Response Values of Earthquake Waves and Structural Assessment of Cultural Heritage

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SUMMARY:
This paper describes the response values of earthquake waves in "the Great East Japan Earthquake" in 2011 and some other earthquakes. In order to know generally how much displacements took place in buildings by earthquakes in many countries, the response values are calculated by non-linear dynamic analysis with the period from 0.1 (sec) to 5.0 (sec), and the shear coefficient of the yielding point equal to 0.2 and 0.1. The hysteresis property is the Peak Oriented Bi-Linear.
Structural assessments of cultural heritages were executed. The cultural heritages were the wooden 2 storeys house with a brick footing and another. In these assessments, the hysteresis properties of each storey were clarified in the values of each mass, stiffness, and shear coefficient of yielding point. So, the author made the non-linear dynamic analysis about the shear system of 3 or 4 mass. Among the earthquake records, the ones which calculated larger response displacement or are close to the cultural heritage were selected
The resolutions are that the response values of some earthquake waves in the 2011 Great East Japan Earthquake are calculated, and so on.

Keywords: response values, earthquake waves, cultural heritage

1. INTRODUCTION

Recent years, many earthquakes occurred and caused severe damage. In these earthquakes, many earthquake waves are recorded, for example, the Great East Japan Earthquake in 2011, South Island of New Zealand in 2011, Offshore BIO - BIO, Chile in 2010, etc. These earthquake waves are very helpful for the structural assessment of buildings including cultural heritages. The reference showed the results of the inelastic dynamic response analysis by Single Degree of Freedom (SDOF). In this paper, the hysteresis property is the Peak Oriented Bi-Linear. A cultural heritage of the wooden 2 storey house with a brick footing was retrofitted and the hysteresis properties of each storey were clarified. Another monument is under retrofitting. This paper describes the results of the inelastic dynamic response analysis by Shear System of 3 Mass and 4 Mass which are not mandated. In these analyses, the matrix equations for the forced vibrations with the inertia damping are solved. The hysteresis properties are also the Peak Oriented Bi-Linear on each storey same as that of SDOF.

2. RESPONSE VALUES

2.1. Earthquake Waves

The earthquake waves are showed in Table 2.1 and Table 2.2. These tables show Earthquake Names, Earthquake Waves Names which mean the measured point names, Direction measured in horizontal, Peak Acceleration which means the maximum of absolute value of strong motion data, Epicentral distance, Date and time in local time, and Remarks of Courtesy. The records of Table 2.1 are used for the calculation with the hysteresis property when the shear coefficient qCy is 0.2. This value of qCy is adopted to the base shear coefficient in the seismic design code, Japan. The records of Table 2.2 are used for the calculation about the cases of qCy(=0.1). This value of qCy is nearly equal to the values

of the seismic design codes in Mexico, USA and other countries.
The records are not corrected in the response analysis.

Table 2.1. Earthquake Waves for qCy* (= 0.2)

Earthquake Names [Earthquake Waves Names in Fig. 2.3.]	Direction (NS: North and South, EW:East and West)	Peak Acceleration (cm/sec^2)	Epicentral distance (km)	Date and Time in Local Time	Remarks of Courtesy for Strong Motion Data
2011 Great East Japan Earthquake					
[2011 THU-1F N192˚E, BRI]	N192˚E*	259	177	11 March 2011. 14:46	BRI
[2011 SND-B2F N164˚E, BRI]	N164˚E	333	175		
[2011 Tsukidate NS, K-NET]	NS	2699.9	175		K-NET, NIED*, Japan
[2011 Sendai NS, K-NET]	NS	1517.2	170		
[2011 Osaki NS, JMA]	NS	549.6	173.8		JMA*
[2011 Wakuya EW, JMA]	EW	438.7	158.5		
2007 Niigata-ken Chuetsu-Oki Earthquake [2007 Kashiwazaki NS, K-NET]	NS	667.9	21.3	16 July 2007. 10:13	JMA
2004 Niigata-ken Chuetsu Earthquake [2004 Ojiya EW, K-NET]	EW	1313.5	7.0	23 Oct. 2004. 17:56	K-NET
1995 Hyogo-ken Nanbu Earthquake [1995 Kobe NS, JMA]	NS	818.0	16.5	17 Jan. 1995. 05:46	JMA
BCJ Level2 <Artificial Earthquake Acceleration Data on Bedrock > [BCJ Level2 (Bedrock)]	—	355.7	---	---	Building Center of Japan

Notes) 1)qCy* : Design Shear Coefficient at Yielding Point referred to Section 3.
2) N192˚E* : Direction in 192 degrees from North to East.
3)NIED* : National Research Institute for Earth Science and Disaster Prevention
4)JMA* : Japan Meteorological Agency

Table 2.2. Earthquake Waves for qCy (= 0.1)

Earthquake Names [Earthquake Waves Names in Fig. 2.4.]	Direction (NS: North and South, EW:East and West)	Peak Acceleration (cm/sec^2)	Epicentral distance (km)	Date and Time in Local Time	Remarks of Courtesy for Strong Motion Data
2011 New Zealand Christchurch Earthquake [2011 HVSC S26W, GeoNet]	S26W*	1646.8	2	22 Feb. 2011. 12:51	GeoNet New Zealand
2010 Chile Earthquake					
[2010 Angol NS, UCS]	NS	910.4	209.3	27 Feb. 2010. 03:34	UCS, Chile CESMD
[2010 CCSP NS, UCS]	NS	639.0	109.1		
[2010 Cons EW, UCS]	EW	627.5	69.7		
[2010 Conc NS, UCS]	NS	394.6	82.4		
2009 Italy L'Aquila Earthquake [2009 AQV EW, Itaca]	EW	662.6	4.9	06 Apr. 2009. 03:32	Itaca, DPC, Italy
2005 Pakistan Earthquake [2005 Abbottabad EW, MSSP]	EW	226.4	50	08 Oct. 2005. 08:50	MSSP-PAEC Pakistan
2003 Algeria Earthquake [2003 Dar El Beida EW, CGS]	EW	537	25	21 May 2003. 19:44	CGS, Algeria
2001 El Salvador Earthquake [2001 La Libertad NS, UCA]	NS	1154.6	75	13 Jan. 2001. 11:33	UCA, San Salvador El Salvador
1994 Northridge Earthquake [1994 Tarzana EW, COSMOS]	EW	1744.5	16.7	17 Jan. 1994. 04:30	COSMOS, USA
1985 Michoacan Mexico Earthquake [1985 Mexico City EW, SCT]	EW	167.9	400	19 Sep. 1985. 07:17	SCT, Mexico
1940 Imperial Valley Earthquake [1940 Elcentro NS, BCJ]	NS	341.7	7～15 (predicted)	18 May 1940. 20:37	BCJ

Notes) 1)S26W : Direction in 26 degrees from South to West
2) GeoNet : Geophysical Networks for Monitoring geological hazards in New Zealand
3) UCS : the University of Chile, Civil Engineering Dept., Santiago, Chile
4) CESMD : Center for Engineering Strong Motion Data
5) Itaca, DPC : ITalian ACcelerometric Archive, Department of Civil Protection of Italy
6) MSSP-PAEC : Micro Seismic Studies Programme, Pakistan Atomic Energy Commission
7) CGS : National Earthquake Engineering Center, Algeria
8) UCA : the University of Central America, San Salvador, El Salvador
9) COSMOS : Consortium of Organizations for Strong-Motion Observation Systems
10) SCT : Ministry of Transportation and Communications, Mexico

2.2. Inelastic Dynamic Response Analysis Method

The following is the overview of the inelastic dynamic response analysis method.

The equation of motion about SDOF (Fig. 2.1) is the following Equation (2.1).

$$m\ddot{x} + c\dot{x} + kx = -m\ddot{y} \qquad (2.1)$$

where

m : mass(N/(cm/sec^2))
c : damping factor(N/(cm/sec))
k : stiffness(N/cm)
x : relative displacement(cm)
$\dot{x}$: relative velocity(cm/sec)
$\ddot{x}$: relative acceleration(cm/sec^2)
$\ddot{y}$: horizontal acceleration record on the ground surface (cm/sec^2)

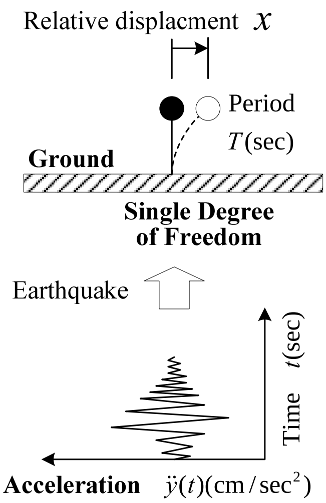
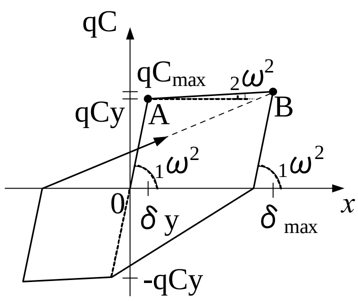


Figure 2.1. Modelling of SDOF

The hysteresis property is the Peak Oriented Bi-linear model in Fig. 2.2. In this model, the shear coefficient qCy at the yielding point is assumed to be 0.2 or 0.1 , the initial ω^2 is assumed to be ${}_1\omega^2$, and the ω^2 after yielding is assumed to be ${}_2\omega^2$ (= $\frac{1}{1000} \times {}_1\omega^2$) which is the degraded stiffness. The value of damping coefficient h is 0.05.



Notes)
(1) qC : Shear Coefficient
$$\left(\mathrm{qC} = \frac{\text{Storey Shear Force (Q)}}{\text{Weight } (m \cdot g)} \right)$$
(2) qCy : Shear Coefficient at the yielding point of A
(qCy = 0.2, or 0.1)
(3) qC_{max} : Shear Coefficient at the peak of B
(4) Q : Storey Shear Force (N) ($= k \cdot x$)
(5) Stiffness Degrading Ratio : 1/1000
$$\left({}_2\omega^2 = \frac{1}{1000} \times {}_1\omega^2 \right)$$
(6) $${}_1\omega^2 = \frac{\text{Initial Stiffness } ({}_1k)}{\text{Mass } (m)}$$
(7) h : Damping Coefficient (= 0.05)
(8) g : Gravity Acceleration (=980 (cm/sec^2))

Figure 2.2. Hysteresis property of Peak Oriented Bilinear Model

2.3. Results of Analysis

Many of earthquake waves are measured and opening to public on Web by BRI, K-NET, JMA, GeoNet and others. In this section, in order to understand how much effects on structures the earthuakes make, the inelastic response spectra were calculated by the inelastic dynamic response analysis for the SDOF. Fig. 2.3 and Fig. 2.4 show the inelastic response spectra of the relative displacement, the shear coefficient, the relative velocity, and the absolute acceleration, by calculation parameter of qCy (= 0.2 and 0.1 respectively) which means the design shear coefficient at yielding point referred to Section 2.2.

Generally, the response displacement of time history analysis is said to be equivalent to the relative displacement between storeys of buildings. The response displacement of SDOF is considered to be the relative displacement of 1 storey building. In this section, the analysis were executed in the period from 0.1 (s) to 5.0 (s), and the spectra show the maximum values of the absolute values in each time history analysis.

The earthquake waves were affected by the ground condition and the structures around the measuring points. On the soft soil condition, the response displacement is considered to be larger in the range of long period.

By the way, according to the building seismic design code in Japan, the required peak response relative displacement at storeys should be less than a value in the time history analysis of high-tall building design or the limit capacity calculation. The time history analysis of high-tall building design of the code usually requires the values of storey drift during earthquake less than 1/100 rad. The limit capacity calculation of the code requires the values of storey drift during earthquake less than 1/75 (rad.) for the other than wooden structures and 1/30 (rad.) for the wooden structures. For example, when the storey height is 3 (m), the required peak response relative displacement at storeys under the state of the collapse limit should be less than 3 (cm) or the displacement under the state of the limit capacity calculation should be less than 4 (cm) for the other than wooden structures and 10 (cm) for the wooden structures.

In this analysis, the earthquake waves are used, among the points of the larger peak acceleration record or the larger response displacement of the earthquakes. Each earthquake wave has the 3 directions acceleration records and the spectra in Fig. 2.3 and Fig. 2.4 show the analysis results of the 1 larger horizontal acceleration record or the 1 larger response displacement of the 2 horizontal directions.

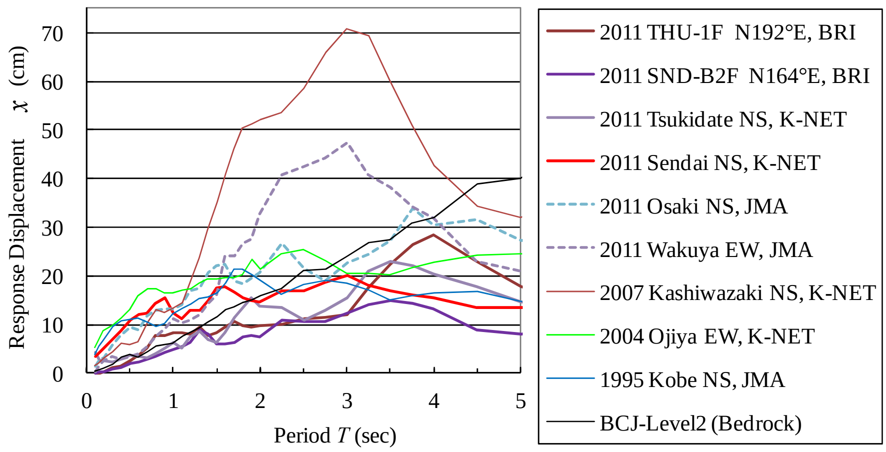


Fig. 2.3-1. Relative Displacement (x)

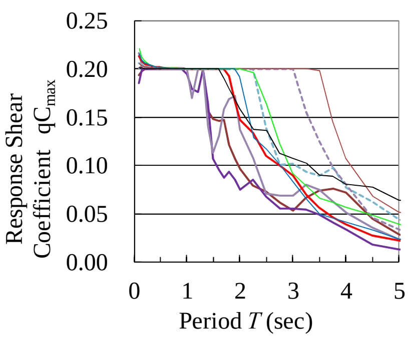


Fig. 2.3-2. Shear Coefficient (qCmax)

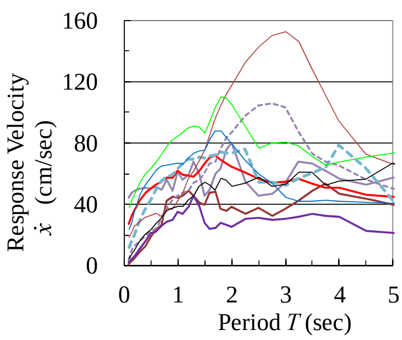


Fig. 2.3-3. Relative Velocity ($\dot{x}$)

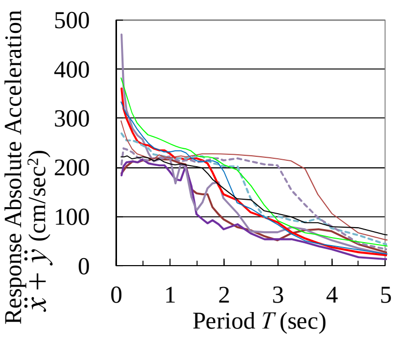


Fig. 2.3-4. Absolute Acceleration ($\ddot{x}+\ddot{y}$)

Figure 2.3. Inelastic Response Spectra (qCy=0.2, h=0.05)

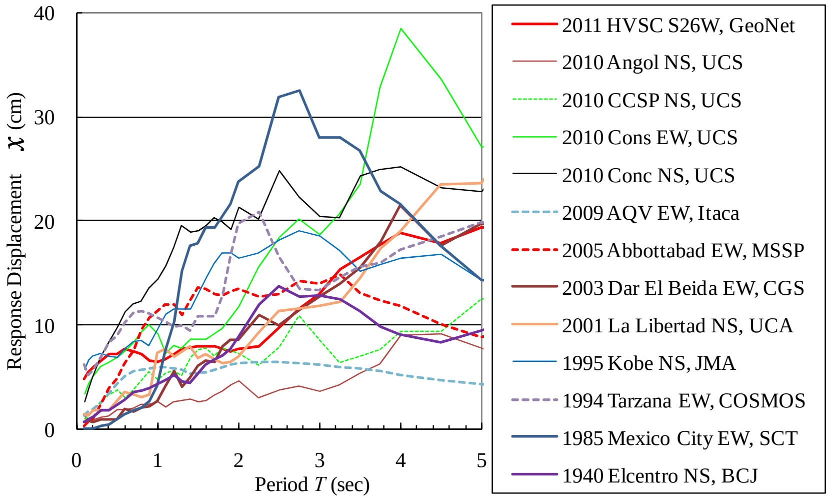


Fig. 2.4-1. Relative Displacement (x)

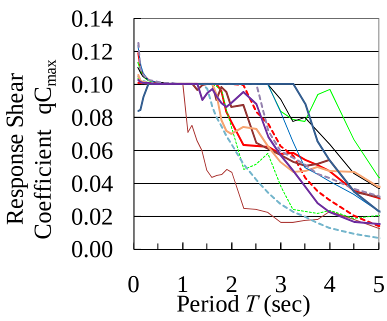


Fig. 2.4-2. Shear Coefficient (qCmax)

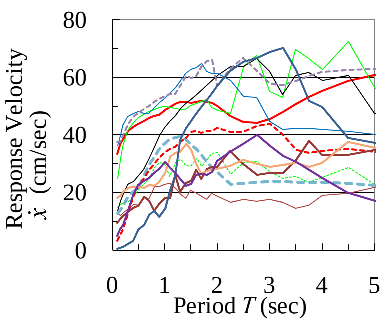


Fig. 2.4-3. Relative Velocity ($\dot{x}$)

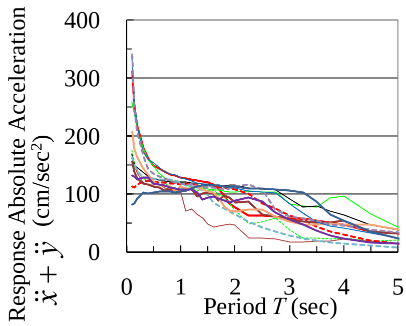


Fig. 2.4-4. Absolute Acceleration ($\ddot{x}+\ddot{y}$)

Figure 2.4. Inelastic Response Spectra (qCy=0.1, h=0.05)

According to Fig. 2.3-1, when the period is about 3 (sec), the maximum of inelastic response relative displacement is analyzed to be about 71 (cm) of [2007 Kashiwazaki NS, K-NET] and the second largest one is about 47 (cm) of [2011 Wakuya EW, JMA].
These measuring points are placed on the soft soil condition and it is one of reason of larger response displacement. When the period is less than 1 (sec), the maximum displacement is about 17 (cm) of [2004 Ojiya EW, K-NET]. According to Fig. 2.3-3, when the period is about 3 (sec), the maximum of inelastic response relative velocity is about 153 (cm/sec) of [2007 Kashiwazaki NS, K-NET].

The response displacements by the earthquake waves of the 2011 Great East Japan Earthquake are less than the others. It is one of reason because the epicentral distances of the 2011 Great East Japan Earthquake are longer than others.

According to Fig. 2.4-1, when the period is about 1 (sec), the maximum of inelastic response relative displacement is analyzed to be about 15 (cm) of [2010 Conc NS, UCS]. When the period is about 2.5 (sec), the maximum is about 32 (cm) of [1985 Mexico City EW, SCT] and when the period is about 4 (sec), the maximum is about 38 (cm) of [2010 Cons EW, UCS].
Comparing the results of Fig. 2.4-1 by the Peak Oriented Bilinear Model and the another results by the Bilinear Model of 14WCEE, the typical differences are shown in [1995 Kobe NS, JMA] at the period of 0.5 (sec) which displacement is decreased from 15 (cm) of the Bilinear Model to 7 (cm) of the Peak Oriented Bilinear Model. That is one of reason because the Peak Oriented Bilinear Model has the many periods during analysis and that decreased the resonance amplitude.

2.4. Comparison between Main Shock and Aftershocks

The 2011 Great East Japan Earthquake has many aftershocks more than 650 times of magnitude 5 and more. Among these aftershocks, some people living in Sendai City said that the aftershock which occurred on 23:32 local time at 7 April 2011 of Magnitude 7.1, was similar shaking to the main shock.

So, the spectra of inelastic response relative displacement in this aftershock are compared with the spectra in the main shock. The points are [Sendai (MYG013)] of K-NET and [Shinmachi, Wakuaya Town] of JMA. In the notes of Fig. 2.5, the dotted line with the name of "0311" shows the spectra of the main shock and the full line with the name of "0407" shows the spectra of the aftershock. According to Fig. 2.5, in the almost periods, the response displacement of the main shock is larger than that of the aftershocks. In some periods, that of the aftershocks is equal or larger than that of the main shock.

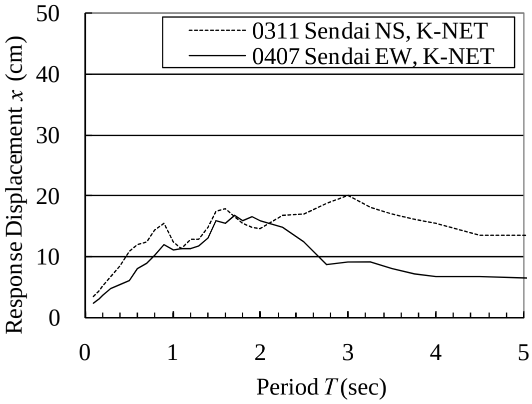


Fig. 2.5-1. 2011 Sendai, K-NET

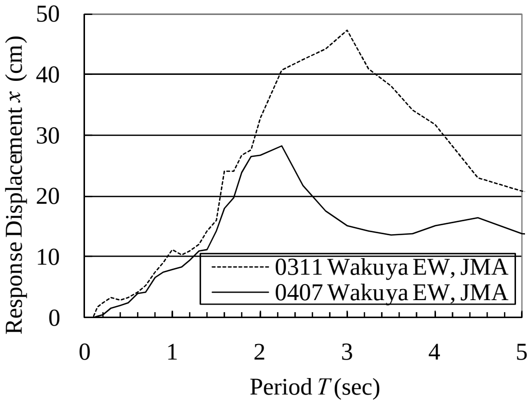


Fig. 2.5-2. 2011 Wakuaya, JMA

Figure 2.5. Comparison of Inelastic Response Spectra in Relative Displacement between Main shock and Aftershock in the 2011 Great East Japan Earthquake (qCy=0.2, h=0.05)

3. STRUCTURAL ASSESSMENT

3.1. Former Kagoshima Spinning Engineer's Residence

Former Kagoshima Spinning Engineer's Residence (*Ijinkan*) is an Important Cultural Property of Japan and it is a 2 storey wooden house with the brick footing (Photo 3.1). According to Manual for the Seismic Safety Assessment of the Important Cultural Properties (Building and other structures), *Ijinkan* had the structural assessment and the retrofitting.

3.1.1. Structural assessment for Ijinkan

The structural assessment for *Ijinkan* was executed (Table 3.1). It had 2 steps of the modelling from each parts to the storey stiffness and strength, and the limit capacity calculation by the equivalent SDOF. At the results, the required seismic safety level was not satisfied and *Ijinkan* needed to be retrofitted. Retrofitting method is the adding wooden shear walls in the wooden lathing walls (Photo 3.2), roofs and floors. And the brick footing is also retrofitted by the reinforced concrete. Finally, the required seismic safety level is satisfied by this retrofitting which means that Storey Drift Angle should be less than 1/60(rad.) during a moderate earthquake and 1/30(rad.) during a major earthquake.

3.1.2. Inelastic dynamic response analysis for Ijinkan

In the structural assessment, the hysteresis property of each storey of *Ijinkan* before and after retrofitting was clarified. According to the hysteresis property, inelastic dynamic response analysis for *Ijinkan* was made.

Photo 3.1. Overview of *Ijinkan* from the North-West

Photo 3.2. Retrofit works of *Ijinkan* (Adding wooden shear walls between studs, inside of wooden lathing walls)

Table 3.1. Overview of Structural Assessment and Retrofit

Structural Assessment or Retrofit	Method
Analysis Method 1 : Analysis Model 1 : Results 1 :	Static 3D Frame Analysis Wooden Lathing Walls simplified to Brace Studs inside of Wooden Lathing Walls not to be considered Joints of columns and beams, and support points assumed to be pin joints Roofs and floors assumed to be rigid plane Storey stiffness and strength of each storey Period T > 1.5 (sec), Shear coefficient qCy < 0.1 on 1st and 2nd floor.
Analysis Method 2 : Analysis Model 2 : Required Seismic Safety Level Results 2 :	Limit Capacity Calculation Equivalent SDOF Seismic Safety without Collapse (Storey Drift Angle should be less than 1/60(rad.) during a moderate earthquake and 1/30(rad.) during a major earthquake) Storey Drift Angle was calculated more than 1/60(rad.) during a moderate earthquake and 1/30(rad.) during a major earthquake.
Retrofit :	Adding wooden shear walls in wooden lathing walls, roofs, and floors. Adding reinforced concrete to footing. Period T < 1.0 (sec), Shear coefficient qCy > 0.2 on 1st and 2nd floor.
Results 3 :	Storey Drift Angle was calculated less than 1/60(rad.) during a moderate earthquake and 1/30(rad.) during a major earthquake.

The analysis model is the shear system of 4 mass shown as Fig. 3.1. This analysis uses the external damping force. The matrix equation of motion in forced vibration with external damping is shown in the equation (3.1). This equation is described to the other equation (3.2) using the n-by-n matrix $[A]$, and n-by-1 matrix $\{a\}$. This equation (3.2) is the linear ordinary differential equation of second order. This general solution $\{x_t\}$ is the complementary function $\{x_c\}$ and particular solution $\{x_p\}$(the equation (3.3)). $\{x_c\}$ is solved by the reduction form of the equation (3.4) which gives the eigenvalue $\{\mu\}$ and the eigenvector $[Z]$. Usually, $\{\mu\}$ and $[Z]$ are complex numbers. The linear combination of these complex numbers and their conjugate complex numbers give the complementary function $\{x_c\}$. $\{x_p\}$ is always solved as same as the method of SDOF and $\{x_t\}$ is also solved by the Dummy Variable Method.

Usually, when $\{\mu\}$ and $[Z]$ are solved, the size of each eigenvector of $[Z]$ is 1. But in this analysis, the size of $[Z]$ are calculated by using the Dummy Variable Method. Moreover, some of $\{\mu\}$ and $[Z]$ are sometime real numbers when the some element numbers of the stiffness matrix are very different from the other element numbers while some storey stiffness are degrading. Finally, in the calculation of $\{x_t\}$, the coupling system of the equation (3.2) are always maintained, by using many of inverse matrix multiplied from the left side of this equation (3.2). All of these inverse matrixes always existed. The value of damping coefficient h is 0.05.

The shear system of 4 mass with the wooden 2 storeys, the roof and the footing shows the clear differences for the response values between the not-retrofitted and the retrofitted. In this analysis, the earthquake waves of 1995 Kobe Earthquake, 2007 Chuetsu-Oki Earthquake and 2011 Great East Japan Earthquake are used in order to compare with the response values. And according to the Fig. 3.2, the response displacement of the retrofitted can follow the structural assessment guideline for cultural heritages.

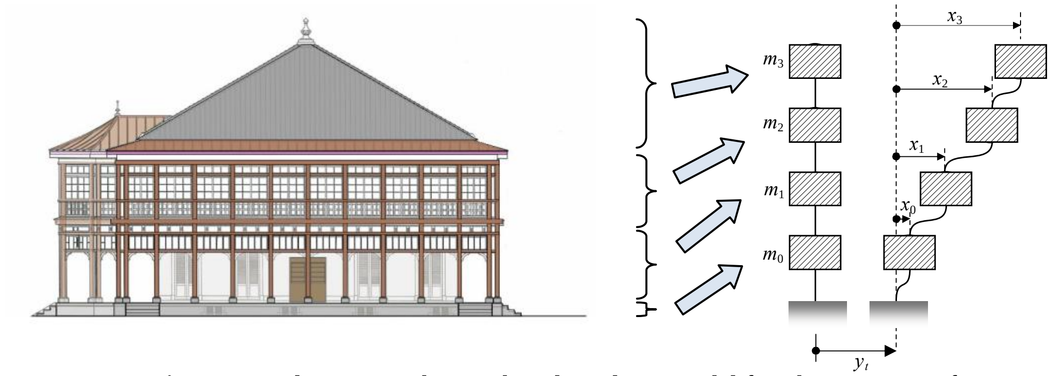


Figure 3.1. Elevation in the North and Analysis Model for Shear System of 4 Mass

$$\begin{bmatrix} m_3 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 \\ 0 & 0 & m_1 & 0 \\ 0 & 0 & 0 & m_0 \end{bmatrix} \begin{Bmatrix} \ddot{x}_3 \\ \ddot{x}_2 \\ \ddot{x}_1 \\ \ddot{x}_0 \end{Bmatrix} + \begin{bmatrix} c_3 & 0 & 0 & 0 \\ 0 & c_2 & 0 & 0 \\ 0 & 0 & c_1 & 0 \\ 0 & 0 & 0 & c_0 \end{bmatrix} \begin{Bmatrix} \dot{x}_3 \\ \dot{x}_2 \\ \dot{x}_1 \\ \dot{x}_0 \end{Bmatrix} + \begin{bmatrix} k_3 & -k_3 & 0 & 0 \\ -k_3 & k_3+k_2 & -k_2 & 0 \\ 0 & -k_2 & k_2+k_1 & -k_1 \\ 0 & 0 & -k_1 & k_1+k_0 \end{bmatrix} \begin{Bmatrix} x_3 \\ x_2 \\ x_1 \\ x_0 \end{Bmatrix} = -\begin{bmatrix} m_3 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 \\ 0 & 0 & m_1 & 0 \\ 0 & 0 & 0 & m_0 \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \\ 1 \\ 1 \end{Bmatrix} \ddot{y}_t \tag{3.1}$$

where

Period of i-th Storey : $T_i = 2\pi \sqrt{\dfrac{\sum_{j=i}^{n} m_j}{k_i}}$ [sec]

Damping Factor of i-th Storey : $c_i = 2h\omega_i m_i = 2hm_i \sqrt{\dfrac{k_i}{\sum_{j=i}^{n} m_j}}$ $\left[\mathrm{N}\dfrac{\mathrm{sec}}{\mathrm{cm}}\right]$

$$[A] = \begin{bmatrix} a_{11} \cdots a_{1j} \cdots a_{1n} \\ \vdots \quad\quad \vdots \quad\quad \vdots \\ a_{n1} \cdots a_{nj} \cdots a_{nn} \end{bmatrix} : \text{n-by-n matrix}, \quad \{a\} = \begin{Bmatrix} a_1 \\ \vdots \\ a_n \end{Bmatrix} : \text{n-by-1 matrix}$$

$$[M]\{\ddot{x}_t\} + [C]\{\dot{x}_t\} + [K]\{x_t\} = -\ddot{y}_t\,[M]\{1\} \tag{3.2}$$

$$\{x_t\} = \{x_c\} + \{x_p\} \tag{3.3}$$

$$[M]\{\ddot{x}_c\} + [C]\{\dot{x}_c\} + [K]\{x_c\} = -\ddot{y}_t\,[M]\{0\} \tag{3.4}$$

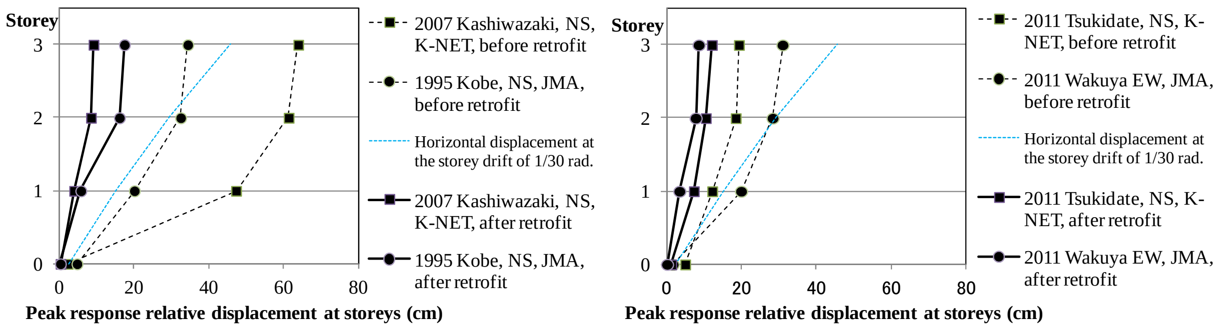


Figure 3.2. Results of inelastic dynamic response analysis for *Ijinkan* in EW direction

3.2. Sant'Agostino in L'Aquila, Italy

In the 2009 Italy L'Aquila Earthquake, Abruzzo, Italy, many churches are damaged. Ministry of Cultural Heritage and Activities, and Department of Civil Protection, Italy listed 45 monuments under retrofitting. The complex monument and church of Sant'Agostino in L'Aquila, Italy is one of these monuments and the very valuable chance was given to see the damage of it in L'Aquila and have some references of it (Photo 3.3). The typical damage of Sant'Agostino is the lantern fallen down to the roof of the next building in the West, and severe damages in the ellipse dome and the walls. In this analysis, some structural identification was made by the shear system of 3 mass (Fig. 3.3). The earthquake waves are shown in Table 3.2. Not only [2009 AQV EW, Itaca], but also [2009 L'Aquila Parking Entrance (AQK) EW, Itaca] and [2009 L'Aquila Castle (AQU) EW, Itaca] are used because the point of AQK is closer to Sant'Agostino and AQU is closest to it. In order to make the structural identification to clarify the physical phenomenon of the lantern fallen down to the West, the EW direction of these earthquake waves were used. The inelastic dynamic response analysis needs the weight, the stiffness and the shear coefficient of each storey. The size of Sant'Agostino were measured based on the Google satellite map, and plan or elevation figures in some references of Sant'Agostino. The Construction Technical Law, Italy is also referred for the material properties.

Photo 3.3-1. View of the North Photo **3.3-2.** View of the West

Photo 3.3. Damage of Sant'Agostino in L'Aquila

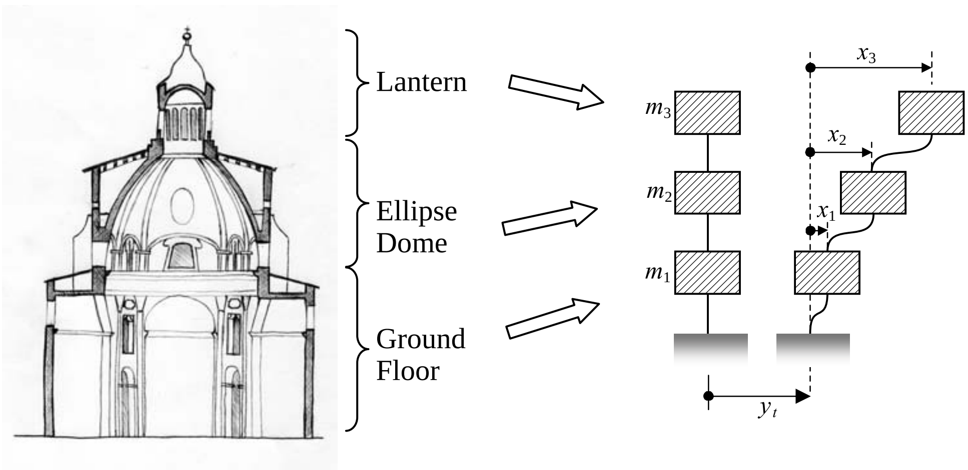


Figure 3.3. Cross section transverse in nearly EW direction and Analysis Model for Shear System of 3 Mass

Table 3.2. Earthquake waves in EW direction of L'Aquila Main shock (Magnitude 5.8)

Date and Time in local time	Station Code	Address (L'Aquila, Abruzzo, Italy)	Peak Acceleration in EW (cm/sec^2)	Epicentral Distance(km)
06 Apr. 2009 03:32:39	AQV	Center of Valley Aterno	662.6	4.9
	AQK	L'Aquila Parking Entrance	323.8	5.7
	AQU*)	L'Aquila Castle	258	6.0

Note) *) The acceleration records of AQU are corrected by subtracting the average of all data.

In the structural identification, some cases of the period and the shear coefficient on each storey are calculated and the one of these results are shown in Table 3.3 and Table 3.4. Table 3.3 shows that all of 3 cases (AQV, AQK and AQU) identified the larger displacement x_3 on the lantern storey to the West. Table 3.4 shows the results if the lantern would be retrofitted, when the qCy_3 is increased from 0.11 to 0.15. This retrofitting method would have valuable effects which would reduce the response displacement x_3 on the lantern storey less than that before retrofitting.

Table 3.3. Structural Properties and Analysis Results before retrofitting

Structure Properties of each Storey						ACC. Records (EW)	Analysis Results of each Storey		
Period (s)			Shear Coefficient (-)				Response Relative Displacement (cm)		
T_1	T_2	T_3	qCy_1	qCy_2	qCy_3		x_1	x_2	x_3
1.00	0.80	0.40	0.07	0.09	0.11	AQV	-4.00	-4.38	-9.42
						AQK	3.25	-5.71	-22.59
						AQU	2.44	4.25	-2.85

Note) x_1, x_2, x_3 : the positive (+) means the displacement to the East, and negative (-) to the West.

Table 3.4. Structural Properties and Analysis Results if retrofitting ($qCy_3 \rightarrow 0.15$)

Structure Properties of each Storey						ACC. Records (EW)	Analysis Results of each Story		
Period (s)			Shear Coefficient (-)				Response Relative Displacement (cm)		
T_1	T_2	T_3	qCy_1	qCy_2	qCy_3		x_1	x_2	x_3
1.00	0.80	0.40	0.07	0.09	0.15	AQV	-3.99	-4.47	-0.82
						AQK	3.25	-5.50	-3.82
						AQU	2.44	4.32	0.97

Note) x_1, x_2, x_3 : the positive (+) means the displacement to the East, and negative (-) to the West.

4. CONCLUSION

The conclusions are as follows;

1. The response displacements of some earthquake waves are mainly large in 2004 Chuetsu Earthquake and 1995 Kobe Earthquake in short periods, and in 2007 Chuetsu-Oki Earthquake in long periods.
2. The response displacements of some earthquake waves in the 2011 Great East Japan Earthquake are similar to the waves in the resolution 1.
3. In the structural assessment of a cultural heritage, the analysis for all storeys with the roof and the footing can estimate the appropriate response values and the effect of retrofitting.
4. The inelastic dynamic response analysis is available for the structural assessment and the structural identification.

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