Observed and Calculated Vibrations of Tall Buildings during a Typhoon

Calcul et mesure de vibrations de bâtiments élevés lors d'un typhon

Beobachtete und berechnete Schwingungen von Hochhäusern während eines Taifuns

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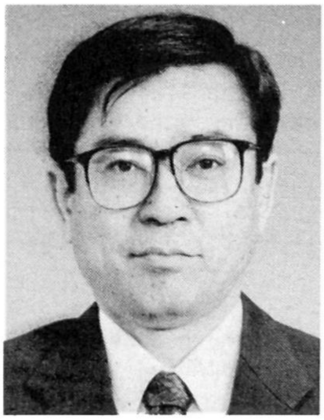

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SUMMARY

Very few studies have been conducted on human response to wind-induced vibrations in tall buildings. An extremely strong typhoon passed through over Tokyo in 1979. The wind-induced vibrations were calculated for ten high-rise buildings, using the wind velocity observed at the time of the typhoon. For three buildings, the calculated vibrations were verified by instrumental observations. An inquiry was made with 2500 employees concerning the human response to the vibrations during the typhoon. The answers were analysed statistically.

RÉSUMÉ

Très peu d'études existent sur les sensations humaines résultant de vibrations de bâtiments élevés, provoquées par le vent. Un typhon violent a eu lieu à Tokyo en 1979. Les vibrations provoquées par le vent ont été calculées pour dix bâtiments élevés, a partir de la vitesse du vent au moment du typhon. Pour trois bâtiments, les vibrations calculées ont été vérifiées à l'aide d'instruments d'observation. Une enquête sur les sensations perçues a été réalisée auprès de 2500 employés se trouvant dans ces bâtiments lors du typhon. Les résultats ont fait l'objet d'une analyse statistique.

ZUSAMMENFASSUNG

Es gibt wenige Untersuchungen über die Auswirkung von Schwingungen, die durch Taifune ausgelöst werden, auf Menschen in Hochhäusern. Ein Taifun mit ausserordentlich starken Windgeschwindigkeiten ereignete sich 1979 in Tokio. Die durch den Wind ausgelösten Schwingungen, wurden bei zehn Hochhäusern auf der Basis der während des Taifuns gemessenen Windgeschwindigkeiten berechnet. Bei drei Hochhäusern wurden die berechneten Schwingungen anhand von Beobachtungen mit Messgeräten überprüft. Rund 2500 Angestellte, die sich während des Taifuns in den Hochhäusern aufhielten, wurden über ihre Empfindungen befragt. Die Antworten wurden statistisch ausgewertet.

1. INTRODUCTION

The interest in wind-induced vibrations has increased recently and analytical methods to predict vibrations have also appeared in the codes of several countries such as Canada and Japan. However, very few studies have been conducted on human responses to wind-induced vibrations due to lack of opportunity for surveys and observations(1).
The Typhoon #7920 hit the Tokyo area on Oct. 19, 1979. This was the strongest wind since the very first tall building, the Kasumigaseki Building (height of 147m), was constructed in Japan in 1968. Utilizing this occurrence, the vibrations of three tall buildings were measured in the form of acceleration or displacement, and by using these records, the reliability of the buffeting theory for predicting wind-induced vibration of buildings was verified.
After the typhoon, questionnaires regarding human responses were distributed to 2,500 personnel working in ten tall office buildings constructed with structural steel frames. The answers to questionnaires allowed the correlation between human reactions and building vibrations to be investigated.

2. BUILDINGS

Table 1 shows the external dimensions and the fundamental periods of the eleven tall buildings included in the study. This includes Building B where only the wind and the vibration of the building were measured. The investigated buildings are spread out in Tokyo city as shown in Fig.1. Among them, five buildings (A~F) are quite close to each other in the Shinjuku New City Center of Tokyo as shown in Fig.2. These eleven buildings are all office buildings except the hotel of Building E. Building K has a radio transmission tower of 150m height on the roof.

Table 1 Dimensions and Fundamental Period (T sec) of Selected Buildings in Tokyo that were Subjected to Strong Wind during Typhoon #7920.

Building		*1,2,3 A	*1,2 B	*3 C	*3 D	*3 E	*3 F	*3 G	*3 H	*3 I	*3 J	*1,2,3 K
Length (m)		59.8	51.4	58.7	66.0	79.8	52	71.7	84.2	51.4	37.5	70.0
Width (m)		45.9	48.4	59.2	30.0	25.9	34	44.1	42.4	48.4	22.5	32.4
Height		211	165	200	193	170	203	226	147	152	65	58 *4
T (sec)	Shortspan-wise	5.07	4.17	5.07	4.76	5.3	5.42	5.97	4.44	4.17	2.4	2.46
	Longspan-wise	5.16	4.17	5.02	4.51	4.4	4.27	4.59	3.60	4.17	2.2	2.37

*1 : Wind observed. *2 : Acceleration or displacement observed.
*3 : Questionnaires distributed. *4 : Height of 150m tower on the roof not included.

3. WIND VELOCITY AND WIND DIRECTION

Extremely strong wind was recorded from about 11:00 to 16:00 on Oct.19, 1979, during Typohoon #7920. The nearest location that the typhoon passed through was about 200 km away from Tokyo and the direction was NW at 13:30.
The wind conditions were observed at the roof of Buildings A and B, and at the top of the radio transmission tower of Building K.
For Buildings A and B, a propeller type anemometer was used to measure the mean and maximum instanteneous wind velocities as well as the wind direction every ten minutes. For Building K, the wind velocity and the wind direction were measured with an ultrasonic anemometer and fourteen time history of fifteen minutes was recorded on magnetic

tape every thirteen minutes from 11:00.
Table 2 shows the maximum wind velocities and wind directions observed, together with the data offered by the Meteorological Agency of Japan (MAJ) located in the central part of Tokyo. Fig.3 shows the records of wind velocity, wind direction and atmospheric pressure measured at Building B.
With consideration given to the locations and the heights of these buildings, the appropriate wind records for each of the ten buildings were respectively selected for analyzing the building vibrations. That is, the wind records of Building B were used for Building A~H, the MAJ data were used for Building J, and those of Building K were used for Building I and K.

Table 2 Maximum Wind Velocity and Wind Direction of Bldgs. A and B (Shinjuku Area) and Bldg. K and MAJ (Central part) of Tokyo.

	Height (m)	Time	Direction	10 minutes-averaged wind velocity(m/s)	Maximum Instantaneous Wind Velocity(m/s)
Building A	237	15:40	SSE	34.9	47.8
Building B	187	14:40	SSW	38.9	52.7
Building K	187	14:50	SSW	24.4	37.4
MAJ *	74.6	13:30	S	17.5	38.2

* : Meteorological Agency of Japan.

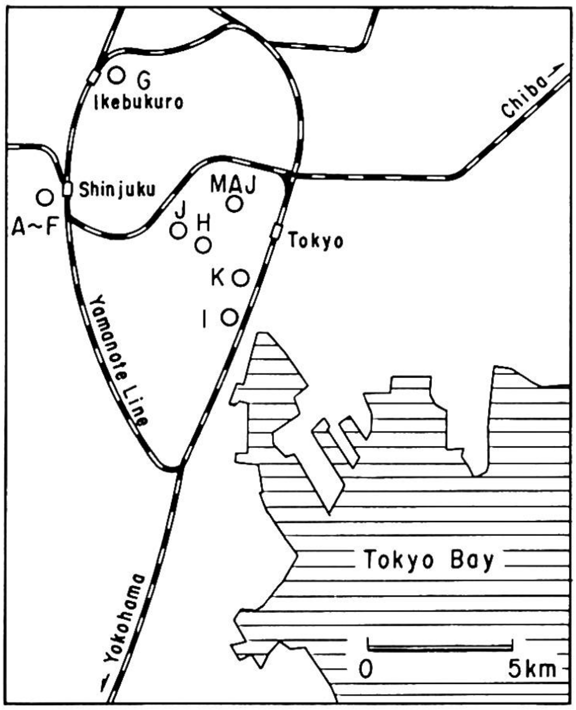


Fig.1 Map of Tokyo and Locations of Buildings Pertained to Wind Induced Vibration Study for Typhoon #7920.

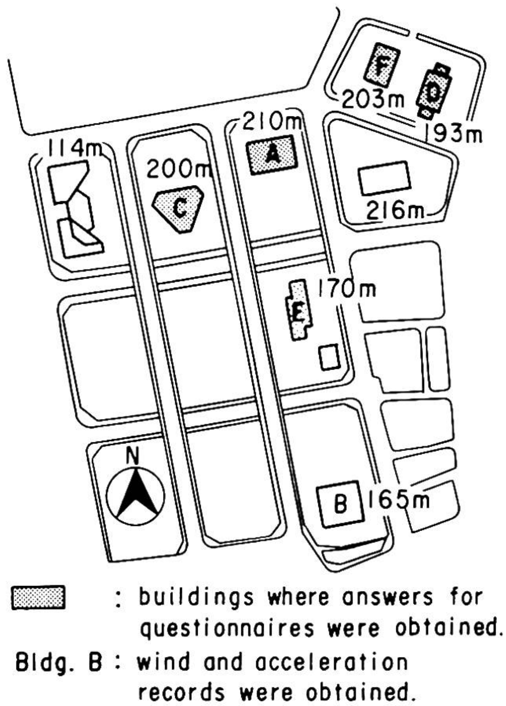


Fig.2 Shinjuku New City District of Tokyo and Locations of Buildings where Questionnaires were Distributed.

4 . MEASUREMENT AND ANALYSIS OF BUILDING VIBRATIONS

(1) Measurement of Building Vibrations

The vibrations were recorded at the top floor of Building A and B, and at the top of the tower on Building K during Typhoon #7920.

(1) At the top floor of Building A, the acceleration was measured with a portable accelerometer (Vibration Technical Co. Ltd : MS1000). Fig. 4 shows one of the records.

(2) At the top floor of Building B, a visual observation of the maximum acceleration was made by using a simple accelerometer (Akashi Seisakusho Co Ltd : ADA-2). The accelerometer detects the acceleration by an instrument in which a ball rolls off a flat plate, when the acceleration becomes greater than 8~ 10 cm/s^2 .

(3) At the tower on Building K, the displacements were measured at 197 m height with an installed made-to-order displacement-meter (Tokyo Sokushin Co. LTD : SA-151). The displacement is recorded through an integral circuit which converts acceleration to displacement. Fig.5 shows the trace of movement at the top of the tower during Typhoon #7920.

(2) Theoretical Method of Analyzing Building Vibrations

The vibrations of the buildings were calculated in the frequency domain.
The equations of the theory are described in detail in the Appendix, along with several assumptions and related empirical findings.
The characteristics of this method were as follows:

(1) The fluctuations of the approach wind velocities and directions were considered as the external forces. The vortex induced forces were, however, not included due to lack of information obtained at the time.

(2) The intensity of wind turbulence is necessary to define the power spectrum of the wind. Therefore, the empirical equations were set up as a function of only the mean and maximum instantaneous wind velocities.

(3) The drag coefficients were obtained by means of wind tunnel tests for each building.

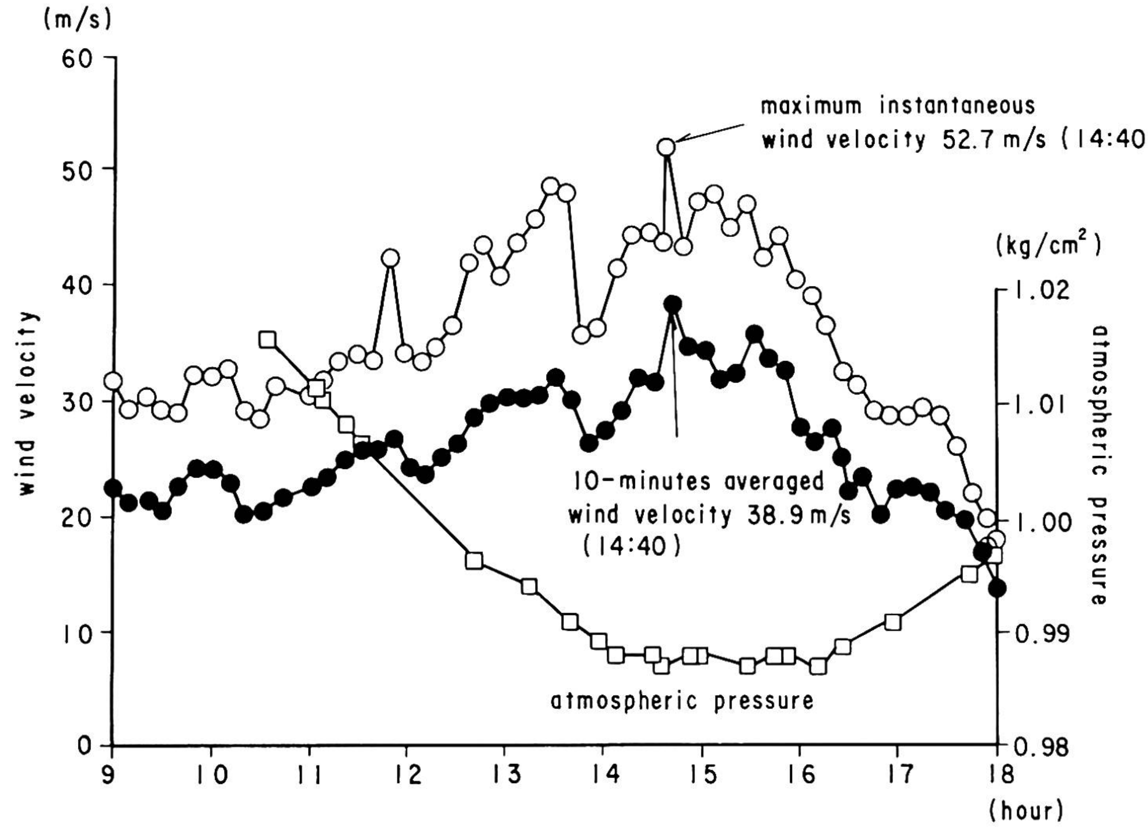


Fig.3 Wind Velocity at 187m Height and Atmospheric Pressure at Building B in Shinjuku Area during Typhoon #7920.

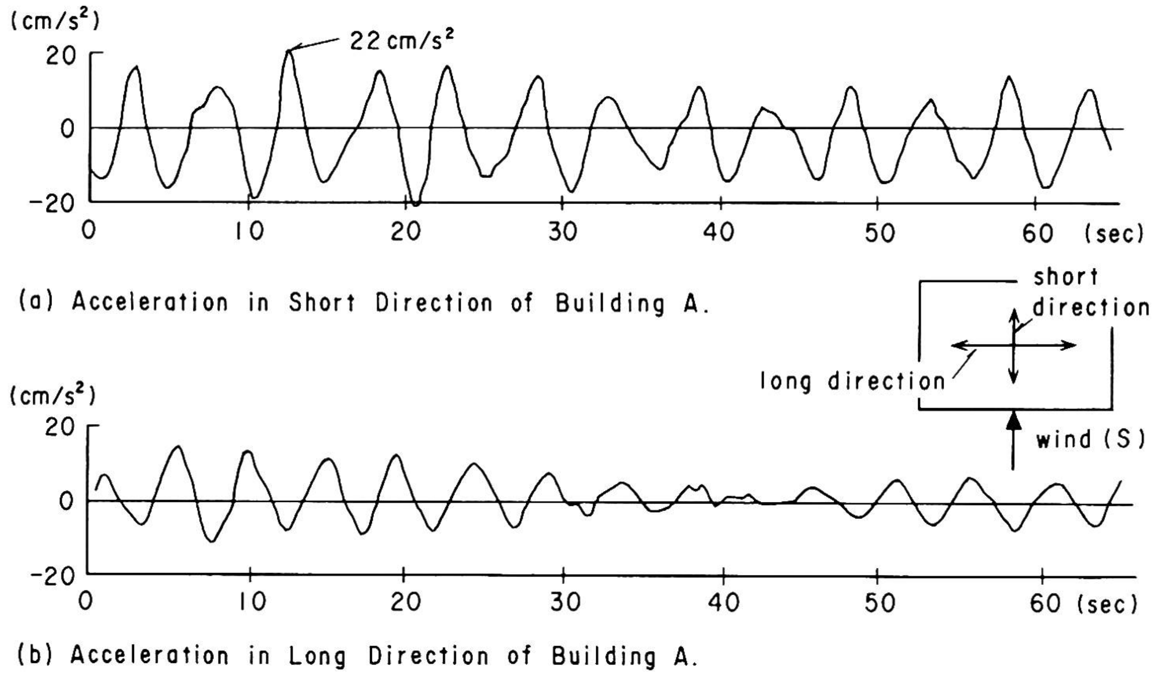


Fig.4 Example of Wind Induced Accelerations as Measured by Portable Accelerometer at the Top Floor of Building A in Shinjuku Area during Typhoon #7920.

5 . QUESTIONNAIRE

The questionnaire had 29 questions, and each person was asked to fill in the form individually. The questionnaire items [1] are classified into three main groups as follows:

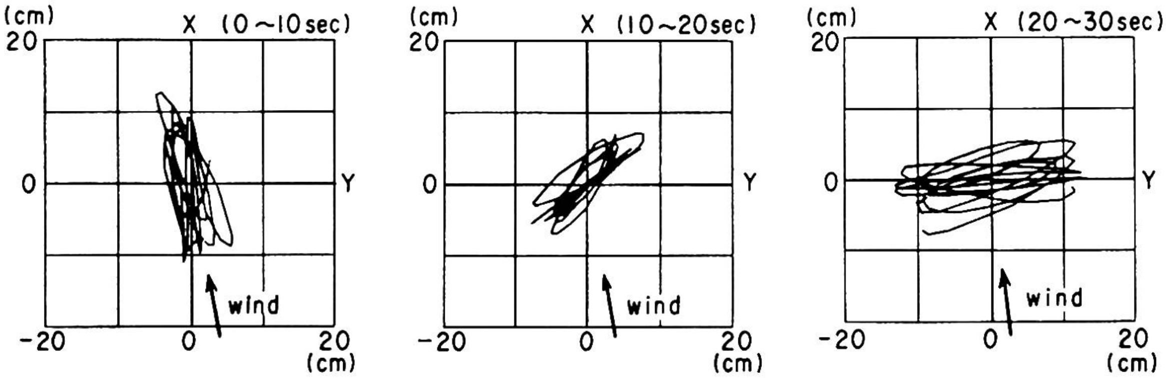
(1) 7 questions regarding the characteristics of the subject group, such as age, sex, working floor, physical condition, etc.
(2) 13 questions concerned with the human response to wind induced vibrations, such as vibration felt, duration of the vibrations, work and physical discomfort, acceptable typhoon recurrence period, etc.
(3) 9 other questions dealt with vibrations similar to this typhoon, for exmple, earthquake experiences, using the elevator, personal opinion, etc.

About 300 ~400 sets of papers containing the questionnaires were distributed to the personnel of ten buildings. Each of the ten buildings was classified into 3 blocks according to the height levels of each building floor. The aimed sampling ratio was 2:3:4 (lower : middle : upper) by height level. The answers were collected from Nov. 12 to Nov. 22, 1979, but of the total number of answers received only about 2,500 were valid.

6 . COMPARISON BETWEEN MEASURED AND COMPUTED VIBRATIONS OF BUILDINGS

(1) Maximum Acceleration at the Top Floor of Building A

The maximum accelerations measured with a portable accelerometer and three acceleration curves are shown in Fig. 6. The wind velocity of abscissa of this figure was measured at Building B. During Typhoon #7920, two records were obtained when the wind directions were SSW and S. The maximum accelerations for the records of SSW were 40 cm/s^2 in both the short and the long directions of the building, when the 10-minutes averaged wind velocity was 38.9m/s. For the record when the wind direction was S, the maximum accelerations were 22 cm/s^2 in the short direction and 20cm/s^2 in the long direction of the building and the 10-minutes averaged wind velocity was 35.1 m/s. Included herein, for information are two other winds of E and ESE which were recorded later during another Typhoon #8115 on Aug. 22, 1981. The maximum accelerations of these records were 4~6 cm/s^2 in the short direction of Building A and 1~4 cm/s^2 in the long direction and the 10-minutes averaged wind velocities were 17~22 m/s. For example, when the wind direction is S and the 10-minutes averaged velocity is 40 m/s, the calculated maximum acceleration is 31 cm/s^2. Meanwhile, the Canadian Code gives the maximum acceleration of 34cm/s^2 in the short direction and 39 cm/s^2 in the long direction of the building. Regarding the Canadian Code, the equivalent wind velocity is assumed to be 38m/s in terms of an hour-averaged velocity at the top of the building.



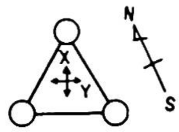


Fig.5 Trace of Movement at Tower Top at 197.4m Height for Building K during Typhoon #7920, when 10 Minutes Averaged Wind Velocity was 23.4 m/s and Wind Direction was SSW.

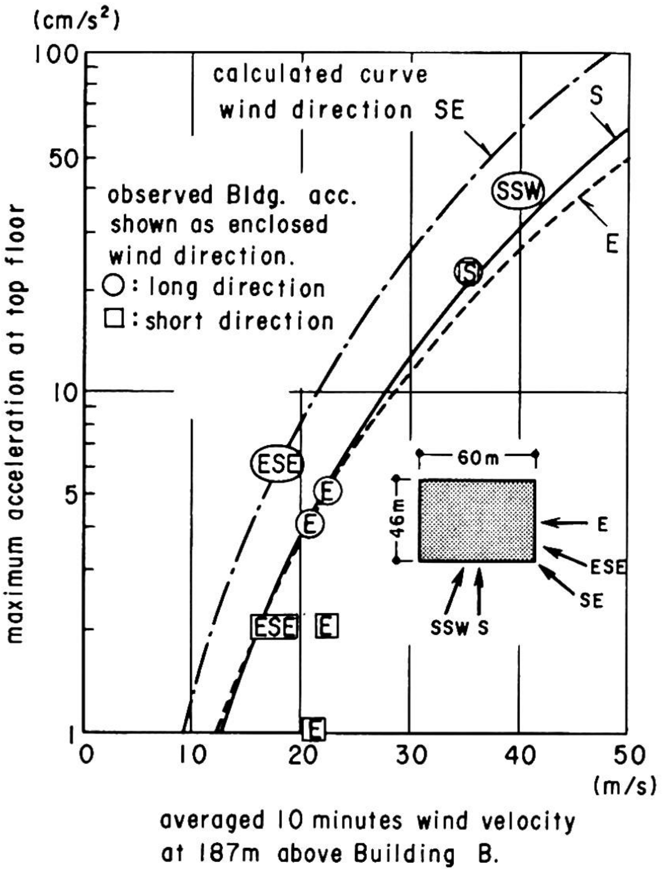
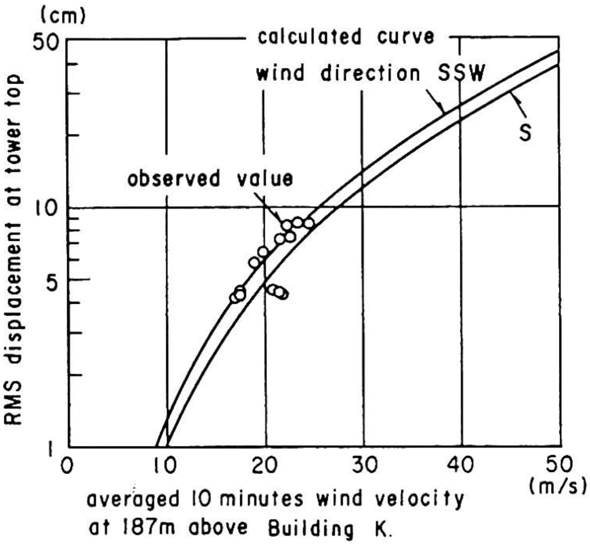
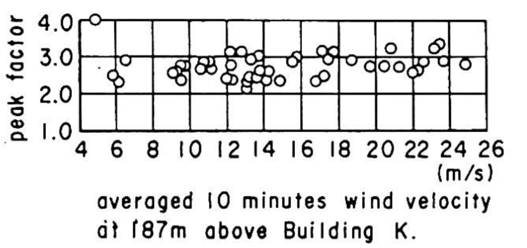


Fig.6 Wind Induced Building Acceleration at Top Floor of Building A in Shinjuku Area during Typhoons #7920(SSW, S) and #8115(E, ESE).



(a) RMS Displacement ($\hat{\sigma}$) of Tower Top (197.4 m height) in Horizontal Direction during Typhoon #7920.



(b) Peak factor ($\hat{g}$) of Tower displacement.

Fig.7 RMS Displacement of Tower Top of Building K with a Made-to-Order Displacement-meter, and with Predicted Curve.

(2) Vibration of Building B

The observation by using a simple visual accelerometer mentioned above also showed the coincidence between the observed and calculated accelerations. The result was that the ball on the plate of this instrument kept rolling off from 13:30, when the acceleration was calculated as 13 cm/s^2. It means that the maximum acceleration was frequently greater than 10cm/s^2. And the ball finally did not fall after 16:20. It means that the actual acceleration was less than 10 cm/s^2, as well as, calculated one.

(3) Vibration of the Tower on top of Building K

Fig.7 shows the root mean square (RMS) displacements measured with a displacement-meter and the calculated ones at the tower top of Building K, where the wind was measured at 187m height during the typhoon. The two curves in the figure correspond to the wind directions of S and SSW in which all calculated RMS displacements during the typhoon are included.

From the foregoing results, it is recognized that the calculated vibrations coincide with the observed ones fairly well and that these accelerations could be used to analyze the answers of the questionnaires.

7. RELATION BETWEEN QUESTIONNAIRE RESULTS AND BUILDING VIBRATIONS

7.1 Regression Analysis

The answers to the questionnaires were, firstly, classified into ten individual buildings, and further, into three height level groups per building. In several buildings, including Building A, where a great number of answers were obtained, the classification by height level was additionally conducted for every five stories.
Table 3 shows an example of the classifications by height level. The accelerations used to analyze the questionnaires are also included.
Allocating the calculated accelerations in the form of logarithm as the variables to analyze the questionnaires, the linear regression analysis for each question was conducted. The number of answers was also considered in the regression analysis.

7.2 Results of Questionnaire

Fig.8 ~10 show the results regarding the items, "Occurrence Frequency of Vibration Felt by Personnel", "Extent of Disturbance due to Building Vibration Experienced by Personnel" and "Duration of Discomfort and Nausea Experienced by Personnel due to Building Vibration". The figures on the left, Fig.8~10(a), represent the results of Building A where the answers were classified by every five stories of the height level and the right side, Fig.8 ~10(b), represent the results of all ten buildings where the answers were classified into three blocks by the height levels of each building floor. The regression lines for each question represent the statistical boundary between two neighbouring zones expressing the cumulative percentages.
It may be recognized from the figures that the degree of disturbance increased as the acceleration became stronger.

(1) Histogram

The histogram was scattered due to the difference in the working conditions of personnel. However, the relation between the histogram and the acceleration was found to be as follows:

i) When the maximum acceleration was 2~3 cm/s^2, the percentage of the personnel who felt the vibration was less than 4 %. When exceeding 4 cm/s^2, most of the personnel felt the building vibration.

ii) When the maximum acceleration was less than 3cm/s^2, disturbances to their work did not occur.

iii) Exceeding 10 cm/s^2, the disturbance became greater abruptly and many of the personnel experienced nausea.

Table 3 Results of Questionnaire Regarding Occurrence Frequency of Vibration Felt by Personnel during Typhoon #7920, as Classified into 3 Groups of Height Levels in Each Building.

Building	Floor	Number of Personnel (1)	(2)	(3)	(4)	(5)	Total	Predicted Max. Acc. (cm/s^2)
A	lower	28	15	16	4	14	77	9.41
	middle	73	21	7	2	2	105	14.77
	upper	126	36	13	3	1	179	21.26
C	lower	29	18	21	6	12	86	9.53
	middle	34	20	10	5	7	76	14.30
	upper	140	63	14	9	13	239	20.48
D	lower	5	4	9	0	10	28	9.02
	middle	6	12	4	0	2	24	13.80
	upper	30	8	3	1	1	43	19.86
E	lower	4	0	2	0	0	6	15.91
	middle	5	1	0	0	0	6	27.48
	upper	22	7	0	0	0	29	50.45
F	lower	38	18	6	4	1	67	11.64
	middle	70	30	3	0	1	104	17.58
	upper	166	33	4	1	3	207	23.78
G	lower	15	10	11	3	20	59	11.01
	middle	12	16	10	4	11	53	17.15
	upper	99	85	35	8	13	240	28.48
H	lower	6	5	12	4	41	68	3.39
	middle	25	30	13	5	8	81	5.27
	upper	28	21	13	8	17	87	6.06
I	lower	0	1	0	0	2	3	10.15
	middle	1	1	0	1	1	4	19.61
	upper	25	9	2	0	0	36	27.45
J	lower	4	11	9	5	52	81	8.21
	middle	18	17	9	0	8	52	13.80
	upper	53	26	6	0	5	90	15.97
K	lower	0	0	1	2	54	57	2.64
	middle	10	24	20	3	45	102	4.00
	upper	12	25	13	2	11	63	4.25

(1) : constantly, (2) : frequently, (3) : occasionally
(4) : seldom, (5) : never felt

(2) Regression line

Depending on the result of the histogram and the regression lines, the percentage of personnel who experienced disturbances in their work and felt nausea at 10 cm/s^2 were as follows:

i) At the maximum acceleration 10 cm/s^2 , the percentage that the personnel experienced difficulty in working was about 5 %.

ii) 10 ~20 % personnel felt nausea all the time at 10 cm/s^2 .

When comparing the regression lines of Building A and all ten buildings, the inclination of Building A was steeper than that of all ten buildings. Besides the personnel working conditions, it is assumed that the calculating method of acceleration has caused the difference in regression lines. This is due to the fact that the calculated accelerations of some of the ten buildings differed from the real ones, because the wind conditions for these buildings were perhaps not appropriate and because the vortex-excited forces might have played a considerable role in building accelerations. Therefore, the accelerations of each building were estimated by using the results of the questionnaires shown in Fig. 8~10, under the assumption that the regression lines of each building were about the same as that of Building A.

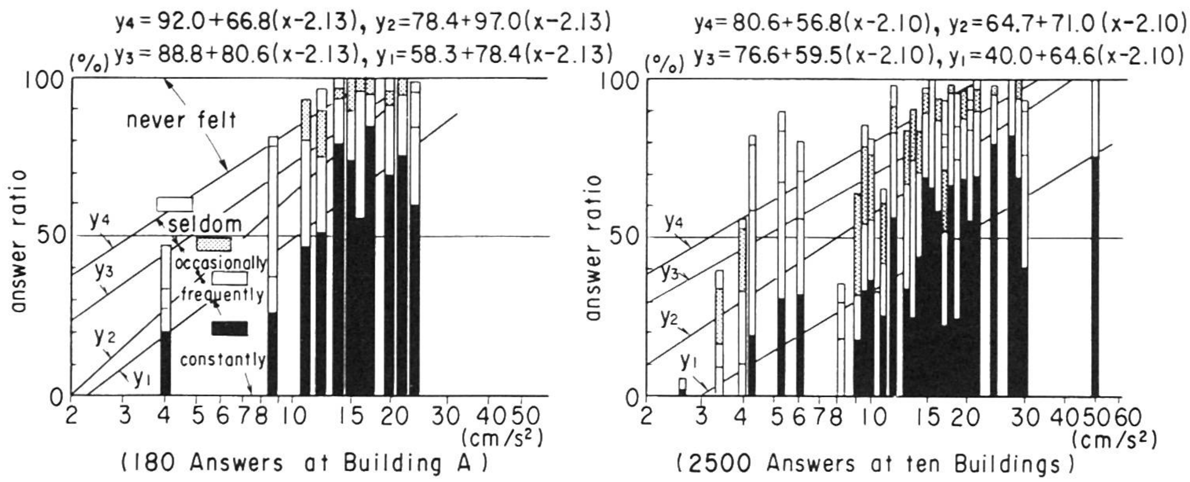


Fig.8 Result of Questionnaire from Bldg.A and ten Buildings.
Re : Occurrence Frequency of Vibration Felt by Personnel during Typhoon #7920.

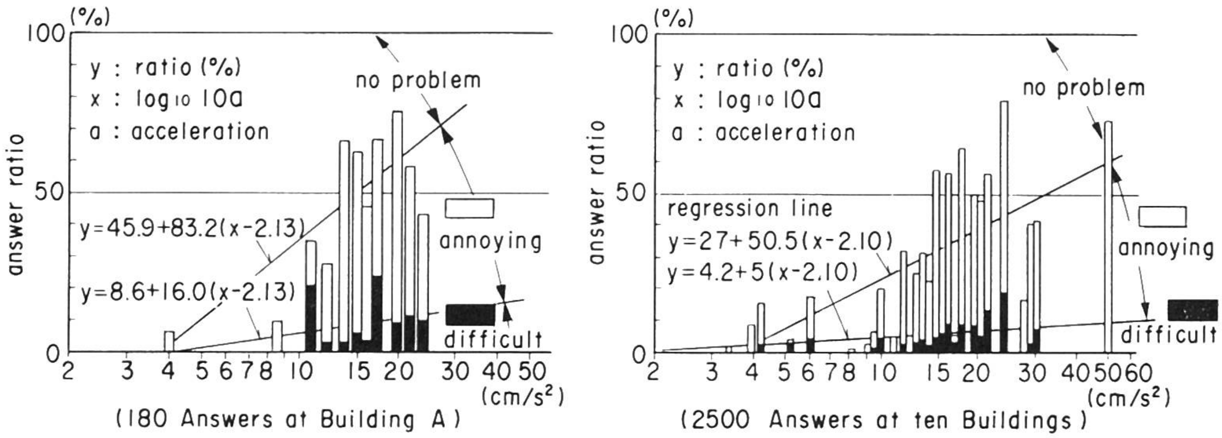


Fig.9 Result of Questionnaire from Bldg.A and ten Buildings.
Re : Extent of Disturbance due to Building Vibration Experienced by Personnel during Typhoon #7920

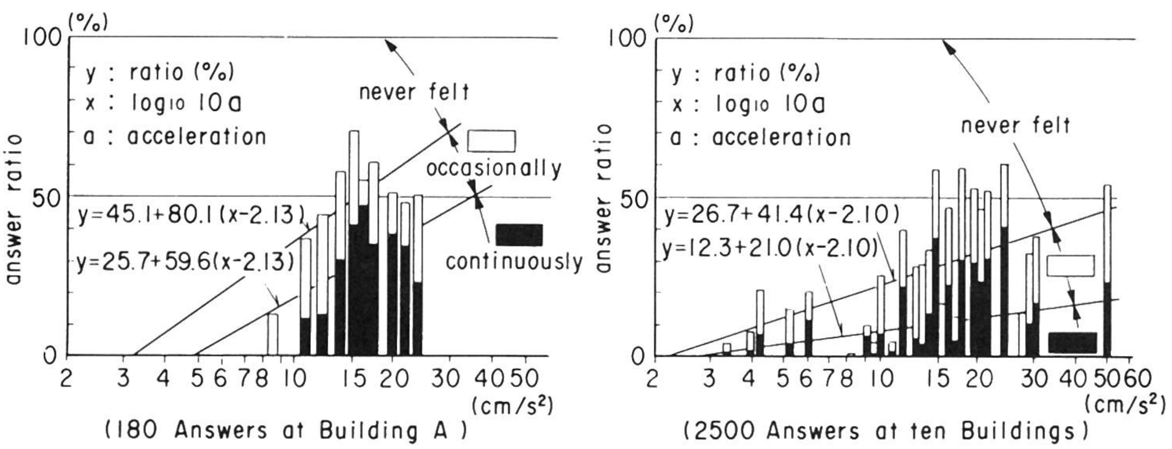


Fig.10 Result of Questionnaire from Bldg.A and ten Buildings.
Re : Duration of Discomfort & Nausea Experienced by Personnel due to Building Vibration during Typhoon #7920.

The estimated results of acceleration can be summarized as follows:

i) At Building G, the inclination ratio of regression line to Building A was least at 0.7. This means that the actual acceleration that occurred in this building was smaller than the calculated one, because this building was not surrounded by other tall buildings and the wind was the strongest in the short direction of building at this time. In other words, the vortex-excited forces were weak at this building and the vibration was mainly induced by the turbulence of the approaching wind.

ii) At Building F, the inclination ratio of regression line was the largest and 3 times that of Building A. This might be caused by the wake turbulence generated in the leeward side of tall buildings surrounding this building. That is, the mean wind velocity and the mean drag coefficient were small but the wind turbulence that was not considered in the prediction of acceleration, shown in Appendix, was large.

iii) At Building H, where the wind blew in the long direction of the building, the inclination ratio of regression line was about 2 times that of Building A. In this case, the vortex-excited forces apparently influenced the vibration.

8. CONCLUDING REMARKS

The results of this investigation are summarized as follows:

(1) The calculated accelerations agreed well with the observed ones.

(2) It was found that there was a linear correlation between logarithmic accelerations and the questionnaire results, which indicated that the human response increased as the acceleration became larger.

(3) The regression lines of questionnaires for Building A and for all ten buildings were not well correlated. The inclination of regression line of Building A was steeper than that deduced from all answers of the ten buildings. This may be understood mainly from the circumstance that the limited wind records had been substituted for the wind conditions on all buildings. The vortex-excited force that was omitted in the prediction might also have played a role in generating acceleration.

(4) From the results of this study, the authors would like to propose that the serviceability threshold should be about 10 cm/s^2 for office buildings when the recurrent period is assumed at five to ten years and the fundmental period is 2 ~6 sec.

ACKNOWLEDGEMENT

The authors wish to express sincere acknowledgements to the building owners and to the personnel who filled in the questionnaires. The initial survey using the questionnaires was prepared by Prof. T. Goto of Hosei University and the authors[1].
The authors conducted the observations of wind and vibrations and the correlation analysis with the collaboration of Dr. Tsugawa, Mr. Horikoshi, Mr. Sanada, Mr. Hongo of the Kajima Institute of Construction Technology.

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APPENDIX

(1) Spectral Modal Method

The equation of motion is represented as :

$$[m]\{\ddot{y}(t)\}+[c]\{\dot{y}(t)\}+[k]\{y(t)\} = \{p(t)\} \quad \cdots\cdots(1)$$

where y is the displacement, p is the fluctuating wind force component, m is the mass of building, c is the coefficient of structural damping, k is the stiffness of building and t is the time. Utilizing the modal method, the displacement at the height of z is represented as:

$$y(z,t)=\sum_r \Phi_r(z)\, q_r(t) \quad \cdots\cdots(2)$$

where Φ is the eigen-vector of building vibration, q is the generalized time function of displacement and suffix r means r-th mode.
Substituting Eq.(2) into Eq.(1), the modal equation of the r-th mode is derived as :

$$\ddot{q}_r(t)+2h_r\omega_r\dot{q}_r(t)+\omega_r^2 q_r(t) = Q_r(t)/M_r \quad \cdots\cdots(3)$$

where,

$$M_r = \{\Phi_r\}^T[m]\{\Phi_r\} \quad \cdots\cdots(4)$$

$$2h_r\omega_r \fallingdotseq \{\Phi_r\}^T[c]\{\Phi_r\} \quad \cdots\cdots(5)$$

$$\omega_r^2 = \{\Phi_r\}^T[k]\{\Phi_r\} \quad \cdots\cdots(6)$$

By transforming in the frequency domain and by using $i=\sqrt{-1}$ and the angular frequency ω, we obtain

$$q_r(i\omega)=Q_r(i\omega) \div \{M_r[-\omega^2+2ih_r\omega_r\omega+\omega_r^2]\} \quad \cdots(8)$$

where $q_r(i\omega)$ and $Q_r(i\omega)$ are the Fourier coefficients of $q_r(t)$ and $Q_r(t)$ respectively.
According to Eq.(2), the variance of displacement, σ_y^2, is represented as:

$$\sigma_y^2(z)=\overline{y^2(z,t)} = \sum_r\sum_s\overline{q_r(t)\,q_s(t)}\,\Phi_r(z)\,\Phi_s(z) \quad \cdots\cdots(9)$$

By assuming that the damping of the building is negligibly small, the non-diagonal elements ($r\neq s$) are omitted. Then, Eq.(9) is reduced as follows:

$$\sigma_y^2(z)=\sum_r\overline{q_r^2(t)}\,\Phi_r^2(z) = (1/2\pi)\sum_r[\Phi_r^2(z)\int_{-\infty}^{\infty}F_{qr}(\omega)d\omega] \quad \cdots\cdots(10)$$

$$F_{qr}(\omega)=F_{qr}(\omega)=q_r(i\omega)\,q_r^*(i\omega) = Q_r(i\omega)\,Q_r^*(i\omega) \div \{M_r^2[(-\omega^2+\omega_r^2)^2+4h_r^2\omega^2\omega_r^2]\} \quad \cdots(11)$$

where, * represents the conjugate property.
$F_{qr}(\omega)$ is the power spectrum of the generalized time function of displacement.
The variance of acceleration, $\sigma_a^2(z)$, is also represented as:

$$\sigma_a^2(z)=(1/2\pi)\cdot\sum_r[\int_{-\infty}^{\infty}\omega^2 F_{qr}(\omega)d\omega\,\Phi_r^2(z)] \quad \cdots\cdots(12)$$

(2) Generalized Power Spectrum of Wind

The power spectrum $F_{Qr}(\omega)$ of generalized wind force is expressed by the cross spectrum of wind loads as :

$$F_{Qr}(\omega)=Q_r(i\omega)\,Q_r^*(i\omega) = \sum_m\sum_n F_p(m,n;i\omega)\,\Phi_r(z_m)\cdot\Phi_r(z_n) \quad \cdots(13)$$

where, $F_p(m,n;i\omega)=p_m(i\omega)\cdot p_n^*(i\omega)$ is the cross spectrum of wind loads, p_m and p_n, on m-th and n-th mass points.

(3) Wind Load

The wind load P is usually represented as :

$$P(t)=C(t)\,q_V(t) \quad \cdots\cdots(14)$$

where C is the drag coefficient of the building, q_V is the velocity pressure of approach wind.
Assuming that the wind velocity is composed of the average and turbulence components, the wind velocity is divided into :

$$V(t)=[(\bar{V}+u(t))^2+v(t)^2]^{1/2} \quad \cdots\cdots(15)$$

where $V(t)$ is the magnitude of wind velocity, which is the scalar of wind velocity vector. $\bar{V}$ is the mean wind velocity, u and v are the turbulence components in alongwind and acrosswind directions respectively.
By omitting second order terms of the wind turbulences, the elements of Eq.(14) are expressed as follows :

$$q_V(t)=(1/2)\rho[(\bar{V}+u(t))^2+v^2(t)] \fallingdotseq (1/2)\rho[\bar{V}^2+2\bar{V}u(t)] \quad \cdots\cdots(16)$$

$$C(t)\fallingdotseq\bar{C}+(dC/d\theta)\,\theta(t) \fallingdotseq \bar{C}+C'[v(t)/\bar{V}] \quad \cdots\cdots(17)$$

where ρ is the density of air. $\bar{C}$ is the drag coefficient for mean wind direction. $\theta(t)\fallingdotseq v(t)/\bar{V}$ is the fluctuation of wind direction and $C'\fallingdotseq d\bar{C}/d\theta$ is the gradient of the drag coefficient around the mean wind direction.
By considering only the wind turbulence components in Eq.(16) and (17), the fluctuating wind load of Eq. (14) is approximated as:

$$p(t)\fallingdotseq(1/2)\rho[2\bar{C}u(t)+C'v(t)]\bar{V} \quad \cdots\cdots(18)$$

(4) Cross Spectrum of Wind Load

Assuming that the wind turbulences, u and v, are independent of each other, the cross spectrum of the fluctuating wind loads on m-th and n-th mass points is expressed as:

$$F_p(m,n;i\omega)\fallingdotseq(1/2\rho)^2\cdot[4\bar{C}_m\bar{C}_n F_u(m,n;i\omega)+C'_m C'_n F_v(m,n;i\omega)]\bar{V}_m\bar{V}_n \quad \cdots\cdots(19)$$

where, $F_u(m,n;i\omega)=u_m(i\omega)\cdot u_n^*(i\omega)$ is the cross spectrum of alongwind velocity, and $F_v(m,n;i\omega)=v_m(i\omega)\cdot v_n^*(i\omega)$ is the one in acrosswind direction, while $u(i\omega)$ and $v(i\omega)$ are the Fourier coefficients of $u(t)$ and $v(t)$ respectively.

Furthermore, the spatial correlation functions of wind are assumed to be the same in both directions of alongwind and acrosswind as that proposed by Dr. Davenport [(2)] for alongwind turbulence. Using the power spectrum of the wind following ESDU [(3)], the cross spectra of wind are approximated as follows;

$$F_u(m,n;i\omega)=F_{uu}(\omega)R(m,n;\omega) \quad \cdots\cdots (20)$$

$$F_v(m,n;i\omega)=F_{vv}(\omega)R(m,n;\omega) \quad \cdots\cdots (21)$$

where, $F_{uu}(\omega)$ and $F_{vv}(\omega)$ are the power spectra in alongwind and acrosswind directions and R is the spatial correlation coefficient.

At some of the buildings related to this study, only 10-minutes averaged wind velocity and maximum gust speed were measured during Typhoon #7920. Therefore, the intensities of wind turbulences to define the power spectra of wind, F_{uu} and F_{vv}, in Eq.(20) and Eq.(21) are deduced as following empirical equations based on six years observations of wind at 187 m height on Building K.

$$\sigma_V \fallingdotseq (G-1)\,\bar{V}/2.61 \quad \cdots\cdots (22)$$

$$\sigma_u \fallingdotseq \sigma_V \fallingdotseq 0.730+0.133\,\bar{V}+0.001\bar{V}^2 \quad \cdots\cdots (23)$$

$$\sigma_v \fallingdotseq 0.905+0.180\,\bar{V} \quad \cdots\cdots (24)$$

where σ_V, RMS value, is the intensity of scalar wind velocity that means the magnitude of wind velocity vector. σ_u and σ_v are also the RMS values of fluctuating wind components in alongwind and acrosswind directions respectively, and G is the gust factor of wind that is the ratio of the maximum gust speed to the mean wind velocity, $\bar{V}$.

<u>(5) The Maximum Acceleration of Building Vibration</u>

According to the buffeting theory mentioned above, the two variances of building accelerations are predicted in short and long directions of the building respectively. The scalar of acceleration vector is necessary for correlation with human responses. However, the scalar acceleration can not be calculated theoretically, using the frequency domain equation of motion. Therefore, in this study, the maximum scalar acceleration of building is defined, for convenience sake, as follows:

$$\hat{\sigma} = (\sigma_S^2 + \sigma_L^2)^{1/2} \quad \cdots\cdots (25)$$

$$y_{max} \fallingdotseq \hat{g}\hat{\sigma} \quad \cdots\cdots (26)$$

where $\hat{\sigma}$ is a tentative RMS scalar acceleration using in Eq. (26), and y_{max} is the maximum amplitude of scalar acceleration. σ_S^2 and σ_L^2 are the variances of building accelerations in short and long directions of the building predicted by the buffeting theory, while $\hat{g}$ is the peak factor for $\hat{\sigma}$.

As for the peak factor $\hat{g}$ in Eq. (26), it was found from the observation of displacement at Building K that the mean value was 2.8 and that 80 percent of the maximum displacement data were less than 3.0 as shown in Fig.7(b). In this report, therefore, the peak factor of $\hat{g}$=3.0 was applied for the calculation of the maximum scalar acceleration.