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Study on dissipation decay in ground coupled vibration -Investigation of response characteristics using observed seismic waves-

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Abstract

Japan is one of the world's major earthquake nations. The earthquake has caused a lot of damage, such as the 2011 off the Pacific coast of Tohoku earthquake. Thereat, the seismic isolation systems have been studied for preventing damages to structure at earthquakes. Though the seismic isolation technology in the buildings is one of the most effective technologies, there are few cases of long-term earthquake response observations and the reports. Accordingly, in order to improve seismic safety, the obtainment of the long-term earthquake response observations with the seismic isolation structures is necessary. In this study, we investigated the Tokyo Denki University Building No.1, which is a base isolated structure mounted on laminated rubber bearings, at Tokyo Senju Campus of Tokyo Denki University. The long-term earthquake response observation has been carried out since December 2011. The long-term earthquake response observations of the eight years have investigated not only to improve seismic safety of the buildings, but also to be applied to the structural health assessment. Because the surface ground of the campus is so soft, the building supported and built on an engineering bed rock (> Vs = 700m / s) at 45m underground using foundation piles. The accelerometers are installed at 45m underground, 2.5m underground, 1m underground, and B1F, 1F, 6F, 14F at the building. The spectrum intensity, the natural frequency and the damping ratio are calculated by using seismic records of the target building, further the relationship among them of the target building is investigated. In addition to the research above mentioned, evaluation of dissipation effect in the soil-structure interaction has been just started to investigate using accelerometer data installed underground. The seismic wave is propagated from the underground, and input seismic wave at superstructure depends on the ground condition. The grip of the response tendency in the underground is important in order to improvement of the earthquake response analysis method. Accordingly, in this report, the tendency of underground response and the interaction with superstructure was investigated. Using the seismic records, the acceleration response in the underground was analyzed and the occurrence of dissipation decay on the ground surface was confirmed. Furthermore, the transfer function and the response spectrum was investigated and the tendency of response in the underground and the tendency of dissipation decay on the ground surface were understood.

Keywords: seismic isolation, earthquake response observation, dissipation decay

1. Background

In Japan, the ratio of the habitable land to the total land area is approximately 30%. The high-rise building has been increased, furthermore the diversification of the construction sites has been investigated. On the other hand, many earthquakes occurred due to the geographical conditions. Accordingly, the damage by the increase in response displacement of the structure with the longer period seismic wave is concerned. For this reason, the seismic response analysis has been performed in the various approaches to investigate the forces and the deformations of the ground and the structures.

In the seismic response analysis, the characteristic of the structure replaces parameters such as mass and stiffness. These values have a major impact on the accuracy of the analysis. The prehension of these values and the input seismic wave is very important. However, the damping coefficient has much uncertainness compared to other parameters. In particular, the dissipation decay on the ground is important for the interaction between the structure and the ground. The recording data is investigated to grasp the vibration characteristics.

2. Purpose

In this study, we investigated the Tokyo Denki University Senju Campus Building No.1, which is recorded ground and underground seismic observations. The acceleration at the seismic waves is investigated, furthermore the response tendency due to the dissipation decay is examined. The lump mass model with the ground property was investigated for the improvement of the seismic safety at the structure.

3. Target building

3.1 Summary of Tokyo Denki University Building 1

Fig.3.1 shows a three-dimensional view of Tokyo Denki University Building No.1, which began operation in April 2012. This building is a seismic isolation structure, which can be used without damage even during a severe earthquake. The dimensions are 87 [m] in front width, 50[m] inside depth, and 60 [m] in height. The base isolation system, which installs seismic isolation devices on B1F pillars, is adopted. The superstructure is a ramen structure with seismic studs of steel construction to 2F to RF including the penthouse. The structures of 1F and B1F are a combination of reinforced concrete construction and steel reinforced concrete construction, furthermore a CFT(: Concrete Filled Steel Tube) construction is adopted at the superstructure pillars. The structure at more than 6F is set back.

For this shape, the drawing force is negligible with the seismic isolation layer, furthermore the horizontal force is equalized at the whole. Accordingly, this building can ensure a balance at the whole. For this shape, the drawing force is negligible with the seismic isolation layer, furthermore the horizontal force is equalized at the whole. Accordingly, this building can ensure a balance at the whole. On the 1st floor, there is a large space called the 100th anniversary hall with a plane scale of 38.4 [m] × 22.7 [m]. Accordingly, the mega truss frame with 2 to 3F as one layer is installed.

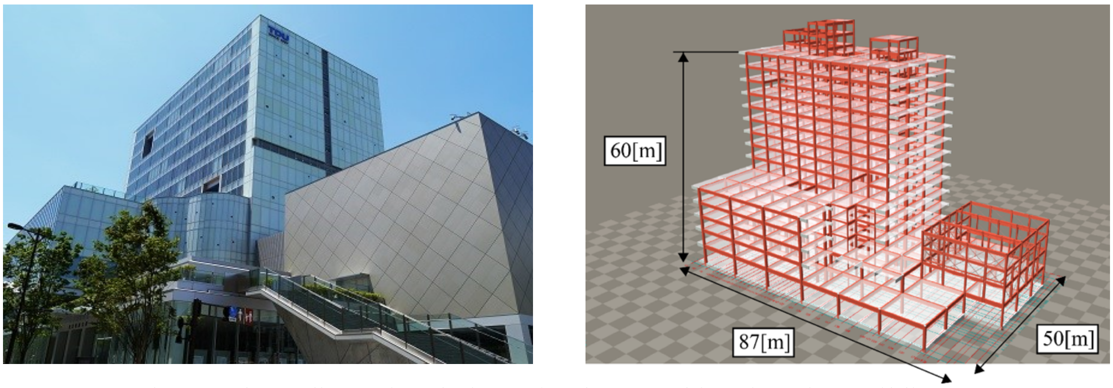
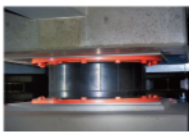


Fig.3.1 Three-dimensional view of Tokyo Denki University Building No. 1

3.2 Seismic isolation structure

Table 3.1 shows the summary of the seismic isolation device, further Fig.3.2 shows the arrangement of the seismic isolation device in the seismic isolation layer. The isolation layer consists of 23 natural rubber bearings, 34 lead rubber bearings, 10 cross linear bearings, 10 oil dampers and 6 lead dampers. In order to prevent torsion of the seismic isolation layer during an earthquake, the seismic isolation devices were arranged. The lead rubber bearings, which has good performance of the restoring force characteristics and damping, are installed on the outer periphery of the seismic isolation layer, further the lead dampers and the oil dampers are installed in the X and Y directions. Furthermore, the oil dampers control the displacement of the seismic isolation layer. The seismic isolation devices are located at different heights near the center of gravity. The cross linear bearings at this part were arranged in order to mitigate the shear forces concentrated near the center of gravity. In the design, the seismic isolation period of these devices is 4.2 [s] at the maximum deformation of the base isolation layer (shear strain 250%), further the yield displacement of the seismic isolation device is 7.33 [mm] with reference to the yield displacement of the lead damper. In the superstructure, the maximum response interlayer deformation angle, which shows the response characteristics in the event of a large earthquake, is suppressed to 1/200 in order to preventing damage of the superstructure, internal and external materials. As the result, this building can be operated continuously.

Table 3.1 – Summary of seismic isolation devices

Seismic isolator	 Lead rubber bearing	 Natural rubber bearing	 Cross linear bearing	 Oil damper	 Lead damper
Article number	LRB-S1000 LRB-S1100	NS100N3	CLB-1560T CLB-2000F	BDS 1201800-B-1	U2426
Unit number	S1000 : 32 S1100 : 2	23	1560T : 7 2000F : 3	10	6

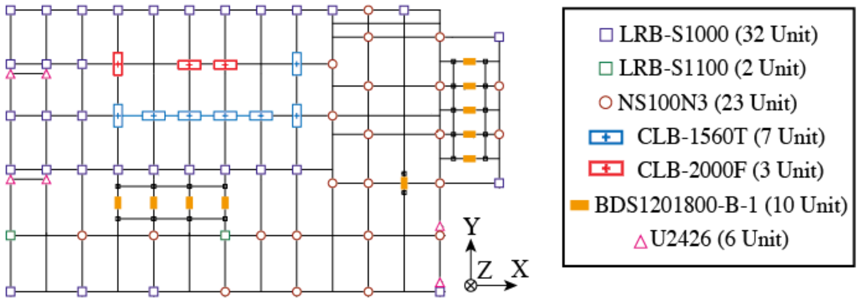


Fig.3.2 Arrangement of seismic isolation devices in seismic isolated layer

3.3 Observation system

Table 3.2 shows the specifications of seismometers which has been installed in the building. The building has floor-mounted servo accelerometer AS-303T1W1 and AS-301, embedded servo accelerometer AS-3250A, relative displacement meter DP-750, marking displacement meter RD-1500, and the pore water pressure gauge EPP-5802-5. In 2017, new accelerometer HM-0013 capable of measuring long-period vibration components was installed on 1F, 6F, and 14F, further a more reliable observation system was established.

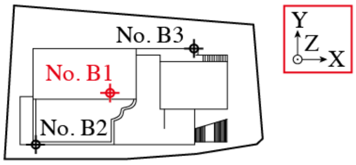
Accelerations in the X, Y, and Z directions at GL-45 [m], GL-2.5 [m], GL-1 [m], 14F, 13F, 6F, and 1F are measured. By measuring the relative displacement between B1F and 1F, the displacement in the X and Y directions of the seismic isolation device is measured.

Table 3.2 – Specifications of various seismometers

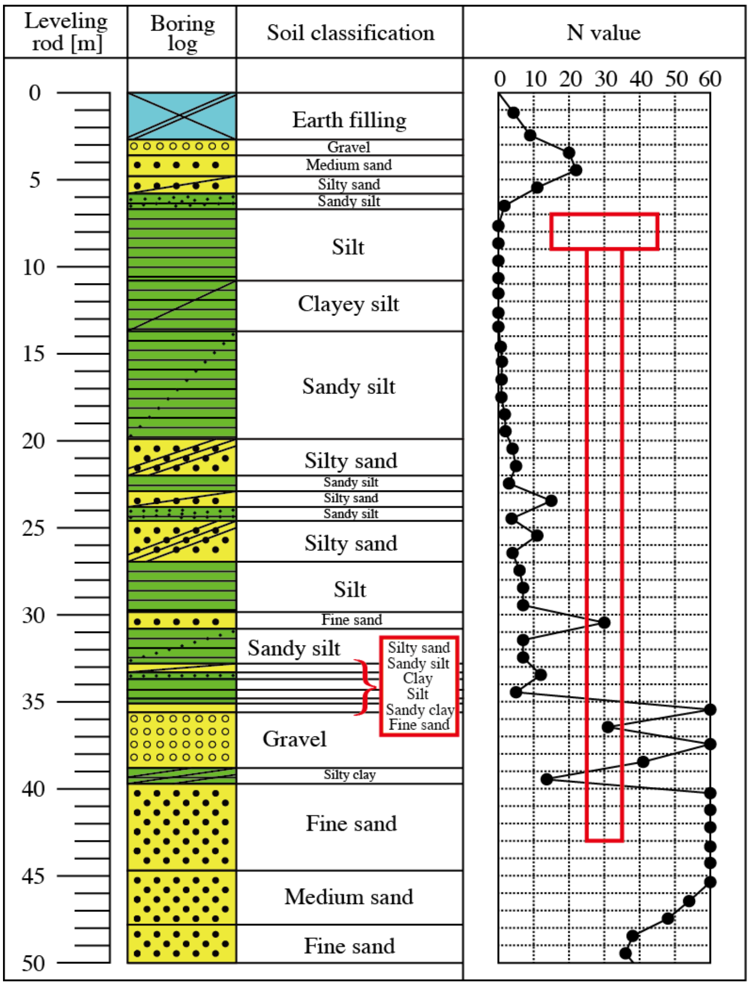
Article number	Observed element	Installation location	Observation direction	Measuring range
AS-303T1W1	Acc.	B1F, 1F, 6F, 14F	X, Y, Z	±20[m/s^2]
AS-301		13F	Z	
HM-0013		1F, 6F, 14F	X, Y, Z	
AS-3250A		GL-1[m] GL-2.5[m] GL-45[m]	X, Y, Z	
DP-750	Disp.	B1F	X or Y	±750[mm]
RD-1500			X, Y	
EPP-5802-5	Pore water pressure	GL-2.5[m]	-	0～0.2 [MPa]

3.4 Ground and foundation structure

During construction, the boring survey and standard penetration test are conducted at three locations on the site. Fig.3.3 shows the drilling location and the boring column diagram on the site. The Boring boring surveys are conducted at three locations. One is near the center of gravity, the others and twoare near the edges. The soil classification from GL-6 [m] to near GL-35 [m], it is almost composed of silt layer. This layer and it is a soft layer with N value of approximately about 3-10. The silt layer is clayey soil formed from soil particles with a particle size of 0.074 to 0.005 [mm]. In addition, around GL-41 [m], a strong ground with an N value of more than 50 composed mainly of fine sand layer appears. For this reason, the piles foundation structure that drives steel pipe piles up to GL-45 [m] is adopted. This building and was it is designed to remain within the elastic range even during a the level level-2 earthquake, taking into account the stress due to the response displacement of the ground. The piles were steel pipe piles with a diameter of 1000 mm, furthermore and a total of 209 piles were driven. The foundation is a concrete foundation with an area of 3000 [m^2] and a thickness of 1.6 [m].



(1) Placement of standard penetration test



(2) Soil profile of No. B1

Fig.3.3 Drilling location and the boring column diagram

4. Observation

In order to confirm the response tendency in the ground, the relatively large seismic record, which was selected among the 158 records obtained from April 2012 to June 2019, is investigated. Two observation records are shown. The maximum accelerations at GL-45, GL-1, B1F, 1F, 6F, and 14F, furthermore the transfer function between GL-45 [m] and B1F in the building is investigated.

4.1 Case1

Fig.4.1 shows record of 5 May 2014 (The Japan Meteorological Agency seismic intensity scale 4). In the X and Y directions, the maximum acceleration at GL-2.5 [m] increases than that at GL-45 [m]. The damping force on the base of B1F from GL-1 [m] is confirmed. This damping force is a dissipation decay. There is a difference in the maximum acceleration tendency between the X and Y directions. In the X direction, the maximum acceleration decreased from GL-2.5 [m] to GL-1 [m]. However, the maximum acceleration in the Y direction increased. In the superstructure, the increase in the Y direction is larger than that in the X direction from 6F to 14F.

In the transfer function, the peak of the response magnification is near 1 [Hz] in the X and Y directions. Accordingly, the primary natural frequency of the underground is approximately 1[Hz]. Also, the peaks are near 3 [Hz], 7 [Hz], and 12 [Hz]. The response magnification near 15[Hz] is larger in the X direction than in the Y direction.

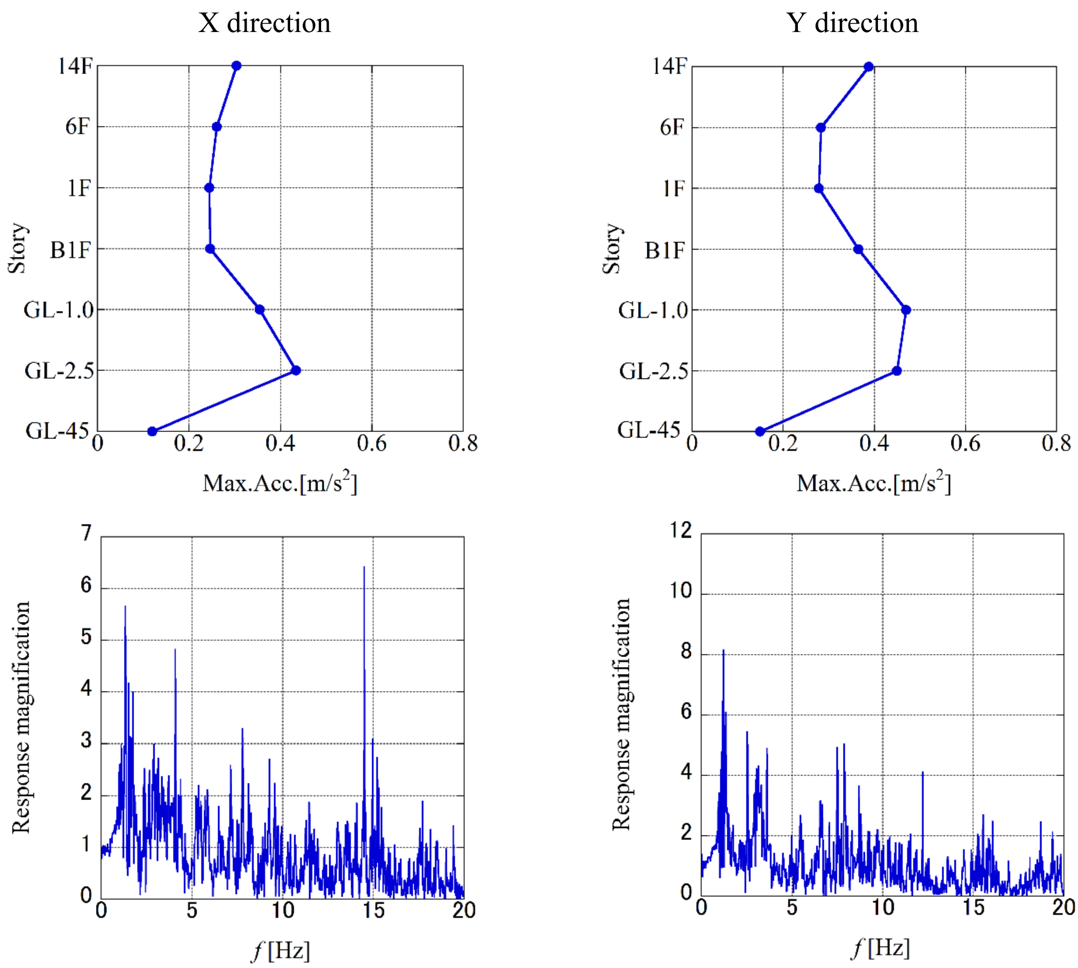


Fig.4.1 Maximum acceleration and transfer function (the record of 5 May 2014)

4.2 Case2

Fig.4.2 shows record of 30 May 2015 (The Japan Meteorological Agency seismic intensity scale 4). In the X and Y directions, the maximum acceleration at GL-2.5 [m] increases than that at GL-45 [m]. From GL-1 [m] to B1F, the maximum acceleration increases in the X direction more than in the Y direction. In the superstructure, the maximum acceleration at 1F increases than that at B1F in the X direction. From B1F to 1F, increasing or decreasing is hardly confirmed in Y direction. From 6F to 14F, the maximum acceleration greatly increases in the Y direction than in the X direction.

The transfer function is similar to Case 1. However, there is no large peak near 15 [Hz] in the X direction. The peaks are also confirmed in the Y direction near 2 [Hz] and 9 [Hz] compared to case 1.

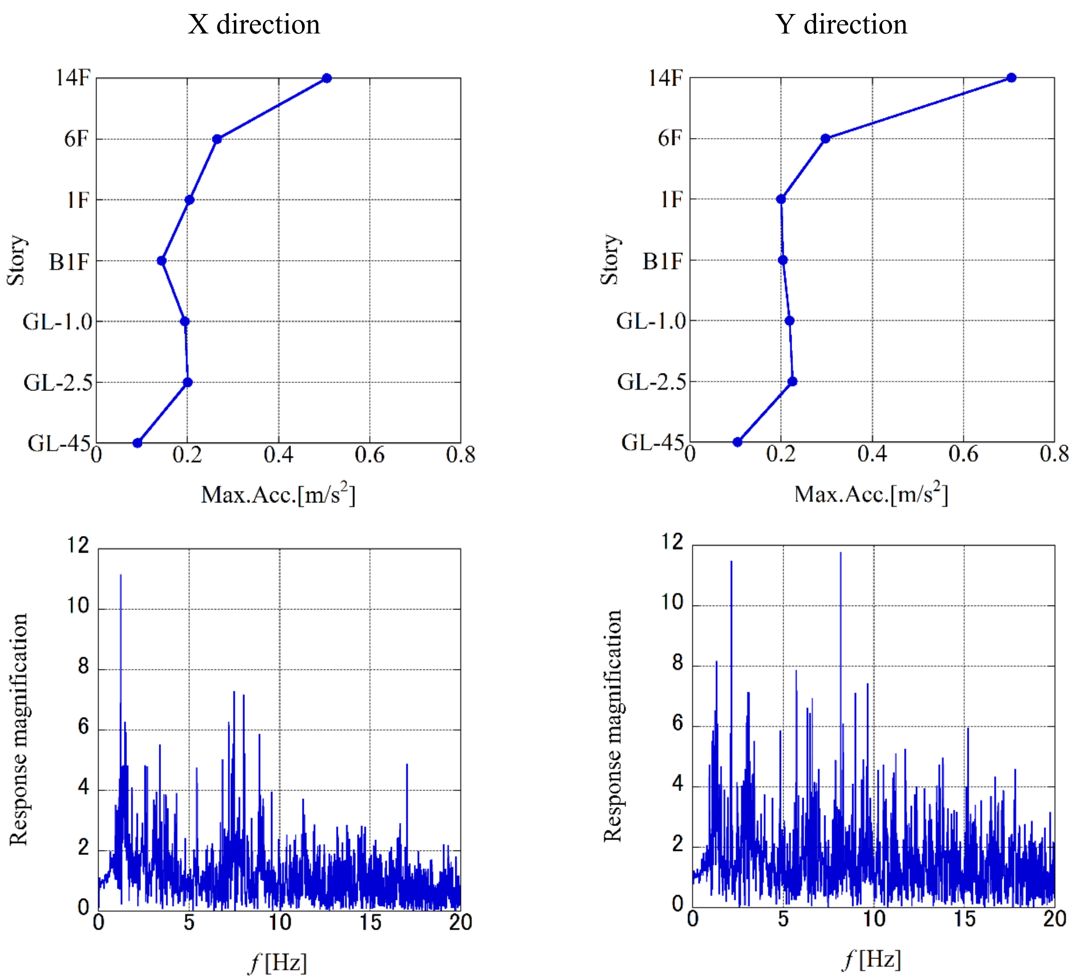


Fig.4.2 Maximum acceleration and transfer function (the record of 30 May 2015)

5. Analysis model

5.1 Modeling

Fig.5.1 shows the analysis model. The seismic analysis was carried out with the lump mass model. One mass point is defined between GL-45 [m] and GL-1 [m] of the building. One mass point is defined between GL-1 [m] and B1F where the dissipation decay occurs most. The superstructure is the 15mass model used in the study of pasts [1]. The mass and stiffness of the superstructure are modified from the values at the time of design from the previous research. The stiffness proportional damping is used for the damping coefficient.

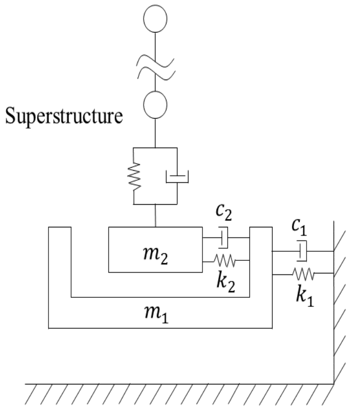


Fig.5.1 Analysis model

5.2 Parameters

The masses m_1, m_2 and stiffnesses k_1, k_2 shown in Fig.6.1 are estimated. The pile foundation structure has been adopted in this building. Accordingly, the underground stiffness was assumed the sum of the stiffness of the pile and the stiffness of the ground. The stiffness of the pile is assumed the flexural stiffness and the shear stiffness. The total stiffness is 209 times one stiffness of the pile because 209 piles are driven in undergroun. The ground stiffness is obtained by the Barkan equation. Eq. (1) to (3) shows the Barkan equation.

$$K_s = \frac{1}{2}\pi G \times \frac{4ab}{f(a,b) - \upsilon g(a,b)} \tag{1}$$

$$f(a,b) = a\ln\left(\frac{b+\sqrt{a^2+b^2}}{a}\right) + b\ln\left(\frac{a+\sqrt{a^2+b^2}}{b}\right) + \frac{1}{3ab}\{(a^3+b^3) - (a^2+b^2)^{\frac{3}{2}}\} \tag{2}$$

$$g(a,b) = a\ln\left(\frac{b+\sqrt{a^2+b^2}}{a}\right) + \frac{1}{3ab}\{(b^2-2a^2)\sqrt{a^2+b^2} - (b^3-2a^3)\} \tag{3}$$

Where $2a$, $2b$ is structure dimensions, G is shear modulus, V_s is shear wave velocity, υ is Poisson's ratio. The long side direction of the building is $2a$, the short side direction is $2b$. In the Barkan equation, the underground is regarded as one layer, the density and Poisson's ratio of the underground is used the average values of the stratum. Furthermore, the shear wave velocity is estimated from the N value of each layer and the average value is used. The equation for estimating the shear wave velocity differs at the soil type. Eq. (4) shows cohesive soil type, Eq. (5) shows sandy soil type.

$$V_s = 100N^{\frac{1}{3}} \tag{4}$$

$$V_s = 80N^{\frac{1}{3}} \tag{5}$$

From the above, k_1, k_2 is determined.

m_1 is calculated with k_1 and the primary natural frequency estimated from the transfer functions in Fig.5.1 and Fig5.2. The primary natural frequency is defined f_1. m_1 is obtained from Eq. (6)

$$m_1 = k_1/(2\pi f_1)^2 \tag{6}$$

m_2 is the mass of the concrete foundation of the building. Table 6.1 shows the estimated values of each parameter.

Table 6.1 – Values of each parameter

	m_1 [kg]	m_2 [kg]	k_1 [N/m]	k_2 [N/m]
Flexural stiffness	1.74×10^7	1.15×10^7	1.12×10^9	1.11×10^7
Shear stiffness	4.47×10^9	1.15×10^7	2.88×10^{11}	2.88×10^{11}

6. Analysis result

Fig.6.1 and Fig.6.2 show the maximum acceleration in the X and Y directions. Fig.6.1 shows the case 1, Fig.6.2 shows the case 2. The blue line shows the observation record, the red line shows the analysis results.

The damping ratio of the superstructure is 2%, which is commonly used as the damping ratio of buildings. The damping ratio of underground is set so that the maximum acceleration in the observation record and the analysis is the closest.

In the observation record, the maximum acceleration at GL-2.5 [m] increases than that at GL-45 [m]. In shear stiffness case, the amplification of the maximum acceleration is properly confirmed in the analysis. However, in the flexural stiffness case, the analysis shows that the maximum acceleration is attenuated. In addition, the value of the maximum acceleration at 1F is closer in the shear stiffness than in the flexural stiffness. The maximum acceleration with the shear stiffness is reproducible more than that with the flexural stiffness. Although the maximum acceleration of the analysis and the record are slightly different in case 1, the maximum acceleration of the analysis and the record are same in case 2. The effective mass and the stiffness in the underground may be changed with the seismic waves. The dissipation decay, which occurred between B1F and GL-1 [m], is confirmed in the observation record. However, the dissipation decay at the analysis is little. In the shear stiffness case, even if the damping ratio between B1F and GL-1 [m] is extremely large, the damping hardly occurs. Because m_2 is only the mass of the concrete foundation, m_2 is much smaller than m_1. Accordingly, the seismic wave in this analysis model hardly decreased from GL-1[m] to B1F.

In order to improve the accuracy of the analysis, the improvement of the method estimating the effective mass is necessary. Furthermore, the investigation of the correlation between the stiffness, the damping ratio and the seismic waves is necessary.

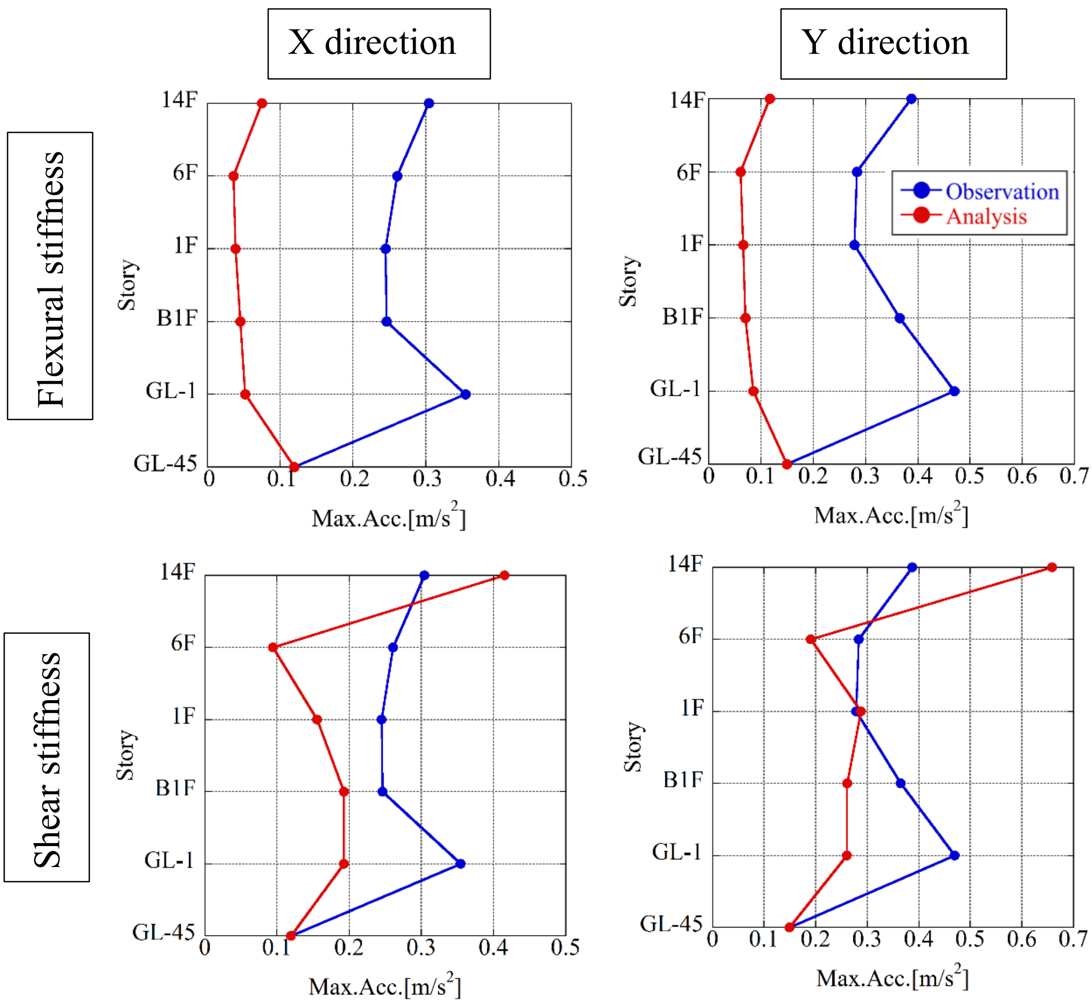


Fig.6.1 Maximum acceleration (the record of 5 May 2014)

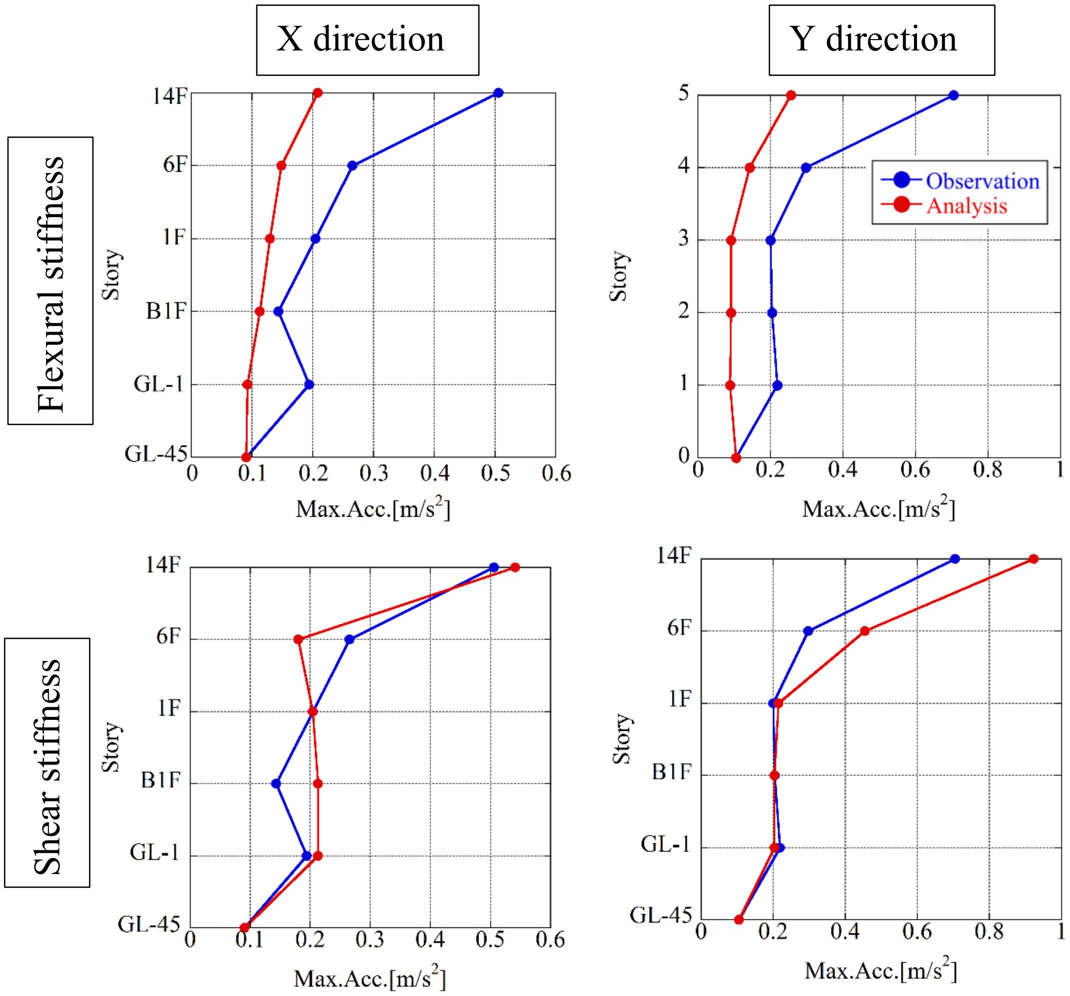


Fig.6.2 Maximum acceleration (the record of 30 May 2015)

7. Conclusion

In this paper, the maximum response acceleration and the transfer function of the underground were investigated with the seismic observation records obtained so far at Tokyo Denki University Building No. 1. In the observation, the decrease of the maximum acceleration on the ground surface was confirmed. The primary natural frequency of the underground was obtained from the transfer function. The underground was a two-mass model, each parameter was estimated. Because the pile foundation is used, the stiffness of the underground is almost stiffness of pile. The seismic analysis was performed with the estimated parameters. The validity of the parameters was examined. The reproducibility of the analysis model is good when shear stiffness case. The analysis model hardly reproduces dissipation decay. In addition, the analysis varies greatly due to the seismic wave. For that reason, the parameters need to be reexamined.

8. References

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