one case (Photo 4.8), a foundation in a building, and several columns on the first floor (or up to the second floor) were left on a site, and the columns fell in the same direction (Photo 4.8).

In most cases, welds between diaphragms with lower flanges and the first-floor columns were fractured and the sections of the columns were exposed. In one building, flanges of the second-floor H-shaped beams were torn. Based on the deformation states near the column bases, it is estimated that a tensile force acted on the first-floor columns and fractured the first-floor column top connections after the first floor was greatly tilted to the same extent as the inclination of the remaining columns.

Photo 4.8 First-floor Columns Falling in the Same Direction

(3) Overturning

One case, in which a whole building including foundation is overturned, was confirmed. Most of the ALC panels of claddings were left (Photo 4.9).

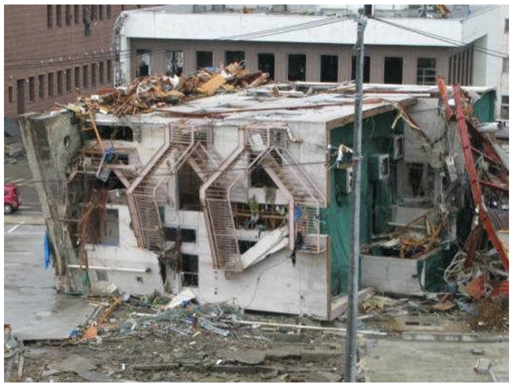


Photo 4.9 Overturning of 3-story Steel Building

(4) Collapse

Damage cases of skeleton collapse include story collapse of the first floor in a 2-story steel building (Photo 4.10) and partial collapse of a warehouse on the coast.

Photo 4.10 Story Collapse of First Floor in 2-story Steel Building

(5) Large residual deformation

Slight tilting was often observed in steel buildings with only a skeleton left. In one case (Photo 4.11), a gabled roof frame building did not collapsed despite large residual deformation.

Photo 4.11 Tilted Gabled Roof Frame

(6) Full fracture and washed away of cladding and internal finishing materials

Cladding materials such as ALC panel were almost fully fractured and washed away, and a steel frame as a skeleton was left. This case was often seen (Photo 4.12). It is considered that an external force that acts on the skeleton became small, due to early washed away of the cladding materials. In the remaining building, slight tilting of the skeleton, member deformation on the face affected by tsunami, or members locally damaged possibly by debris impact, was observed.

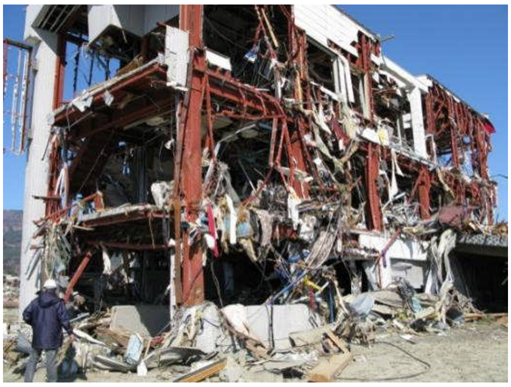


Photo 4.12 Remaining 3-story Steel Building

In another damage case, openings on the face affected by tsunami and on its opposite face, or transverse faces were greatly damaged and fractured possibly due to stream runoff.

4.3.3 Damage to wood houses

Damage patterns of wood houses that were caused by tsunami are considered to be greatly related to a maximum inundation depth. In the case of a maximum inundation depth more than about 6 m (equivalent to a height of eaves of 2-story wood house), the number of 1-story and 2-story Wood houses that remained was almost zero. A damage pattern case where a superstructure in a house was washed away with only its foundation and ground sill left (Photo 4.13) or where the superstructure and the ground sill were washed away with only the foundation left was frequently confirmed.

Photo 4.13 Remaining Ground Sill after Washout of Superstructure

In the case of a maximum inundation depth of about 1 m, most of wood houses remained. Some wood houses were damaged possibly due to debris impact. In the case of a maximum inundation depth of about 1 to 6 m, some wood houses remained. Some wood houses behind the relatively stronger building for tsunami wave force such as a reinforced concrete building remained. This is possibly due to that a tsunami wave force was significantly alleviated by the building not washed away (Photo 4.14).

Photo 4.14 Remaining Wood Houses Behind Remaining Building

In addition to these cases, a tsunami wave force was reduced possibly due to many openings in the direction affected by tsunami, or a wooden house remained despite washed away of columns and external walls in the corner of the building. Several houses that have a reinforced concrete piloti on the first floor, or a mixture of wooden and reinforced concrete structures, remained (Photo 4.15).

Photo 4.15 Remaining Mixed Building with Reinforced Concrete Piloti on First Floor

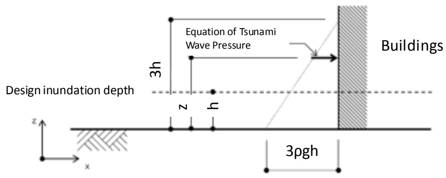
4.4 Database for Investigated Buildings

Outer dimensions of about 100 buildings and dimensions of their skeletons were measured in the site investigation. Maximum inundation depths were measured from tsunami traces on surveyed buildings and surrounding buildings. These measurement results were integrated into a database for investigated buildings. Building name, address, purpose, construction year, designation as tsunami evacuation building, structure category, number of stories, outer dimension, distance from seacoast (river), GPS position, altitude, surrounding circumstances, damage situations, etc., were included in the database. In addition, photos of investigated buildings that were taken from four directions where possible were attached to the database. Based on the database, we estimated strengths of the buildings and tsunami loads on them, and are evaluating whether the estimated values are consistent with the damage situations.

4.5 Discussion on Guidelines for Tsunami Evacuation Buildings by the Cabinet Office of Japan

The Cabinet Office made guidelines for tsunami evacuation buildings in 2005 [4.3]. After the Tohoku earthquake, the committee on structural design for tsunami evacuation buildings (Leader: Prof. Y. Nakano, University of Tokyo) was organized in order to validate the equation of tsunami wave pressure on buildings, using the database for 52 buildings and 44 structures investigated in inundation areas due to the tsunami.

The NILIM announced the results of the validation of the equation of tsunami wave pressure on buildings based on the interim committee report in August, 2011 [4.3]. The general content of the interim result is as follows.



Equation of Tsunami Wave Pressure: $q_x = \rho\, g\, (3h - z)$ (1)

q_x: design tsunami wave pressure (kN/m^2)
ρ: density of water (t/m^3)
g: gravitational acceleration (m/s^2)
h: design inundation depth (m)
z: height from the ground ($0 \leqq z \leqq 3h$) (m)

Fig. 4.1 Design Inundation Depth and Tsunami Wave Pressure of Guidelines for Tsunami Evacuation Buildings (The Cabinet Office) [4.2]

In the Cabinet Office guidelines, the design tsunami load for the tsunami evacuation buildings was adopted as the tsunami wave pressure eq. (1), hydrostatic pressure of 3 times design inundation depth, based on the maximum envelope of the experimental result in Japan because of the following reasons:

1) Simple equation of the design tsunami load
2) Safer estimation for the design tsunami load including hydrodynamic effects
3) Lack of effective data for the design tsunami load

The strength of reinforced concrete buildings and structures was calculated from the database to be converted into the depth of the hydrostatic pressure equivalent to the strength of the buildings and structures under the following conditions.

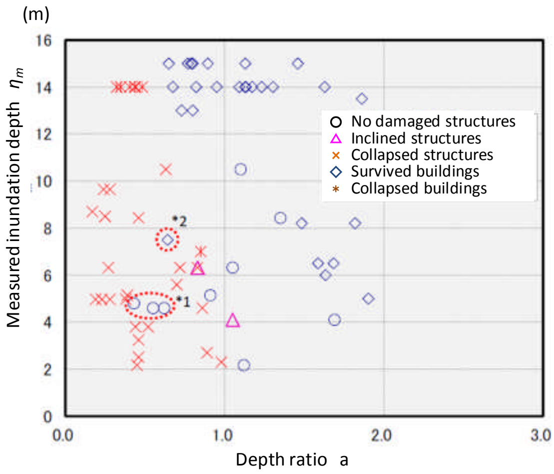


Fig. 4.2 Depth Ratio of Investigated Buildings with Some Obstacles for Tsunami Effect [4.3]

*1: structures paralleled to the tsunami direction

*2: large openings in side faces of a building

1) Mass per unit area for reinforced concrete building structures is 14 (kN/m^2).
2) Compressive strength of a concrete is 21 (N/mm^2) and yielding stress of a round bar is 294 (N/mm^2).
3) Lateral load carrying capacity of reinforced concrete building structures is roughly estimated with total section area and average ultimate shear stress of columns (σ=1 N/mm^2) and structural walls (σ=3 N/mm^2) according to the Japanese seismic evaluation of existing reinforced concrete building structures.
4) Tsunami load is simply reduced in proportion to a ratio of openings to total elevation area affected by tsunami.

A depth ratio α is defined as eq. (2).

$$\alpha = \frac{\eta_s}{\eta_m} \qquad (2)$$

α: depth ratio

η_s: depth of the hydrostatic pressure equivalent to the strength of the buildings and structures (m)

η_m: measured inundation depth (m)

Fig. 4.2 plots damage situations of buildings and structures with some obstacles for tsunami effect for α and η_m. A boundary of damage situations of buildings and structures can be considered at the depth ratio α =1 except *1 and *2 in Fig. 4.2 when the inundation depth was measured less than about 10m. Many buildings at the depth ratio α much less than 1 remain when the inundation depth was measured more than 10m.

This section classified the damage patterns for different structural categories and briefly discussed the factors that had caused various types of damage. Based on the results of the relevant investigation, we are now conducting an additional field investigation as required and collecting design documents for damaged buildings, while further evaluating the effects of building openings and buoyancy and proceeding with the elucidation of mechanisms for causing damage and the identification of tsunami loads on buildings.

5. ACTIVITIES ON DEVELOPING COUNTERMEASURES FOR MINIMIZING THE EFFECT OF THE LONG-PERIOD GROUND MOTIONS TO BUILDING STRUCTURES

5.1 Background of the Study

A large scale of fire had broken out to the large oil storage tanks in Tomakomai, Hokkaido after the 2003 Tokachi-Oki earthquake. The long-period motions caused by the large magnitude earthquake and enlarged by the deep sedimentary basin structures under the Tomakomai region amplified and elongated the motions acting to the tanks and the excessive sloshing of the liquid surface occurred and the liquid (oil) overflowed the tanks and the liquid was ignitioned fire.

After this earthquake, the long-period motions featured with slow, cyclic and long duration recalled the concerns on the future large earthquakes to occur in the Pacific coast of Japan generating such long-period motions and the excessive responses to long-period building structures such as super high-rise and seismically isolated (SI) buildings.

Meanwhile, large earthquakes such as Nankai, Tonankai and Tokai earthquakes are supposed to occur on subduction zones around Japan in near future. Therefore, we have serious concerns on structural damage due to the long-period ground motions generated by those large earthquakes.

The Central Disaster Management Council, the Cabinet Office, had set up many committees on establishing countermeasures for respective large influential earthquakes. They had also started the study on the effect of the long-period earthquake motions for several subduction-zone earthquakes and estimated the ground motions and subsequent damage for urbanized areas in Japan.[5.1]

The Headquarters for Earthquake Research Promotion (HERP) established in the Ministry of Education, Culture, Sports, Science and Technology (MEXT) after 1995 Kobe earthquake had also started the study on the hazard maps showing the long-period motion estimates that are based on the simulations with possible earthquake source rupture process based on experience during the past large earthquakes and the estimated underground structures. The preliminary maps for the Tonankai, Tokai and Miyagi-Ken-Oki earthquakes were publicized in September, 2009. [5.2] Preparation of the additional long-period motion maps on some other subduction-zone earthquakes are now in progress for publication by the HERP.

The Architectural Institute of Japan (AIJ) had also conducted the extensive study on safety measures for building structures against the long-period earthquake motions. [5.3]
Under these various study activities including those currently in progress as background, the MLIT and the NILIM initiated the funding [1.4] to maintain and promote the enhancement of the building codes and one of them was the development of the design long-period earthquake motions. Under this funding, the selected working team and the BRI have been doing the cooperative study on the long-period motions. Various kinds of assistance were given from experts on strong ground motion and structural engineers.

The cooperative study had continued from fiscal years 2008 to 2010 and a research report was published by the BRI as product of the studies.[5.4]

Based on the study, the NILIM prepared a tentative new proposal of some methodology to evaluate the long-period motions for new and existing high-rise buildings in December 2010 and invited the public comments from the MLIT (Tentative New Proposal by MLIT and NILIM).

These are the recent progress on the evaluation of long-period motions. Here, we would like to briefly show the project for developing the countermeasures for minimizing the effect of the long-period motions that MLIT and NILIM proposed.

5.2 Brief History of Design Earthquake Motions for Super High-rise Buildings in Japan

In Japan, the construction of high-rise buildings over 60 meters (hereafter, referred to as HR building) started in the late 60's. The recorded motions available for design at the time were very few both in number and quality. Therefore, an amplitude-magnified recorded motions were mainly used as well as the well-known records such as El Centro NS component from 1940 Imperial Valley earthquake or Taft EW component from 1952 Kern County earthquake that were already in use. The maximum amplitude levels used for magnification were 200-300 cm/s^2 for elastic design, and 300-500cm/s^2 for elastic-plastic design. Afterwards, the scaling with maximum velocity amplitude level was replaced as appropriate for relatively longer-period dominant ground motions. The scaling with maximum velocity amplitude with 25cm/s for damage-protection design and 50cm/s for collapse-protection was established in mid 80's. [5.5] The scaling scheme is still maintained for parts of the design motions. In mid-90's, a project left a fruit that proposed a design motions for the HR buildings. The project was conducted as a cooperative research work between the Building Center (BCJ) of Japan and the BRI of Japan. In the proposal, the design motion was defined at the outcropped hard soil surface that was referred to as 'Engineering Bedrock'. This motion is named as BCJ-wave. The pseudo velocity response spectrum is shown in Fig. 5.1. [5.6]

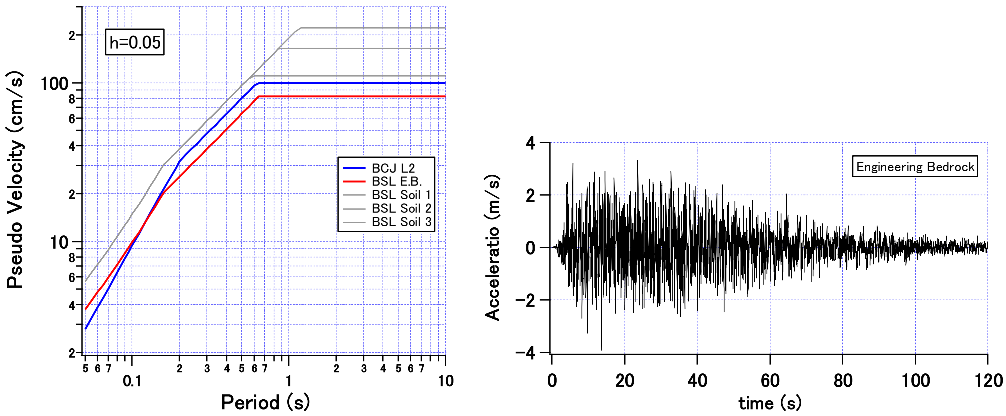


Fig. 5.1 BCJ (L2) and Building Standard Law Design Spectra and Building Standard Law Wave for Engineering Bedrock

The Building Standard Law of Japan was partly revised in 2000, consequently, the design spectrum for response history analysis was officially added as one of its notifications. The design spectrum was derived corresponding to the conventional design seismic force and was lower than the previously shown BCJ-wave spectrum as also shown in Fig. 5.1. In Fig. 5.1, an example of notification-spectrum compatible waveform is shown. The notification also requests site-specific motions referred to as 'site-wave' considering the earthquake environment of the construction site such as nearby influential active fault, etc. (CAO, 2008 [5.1], AIJ, 2007 [5.2], etc.)

The problems are summarized as follows;

(1) The longer-natural period buildings such as high-rise and seismically isolated buildings that have been thought as advantageous to strong earthquake motions turned to be vulnerable to long-period motions.
(2) The long duration time is important for buildings with low-damping and/or equipped with cumulative energy dissipation devices.
(3) Many such buildings have been built on large cities in Japan. Most of those are located on large deep sedimentary basins.
(4) Recorded long-period motions are still insufficient including building responses although the nationwide seismometer networks have been established after the 1995 Kobe earthquake.
(5) Urgently needed is the evaluation methodology of ground motions with large earthquakes expected to occur in near future for confirming the safety of many existing and new structures.

5.3 New Methodology for Design Long-period Ground Motions

The research on the evaluation of long-period motions has widely been conducted using theoretical method such as the 3D-FDM. On the other hand, the researches with the empirical evaluation of the long-period motions are very few to date. Kataoka (2008) [5.7] showed the attenuation formula for evaluating the response spectral properties. However, almost no research work has targeted on the time history generation.

Considering the usefulness of the formula and expecting the data accumulation in future, the empirical method will become much more useful in engineering sense. In addition, the evaluated motion with the empirical method will be useful enough to judge the plausibility of the theoretical method.

Here, we used nationwide many ground motion records to make an empirical model to predict the ground motion with 0.1 to 10 second period range. Furthermore, based on this formula, we investigated the method to construct the long-period ground motion time histories generated by hypothetical large future earthquakes. [5.8].

5.3.1 Attenuation formula for acceleration response spectra with 5% damping in longer period

The data used here are selected from JMA87, JMA95, K-NET, and KiK-net, etc.

The selecting criteria of records are as follows,
1) Subduction Type :Mj>6.5 for hypocentral distance<400km (Mj:JMA magnitude)
2) Crustal Type: Mj>6.0 for hypocentral distance<350km
3) Hypocentral depth<60km

The location of earthquake epicenters and their magnitudes are shown in Fig. 5.2. It is seen that the subduction type earthquakes are mainly on the Pacific Ocean side.

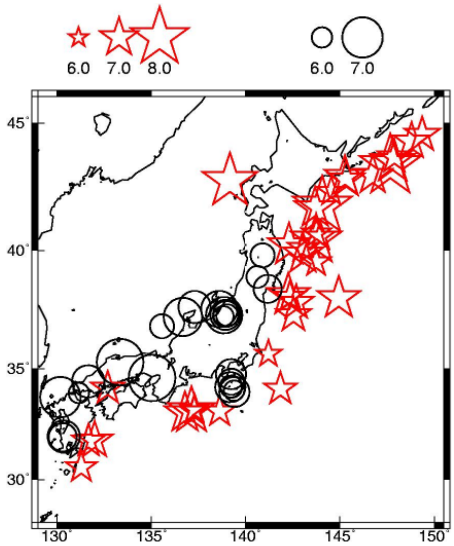


Fig. 5.2 Location of Earthquake Epicenters Used

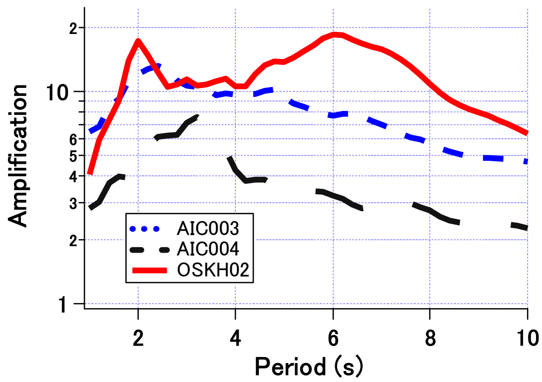


Fig. 5.3 Amplification Factors $c_j(T)$ of OSKH02 in Osaka, AIC003 and AIC004 in Aichi

In the least square analysis, the 5% damping acc. response spectra is related with the moment magnitude and the shortest distance from recording station to the assumed source area of each recorded event, i.e.,

$$\log_{10} S_a(T) = a(T)M_w + b(T)R \\ -\log_{10}(R^{p(T)} + d(T)10^{0.5M_w}) + c(T) + c_j(T)$$

where, M_w is the moment magnitude and R is the shortest distance in kilometer from the recording site to the source area, and a(T), b(T), d(T), p(T), c(T), cj(T) are coefficients to be determined with the least squares analysis. The coefficient c(T) is assumed to be the site amplification factor for KiK-net FKSH19 station which is regarded as benchmark station on the seismic bedrock and cj(T) is a site amplification factor for the j-th recoding station. The least square analysis was conducted separately for subduction and crustal earthquake type datasets. However, the final cj(T) was taken as a weighted average of cj(T) coefficients for both cases with data number taken as weight. Therefore, the cj(T) is treated as identical for subduction and crustal earthquakes. Fig. 5.3 shows the site amplification factors for three sites, OSKH02, AIC003, and AIC004. It is seen that the site amplification factor that shows the amplification from seismic bedrock to surface becomes nearly 20 at around 6 second for OSKH02 site, located at the coast of the Osaka Bay. The sites AIC003 and AIC004 are both in the Aichi Prefecture, but it is seen that the site amplification factors are different within the Nobi plain. In addition, the site amplification map for whole Japan area is also shown in Fig. 5.4. It is seen that the factor takes large value for area such as Kanto, Osaka, Nobi, Niigata, Sakata, Ishikari-Yufutsu, and Tokachi plains that holds thick overlying soil medium.

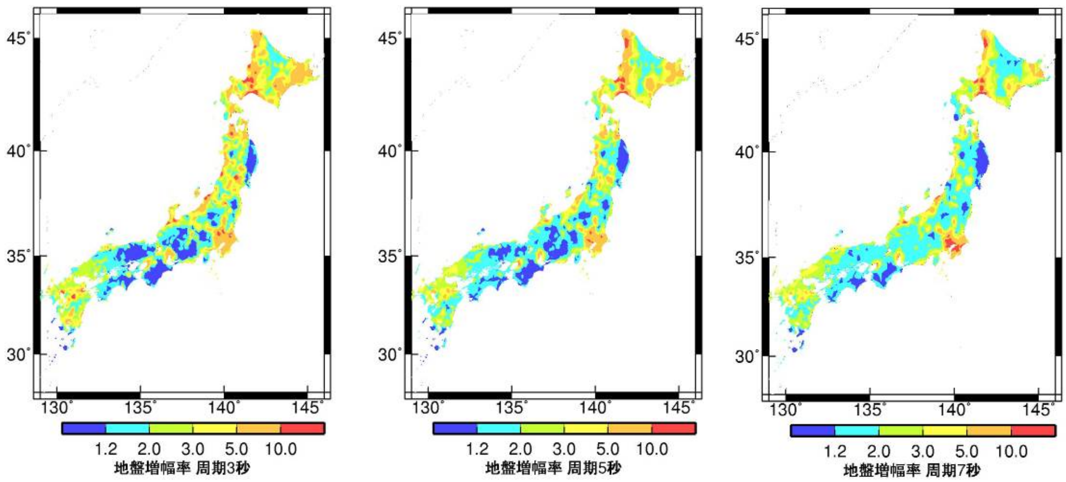


Fig. 5.4 Nationwide Distribution of Amplification Factors $c_j(T)$ with Periods of 3, 5, and 7 Second

5.3.2 Empirical formula for frequency-dependent average and variance of narrow-band group delay time

The average value μ_{tgr} of the group delay time corresponds to the gravity center of arriving time of wave group in a narrowband. The standard deviation σ_{tgr} of the group delay time corresponds to the scatter of the arriving time that is the duration time of the wave group in the narrowband. [5.9] Since the group delay time is the first derivative of the Fourier phase spectra, once the initial phase angle is fixed, the other phase angles are calculated recursively, assuming a normal distribution with the average and standard deviation values within the narrowband. The method holds an advantage to realize the spectral non-stationarity of the wave seemingly caused by the dispersion of surface waves. The average values are corrected so that the rupture initiation time should be zero.

Since both of the average group delay time μ_{tgr}, and the standard deviation σ_{tgr} of group delay time can be related with the source property, path effect and the site characteristic, both μ_{tgr} and σ_{tgr}^2 were eventually related in the following relationship.

$$Y(f) = A(f)M_0^{1/3} + B(f)X + C_j(f)$$

where, Y(f) is either μ_{tgr} or σ_{tgr}^{2}, M_0 is seismic moment in dyne-cm, X is the hypo-central distance in kilometer, the 'f' is frequency in Hz. A(f), B(f) and Cj(f) are determined by the least square analysis. The site coefficient for μ_{tgr} and σ_{tgr}^{2} with horizontal component for OSKH02, AIC003, AIC004 sites are shown in Fig. 5.5. It is seen from the figure, both site coefficients become larger for longer period.

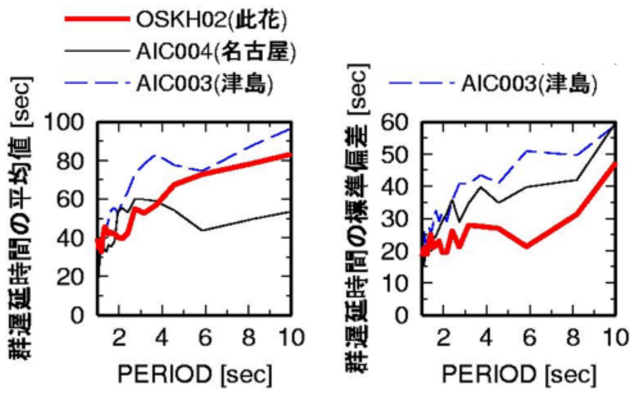


Fig. 5.5 Site Coefficient with Average (left) and Standard Deviation (right) of Group Delay Time

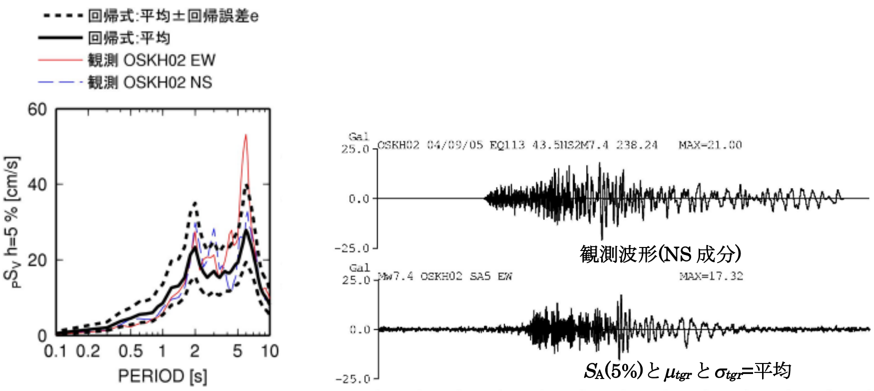


Fig. 5.6. Comparison of pSv and Waveform between Recorded and Simulated Motions for OSKH02

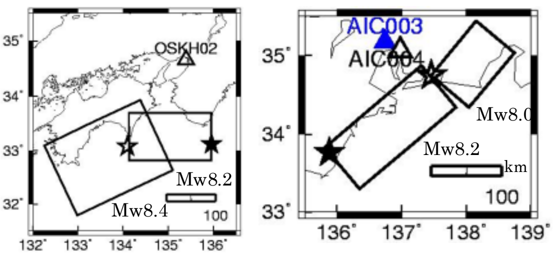
5.3.3 Generation of waves with empirical formula

For generation of the time history, the Fourier phase angles are firstly determined using the regression formula with μ_{tgr} and σ_{tgr} and giving initial phase angles and random numbers with normal distribution of μ_{tgr} and σ_{tgr}. Then, the 5% damping acc. response spectrum are determined with the attenuation formula. The Fourier amplitudes will be corrected so that the generated wave holds the acc. Response spectrum by correcting the Fourier amplitudes cyclically. [5.10]

At first, the method was examined with its validity by simulating the recorded motions during the 2004 Off-Kii-Peninsula earthquake (Mw=7.4). Fig. 5.6 shows the comparison of response spectra and waveform between recorded and generated waves for the 2004 earthquake for the OSKH02 site. The duration time of generated velocity waveform is less than the recorded, when the average values with the group delay time standard deviation is used. These comparison shows the method can reproduce the recorded motion when a regression error is taken into account appropriately.

We generated the waves for Nankai (Mw=8.5) earthquake with site OSKH02, and for Tokai-Tonankai (Mw=8.3) earthquake with sites AIC003 and AIC004 sites. The macroscopic fault plain modeled as a rectangle for Nankai earthquake [5.11] and the rupture initiation points and the location of the predicting site are indicated in Fig. 5.7(a). In Fig. 5.7(b), the rectangular fault plain model for Tokai-Tonankai earthquake based on Sato, et. al. (2006) [5.12] etc. are shown. When the multiple plain fault model is assumed, each generated wave from single component fault model will be added considering rupture time differences to get the final total waveform.

The pseudo response spectra for the Nankai earthquake using the method was shown in Fig. 5.8, and compared with the preceding simulations by Kamae (2006) [5.13], Tsurugi (2005) and Sekiguchi (2006) [5.14]. Kamae's result includes components longer than 2.5 second. Other simulations include short period components. The generated velocity waveforms are also shown in Fig. 5.9.



(a) Nankai Eq. (b) Tonankai-Tokai Eq.

★ First rupture, ☆ Second rupture

Fig. 5.7 Source Models used for Nankai and Tonankai-Tokai Earthquakes (Tsurugi, 2005, Sato, 2006)

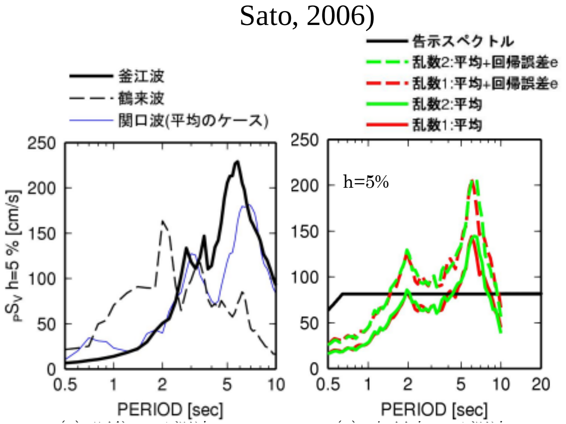


Fig. 5.8 Comparison of pSv with Other Research Results (left) and This Study (right)

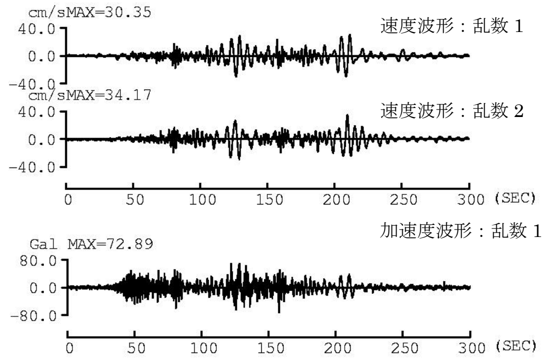


Fig. 5.9. Velocity (two random number cases) and Acceleration Waveforms with OSKH02 Site for Hypothetical Nankai Earthquake

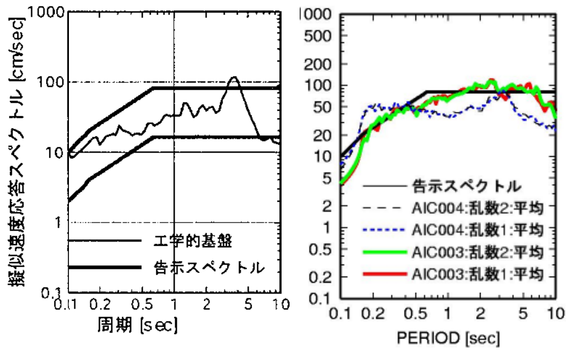


Fig. 5.10 Comparison of pSv's for Tonankai-Tokai Earthquake between Previous Study (left) with NST site at downtown Nagoya and this study (right) with AIC003 and AIC004 sites

In addition, the pseudo velocity response spectra for AIC003 and AIC004 for Tonankai-Tokai earthquake were compared in Fig. 5.10 with the simulated wave for NST site, located at downtown Nagoya that was estimated with so-called hybrid procedure. It is seen that the spectral levels at longer period for AIC004 and NST are comparable. In Fig. 5.11, the predicted waveforms for both sites are also compared. The maximum velocity amplitudes and the effective duration times for both waves are almost equivalent.

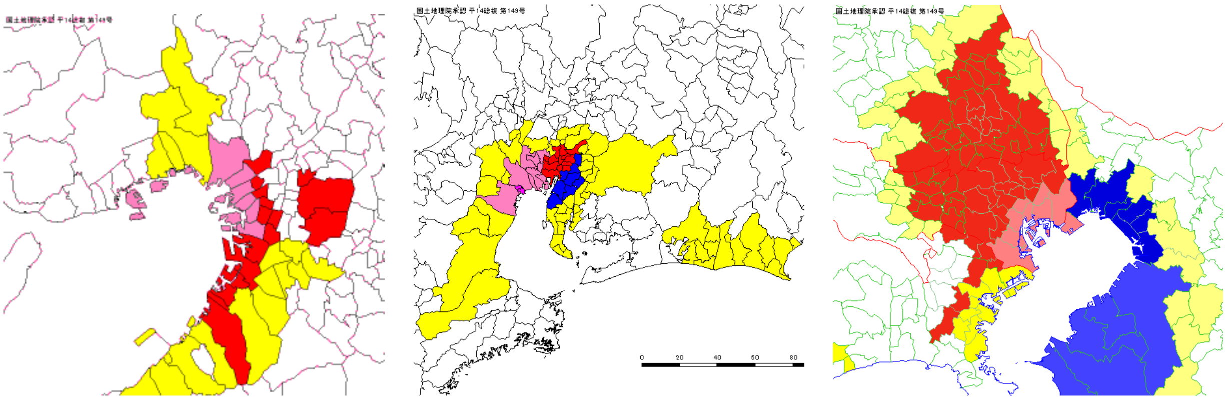


Fig.5.11 Selected areas from Tokyo, Aichi and Osaka and those area was divided into 9 sub-divided areas. Each sub-area is given design spectrum and corresponding time history to represent the sub-area.

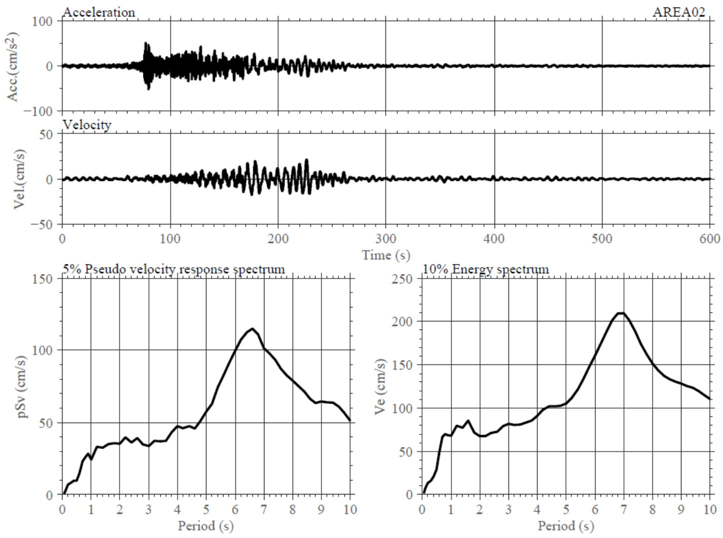


Fig.5.12 Assigned long-period motion for area No.2 in Tokyo corresponding to the pink area for the rightmost map in Fig. 5.11. The acceleration and velocity waveforms are given above. The 5% damping pseudo response spectrum and the 10% damping energy spectrum are given in the lower figure.

5.4 Tentative New Proposal for Countermeasures on Long-period Earthquake Motions by MLIT and NILIM

Based on the studies introduced in the preceding sections, the MLIT and the NILIM have made a so-called tentative new proposal on the countermeasures of seismic safety for high-rise buildings in December, 2010. This section briefly describes about it.

As explained in 5.2, the design earthquake motions for high-rise buildings in Japan have been changing to date. Therefore, the design motions for HRs in early days do not necessarily meet the standards of today, since there were not the concept of the design long-period motion at that time. Therefore, these buildings need to be verified their safety with the newly evaluated design motions.

5.4.1 Earthquakes, evaluation of long-period motions, and zonings

The Tonankai, the Tokai and the Miyagi-Ken-Oki earthquakes were selected for the evaluation of the long-period motions. The earthquake magnitudes and epicenters for these three earthquakes were based on the HERP's specification, with which they also made long-period motion maps.

The highly urbanized area in Tokyo, Aichi, and Osaka was selected and made 9 zonings were made that represent the surrounding area in view of the long-period motions as shown in Fig.5.11. The Tokyo Metropolitan area has 4 zonings that represent Tokyo Bay, Tokyo uptown, Yokohama and Chiba sub-areas. The Aichi area was represented by three zonings, the western, the central and the eastern sub-areas. Finally, the Osaka area was represented by two zonings, both facing on the Osaka Bay also shown in Fig. 5.11. Depending on the sub-area that each of the construction sites belongs, a long-period motion is assigned for the safety check. For example, area-2 in Tokyo (red area) is given the motion given in Fig. 5.12.

The 9 zonings are made specifically for the three earthquakes.

For earthquakes other than these, the proposal includes how to generate the long-period motions for the specific site.

5.4.2 Application of the tentative new proposals by the MLIT and the NILIM

For new high-rise buildings, re-evaluation of earthquake safety is requested using the assigned long-period motion from aforementioned three large earthquakes.

Some consideration is requested for protecting the furniture and utensils, such as copy machines from overturning.

Some technical information for the promotion with affordable design consideration will be provided, for excess long-period motions with multiple earthquake source events as well as the motions from single event mentioned above,

For existing high-rise or seismically isolated buildings that already acquired the minister's approval, the assigned motion is compared with the actually used design motions and if the assigned one even partly exceeds the used one, the building is requested for re-evaluation of the seismic safety with the newly assigned long-period motions and for strengthening if necessary.

5.4.3 The MLIT/NILIM proposals and re-start of the study

The MLIT/NILIM proposal immediately invited the public comments on December 21st until the end of February, 2011. Many comments were collected. The brief summary of the comments are disclosed by the MLIT. It says,

1. Agrees with the MLIT/NILIM actions for the long-period ground motions
2. Some other earthquakes that control the area such as the Nankai earthquake should be considered.
3. Some alleviated countermeasure or financial assistance will be recommended, because the retroactive application is too strict.
4. The design criteria in building performance should be presented as well as the input motions.

It is taken into consideration that these are summarized comments collected before the Tohoku earthquake occurred.

In addition, huge number of strong motion records is obtained by many public and private earthquake observation networks. Some verification study will be crucial using these data to enhance the reliability of the methodology that can account for the recorded data.

5.5 Conclusions

In this paper, a method was proposed that would produce broadband long-period (0.1-10 sec.) earthquake motion waveforms using earthquake parameters, such as, the seismic moment, the macroscopic fault plain model, rupture initiation position, time difference among ruptures for multiple events.

It was suggested that the empirical formula for long-period earthquake motions even in spectral property was very few, however, the method presented here will be useful in case of generating earthquake motion time history.

The simulated waves using the proposed method was in general comparable to the results in the preceding research results, although the number of the long-period earthquake records are in general very small and the situation makes the statistical study using those data unreliable and also that there are some cases in which the recording time is not sufficient for the purpose.
In view of that these research results will be applied to the specific field, seismic design of buildings, it is necessary to make further studies in parallel on the earthquake responses of the HR buildings and/or the base-isolated buildings to these simulated motions and their uncertainties.

In addition, the MLIT/NILIM tentative new proposal for the countermeasures for the long-period ground motions was introduced. The proposal will be upgraded using the new data from the Tohoku earthquake.

6. COPING ACTIVITIES BY NILIM AND BRI

From the Tohoku earthquake, the following issues are obtained to be countermeasured administratively for example by modifying current building technical standards.

(1) Evaluation of load effect by tsunami
(2) Countermeasure for falling down of non-structural elements, especially of ceilings
(3) Long-period earthquake ground motion countermeasure for buildings with long natural periods
(4) Liquefaction countermeasure for residential land of detached housings

In addition to above issues, cracks in the damping devices (lead damper) of the seismically isolated buildings were observed under a number of reversed cycles of loadings with small displacement amplitude even at places of rather small ground motions, with no severe damage in building structures. Lead rubber bearing which is made of rubber layers and stiffening plates with lead plug inserted is thought to be investigated like cracked exposed lead damper. The Japan Society of Seismic Isolation has started the survey and investigation on this issue on the request by MLIT and NILIM utilizing the grant-in-aid mentioned later, thus the progress of the study should be carefully observed.

The Building Structural Code Committee[5] chaired by Tetsuo Kubo, University of Tokyo, established in NILIM will investigate these issues as shown in Fig. 6.1. Where, MLIT and NILIM identify study items and invite those ("execution team") who serve as volunteer for the work on study items by utilizing the grant-in-aid [1.4] for maintenance and promotion of building codes. BRI is appointed to work together with the execution team.

[5] Draft of the building structural codes prepared by NILIM will be reviewed from the technical point of view by the Building Structural Code Committee. The modified draft by NILIM based on the review will be sent and will be put into Enforcement order or Notification by MLIT.

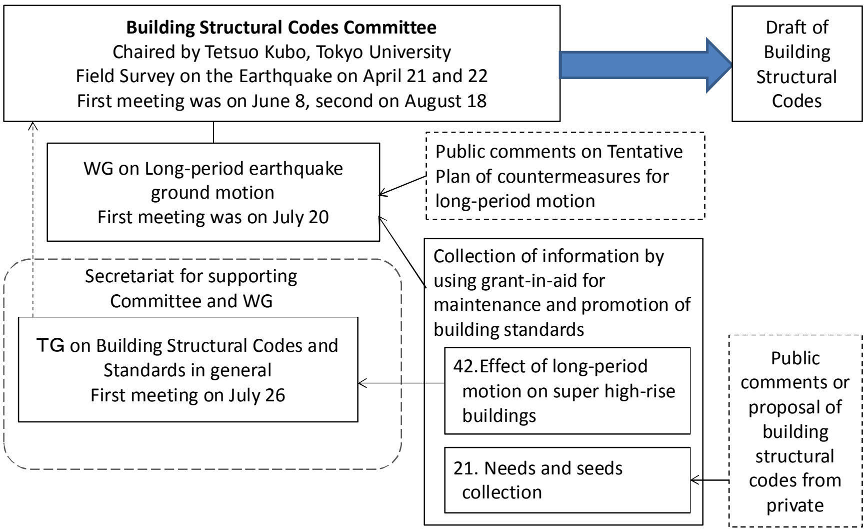


Fig.6.1 Building Structural Codes Committee

(1) Outline of evaluation of load effect by tsunami

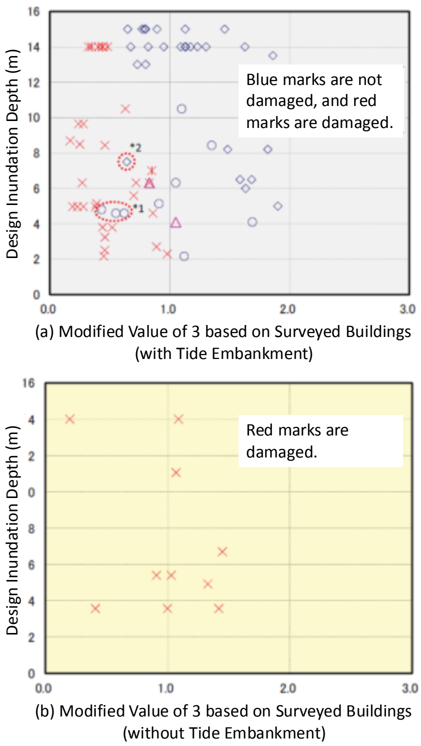
Miyagi Prefecture and Ishinomaki City assigned building restrictions in the damaged urban area based on the Article 84 of the Building Standard Law[6]. This assignment is active for 8 months at the maximum by the Special Law which came into force after the Tohoku earthquake. At the time of reconstruction of these areas, it is surly requested to reconstruct safe urban area even for another tsunami. Therefore, the technical information on the load effect by tsunami, which can be used in designing new buildings, is urgently in need.

Investigation on the building structural codes in the area of tsunami danger was started after the earthquake by the execution team of the Institute of Industrial Science, Tokyo University lead by Yoshiaki Nakano. The study items are 1) verification of structural design method for tsunami evacuation building, and 2) building restrictions, which should be, in the area of tsunami danger. In this section, 1) will be explained briefly.

In the existing guidelines [1.3] on the structural design method for tsunami evacuation buildings, as shown in Fig. 4.2, the tsunami load is estimated as the hydrostatic pressure acting on one side of the building whose inundation depth is 3 times the design tsunami inundation depth. Here, the value of 3 was determined from the laboratory test utilizing waterway, which was validated later by the field survey of tsunami damage by the 2004 Indian Ocean Tsunami Disaster [6.1]. However, the inundation depth of this time tsunami reaches about 15m or more which is far above the previous study and the topographical shape of rias coastline has not yet been verified. According to the interim report by the execution team, 44 examples of simple structures and 52 examples of buildings which subjected tsunami damage were collected and used for verification of the value. Fig. 6.2 shows the comparison of the measured tsunami inundation depth and the estimated value for simple structures and buildings with and/or without tide embankment etc. As seen from the figure, the value of 3 seems to be reduced drastically in case of with tide embankment, and the value becomes less than 1 in case of higher tsunami inundation depth such as over 10m. It is explained in the interim report that the load

[6] (Building Restrictions in Afflicted Urban Areas) Article 84. In case of a disaster in an urban area, the special administrative agency may, if it deems it necessary for the city planning or the land readjustment work under the Land Readjustment Law, designate areas and restrict or prohibit the construction of buildings therein, for a limited period of time within one month from the day of the occurrence of the disaster.

effect by tsunami is the largest not at the highest inundation depth, so the effective tsunami inundation depth can be reduced in case of high depth.



(a) Modified Value of 3 based on Surveyed Buildings (with Tide Embankment)

(b) Modified Value of 3 based on Surveyed Buildings (without Tide Embankment)

Fig.6.2 Comparison of the Measured Tsunami Inundation Depth and the Estimated Value

In the latter half of this fiscal year, study on the buoyant force and the pile resistance through the study on the overturned buildings, study on the effect of the openings of the buildings on tsunami force, and study on the impact force by the debris to the buildings will be carried out in detail. The report will be planned to be summarized by March, 2012.

(2) Outline of countermeasure for falling down of non-structural elements, especially of ceilings

After the 2001 Geiyo earthquake, in which many ceilings fell down in the large space gymnasiums stood in the urban park, the technical advice to keep appropriate clearance between ceiling and surrounding structure and to install braces on hanging bolts was send from MLIT to the special administrative agency[7]. An additional technical advice to keep clearance even between different stiffness ceilings was send after the 2003 Tokachi-Oki earthquake, and an appropriate strengthening of clip members which connect ceilings to hanging bolts was indicated after the 2005 Miyagi earthquake. Fig. 6.3 shows schematically these technical advices. In the figure, red circle potions should be taken care in the design of ceilings. From the fiscal year of 2008, vibration measurement, time history analysis, and vibration test using shaker on gymnasiums were performed in order to prepare concrete design manuals for ceilings against falling down.

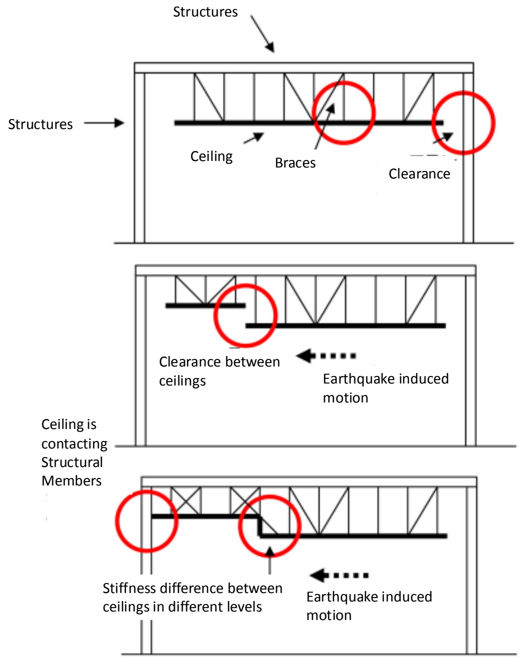


Fig.6.3 Schematic Explanation of Technical Advice on Details of Ceilings

In the Tohoku earthquake, building collapse was rarely observed but many ceilings falling down was reported at the places with not large earthquake motions and even death due to falling down of ceilings was occurred for example in the Imperial Crown Style building in Chiyoda Ward, Tokyo.

Therefore, in order to extensively study the ceiling falling down including effectiveness study on the existing technical advices, the

[7] The head of a city, town, or village for the area of a city, town, or village having building officials, or the prefectural governor for the area of other cities, towns, or village.

investigation on establishing building structural codes for non-structural elements based on the earthquake damage was initiated by the execution team of Building Performance Standardization Association. The study items are 1) collection and classification of ceiling damages in the Tohoku earthquake, and 2) building structural codes, which should be, for non-structural elements especially for ceilings. In the interim report, 151 examples of ceilings falling down are collected through the questionnaire to the special administrative agency and field survey was made on the selected 10 examples. Fig. 6.4 summarizes the shape of the damaged ceilings in these buildings.

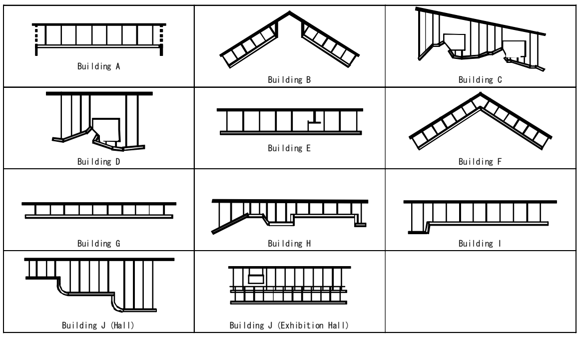


Fig.6.4 Patterns of Ceilings Surveyed

During the next six months after the interim report, concrete countermeasure methods for existing ceilings and safety calculation methods for new ceilings will be considered.

(3) Outline of long-period earthquake ground motion countermeasure for buildings with long natural periods

Prediction map of long-period earthquake ground motions in 2009 (draft) was announced from the HERP, MEXT, in which earthquake motions for expected Tokai earthquake, Tonankai earthquake, and Miyagi-Ken-Oki earthquake are predicted, which raised a social concern on the long-period earthquake ground motions caused by mega-earthquakes at the subduction zone near trench.

The MLIT and the NILIM has been working for the practical solutions for prediction of long-period earthquake ground motions based on observation [5.4] with the assistance by BRI, because the motions predicted by HERP just include the component of motions with 3.5 seconds or longer and so the higher mode of response of the buildings cannot be represented. The result was announced as the tentative new proposal [1.1] and public comments were collected. Many comments say that Nankai and combined Tonankai-Nankai earthquakes should also be included in the scope of the tentative new proposal as they affect more on the buildings in Nagoya and Osaka Areas than those in the current scope.

Meanwhile, the Tohoku earthquake is much larger than the prediction by HERP and good quality strong motion observations became available. Therefore, the investigation of the effect of long-period earthquake ground motions on the buildings with long natural periods is started by the execution team of Osaki Laboratory, Shimizu Cooperation.

The study items are 1) validation of prediction method (tentative new proposal) utilizing new observation data for long-period earthquake ground motions, 2) preparation of long-period earthquake ground motions at principal places for Nankai and combined Tonankai-Nankai earthquakes, and 3) earthquake response calculation of super high-rise buildings by the prepared motions. In this section, the outline of the validation study in 1) will be explained briefly.

As the Tohoku earthquake occurred in the northeastern part of Japan and the amplifying accretionary wedge[8] is not on the propagation path, the amplification of long-period component in Tokyo Metropolitan area is expected to be a bit small, which agreed with the observation results. Many good quality observed ground motions at various places can be used to verify the proposed site coefficient in the tentative new proposal for long-period earthquake ground motion prediction based on observation data. The response observations of super high-rise buildings in Tokyo, Nagoya and Osaka, can also be used for validation study. HERP is planning to

8 Thick sediment along Nankai Trough is thought to amplify the long-period component of ground motion.

develop source model even for combined earthquakes by the next spring, which can be made use of brushing up the tentative new proposal.

(4) Outline of liquefaction countermeasure for residential land of detached housings

Structural calculation is released for wood houses in the Building Standard Law. Thus, the liquefaction countermeasures cannot be considered in the building construction for the detached housings. In fact, it was thought the liquefaction countermeasure is the issue of the land development and not of the housing construction.

To answer for the social demand in the consumer protection on liquefaction of residential land, a study is planned on the application of the system of performance indication[9] of detached housings.

It is the investigation on indicator of liquefaction information for detached housings, and the study items are 1) study on the validation of existing liquefaction prediction methods and countermeasuring construction methods, 2) study on the information indicator for liquefaction, and 3) knowledge and information collection on liquefaction prediction and countermeasure methods in the previous investigation and technology development. The execution team is now under the selection as of August 19, 2011.

The liquefaction phenomenon is a big issue not only for detached housings but also for civil infrastructures related to sewerage, river, road, and harbor. In MLIT, technical examination council [6.2] on liquefaction countermeasures was established and the technical information in each field is continuously exchanged. Urayasu where subject to devastating liquefaction damage established technical examination and investigation council [6.3] on liquefaction countermeasures, and Tokyo Metropolitan also established similar council [6.4]. Each council has just started its study, but it is expected that the technical knowledge obtained in each council be unified into common goal. Here, it should be noted that the Swedish weight sounding test method may be the available foundation investigation tool considering the cost in the field of residential land, thus the technical information in the civil infrastructure field need some translation as the investigation tool is generally in different.

7. CONCLUSIONS

This paper presents outlines of the strong motion observations recorded by the BRI strong motion observation network etc. at in and out of the buildings of various structural types, and of the motion induced and tsunami induced building damage field-surveyed by NILIM and BRI in the occasion of the 2011 off the Pacific coast of Tohoku earthquake (the Great East Japan Earthquake). “Tsunami”, “non-structural elements”, “long-period earthquake ground motion”, and “liquefaction” are identified as important administrative issues to be urgently countermeasured in the building structural codes, and lastly the outline of on-going coping activities on these issues by NILIM and BRI collaborated with the administration is introduced.

8. ACKNOWLEDGMENTS

We would express deepest condolence to the victims of the earthquake and tsunami and their family members as well as those who are affected. We also would like to give hearty thanks for the outpouring of support and solidarity given by all parts of the world to the Japanese people.

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