Evaluation of the Voorhof II building refurbishment: a dynamic behaviour view-point

G.P.C. van Oosterhout
TNO Building and Construction Research,
P.O. Box, 2600 AA Delft, The Netherlands

The paper discusses the effectiveness of stiffening an existing building to improve the dynamic behaviour. The evaluation was part of a Ph.D. studies on the wind-induced dynamic behaviour of tall buildings. Sensitivity analyses by means of a computer model suggested that stiffening tall buildings may not be effective to increase human comfort in relation to wind-induced vibrations. The dynamic behaviour of the test-case building was assessed by measurements before and after a major refurbishment, that included stiffening the existing steel frames. The evaluation of the measurement confirmed increasing the stiffness of tall buildings is an ineffective measure to increase human comfort.

Key words: **tall buildings, dynamic behaviour.**

1 Introduction

For more than a century, tall buildings have been designed and erected. In that period, the knowledge about the behaviour of tall structures has grown substantially. Most of the research has been focused on structural behaviour related to ultimate limit states, as the potential hazard for occupants is evident. Nowadays, ultimate limit state design is common in the design of tall buildings.
In research and design relatively little attention has been paid to serviceability conditions, although these conditions will apply during the majority of the life-span of a building. As there are no lives at stake, the need to research and codify the behaviour of a building in everyday situations is not felt to be a pressing issue. Nevertheless, serviceability requirements have become important design issues for tall buildings. In this respect some developments in the building industry are relevant:

- Application of high-strength materials, like high-strength concrete.
- A decrease in partial safety factors for self-weight in Building Codes and guidelines.
- A decrease in the number of non-structural elements, like partition-walls, that are firmly attached to the main structure. Current building practice uses flexible connections between main structural elements and non-structural elements.

Due to these developments, modern tall buildings are relatively light and flexible, which makes them sensitive to dynamic loads. A major concern is that wind-induced vibrations might cause discomfort for the occupants of a tall building. Therefore, attention should be devoted to the control of vibrations in the design process.

HERON, Vol. 42, No. 2 (1997) ISSN 0046-7316

The importance of serviceability as a limit state can be illustrated by the costs of retrofitting. This paper reports on a tall building, Voorhof II, that was erected in Delft (The Netherlands) in 1966. The building has been serving as a student dormitory since then. Voorhof II got a bad reputation due to its lack of serviceability. The sway of the building during storms was strongly felt and cracks appeared in walls. Measurements of the accelerations at the top floors supported the occupants' complaints, since the recorded accelerations exceeded the tolerance limits set up by ISO [3].
The behaviour of this building was assessed as part of a Ph.D. studies on the wind-induced dynamic behaviour of tall buildings. The studies aimed at a tool that reliably assesses the wind-induced dynamic behaviour of tall buildings, supporting the design process of such structures from the preliminary design stage. As a result, a computer program, SkyDyFe, was developed. This program was also used in the evaluation of the Voorhof II building's behaviour.
In the following section a description of the building is given. The structural system and the building elements, such as partition walls, floors and facade, will be described. Subsequently, a list of deficiencies, based on a survey in Winter 1985, is given.
Several measures to improve the performance were proposed in 1986. They are presented in section 4. It was decided to stiffen the steel structure by the addition of four concrete walls. The dynamic behaviour of the retrofitted building was examined in 1994. Results of that re-examination are presented in section 6. The effectiveness of the selected structural measure is assessed in section 7. Finally, the uncertainties in predicting the dynamic behaviour of tall buildings are assessed in section 8 by comparison of the measured and calculated response of the Voorhof II building.

2 Description of the Voorhof II building

Voorhof II (1966) is a 17-storey student dormitory and is surrounded by similar buildings. Nowadays, the building is also known as the E. du Perron building. The quarter was built in the sixties to satisfy the growing demand for housing in Delft. The building is 51.3 m tall and has a rectangular plan with dimensions 80.8×14.2 m^2. Figure 1 displays a front view of the dormitory in its present state. An extensive description of the building and its history is given in [6]. Herein a brief summary is given.
The structure originally consisted of steel frames in both directions. In x-direction, there are 20 bays that are 4 m deep, divided by frames spanning 10.6 m. Six frames are braced over the full width. These braced frames can be recognised in the plan in Figure 2, being the thick lines. In y-direction the lateral resistance is provided by four braced frames as is also shown in Figure 2 by thick lines. Floors are made of reinforced concrete, thus acting as stiff diaphragms.
The main non-structural elements are the partition walls and the facade elements. The partition walls are fabricated from three layers: insulation covered by two layers of cellular concrete. The east and west facades consist of cellular concrete walls with steel sheeting attached to it. The (long) north and south facades originally consisted of wooden frames, which were replaced by prefabricated cladding elements during the refurbishment.

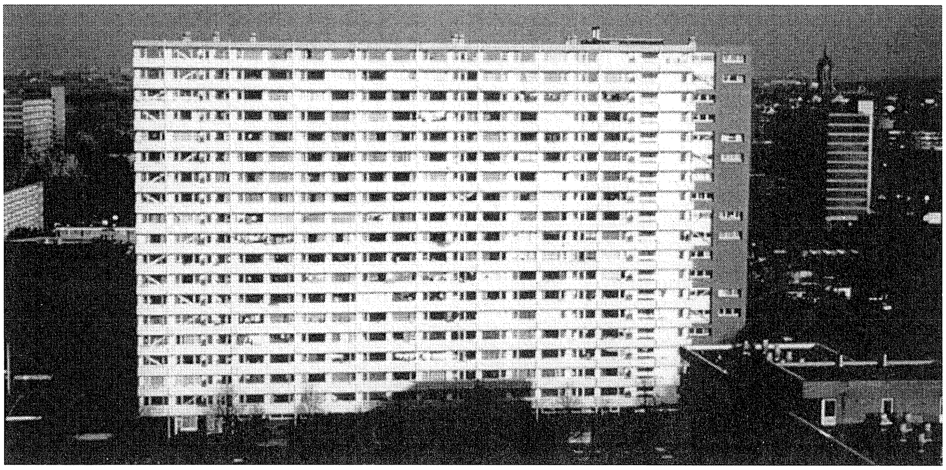

Fig. 1. Front view of the Voorhof II building.

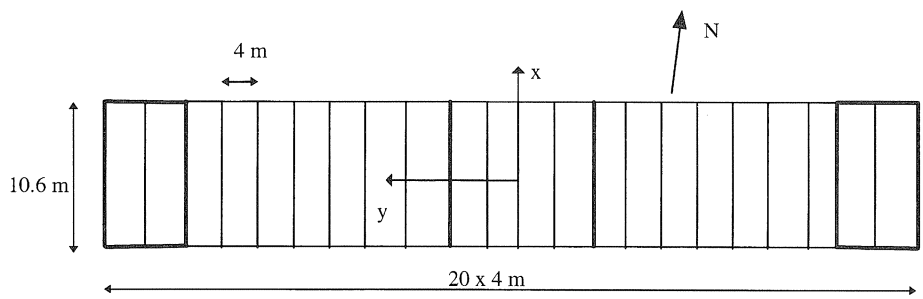

Fig. 2. Plan of the building with main structural elements: thin lines are frames, thick lines are braced frames.

3 Deficiencies due to sway of the building

Many deficiencies were reported in a survey that was carried out in 1985 [7]. This led, together with a need to change the lay-out of the rooms, to the refurbishment of the student dormitory.
The following problems were reported and are all related to the sway of the building:

- All partition walls were cracked.
- Several partition walls had moved relative to adjacent walls.
- Tiles had cracked or came loose.
- The connections between partition walls and window frames were poor.
- Several partition walls contained horizontal cracks, causing them to stand loose.
- Doors jammed frequently.
- A crack along the longitudinal axis appeared in the floor finish.
- Some of the bathrooms were leaking.
- Occupants on the top floors complained of nausea.

The latter complaint was supported by measurements of the dynamic response to a storm on 14 January 1986. The measurements were conducted in the afternoon [2]. From 15.00 to 17.00, the response was recorded by means of accelerometers, which were placed as shown in Figure 3 [2]. A detailed description of the test procedure is given in [6].
The hourly mean wind speed at the top, based on data from the nearby Rotterdam Airport, was 14.3 m/s. The storm reached its peak at about 16.00 hours. At that time, the 10 minute mean wind speed at the top was calculated at 18.3 m/s.

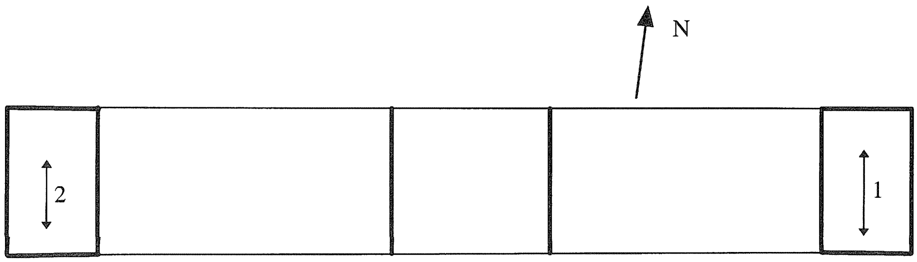

Fig. 3. Plan of the building with position and direction of accelerometer 1 and 2 .

Unfortunately, most of the response power spectra that were calculated from the time traces were rather rough due to a lack of resolution. However, there were a few spectra of adequate resolution to identify the lowest natural frequency at 0.63 Hz. Other resonant peaks appeared at 2.28 Hz and 4.42 Hz.
The phase difference between the recordings of the accelerometers at the east-side and west-side of the building identified a torsional vibration in the frequency range 0.64 to 0.75 Hz. Mode shapes were not presented in [2].
Due to the lack of resolution in the spectra, the rms acceleration can not be determined from them. Fortunately, a few time-traces of the response during the peak in the storm were available.
The highest peak in 10 minutes was 70 mm/s^2. The peak factor for f_e = 0.63 Hz and T = 600 s is 3.61. From these figures the rms acceleration due to a storm with a mean wind speed of 18.3 m/s at the top of the building can be determined to be 19 mm/s^2. This number has been checked with a rough calculation based on the inaccurate spectra that were available. This calculation indicates a rms acceleration in the range 10–30 mm/s^2. So, 19 mm/s^2 might be considered a reasonable value for the rms acceleration during the test.
Due to the poor resolution of the spectra, determination of the damping ratio from these plots is virtually impossible.

4 Improving the dynamic behaviour

Following the decision to refurbish the building, a study was conducted to improve the dynamic behaviour of the building [2]. Six alternatives were developed, based on the measured response in the first mode.

1. *The removal of several top-floors.* There are two advantages to this alternative. Firstly, the total wind load will be lower. Secondly, the natural frequency will increase due to the decrease of mass and height. Consequently, the deflection and acceleration at the top will decrease.
2. *The addition of concrete walls at the east- and west-side of the building.* For this purpose a new foundation should be built to carry the extra loads.
3. *The addition of two external concrete cores.* These cores may be placed near the existing stair cases, as shown in Figure 4. This alternative also requires an extra foundation.
4. *Deepening the steel frames.* The result will be that diagonals will separate the balcony into bay-widths of 4 m. This alternative does not reduce the acceleration significantly.
5. *Active damping.* A dynamic damper placed on the top-floor of the building can effectively reduce the sway of the building. The applied mass per frame will be maximum 15000 kg per frame, according to [2].
6. *Stiffening the four existing steel frames by addition of four concrete walls.* The walls are placed directly next to the braced frames. The extra foundation can be placed within the existing building or partly outside the building.

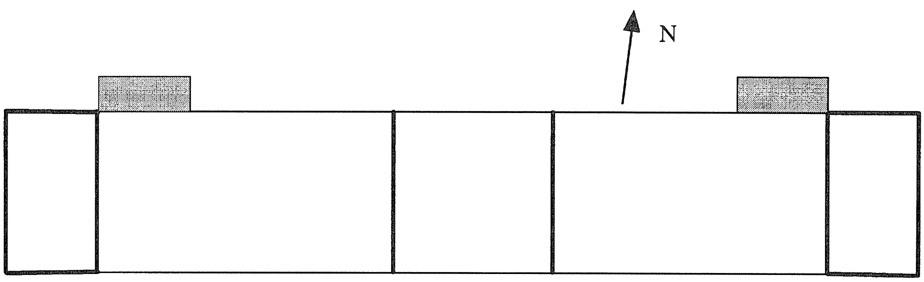

Fig. 4. Plan of the building with additional cores (the grey areas).

The effects of alternatives 1, 2, 3 and 6 on the dynamic behaviour of the building have been evaluated with SkyDyFe. The rms acceleration (σ_a) is calculated according to the requirements of ISO 6897 [3], i.e., wind loads are based on a five-year return period wind speed. The calculated responses are summarised in Table 1. A graphical comparison with the criterion in ISO 6897 is displayed in Figure 5. The plot shows that most of the alternatives perform rather poorly. Only removal of the top 5 floors or the addition of two external cores (both alternatives are not shown in the graph) mitigate the building's behaviour into a region of low, well-accepted acceleration levels.

Table 1. Effects of alternatives on the dynamic behaviour. The effective mass is based on a linear fundamental mode shape.

Alternatives	k_e (MN/m)	m_e (10^6 kg)	ζ (%)	f_e (Hz)	σ_a (mm/s^2)
1. Removing top floors					
17 floors (actual situation)	56	3.6	0.95	0.63	47
14 floors	119	3.0	1.42	1.00	22
12 floors	189	2.6	1.54	1.37	15
2. Addition of two walls	102	3.9	1.42	0.81	26
3. Addition of two cores	680	4.1	2.04	2.04	7
6. Stiffening of steel bracing	111	4.1	1.38	0.83	25

It is important to note that the calculations in [2] were based on the measured response as described in the previous subsection. The quality of the measurements was poor and conclusions that are drawn solely on these results should be considered with caution.

The most economical alternative, stiffening the four existing steel frames, was chosen to improve the serviceability of the building. An important advantage of this solution was that the east- and west-facade could remain unchanged, thus keeping the building weather-tight during the refurbishment. According to Figure 5, this alternative provides just satisfactory behaviour, as is supported by the measurements after refurbishment. In [2] it was already suggested that most probably occupants were still able to strongly perceive motion during severe storms when this alternative would be selected.

It has already been said that the test results were inaccurate. Damping figures could not be identified with confidence. Therefore, the damping ratios that are stated in Table 1 were estimated with the following equation:

$$\zeta = 0.01 f_e + C_e \frac{\hat{u}_h}{h} (f_e \text{ in Hz}) \tag{1}$$

where C_e is an energy dissipation factor. Equation (1) takes account of the displacement ($\hat{u}_h$) dependency of the damping ratio. Jeary was the first to propose an equation of this form in [4].) When C_e is low, the damping mechanisms in the building are only weakly dependent on the displacement. Before refurbishment, when many partition walls were cracked or loose, C_e must have been small. For the comparison purposes of Table 1, values for C_e were fitted to the response measurements. The fitting resulted in a value of 15 before refurbishment and 75 after refurbishment. A deeper discussion on the values that apply to the energy dissipation factor may be found in reference [5].

The uncertainties in the response assessment procedure are discussed in detail in section 8.

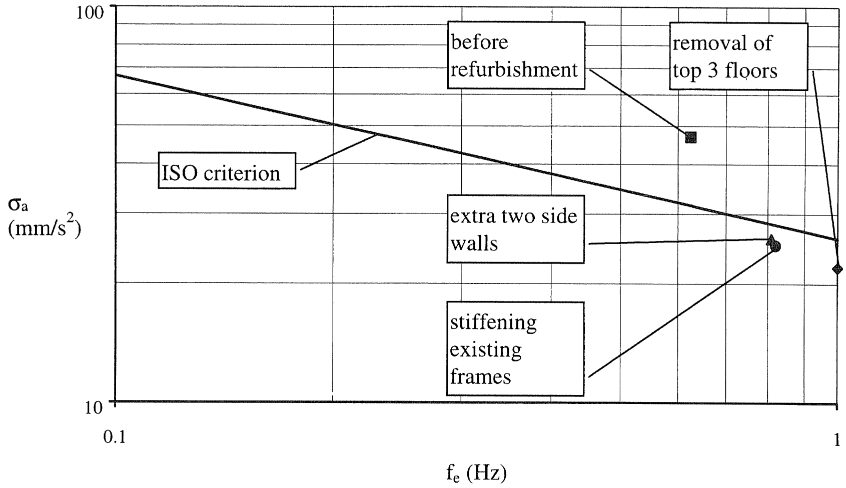

Fig. 5. Proposed alternatives versus ISO-criterion [3]. Alternatives with natural frequencies above 1 Hz are not displayed.

5 Summary of SkyDyFe analyses

Sensitivity analyses have been performed by SkyDyFe to evaluate the effectiveness of the main design parameters in tall building dynamics: mass, stiffness and damping. In Figure 6 the effects of the three parameters mass, stiffness and damping on the acceleration are plotted in the ISO 6897 graph for a squat and a slender building. The arrows point in the direction of increasing quantities. Figure 6 indicates that stiffening the structure of a tall building may not be effective in reducing the accelerations. The following proportionalities were found in the sensitivity analyses [5]

$$\sigma_a \propto (\bar{v}(h))^{2(2-\varepsilon)} k_e^{-1+\varepsilon} m_e^{-\varepsilon} \zeta^{-\frac{1}{2}} \qquad \frac{1}{6} \leq \varepsilon \leq 1 \tag{2}$$

where

$\bar{v}(h)$ = the mean wind velocity at the top of the building [m/s],
m_e = effective mass [kg],
k_e = effective spring stiffness [N/m],
ε = a proportionality factor,
ζ = damping ratio.

For normal buildings, with natural frequencies near 1 Hz or above ε is close to 1/6. Thus, an increase of the stiffness might be considered as an effective tool to reduce the accelerations (See Building B in Figure 6). However, for slender, tall buildings, with natural frequencies well below 1 Hz, ε is going to 1/3 or less. Consequently, the influence of the stiffness k_e on the response is very small.

Even more remarkable is the effect on the ISO criterion. If ε is low, an increase in the stiffness may result in a shift from the admissible to the inadmissible domain (see Building A in Figure 6). The point is that an increase of the stiffness also leads to an increase of the natural frequency. Similarly, drift limits do not guarantee low acceleration levels, since they only effect the stiffness of the structure. Therefore, the wide-spread belief that drift limits cover all aspects of serviceability is incorrect for tall buildings.
The structural mass m always helps to reduce the vibration level, but more significant for buildings with high aspect ratio. For increasing m also the structural natural frequency is reduced, which makes the effect even stronger. Damping always has a beneficial effect on vibrations. For high-rise buildings this could become an important design option. One might think of artificial damping, as well as the exploration of the damping capacity of non-structural elements.

The reader may note that the design alternative that was chosen in the Voorhof II refurbishment is rather ineffective according to Figure 6. The measurements of the dynamic behaviour of Voorhof II after refurbishment, therefore, will be an interesting validation of the theory that was used in Sky-DyFe.

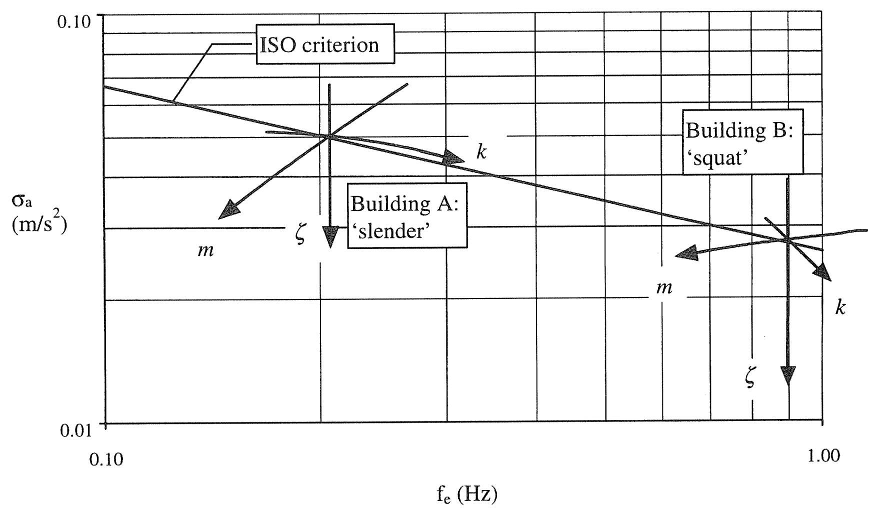

Fig. 6. *ISO acceleration criterion [3] and effects of the main dynamic parameters effective stiffness (k_e), effective mass (m_e) and damping (ζ). Arrows point in direction of increasing quantity. Building A is a "slender" structure being 150 m high and 20 m deep. Building B is a 'squat' structure being 50 m high and 14 m deep.*

6 Dynamic behaviour after refurbishment

As described earlier, tenants complained for a number of years about the serviceability of the building. It is therefore interesting to see if the building after refurbishment has improved its performance. On 31 October 1994, response measurements were taken. During the tests the hourly mean wind speed at the top was 11 m/s. An extensive description of the test procedure and the equipment is given in [6].

The building was equipped with 2 accelerometers, which measured the horizontal accelerations of the building at the 17th floor. The accelerometers were placed similar to the previous tests in 1986 (see Figure 3).

Figure 7 shows reduced spectra that were derived from the response measured by accelerometers 1 and 2. The position of the accelerometers in the plan can be appreciated from Figure 3. The fundamental natural frequency of the NS mode has increased from 0.63 Hz to 0.85 Hz. In Figure 8 the according mode shape in plan is shown.

Remarkable is the resonant frequency of 0.77 Hz in the accelerometer 2 spectrum. The top mode shape in Figure 8 suggests that this is the lowest torsional natural frequency. Note that both modes in Figure 8 exhibit some coupling, which indicates that the centre of inertia is close to the east side of the building.

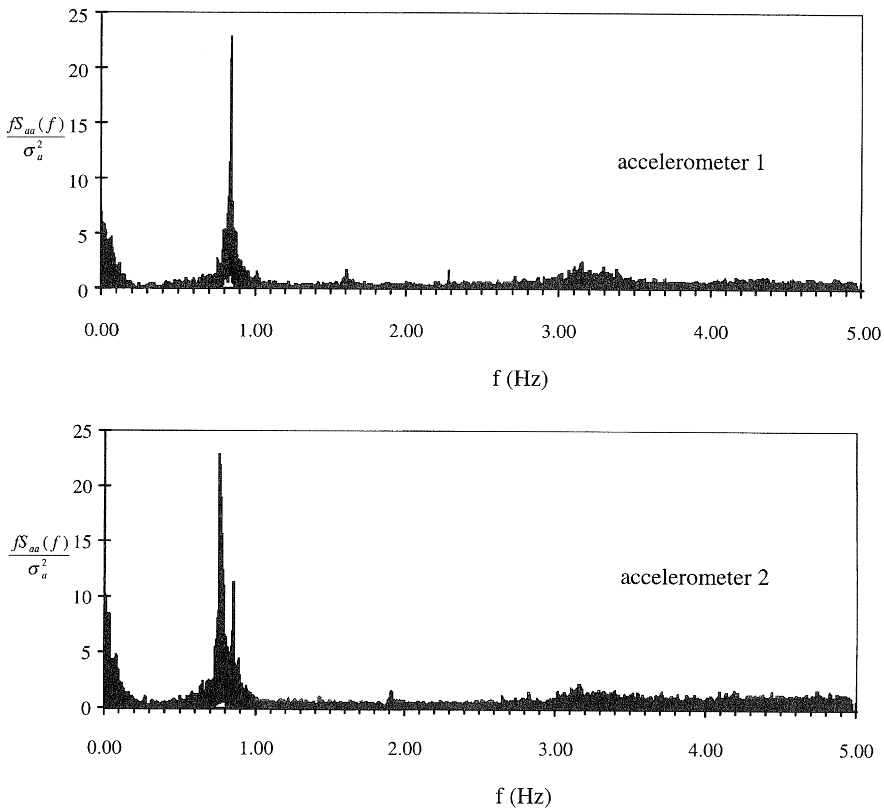

Fig. 7. Power spectra of accelerometer at east (nr. 1) and west (nr. 2) side [6].

A rms acceleration of 1.1 mm/s^2 is found. Retrieval of the damping ratio appeared to be difficult, since the resolution of the spectra did not meet the requirements for a reliable use of the half power bandwidth method as a damping predictor (a discussion on disadvantages of the half power bandwidth method as predictor of the damping ratio may be found in [4]).The spectra suggest values between 1 and 2.5% of the critical damping. For reasons that will be outlined in section 8, it is not possible to improve the estimate of the damping by comparison of the measurements to predictions.

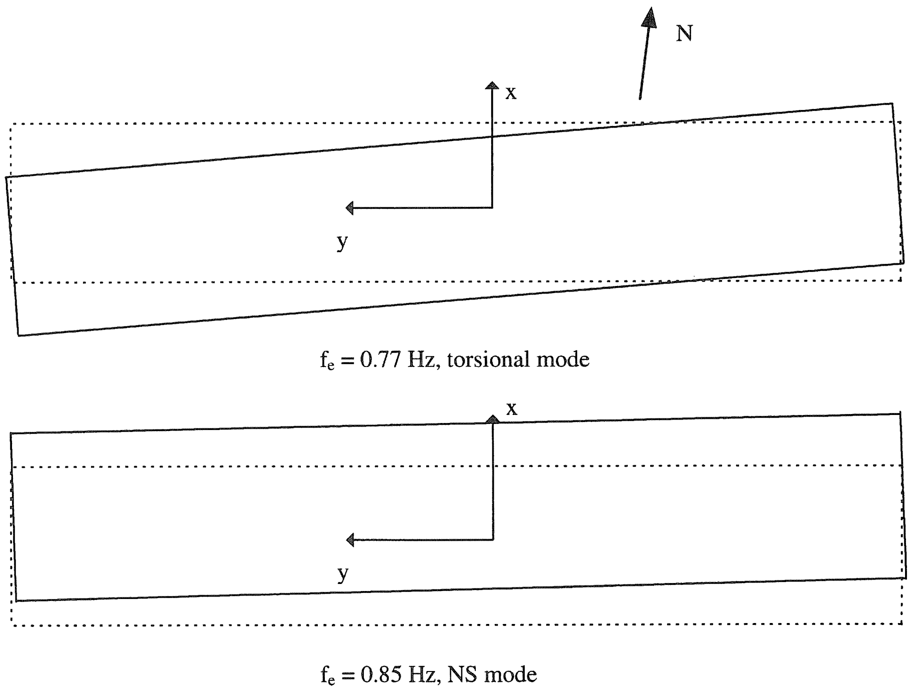

Fig. 8. Two fundamental mode shapes in plan: at the top the torsional mode, at the bottom the translational mode.

7 Stiffness versus acceleration

It is often suggested that commonly used drift limits guarantee acceleration levels that are admissible. The sensitivity analysis in section 5 showed that this may be true for low-rise structures, but for tall buildings things are different. The Voorhof II refurbishment is a good illustration of this fact.

The effective stiffnesses are summarised in Table 2. The effective stiffness of the bare frame was determined from a 2D finite element analysis [6]. The stiffness of the whole building is derived from the test data.

Table 2. Effective stiffness of Voorhof II before and after refurbishment [6].

	Before (MN/m)	after (MN/m)
bare structure (calculated)	28	80
non-structural contribution	28	35
whole building (measured)	56	115

The main objective of the refurbishment was to increase the stiffness of the structure. From Table 2 it may be appreciated that the stiffness has increased to more than 200% of the original value. Nevertheless, the increase in human comfort is small. The second effect of the refurbishment, a 10% increase in the mass is relatively more effective, as is illustrated in Figure 9.
In section 4, the effect of the refurbishment on the energy dissipation has been fitted to the measured response in order to compare the design alternatives. Before refurbishment the factor C_e in damping predictor (2.44) was fitted to 15. After refurbishment $C_e = 75$ gave a good fit. Accordingly, the damping ratio increases from 1.0 to 1.3 %. This beneficial effect on the human comfort is apparent from Figure 9.
In conclusion, there is no rationale for the belief that deflection criteria guarantee adequate dynamic performance. This confirms the sensitivity analyses performed in SkyDyFe, which indicated that stiffening is not very effective in improving the dynamic behaviour of tall buildings. Consequently, the refurbishment of Voorhof II has not been very effective in increasing the comfort of the occupants. This conclusion is supported by the occupants' perception: people do still perceive motion even during moderate storms.

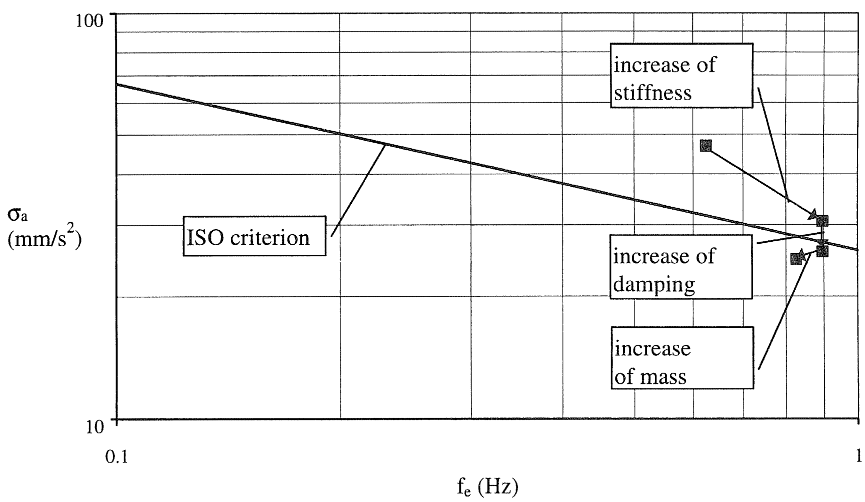

Fig. 9. Three effects of selected refurbishment alternative, added stiffness, added damping and added mass, on human comfort.

8 Comparison of measured and predicted response: an assessment of uncertainties in the prediction of wind-induced acceleration in tall buildings

In this section the sensitivity of acceleration to uncertainties in the parameters shear velocity, aerodynamic roughness, stiffness, mass and damping is reviewed. The determination of most of these parameters is a process full of uncertainties.

The Voorhof II building provides an interesting opportunity to compare predicted and measured dynamic behaviour. In section 3, a rms response of 19 mm/s^2 due to a hourly mean wind speed during the tests of 14.3 m/s was reported. Using the building parameters that were presented in section 6 and terrain characteristics that are z_0 = 1.0 m and d_0 = 10 m, a rms response of 7.2 mm/s^2 is found. This result is not even close to 19 mm/s^2.

It has already been pointed out that the resolution of the tests was low. Therefore, there are some inherent uncertainties, like the actual damping ratio and the value of the rms acceleration. There are many sources of deviation between measurement and prediction. Roughly, there are two groups: a group of parameters related to the wind and another group of parameters related to the building. These groups will now subsequently be discussed.

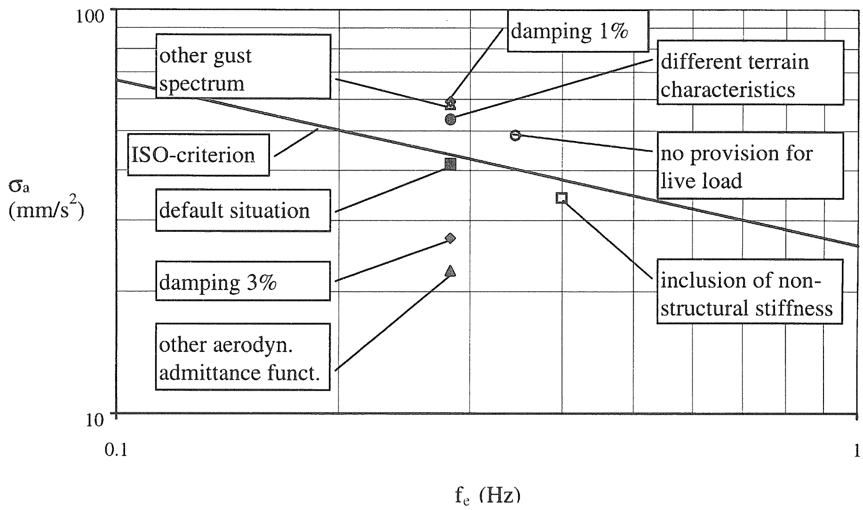

Fig. 10. Effects of uncertainties in response parameters on human comfort. Human comfort criterion according to ISO 6897 [3]. Default situation is a 120 m high building.

Voorhof II is excited by the wind: a random load both in time and space. Thus, there are inherent uncertainties in the loading that is assumed to work on the structure. Measurements of Voorhof II were recorded during moderate storms. The wind loads on the building are estimated according to the well-established theory of along-wind response as introduced by Davenport in 1961 [1]. This procedure introduces many uncertainties in the assessment of the dynamic loads on the structure.

The mean wind speed that is used in the reduced spectrum and the aerodynamic admittance function is based on data of a (close) wind station. In general, the conditions of the surroundings differ from the conditions at the station. Therefore, the wind station data is transformed with the help of the aerodynamic roughness and shear velocity. This transformation process is a source of deviation. For example, typical values of the aerodynamic roughness in urban environments are 0.7 to 2.0 m. Transformation of the annual return wind speed at a typical Dutch wind station using 0.7 m for the aerodynamic roughness will yield a shear velocity of 2.32 m/s (see [8] for a derivation). For an urban environment the aerodynamic roughness can be in the range 0.7-2.0 m. The calculations that are summarised in Table 1 used z_0 = 1.0 m, d_0 =10 m and v* = 1.54 m/s. If z_0 = 0.7 m had been used, the response would have dropped to 90% of the originally calculated value. If the aerodynamic roughness is 2.0 m, a shear velocity of 2.93 m/s is found. Data point "different terrain characteristics" in Figure 10 shows the combined effect on the acceleration. In this case a deviation of about 35 % due to uncertainties in terrain characteristics is found. Thus, the response would have increased to 135% of the originally calculated value if z_0 = 2.0 m would have been used.
The other estimated quantity is the average height of the surrounding buildings d_0. This parameter is normally estimated "on the eye" and large errors may be expected. A value of 10 m was used in the calculations. It is interesting to see what the response is when the actual value is 50 or 200% of the value that was used. For d_0 = 5 m the response grows to 108%. An average building height of 20 m yields a drop to 83%.

Once the wind speed and its associated parameters have been determined, the next step is to derive the wind load spectrum from the spectrum of the speed fluctuations and the aerodynamic admittance or coherence of the pressures. Although the theory of along-wind response nowadays is well established, no agreement on the spectrum or the coherence function is yet achieved. Because of this disagreement, deviation in the response of about 35% and 50% due to differences in spectrum and aerodynamic admittance respectively were found in [5]. These deviations are illustrated in Figure 10.

The effects of the structural parameters, namely stiffness, mass and damping, on human comfort are also shown in Figure 10. The stiffness and the damping ratio exhibit the largest uncertainties.
The only one property of the Voorhof II building that is known with great confidence is the lowest natural frequency, which was determined with 0.01 Hz accuracy.
The mass is calculated from structural drawings. There are several contributions to the total mass of a building: the structural system, the floors, the facade, finishes and live load. The live load is an important source of uncertainty in the determination of the mass, since there is no evidence that the full live load will be present. As an illustration, data point "no provision for live load" in Figure 10 shows what the effect of an "empty" building on the human comfort is. It seems that it is unconservative to take account of added mass by live load in a dynamic analysis.
Besides errors in the calculation, there is another error source in the determination of the effective mass and that is the estimate of the mode shape. It is normally assumed that the mode shape of tall buildings can be approximated by a straight line. If the mode shape was pure bending, the response would be slightly higher. However, if the mode shape was pure shear the response would have been 83% of the level for a linear mode shape.

It seems to be reasonable to say that a ± 20% deviation in the total mass can occur due to building use that differs from the assumptions in the design. Since the natural frequency of the Voorhof II building was determined accurately, there is also a ± 20% deviation in the stiffness. If the mass decreases 20 % the acceleration therefore grows to 125%. In the other hand, a 20 % increase let the acceleration drop to 85% of the originally calculated value.

On the other hand, the determination of the stiffness is also a process with inherent errors.
The contribution of non-structural elements to the stiffness is, in general, neglected. However, in serviceability limit states these elements do add to the stiffness. For example, measurements on an eight-storey building in England [5] revealed that the stiffness of non-structural elements can be more than twice the stiffness provided by the structural system. The effect on the estimation of human comfort is illustrated in Figure 10 (data point "inclusion of non-structural stiffness"). A non-structural stiffness that equals the stiffness of the structural system is assumed. The rms acceleration will drop, but the human comfort will stay at the same level, since the natural frequency will increase.

Finally, the damping ratio is often assumed to be a constant, like 1% of critical for steel buildings. However, non-linear mechanisms are involved in the energy dissipation in buildings, for example, due to hysteresis in non-structural elements. As a consequence, measurements of the damping ratio in a steel building often indicate 2 % of critical or more. It is clear from Figure 10, in which the default situation has 2% damping and two data points for 1 and 3% damping are plotted, that the difference between actual and calculated response will be large accordingly.
In section 3 it appeared that the damping could not be determined from the test results. Comparison of calculated and measured response should indicate a reasonable value for the damping. However, there are so many uncertainties and related deviations in the response that a reliable value for the damping of this building in its lowest mode of vibration can not be set. Probably the damping was somewhere between 0.5 and 1.0% at the time of the test. Using one of the boundary values causes a variation in response of 40%.

Resuming, it appears that there are many uncertainties in the assessment of the Voorhof II behaviour, so that it is not an easy task to fit calculated to measured response. This study has clearly shown that there is a need to quantify these uncertainties in order to improve the serviceability design of tall buildings.

9 Summary

The actions that were taken to improve the dynamic behaviour of the Voorhof Building have been evaluated as part of a Ph.D. studies. The evaluation confirmed some of the conclusions that were drawn from the theoretical part of the before mentioned Ph.D. studies. It has been shown that:

- The Voorhof II case-study illustrated the possible economic consequences of poor dynamic performance from a serviceability point of view. Both deflection and motion limits were exceeded in the original configuration. An expensive refurbishment that included the addition of four concrete walls to stiffen the structure, was required.

- Measurements of the dynamic behaviour of Voorhof II before and after the refurbishment revealed that the addition of stiffness by concrete walls was rather ineffective in improving the comfort of the occupants.
- Increasing the stiffness of tall buildings is an ineffective measure to increase human comfort. Consequently, drift limits do not cover human comfort issues, although this is a wide-spread belief.
- Damping is always beneficial in reducing both dynamic displacements and accelerations.
- The effective mass is an efficient tool to improve the dynamic performance of a tall building. Human comfort can be improved significantly.
- Uncertainties in the determination of the characteristics that describe turbulent winds, namely the parameters aerodynamic roughness and shear velocity, might lead to very large deviations in the response. This is equally true for the uncertainties in structural parameters.There is a need to quantify these uncertainties in order to improve the serviceability design of tall buildings.
- It may be unconservative to add the live load to the total mass of a tall building: extra mass has a beneficial effect on human comfort.

References

[1] A.G. Davenport, The spectrum of horizontal gustiness near the ground in high winds, Quarterly Journal of the Royal Meteorological Society, vol. 87, p. 194-211, 1961.

[2] Van Dorsser B.V., Survey of the low-frequency vibrations of the high-rise buildings at the Roland Holstlaan (Voorhof II) due to wind loads (in Dutch), reports. 5070.A, 5070.B and 5070.C, 1986.

[3] ISO 6897, Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0.063 to 1 Hz), ISO, 1984.

[4] A.P. Jeary, The dynamic behaviour of tall buildings, Ph.D. thesis, Faculty of Science of the University of London, 1981.

[5] G.P.C. van Oosterhout, Wind-induced dynamic behaviour of tall buildings, Ph.D. thesis Delft University of Technology, Delft, 1996.

[6] M.J.A. Pols, The relation between stiffness and serviceability of tall buildings, M.Sc. thesis Faculty of Civil Engineering, Delft University of Technology, 1995.

[7] Records of SDSH building maintenance committee (in Dutch), 1985.

[8] P.C. van Staalduinen, Background of the wind loads according to NEN 6702 (in Dutch), TNO-report B-90-483, 1990