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Field measurement of damping and natural frequency of an actual steel-framed building over a wide range of amplitudes

N. Fukuwa*,[a,1], R. Nishizaka[a,2], S. Yagi[a,3], K. Tanaka[b,4], Y. Tamura[c,5]

[a]*Department of Architecture, School of Engineering, Nagoya University, Furo-Cho, Chikusa-Ku, Nagoya 464-01, Japan*

[b]*Computational Engineering Department, Technical Research Institute, Obayashi Corporation, 640, Shimokiyoto 4-Chome, Kiyose, Tokyo 204, Japan*

[c]*Department of Architecture Engineering, Faculty of Engineering, Tokyo Institute of Polytechnics, 1583, Iiyama, Atsugi, Kanagawa 243-02, Japan*

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Abstract

In order to understand the dynamic characteristics of a prefabricated steel building, several vibration tests with a wide range of amplitudes were carried out. Under excitation with relatively small amplitude levels, four different construction stages are prepared. The dynamic properties such as natural frequencies and damping ratios are evaluated using several identification methods based on observation records. The variability in the results due to the differences in identification methods and excitation conditions is examined. By comparing the results at different construction stages, the effect of non-structural members on the structural stiffness and damping ratio is studied. Using a mechanical exciter, forced vibration tests were carried out for several high excitation levels (100–3000 kgf). Free vibration tests using a wire cutting method were also carried out for several initial displacement levels (5–25 tonf, as initial tensile force levels). Based on the results of these vibration tests with various amplitude levels, the effect of non-structural members on dynamic characteristics is studied from the view point of amplitude dependence.

Keywords: Amplitude dependence; Damping ratio; Field measurement; Natural frequency; Non-structural member; Prefabricated steel building; Random decrement technique; Sweep excitation test; Transfer function

*Corresponding author.

[1]Associate Professor.

[2]Graduate student (JSPS Research fellow).

[3]Graduate student.

[4]Research Engineer.

[5]Professor.

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1. Introduction

In Japan, large earthquakes occurred frequently during 1993–1995, and a lot of buildings were severely damaged, especially by the recent Hyogo-Ken-Nanbu earthquake. However, the prefabricated steel buildings were only slightly damaged in spite of the large seismic ground motions. According to the latest survey in some areas of Ashiya and Nishinomiya cities, 90% of conventional wooden houses were damaged and 30% of them collapsed. In contrast, only 30% of prefabricated houses were slightly damaged. It is well-known that prefabricated steel buildings have a lot of non-structural members, and it is considered that the effect of such non-structural members against the seismic forces is remarkable. Since prefabricated buildings are one of the most popular housing in Japan, it is of urgent necessity to clarify the dynamic properties of them under excitation with large amplitude levels.

Recently, many slender buildings have been built in Japan on soft soil and small spaces, because of the lack of land and because of the subdivision of sites due to the sharp rise of land prices. In many cases, these buildings are built in an urban area, along a main street or a railroad, and a lot of problems caused by traffic or other environmental vibrations are reported. Therefore, it is also very important to study the dynamic properties of prefabricated buildings under excitation with small amplitude levels.

In spite of the urgent necessity and the importance, there are few reports of studies on the dynamic properties of prefabricated buildings. Because of the complexity of the plan of structures and because of the great contribution of non-structural members to the structural stiffness, it is difficult to analytically understand their properties.

In this paper, it is clarified that the stiffness of non-structural members plays an important role for the dynamic properties of the structure, and that the amplitude dependence of the non-structural members on the dynamic properties is remarkable, based on the vibration tests of full-scale three stories steel prefabricated buildings in a wide range of amplitude.

2. Summary of vibration tests, buildings and soil condition

The Fourier amplitude spectrum of microtremor record observed on this ground (vertical direction) is shown in Fig. 1. The predominant frequency of the ground is about 3.2 Hz. According to the result of the standard penetration test conducted at the construction site, the surface soil is fill-up bank with a thickness of about 1.5 m. Below this layer, there is a sand stratum with an N-value of about 38. It is recognized that the soil condition of this site is relatively stiff.

The varieties of the building types, the vibration tests and the purposes of them are summarized in Table 1. Three types (A, B, C) of full-scale three story prefabricated steel buildings are constructed for the series of tests. The external view of the building is shown in Fig. 2, and its plans and elevations are shown in Fig. 3. In the NS direction, there are plenty of openings and the length is short. On the other hand, in the EW direction, there are no openings and the length is long. The external walls consist of

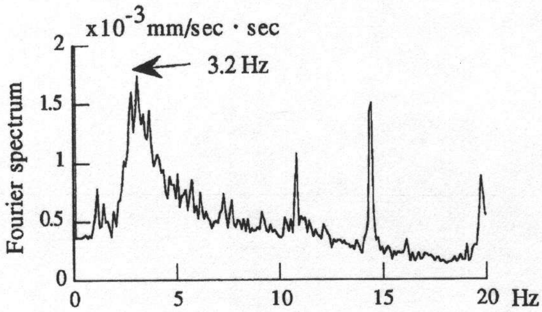
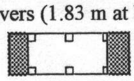
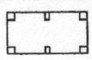


Fig. 1. Fourier spectrum of microtremor record (UD direction).

Table 1
Outline of experiment

	A	B	C
Height		9.325 m	
Stories		3 stories	
Structure		Steel frame structure of 1 x 2 spans = 3.66 m	
Plan	3.944 m x 9.43 m (1F) 3.944 m x 11.26 m (2F~)	3.944 m x 7.604 m	
Structural characteristics	with stairs with cantilevers (1.83 m at both sides) 	without stairs without cantilevers 	
	with partition walls at 4th construction stage	exciter of weight 2 tonf on 3rd floor	-
		ALC panels are used for outer walls and floor slabs / RC continuous footing foundation	
Objectives of experiment	the effect of non-structural members from the tests at different constructin stages	the effect of amplitude dependency in large amplitude level	
Methods of tests		microtremor test/sweep exitation test using electromagnetic exciter / free vibration test	
	-	sweep exitation using mechanical exciter	free vibration test using wire cutting method

ALC (autoclaved lightweight concrete) panels with a thickness of 100 mm, which do not bear the shear stiffness of the structure under strong ground motion. A building of type A is tested at four different construction stages as shown in Table 2. Buildings of type B and C do not have stairs or cantilevers. The weight of each floor for all the types of buildings is also shown in Table 2. Table 2 shows that the weight of ALC external walls accounts for about 68% of the weight increase from the bare steel frame. An electromagnetic exciter or a mechanical exciter is installed on the third floor slab. In Fig. 4, the arrangement of the wire cutting test for the building of type C is illustrated.

Fig. 2. Photo of the building.

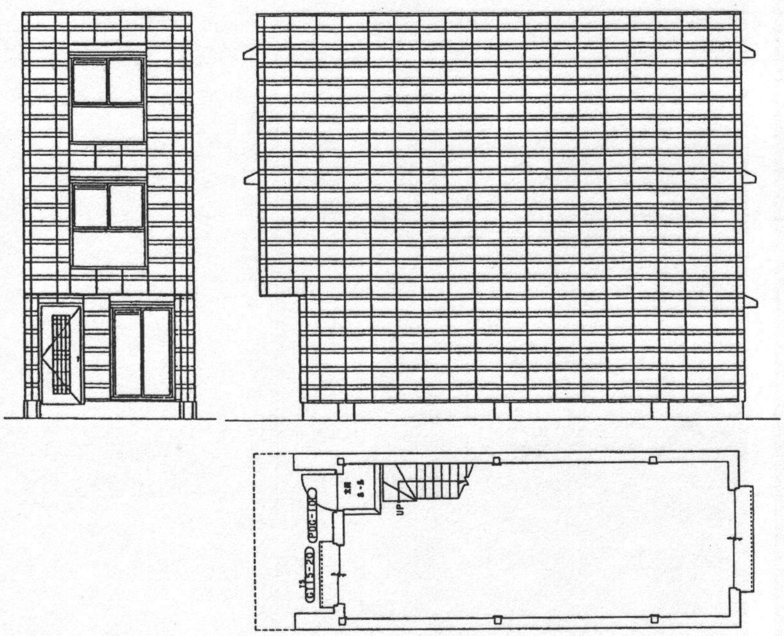
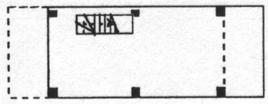
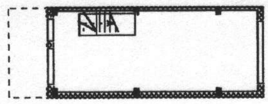
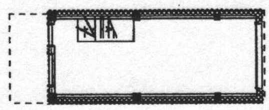
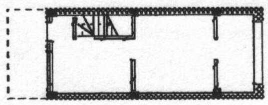
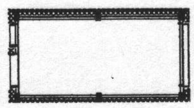


Fig. 3. Plans and elevations.

Table 2
Construction stage and weight of each floor

Type	Stage	Structure	Weight (ton) 1F	2F	3F	RF	Total	Plan of first story
A	1	Steel frame + ALC floor slab + stair	33.1	4.9	4.8	4.0	46.7	
	2	+ ALC wall panel	35.1	10.6	10.7	7.3	63.7	
	3	+ inner plaster board + ceiling	35.2	12.2	12.4	8.9	68.8	
	4	+ partition wall	35.5	13.0	13.9	9.3	71.7	
B		Steel frame + ALC floor slab + ALC wall panel + inner plaster board + ceiling	35.2	9.1	12.2	7.0	63.5	
C	3		35.2	9.1	9.1	7.0	60.4	

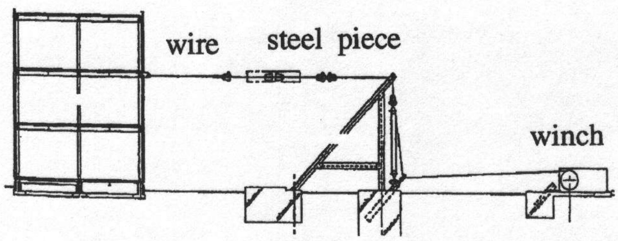


Fig. 4. Free vibration test using the wire cutting method.

3. Examination of identification methods (type A)

Natural frequencies and damping ratios are estimated by many kinds of identification methods and excitation conditions. Since it is generally pointed out that the estimated values vary from method to method, the appropriate methods for the buildings are selected based on the records of the NS direction at the fourth construction stage on the building of type A.

3.1. Identification based on free vibration test

Dynamic properties of the soil–structure system are discussed using the records obtained from the free vibration test. The building is excited harmonically at its fundamental resonance frequency by an electromagnetic exciter installed on the third floor. When steady state motion is obtained, the exciter is stopped and the free vibration response of the building is observed. The free vibration response and its Fourier spectrum are shown in Figs. 5 and 6, respectively. The natural frequency of the building is about 2.6 Hz. According to the free-vibration decay method based on the logarithmic decrement as shown in Fig. 7, the damping ratio is estimated to be

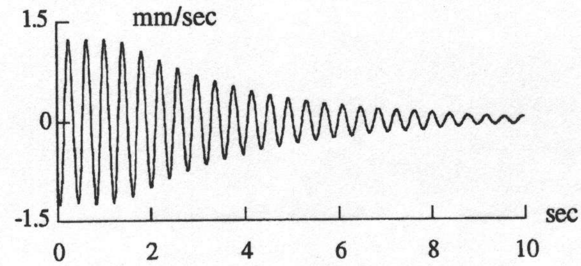


Fig. 5. Free vibration response.

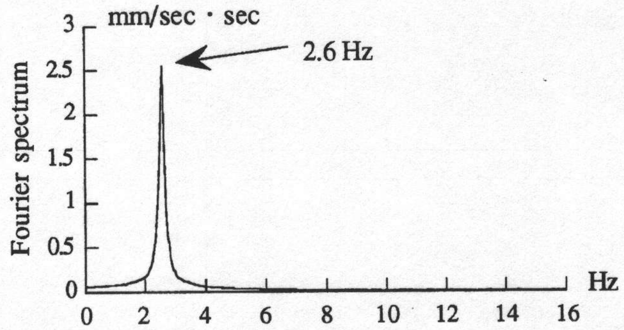


Fig. 6. Fourier spectrum of free vibration response.

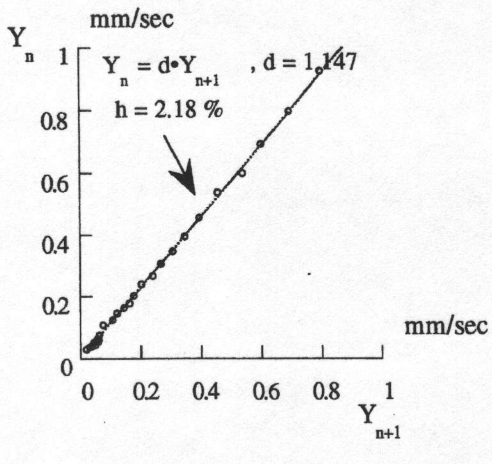


Fig. 7. Damping ratio based on the free vibration decay method.

2.18%. As the length of the record used in the estimation becomes longer, the damping ratio becomes smaller as shown in Fig. 8. Applying the identification method based on the envelope of a free vibration response, the damping curve regressed is obtained as shown in Fig. 9. The damping ratio varies from 1.98% to 2.18% depending on the length of the record used in the estimation. Generally, it has been pointed out that amplitude dependence of damping ratio is observed in structures with many non-structural members [1]. In Fig. 10, natural frequencies and damping ratios which are

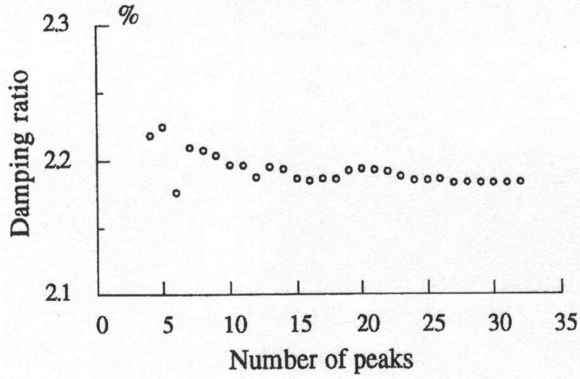


Fig. 8. Effect of number of peaks.

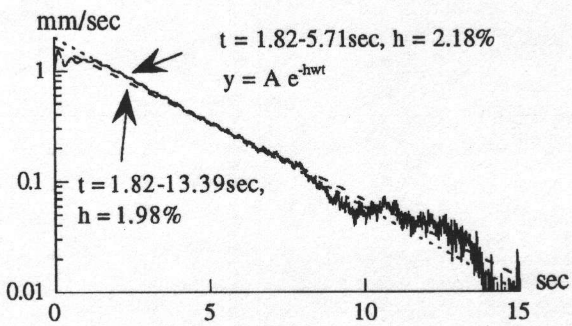


Fig. 9. Envelope of free vibration response and regressed damping curve.

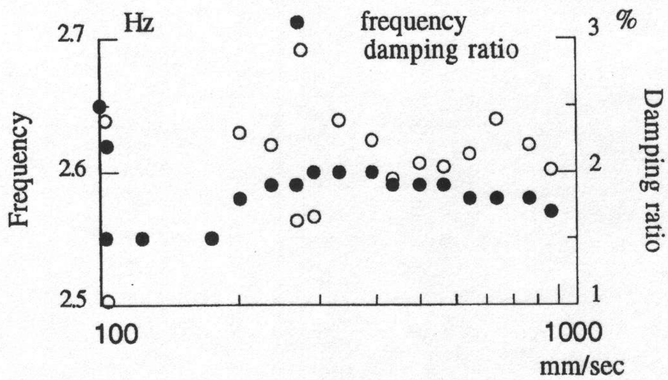


Fig. 10. Natural frequencies and damping ratios based on the moving average method.

estimated by taking the averages of three adjoining peaks are plotted [2]. Fig. 10 shows that as amplitude becomes larger, natural frequency decreases and damping ratio increases.

3.2. *Identification based on random decrement (RD) method using microtremor records [3]*

A microtremor record obtained on the third floor and its Fourier spectrum are shown in Figs. 11 and 12, respectively. A band-pass filter with width shown in Fig. 12 (with tapers of cosine form whose width is 1.0 Hz at both sides) is applied to the record shown in Fig. 11. The free vibration response with the primary natural frequency is generated by using the RD method as shown in Fig. 13. Using the free-vibration decay

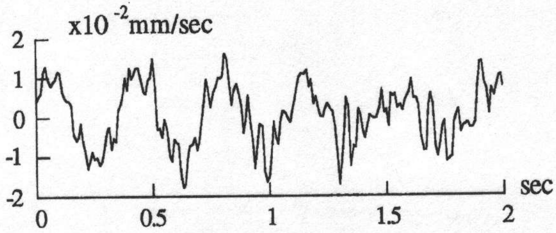


Fig. 11. Microtremor record.

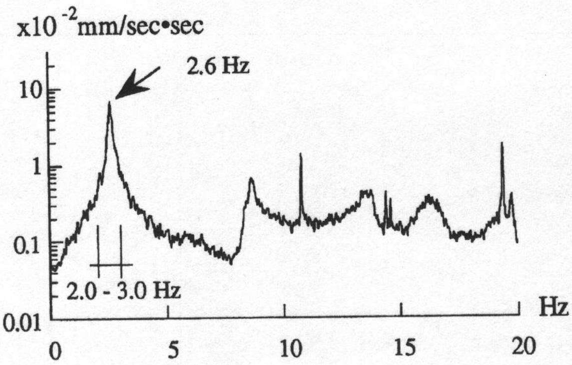


Fig. 12. Fourier spectrum of the microtremor record.

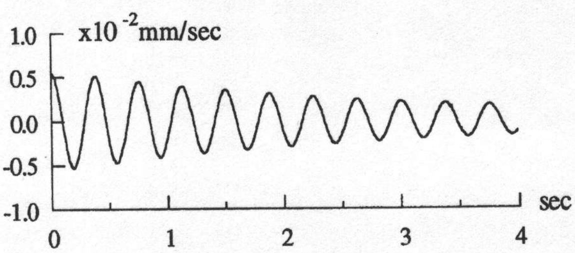


Fig. 13. Generated free vibration response of primary mode.

method for the primary mode, it is estimated that the natural frequency is 2.7 Hz and the damping ratio is 1.6%. When the RD method is used, it is important to examine the number of stacks. Fig. 14 shows that about 2000 stacks are needed.

3.3. Identification based on sweep excitation test

Sweep excitation test is carried out by the electromagnetic exciter installed on the third floor. The resonance curves and their phase differences for the two different levels of exciting forces (4.4, 35 kgf) are shown in Figs. 15a and 15b, respectively. For these results, half-power method, resonant amplification method, phase gradient method and spectrum curve fitting method are applied, and the estimated natural frequencies and damping ratios are examined. The results of the tests with an exciting force of 4.4 kgf are arranged in Table 3. There is little difference among the estimated natural frequencies by various identification methods. On the contrary, there are remarkable differences among the estimated damping ratios. With larger levels of exciting force, the amplitude dependence is observed; the peak frequency becomes smaller and the response amplitude at the frequency becomes lower.

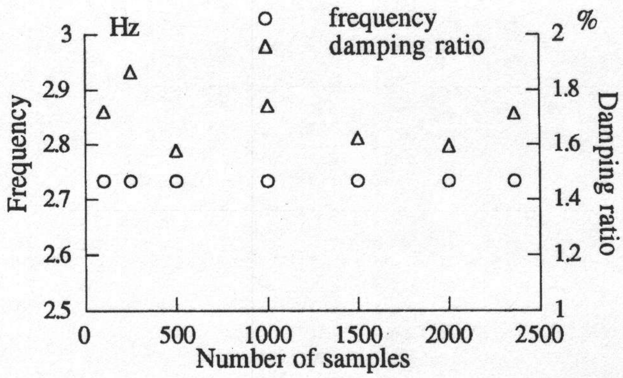


Fig. 14. Effect of number of stacks on natural frequency and damping ratio for primary mode.

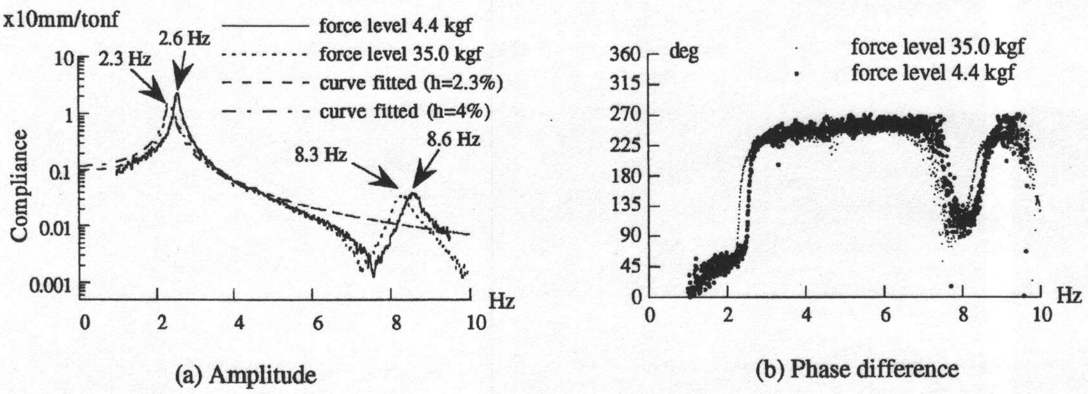


Fig. 15. Resonance curves for two different levels of exciting force.

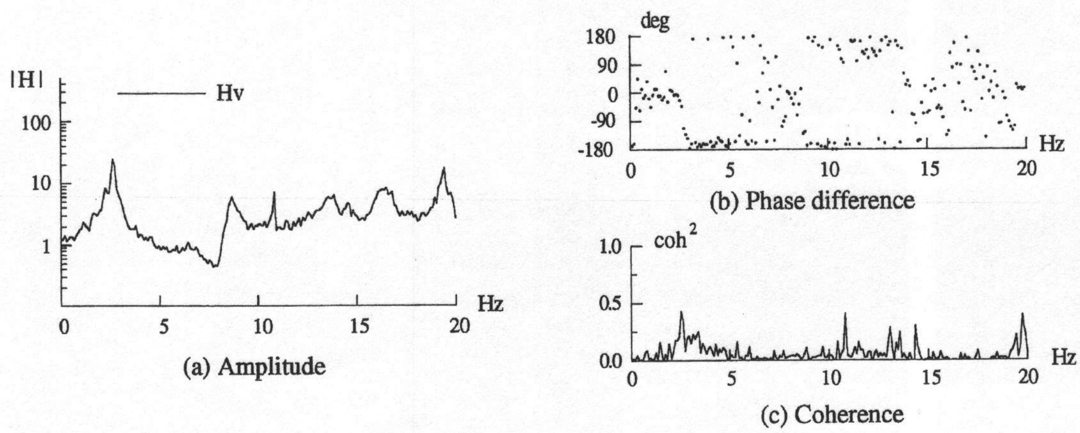


Fig. 16. Transfer function of third floor over foundation.

3.4. *Identification based on transfer function using microtremor records*

The microtremor record with a duration of 480 s is divided into 40 samples with a length of 12 s. Each sample is normalized by the r.m.s.-value of the first floor, and the ensemble average of power spectra is evaluated. Using this ensemble average, Hv estimation is carried out [4]. In Fig. 16, the transfer function of the third floor over the first floor is shown with the coherence function. Observing the phase and the coherence together in Fig. 16, the disturbances might exist besides the input from the soil. In such a case, the damping ratio tends to be overestimated.

3.5. *Differences of dynamic properties due to excitation conditions and identification methods*

Natural frequencies and damping ratios estimated by each identification method are summarized in Table 3. There is no difference among identification methods to estimate natural frequency, when the amplitude is at the same level. However, the estimated values of damping ratio are scattered among the methods. It is concluded that the results by the method using the regressed damping curve of a free vibration response and those by the spectrum curve fitting method based on the transfer function using microtremor records are not desirable. In the following sections, the free-vibration decay method is applied to estimate the damping ratio, since it can provide the average results. When studying the amplitude dependence of dynamic properties, the moving average method using free vibration responses is used.

4. Effects of non-structural members on dynamic properties (type A)

The effects of non-structural members on dynamic properties of the building in relatively small response amplitude levels (0.1 μm–0.6 mm) are examined. Four kinds

Table 3
Natural frequencies and damping ratios

	Natural frequency (Hz)	Damping ratio (%)
Free vibration test	2.6	
Free-vibration decay method		2.2
Damping curve method		1.98–2.26
Moving average method		1.9–4.1
Sweep excitation test	2.6	
Half-power method		2.26
Resonant amplification method		2.16
Phase gradient method		2.85
Curve fitting method for amplitude		2.2
Curve fitting method for phase		1.8
Microtremor test	2.6	
RD method and free-vibration decay method		1.75
Hv estimation and half-power method		3.64

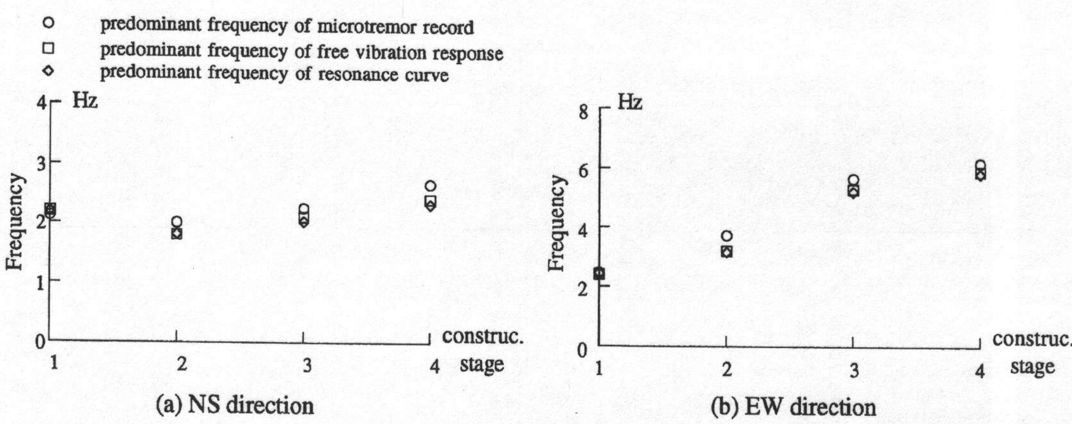


Fig. 17. Natural frequencies for different construction stages.

of construction stages are examined for the building of type A as shown in Table 2. The estimated natural frequency and damping ratio at each construction stage are shown in Figs. 17 and 18, respectively. In Fig. 19, the ratios of natural frequency, weight and stiffness of the building at each construction stage to those of bare steel frames are shown.

At the first construction stage, the building consists of pure steel frames and ALC floor slabs. There are no non-structural members except for steel stairs. At this stage, the natural frequencies are nearly identical in the NS and EW directions, and the damping ratio is very small.

At the second construction stage, ALC external walls are added. The difference in dynamic properties between the NS and EW directions is remarkable, because of the difference of the quantity of effective external walls. Fig. 19 shows that in the NS

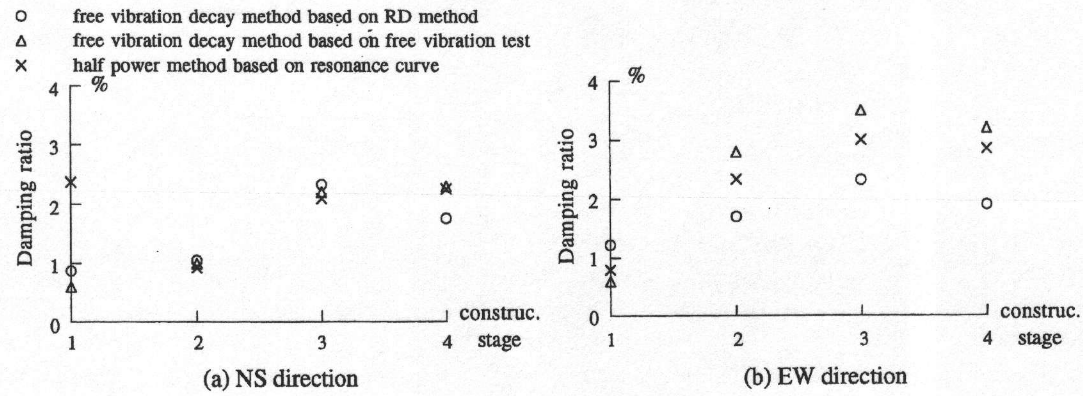


Fig. 18. Damping ratios for different construction stages.

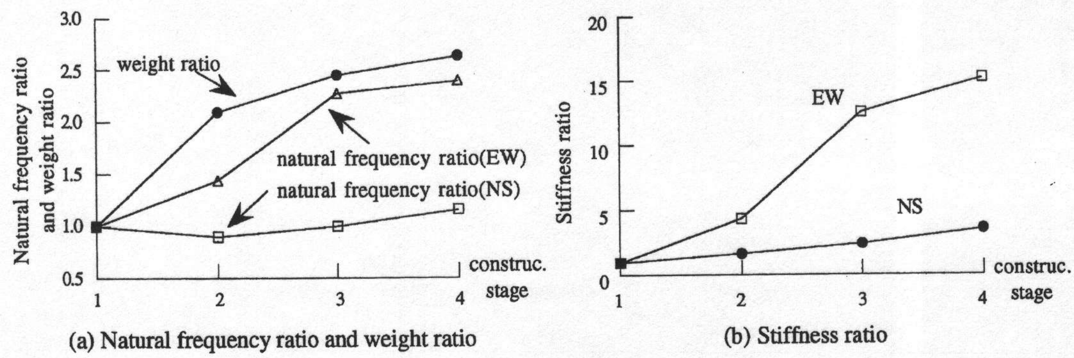


Fig. 19. Ratios of natural frequency, weight and stiffness at each construction stage to those of the first construction stage.

direction, the effect of the weight increase is more dominant than the effect of the stiffness increase; accordingly, the natural frequency decreases by less than 10%. On the other hand, in the EW direction, the natural frequency increases by 50%. The weight of the structure except for the foundation increases by 200% because of additional external walls, and the stiffness of external walls corresponds to 60% of that of the bare steel frames in the NS direction and 350% in the EW direction. The increase of the damping effect partially caused by the friction of ALC panels is remarkable. The degree of the increase of damping ratio corresponds to the amount of external walls.

At the third construction stage, inner plaster boards and ceilings are installed, leading to the increase of both natural frequency and damping ratio. The weight does not increase very much from the second stage. The stiffness of the building with non-structural members is about 1.8 times larger than that of bare steel frames in the NS direction and about 12.3 times in the EW direction.

At the fourth construction stage, partition walls are installed in the NS direction. The increase of natural frequency is large in the NS direction; however, the damping ratio does not vary much. At this stage, the stiffness of this building is about 4 times that of the bare steel frames in the NS direction and about 15 times in the EW direction.

These results show that the stiffness of non-structural members plays an important role under excitation with small amplitude levels such as traffic vibration problems. The difference of dynamic properties between the NS and EW directions is caused by the difference in the arrangement of the non-structural members. The natural frequency of a building can be adjusted relatively easily, and, accordingly, the vibration of the building can be controlled by arranging non-structural members appropriately.

5. Effects of non-structural members on amplitude dependence (type A)

Amplitude dependences of natural frequencies and damping ratios on the non-structural members are examined under excitation with relatively small amplitude levels. The natural frequencies and damping ratios of the building estimated using the moving average method for the free vibration response are shown in Figs. 20 and 21, respectively. The dynamic properties estimated from microtremor records by the RD

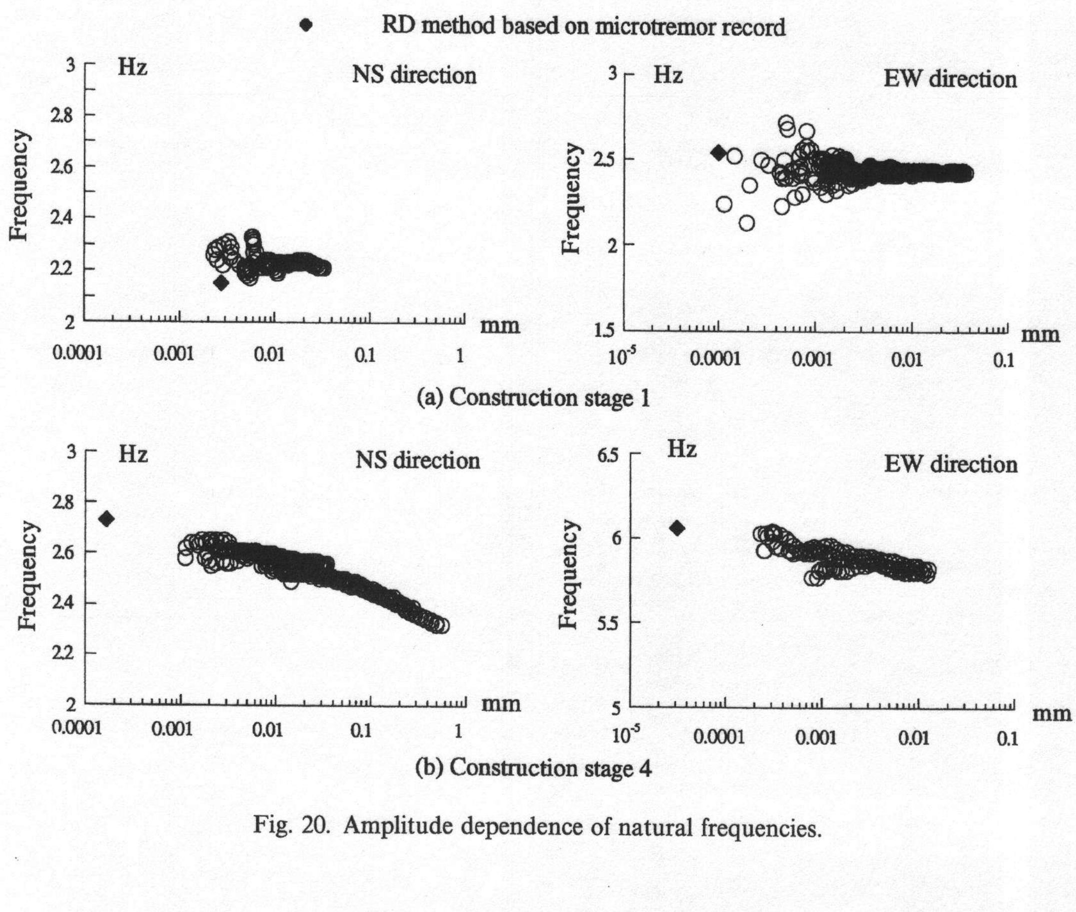


Fig. 20. Amplitude dependence of natural frequencies.

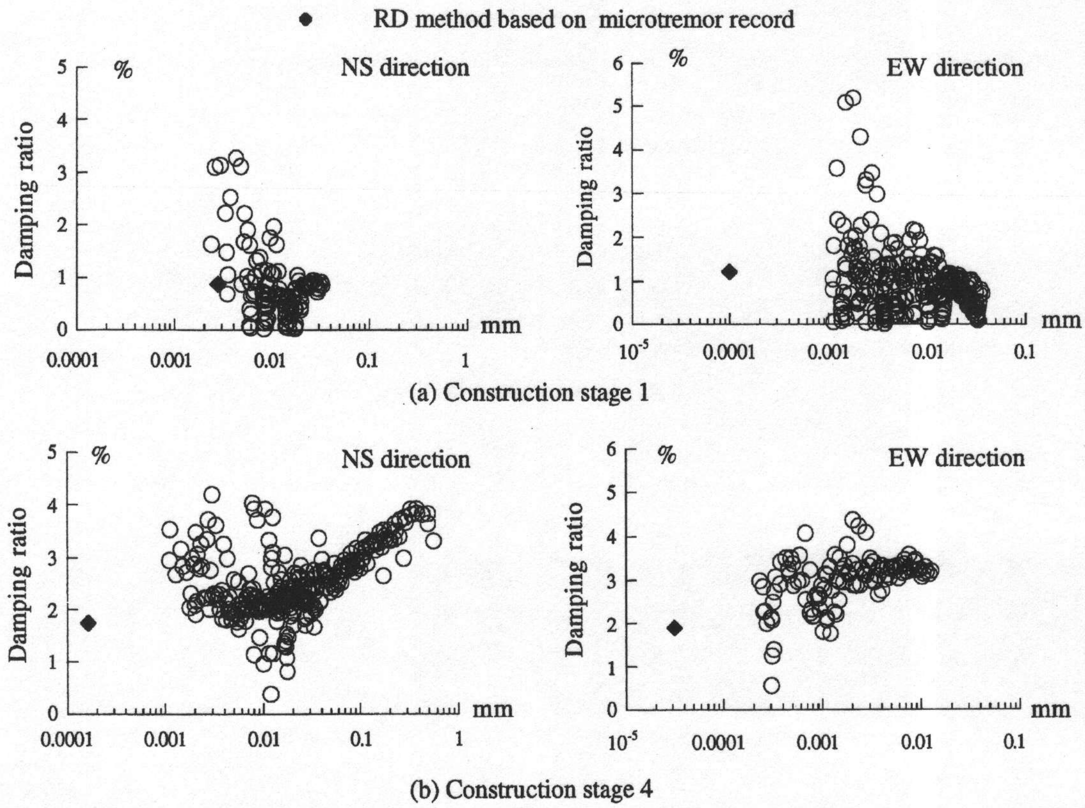


Fig. 21. Amplitude dependence of damping ratios.

method are also plotted in these figures. The natural frequency does not depend on the amplitude at the first stage, while it does at the fourth stage. As the amplitude level increases, the natural frequency decreases remarkably. Although at the first stage, the estimated damping ratios are scattered, little dependence of the damping ratio on amplitude levels is observed. At the fourth stage, as the response amplitude level increases, the damping ratio increases. These results are similar to that obtained from the tests of a high-rise building [2]. These results show that the non-structural members contribute not only to the increase of stiffness and damping ratio but also to the amplitude dependences on dynamic properties, when the excitation level is relatively small.

6. Eigenproperties in the range of large amplitude levels (types B and C)

Two kinds of vibration tests (the forced vibration test using a mechanical exciter and the free vibration test using the wire cutting method) were carried out in order to identify the dynamic properties of the building under excitation with large amplitude levels such as large earthquakes.

6.1. *Forced steady state excitation tests using mechanical exciter (type B)*

A building of type B is excited with three levels in both directions, and the responses of the building to the steady state excitation are observed. A building of type B corresponds to the third construction stage of type A, except for the steel stairs and cantilevers.

Fig. 22 illustrates the ensemble averages of power spectra of microtremor records on the third floor observed before the forced vibration tests. In Fig. 23, the response

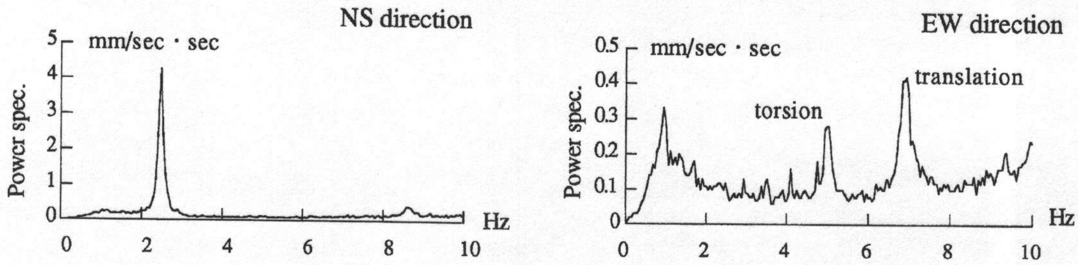


Fig. 22. Ensemble average of power spectra of microtremor record.

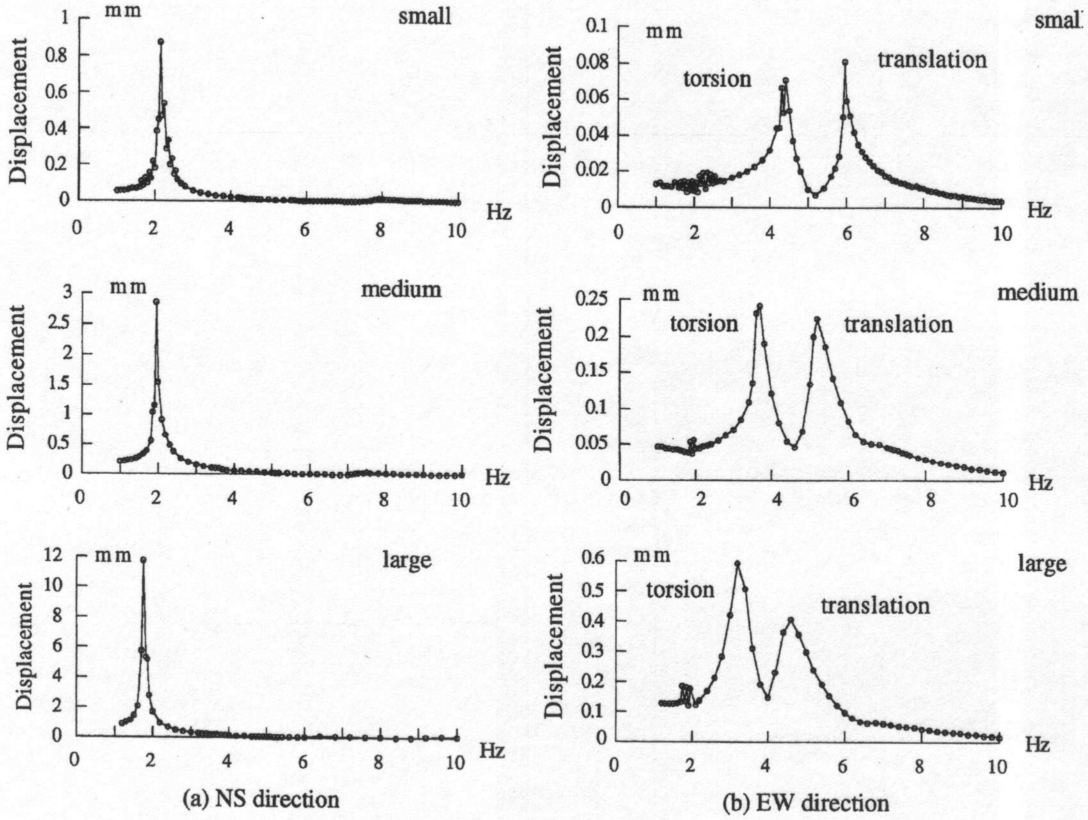


Fig. 23. Response curves subjected to constant sinusoidal excitation force.

curves obtained from the forced vibration tests are shown for different excitation levels. The predominant frequency decreases as the response amplitude of the building increases. It is expected that under excitation with a larger amplitude level, the contribution of non-structural members to the stiffness is smaller than that with a small amplitude level. The relation between response amplitude and resonance frequency and the relation between response amplitude and damping ratio obtained from Fig. 23 are shown in Figs. 24 and 25, respectively.

After the forced vibration tests, damage to the inner walls was observed. To clarify the effect of such damage to the non-structural members, the dynamic properties of the building before the forced vibration tests are compared with those after the tests. The free vibration responses obtained before and after the forced vibration tests and their Fourier spectra are shown in Fig. 26. Both responses are caused by almost the same small excitation force levels. The predominant frequencies in both directions decrease, and it is expected that the contribution of non-structural members to the

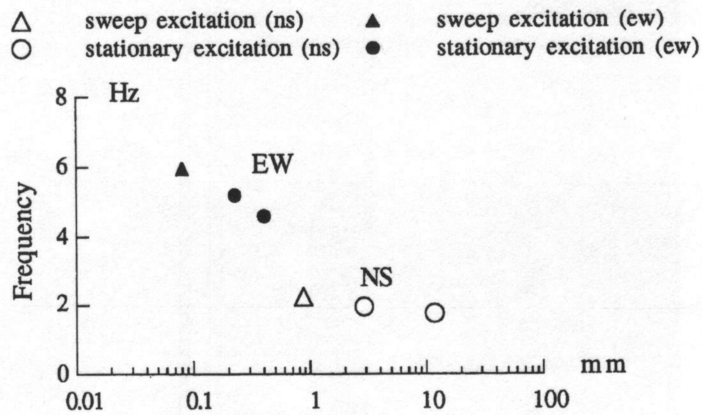


Fig. 24. Response amplitude–natural frequency relation based on forced excitation test.

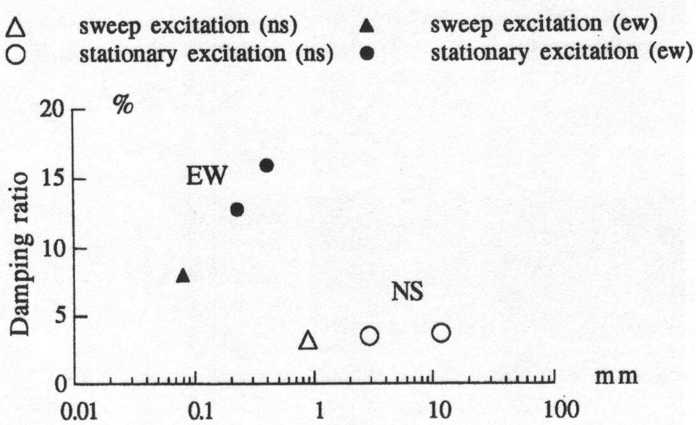


Fig. 25. Response amplitude–damping ratio relation based on forced excitation test.

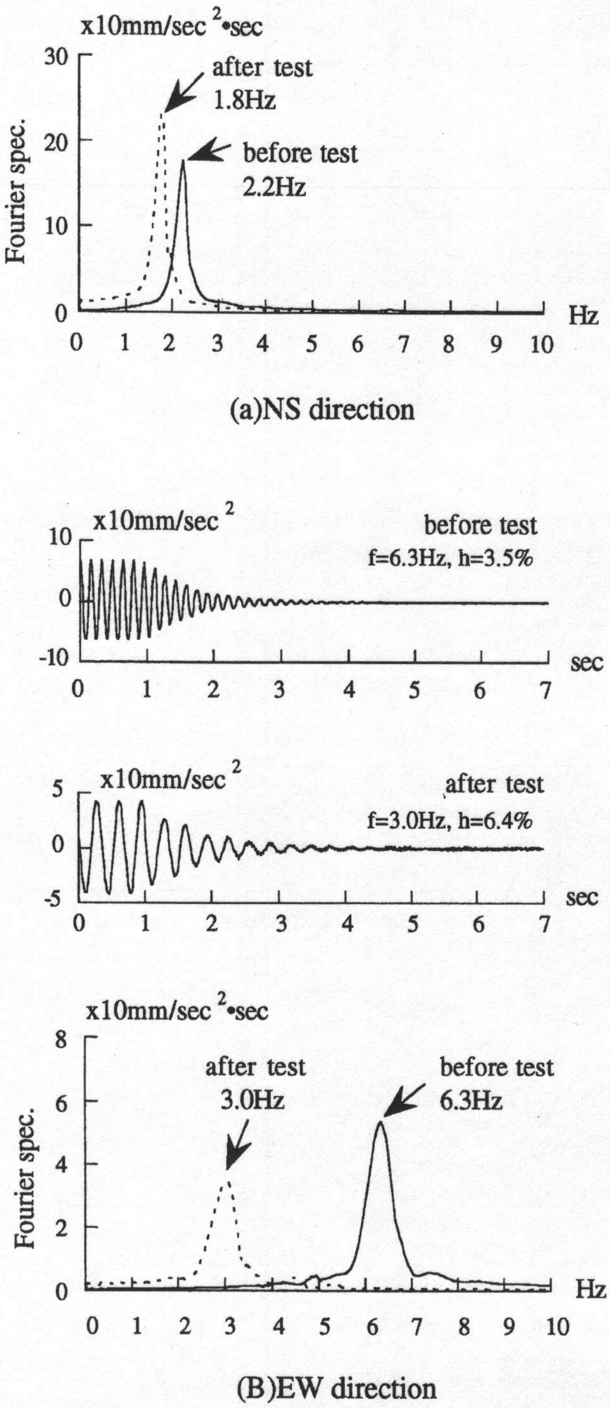


Fig. 26. Difference of free vibration responses between before test and after test.

stiffness declines because of the damage to them. This phenomenon is remarkable especially in the EW direction in which there are a large quantity of nonstructural members, and the increase of damping ratio is also observed in this direction.

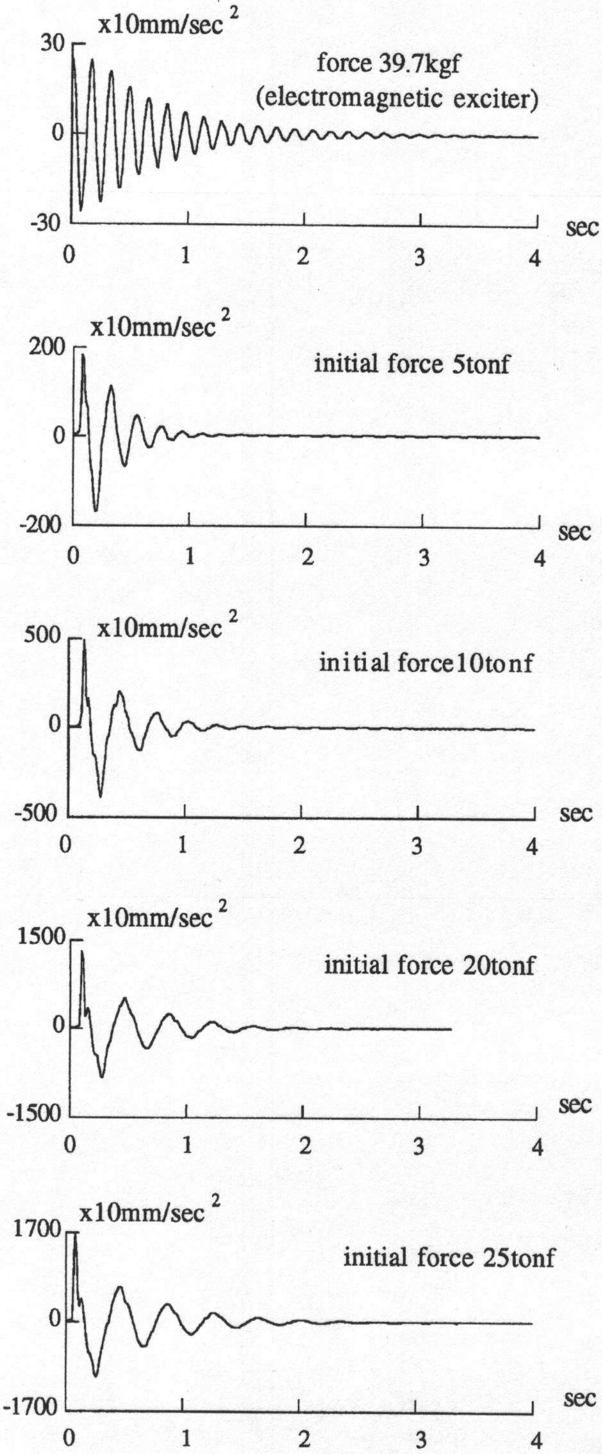


Fig. 27. Free vibration response for different amplitude levels.

6.2. *Free vibration tests using wire cutting method (type C)*

The free vibration tests in the EW direction using the wire cutting method are carried out for a building of type C. Four initial tensile load levels (5, 10, 20, 25 tonf) are prepared, and after the steel piece is broken (see Fig. 4), the free vibration responses of the building are observed.

The free vibration responses on the roof obtained from the tests for each tensile load level are shown in Fig. 27. The free vibration response caused by the electromagnetic exciter is also illustrated in Fig. 27. In Figs. 28 and 29, the relation between initial displacement and natural frequency and the relation between initial displacement and damping ratio are shown, respectively. The decrease of natural frequency is observed as the initial displacement increases. Unlike the previous results under excitation with small amplitude, the damping ratio decreases as the response increases above

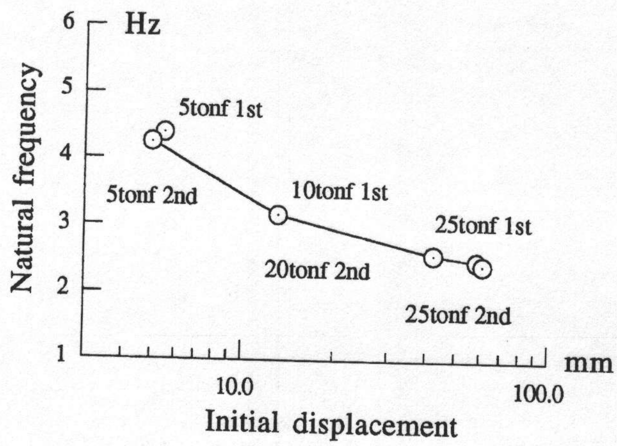


Fig. 28. Initial displacement–natural frequency relation based on the wire cutting method.

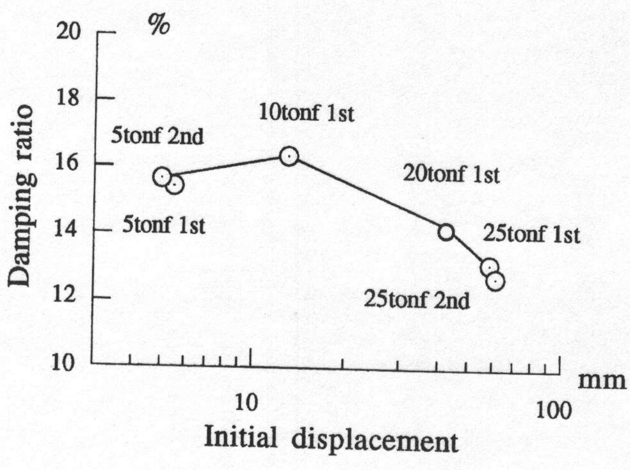


Fig. 29. Initial displacement–damping ratio relation based on the wire cutting method.

10–20 mm, although it increases at first. In order to examine the amplitude dependence in detail, the natural frequencies and the damping ratios estimated from the free vibration responses by the moving average method are shown in Figs. 30 and 31. These figures show similar amplitude dependence of dynamic properties to that observed in Figs. 28 and 29. It should be noted that the natural frequency decreases as the tests with large load which would cause damage to the non-structural members are repeated.

The static loading test from 0 to 50 tonf is also carried out. After all the tests finished, the microtremor response on the third floor is observed to clarify the effect of the damage to the non-structural members.

The free vibration responses on the third floor generated by the RD method before the series of tests are compared with those after the tests in Fig. 32. In Fig. 32, the

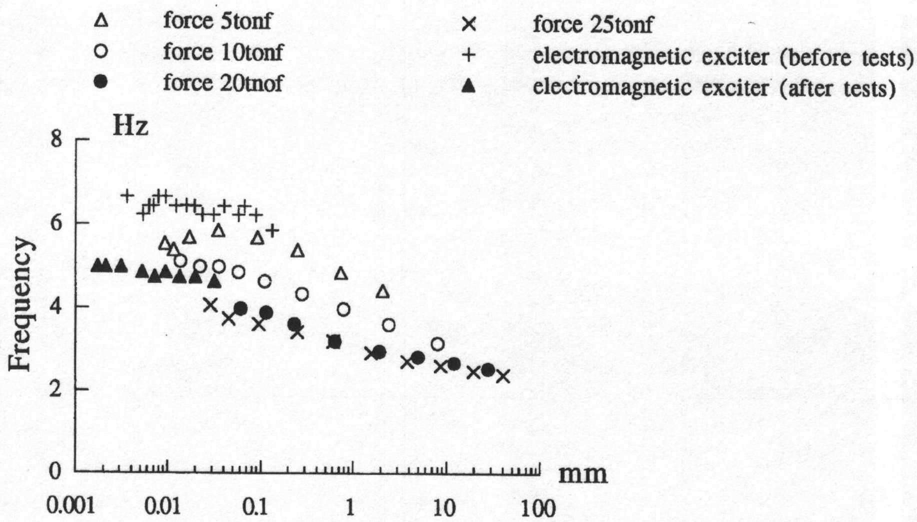


Fig. 30. Amplitude dependence of natural frequencies based on free vibration tests.

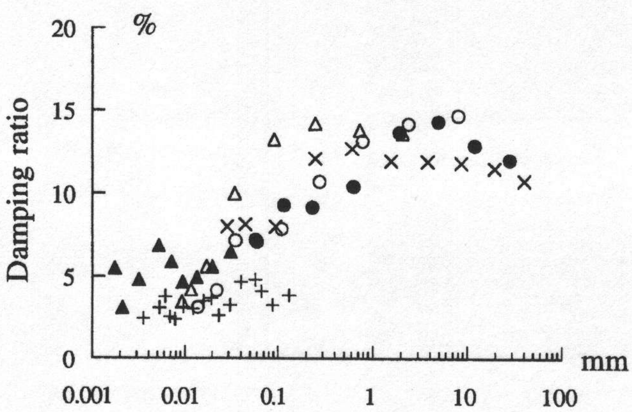


Fig. 31. Amplitude dependence of damping ratios based on free vibration tests. The symbols have the same meaning as in Fig. 30.

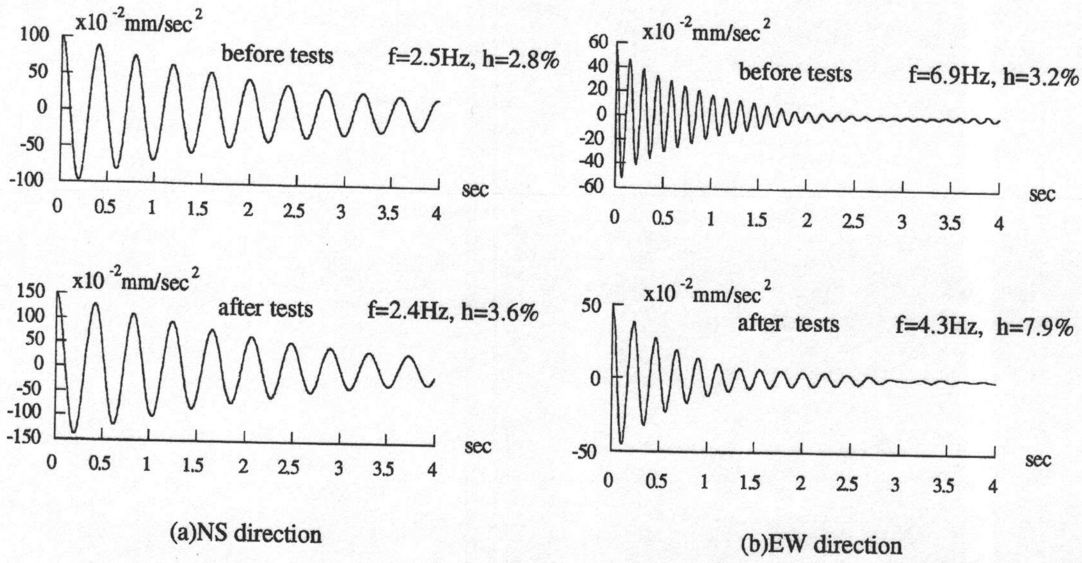


Fig. 32. Free vibration response generated by the RD method.

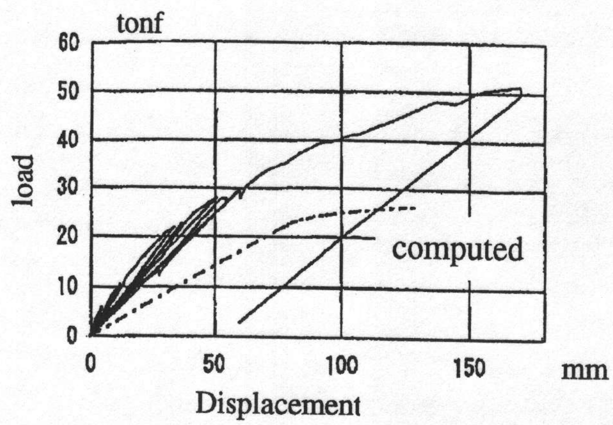


Fig. 33. Displacement–load relation based on static loading tests.

estimated natural frequencies and damping ratios are also shown. In the EW direction in which all the tests are carried out, the decrease of natural frequency is large; on the other hand, it is small in the NS direction. Based on the results of the static loading test, the load–displacement relation and the load–tangent modulus relation are compared with those calculated using the bare steel frames in Figs. 33 and 34, respectively. It is shown in Fig. 33 that the steel frames of this building are in the elastic region at the load of 25 tonf, though the ultimate strength of the building assumed in the design is 25 tonf. Fig. 34 shows that after the static loading test, the non-structural members are severely damaged; however, the stiffness of this building remains to be about 2 times larger than that of the bare steel frames.

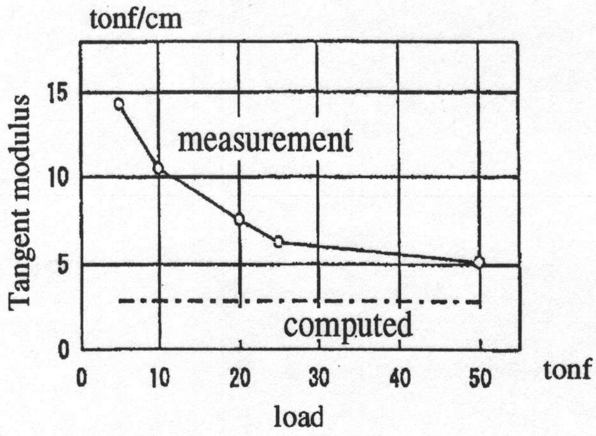


Fig. 34. Load–tangent modulus relation based on static loading tests.

7. Conclusions

The results of the series of vibration tests over a wide range of amplitudes using the full-scale buildings can be summarized as follows.

(i) Since the values of damping ratio varies with the identification methods and excitation conditions, it is important to select appropriate methods.

(ii) Putting the non-structural members such as ALC walls, inner plaster boards and partition walls on the bare steel frames, the natural frequency increases from 2.4 to 6.1 Hz in the EW direction at relatively small response levels. Taking the increase of the weight of building into account, the non-structural members provide 14 times larger stiffness than structural members. The damping ratio also increases with the non-structural members.

(iii) The dynamic properties strongly depend on the response amplitudes when the non-structural members exist.

(iv) In the experience of large response, the natural frequency of the building decreases from 6.9 to 4.3 Hz in the EW direction. This large decrease is due to the damage by large deformation to a lot of non-structural members arranged in the EW direction.

When the response amplitude level is small, the non-structural members play an important role on the dynamic properties of a building. In the range of large amplitude levels, the contribution of non-structural members to the stiffness decreases. However, the stiffness and the ultimate strength become almost twice as large as those of the bare steel frames and large damping can also be expected. It is necessary to evaluate properly the effects of non-structural members depending on amplitude levels, when the dynamic structural model of a prefabricated building is established.

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