RESPONSE OF TALL BUILDINGS TO WEAK LONG DISTANCE EARTHQUAKES

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ABSTRACT

In the last decade, two tall buildings in Singapore were instrumented with accelerometers and anemometers for the original purpose of identifying the characteristics and effects of wind loading. During the monitoring it became clear that the largest acceleration responses should result from ground motions due to earthquakes having magnitudes between 6 and 8 and epicentres at least 350km distant. The paper describes the strategy for identifying and capturing the signals from distant tremors, which depends on tracking the RMS response levels in the second vibration mode. Characteristics of some recorded signals are given. While response levels are generally small, the frequency content coincides with the range of fundamental mode frequencies for high rise residential buildings. The validity of using a tall building as a 'weak-motion' seismograph is discussed by considering the mode shape of the building and the measured transfer function between basement and roof responses.

KEYWORDS

Long distance, transmissibility function, tall building, monitoring

DYNAMIC RESPONSE MONITORING OF TALL BUILDINGS IN SINGAPORE

Design for lateral dynamic loading of tall buildings would normally consider effects of wind and tremors, depending on the location; for Singapore the dominant form of loading with dynamic effect is assumed to be wind. By law in Singapore wind loading is governed by CP3[1], which gives methods for calculating the maximum gust wind loads, but does not provide for dynamic excitation, or by BS6399[2]. Design gust speeds of up to 35m/sec are commonly used which are believed to be over-conservative.

Local building design must also necessarily provide for accidental lateral load such as specified in BS8110[3] which can be taken as 1.5% of dead weight applied laterally at each level. There is no specific provision for earthquake resistance and it is assumed[4] that earthquake loads will be taken care of by the 1.5% notional horizontal load (NHL) provision.

Singapore is subject neither to very strong winds nor to powerful earthquakes. So far the strongest earthquake loads are purely from the frequent earthquakes occurring in neighbouring Indonesia, at least 350km distant, while the strongest wind loads are from highly turbulent squalls and localised storms. Neither type of loading is either well understood or adequately covered by loading codes, an unusual combination with no known precedent.

Estimating the probabilities and recurrence rates of strong loading events requires analysis of many years of data and is a separate exercise; some recent conclusions about regional seismicity are presented in the next section. Apart from the peak values of loading, building response depends on the frequency content and spatial distribution of the loading and the dynamic characteristics of the buildings.

There is a strong case for instrumentation of structures to help understand what the loads are and how they structures respond to them. Dynamic characteristics of buildings are estimated (with variable accuracy) by structural analysis which can only be validated experimentally. Measurement of actual wind load and its effect is only possible through instrumentation, and while magnitudes and spectral distribution of local ground motions due to distant tremors can be estimated using geophysical models[5], the most reliable method of identification is, once again, by instrumentation. Many valuable data about characteristics of earthquakes and more particularly of the superstructure and foundations have been obtained by past instrumentation exercises on full-scale buildings[6-12]. This paper presents some findings on seismic influences obtained through measurement and instrumentation of two tall buildings in Singapore.

SINGAPORE SEISMICITY

Although Singapore and Peninsular Malaysia are in an a-seismic area, an active earthquake belt, comprising the 1500km long Sumatra Fault system and the subduction zone, is about 350 km away at the closest point. Fig. 1 shows the epicentre locations of numerous earthquakes occurring in this region between 1960 and 1995. In the figure, the large circle centred at Singapore with 500 km radius is included for reference, and the size of small circles is proportional to earthquake magnitude. Very large earthquakes have been generated along the interface between the two plates in the system, for example the great earthquake in 1833 was estimated to have a moment magnitude of around 8.7 causing an estimated 500 km long rupture.

The size, location and timing of earthquakes are generally erratic. A recent study[13] showed that there have been no great earthquakes in the last half century. The last two greatest earthquakes in Sumatra both occurred in the previous century, when there were practically no high-rise structures or reclaimed land. Therefore, historically earthquake hazard has hardly been an issue for the Malay Peninsula, with earthquake resistant design required only for some more important structures.

Although there has never been any earthquake damage to Peninsular Malaysia and Singapore, ground tremors have been reported many times, with a significant increase in frequency of reports over the last three decades, corresponding to an age of high rise construction. Until very recent exercises such as described here, there have been no seismographic stations in Singapore and the ground motion reports had therefore been based solely on local newspaper reporting and the anecdotal history of Singapore

SEISMIC MONITORING IN SINGAPORE

For the specific purpose of acquiring data to characterise ground motions in Singapore, Meteorological Service Singapore (MSS) established a national seismic system (http://www.gov.sg/metsin/eqr.html) in September 1996. Before this, no recorded ground motion time histories were available. The MSS system comprises five hard rock seismograph stations with seismometer and strong motion accelerometer arrays. One of these has very broad band and short period seismometers and is a Global Seismographic Network (GSN) recording station. In addition two ground motion stations are located at soft soil sites. Each of these has surface seismometers as well as arrays of strong motion accelerometers at surface and at two depths of borehole.

Since 1999 two seismograph stations have been independently installed. One is a surface accelerometer array and seismometer on NTU campus, the other is a set of surface and downhole strong motion accelerometers in a new land reclamation site to be used as a port facility.

In addition a residential building (condominium) and tall office tower have been instrumented with biaxial accelerometers and the experiences with these two projects are described here. In this paper, the two buildings are first described and their experimentally determined modal characteristics presented. The instrumentation systems used in each building are then described together with the strategy that has evolved for efficiently capturing tremor-induced response. Some of the recorded signals and their characteristics are presented. Finally, based on measurements and consideration of the transmissibility function between ground and roof level of the office tower it is concluded that signals recorded at basement level (but triggered by rooftop motions) are almost certainly representative of bedrock motion.

DESRIPTION OF INSTRUMENTED CONDOMINIUM

The 26-storey condominium tower studied[14] is a private residential apartment block constructed circa 1982 as a conventional reinforced concrete frame structure approximately 38m by 33m in plan. The plan view (Fig. 2) shows two residential halves of the building connected through a lift/stair core forming a natural 'Lateral' axis of symmetry and an orthogonal 'Longitudinal' axis. No details of foundation are available and the local soil is alluvium, known as Kallang Formation, with marine clay at foundation depth.

Above the flat roof at 86m is an observation tower housing lift machinery and supporting a cantilevered enclosed observation platform at 91m. The rectangular outlines of the tower and the overhanging observation platform are shown in dashed line superimposed on the floor plan in Fig. 2.

DESCRIPTION OF INSTRUMENTED OFFICE TOWER:REPUBLIC PLAZA

A full description of this 280m, sixty-six storey office tower is given elsewhere[15]. Fig. 3 shows a perspective view of the building and Fig. 4 shows a typical cross section at lower levels. The tower has a frame-tube structural system with an internal core wall connected to a ring of external columns by horizontal steel framing system at every floor.

The reinforced concrete (RC) central core wall maintains a plan area approximately 22m square for almost the full height of the building. The perimeter of the building comprises eight steel tube columns with diameters up to 1.22m and occupies a square with dimension 45m at ground level. There are two tapering sections where mechanical equipment is located in double-height storeys and where outriggers are installed to enhance the rigidity of the building frame under lateral loads.

The column bases are bolted to the foundation at basement level (B1) where they sit on a deep stiff foundation system[16]. This comprises six inner caissons founded up to 62m deep in boulder clay and connected by a 5.5m thick concrete mat, and eight exterior caissons founded up to 40m deep and linked by deep transfer beams. All caissons are 5m diameter.

The building symmetry suggests a pair of axes, labeled A and B as shown in Fig. 4, for measurement of building vibration. The arrangement of lift shafts and core wall openings up to the highest level suggests that B will be the stiffer direction. Acceleration limit of 0.01g ($g=9.8m.s^{-2}$) was used in the design and the fundamental mode period was estimated as 7 seconds (0.14Hz).

DYNAMIC CHARACTERSTICS OBTAINED THROUGH AMBIENT VIBRATION SURVEYS

Natural frequencies, mode shapes and damping ratios for a set of vibration modes were estimated for each building[14,17-20] via ambient vibration survey in which low level building response to wind and machinery was recorded at different levels throughout the building and at different locations (in plan) at certain levels.

Frequency and damping estimates for condominium and Republic Plaza are shown in Tables 1 and 2 respectively. The frequency estimates were obtained by curve fitting procedures and are accurate to better than 0.5% (one standard deviation). The damping values are the lowest values obtained over many series of measurements and are believed to be accurate to within approximately 10% for low levels of response i.e. up to $15mm.s^{-2}$.

Table 1 Modal parameters for condominium

Direction	Lateral			Longitudinal		
Mode r	Lat1	Lat2	Lat3	Long1	Long2	Long3
Frequency f_r/Hz	0.65	2.34	4.60	0.70	2.45	5.05
Damping ζ_r /%	0.9	0.8	-	0.7	1.0	-

Table 2 Modal parameters for Republic Plaza (in 1997 when occupied)

Direction	A			B		
Mode r	A1	A2	A3	B1	B2	B3
Frequency f_r/Hz	0.184	0.676	1.49	0.194	0.726	1.69
Damping ζ_r /%	0.66	0.85	0.87	0.70	0.52	0.77

Mode shapes (in elevation) are shown for Lateral direction of the condominium and for A direction of Republic Plaza in Figs. 5 and 6 respectively. The mode shapes ψ as discussed in this paper have been conventionally normalised to $\psi = 1$ at roof level for each mode and absolute strength of any mode is not intended to be represented. Modal ordinates obtained during the ambient vibration surveys are generally accurate to better than 10% for one standard deviation, the low scatter in the curves supports this. The mode shapes shown represent the relevant translational components of the rather complex vibration patterns of these buildings which are described in more detail elsewhere[14,17-21].

Fig. 5 clearly shows that at ground level, the condominium is responding significantly in mode 3 and to a lesser extent in lower modes. The numerical values of ground floor (level 1) modal ordinates for modes in Lateral direction are given in Table 3

Table 3 Ground level modal ordinates for condominium

Mode	Lat1	Lat2	Lat3
Level 1 modal ordinate ψ_1	0.016	-0.17	0.8

For Republic Plaza the basement level response is much smaller, and could not be defined reliably until a monitoring system with accelerometers at roof and basement levels had been installed and several hundred relatively strong wind-induced response signals had been recorded. The numerical values of modal ordinates for A-direction modes at 3.5m above top surface of the foundation mat at level B1 are given in Table 4 and are accurate to three decimal places.

Table 4 Basement 1 level A-direction modal ordinates for Republic Plaza

Mode	A1	A2	A3
Basement 1 modal ordinate ψ_{B1}	0.006	-0.010	0.016

The very low level of response at foundation level for Republic Plaza indicates that the deep foundation system[16] is stiff as designed and locks the building rigidly into the ground. The recording system used to obtain these signals is described in the next section.

INSTRUMENTATION SYSTEMS

As part of a project to characterise wind loading and dynamic response of tall buildings in Singapore, an instrumentation system was installed at the condominium between December 1994 and March 1996. The system is described elsewhere[20] in detail and comprised a pair of QA-700 servo-accelerometers arranged in Longitudinal and Lateral horizontal directions in the observation tower, an anemometer rising above the observation tower, a signal conditioner for the accelerometers, a PC with 12-bit analog digital converter (ADC) and acquisition software writing data to an optical disk.

The software was designed to acquire 8192-sample 'segments' of data at 9.4Hz capturing building response in the first two modes in each direction. For each record, values of maximum and variance of wind speed, mean wind direction and band-limited RMS acceleration response in the fundamental ‘Lateral’ and ‘Longitudinal’ modes were recorded in statistic files. For strong winds the original digitised segment was saved, otherwise discarded. Statistic files and data segments were written to optical disks. Measures taken to avoid data loss due to power failure included automatic closure of data files with imminent mains failure indicated by the uninterrupted power supply (UPS), and automatic acquisition restart on power resume. Acceleration signals were low pass (anti-alias) filtered and amplified to 1260V/g into an ADC range of ±2.5V, corresponding to approximately $\pm 20 mm.s^{-2}$.

When the condominium project closed the components of the system were moved to Republic Plaza as part of a more elaborate system operational from October 1996, starting with the pair of accelerometers aligned in A and B directions at roof level (level 65). A second pair of QA-700 accelerometers were added at basement (B1) level followed by three-component propeller anemometers on the Eastern and Western corners of the roof parapet. Signal cables at roof and basement routed to a small enclosure at level 65 were built-in during construction of the building.

Acceleration signal ranges are set to $\pm 10 mm.s^{-2}$ at basement and $\pm 50 mm.s^{-2}$ at level 65. For the 12 bit ADC, best resolution is $5 \mu m.s^{-2}$, which is matched to the accelerometer resolution threshold stated to be 'better than 1μg'.

The final system comprising four accelerometers and two 3-component anemometers has been in operation since May 1997, with progressive upgrades. Hardware improvements to minimise data loss due to failure of equipment with moving parts have included use of a solid state disk drive for DOS operating system and acquisition software, redundant disk drives for data storage and ultrasonic anemometers instead of propeller type. In addition the system is accessible by telephone line for status check, software upgrades, data download and PC clock correction. A separate telephone line is used for hardware reset in case of (rare) system crash.

The acquisition software not only records response to very strong winds, but also has been modified to improve the ability of the system to capture response signals due to tremors without overloading the system with irrelevant raw data.

The signal conditioning system was custom built by University of Bristol (UK) and together with the QA-700s has continued to provide signal stability with excellent resolution of the very low levels of acceleration response. The software has been developed at NTU and, as described in the next section, is being progressively optimised to 'skim off' the best raw data and provide a comprehensive and manageable account of performance of the building as well as the monitoring system itself.

STRATEGY FOR CAPTURING TREMOR DATA

In the acquisition system now at Republic Plaza, signals are digitised at 60Hz per channel into 1024 point short segments and mean and variance levels computed. For acceleration signals, modal RMS values are computed for narrow frequency bands centred on known natural frequencies. Mean and variance values for wind and modal RMS for accelerations are compared with set thresholds and a corresponding bit of a trigger word is set if any threshold is exceeded.

Data are digitally filtered and re-sampled at 7.5Hz for all channels into arrays representing 543-second segments of data and the statistic parameters (mean, variance, modal RMS) and trigger word value for the long segments are computed and saved. A data segment is saved if the trigger word is not zero. The trigger word shows why the event was saved, a vital debugging aid to devising the best method of capturing the useful data.

Fast sampling and with decimation (re-sampling) is done for three reasons:

1. Without using expensive simultaneous sample and hold acquisition cards the maximum time between first and last channels in a single ADC scan is minimised.
2. The raw 60Hz basement signals can be saved for suspected tremors. For the majority of signals, which are wind-induced, high frequency basement response is of no interest.
3. In some commercial spectrum analysers a single fixed (high) frequency anti-alias filter is employed together with a fixed high frequency sample rate. Subsequent digital filtering before resampling is software-adjustable to any desirable characteristic. The practice is copied here.

Finally, while not dependent on fast sampling, the triggering mechanism as described below relies on thresholding of RMS response levels in short lived transients with relatively high crest factors. There is a greater chance to capture such signals in short (17 second) records than to capture them according to lower modal RMS levels smoothed over 543 seconds.

Unlike conventional seismographs, using classical level threshold triggers on acceleration data is not a good strategy. The signal levels are in the μg range and signals are subject to thermal drift. Even using adaptive triggering with trend removal and filtering, different kinds of electrical noise spikes and colouring can lead to raw data overload.

True building response will be identified by significant roof level response at modal frequencies and therefore modal RMS levels for level 65 are used as the principal detector of dynamic building response to wind and tremors.

Fig. 7, data for September 1999, shows how level 65 modal RMS levels can be used for detection. Examination of A-direction response in fundamental mode (A1) and second mode (A2) shows a daily cycle of variation in modal RMS values, particularly for A2, probably due to lift and air conditioning machinery, and an obvious reduction of mechanical activity over a weekend.

There are also strong levels of response in mode A1 on days 10, 15 and 19, due to strong winds. For mode A2, the single spike on day 18 is due to a Ms5.2 earthquake epicentred 590km from the building.

There are uncharacteristic spikes in both A1 and A2 around 2AM on 21st September that certainly correspond to the Ji Ji (Taiwan) earthquake.

Examination of raw data and statistical properties of a number of tremors shows that the second translational modes of the building in each direction are most strongly excited, whereas even a strong wind will not increase significantly the second mode response compared to the usual background level of 75-125$\mu m.s^{-2}$. Hence an isolated second mode RMS spike strongly suggests a tremor. Since the background noise varies down to a minimum 10$\mu m.s^{-2}$ overnight, an isolated spike within the daily noise range at these times also suggests a (weak) tremor. To capture even the weak tremors at night, the detection strategy has been modified to compare critical modal RMS values in mode 2 for each direction against a moving average, similar to short time average/long time average (STA/LTA) algorithms used in other seismometers[22].

Some of the captured signals are described in the next section.

SUMMARY OF CAPTURED SIGNALS AT REPUBLIC PLAZA

Table 5 summarises signals recorded to date (June 2000) based on information given at the United States Geological Survey web site http://wwwneic.cr.usgs.gov/. Times are UTC (Greenwich Mean Time; Singapore is GMT+8 hours) and distances are from Singapore at Latitude 1.3°, Longitude 103.8°. Surface magnitudes (Ms), peak basement (B1) accelerations and peak and standard deviation (σ) for roof (level 65) acceleration response are given.

Table 5 Summary of detected long-distance earthquakes, 1997-2000

	Timing		distance	Magnitude and building response / $mm.s^{-2}$				
	GMT	Date	km	Ms	max(B1)	max(65)	σ(65)	1.5%W/V_{pk}
1	08:05:48	17 Mar 1997	892	6.4	0.27	3.5	0.8	253
2	22:14:18	18 May 1997	572	5.4	0.44	2.4	0.35	400
3	11:24:37	07 Jul 1997	695	5.9	0.64	3.2	0.60	257
4	**07:15:00**	**20 Aug 1997**	**883**	**6.0**	**0.81**	**8.6**	**1.44**	**82**
5	05:46:57	18 Dec 1997	591	5.7	0.49	3.9	0.51	256
6	**14:10:32**	**29 Nov 1998**	**2372**	**8.3**	**0.61**	**3.2**	**0.47**	**192**
7	04:35:57	18 Feb 1999	494	5.1	1.04	.6.4	0.70	175
8	00:16:52	14 Aug 1999	798	6.5	0.37	4.0	0.77	167
9	12:52:35	18 Sep 1999	590	5.2	0.53	4.0	0.78	162
10	**18:05:43**	**11 Nov 1999**	**397**	**6.2**	**0.87**	**4.5**	**0.84**	**148**
11	14:14:57	21 Dec 1999	918	6.6	0.57	7.9	1.51	79
12	16:28:25	4 June 2000	685	8.0	7.35	47.4	7.25	13

The list does not include some of the strongest tremors felt in Singapore in the last decade:

15 Feb 1994	(693km)	Ms 7.0
06 Oct 1995	(453km)	Ms 7.1
08 Nov 1995	(973km)	Ms 7.1
01 Apr 1998	(543km)	Ms 7.0

The first two predated the instrumentation at Republic Plaza and the last one was not captured due hard disk failure. Signals recorded during and after the Bengkulu earthquake of June 2000 (event 12) are the subject of separate research involving data from other seismic stations.

All events except 1 and 6 occurred in Sumatra. Records 4,6 and 10, highlighted in Table 5 originated from Central Aceh (North Sumatra), Taliabu (off Sulawesi) and Central Sumatra and are shown in Figs. 8,9 and 10 respectively illustrating events at (relatively) medium, very long and close range. The Aceh signal is typical of other Sumatran records, lasting several minutes and inducing a relatively strong second mode response. The Taliabu (record 6) signal is for the strongest and most distant event and is characterised by a strong low frequency response. Double integration reveals that the building, presumably together with surrounding areas, experienced a slow rigid body oscillation of up to 1cm.

Fig. 11 shows the 1% response spectra derived from the basement signals of records 4,6 and 10. Spectra for other records have strong peaks below 1Hz, and for all but the closest records the response is limited to 0-2Hz range.

Since the response records are available, the building mass distribution is known from construction data provided by the contractor, and the prototype mode shapes are available, the actual base shear $V(t)$ and corresponding absolute maxima V_{pk} can be estimated by summing contributions over modes *r*:

$$V(t) = \sum_r L_r \ddot{x}_r(t) \qquad (1),$$

where

$L_r = \sum_k {}^r\psi_k m_k$ the summed product of mode *r* ordinates ${}^r\psi_k$ with lumped masses m_k over storeys (levels) k. Mode shapes ${}^r\psi$ are normalised to unity at the roof.

$\ddot{x}_r(t)$ is the component of roof level acceleration response in mode *r*, obtained by digital band-pass filter in the acquisition software.

Invariably the largest contributor to V_{pk} in these tremors is the second mode. Overturning moment can be similarly determined and is invariably dominated by fundamental mode contribution. The largest base shear value over all the records, 0.804MN represents 0.11% of total weight W which is only a small fraction of the 1.5%W notional horizontal load (NHL) value. Scale factors on the signals to bring the measured V_{pk} for the recorded tremors values up to NHL are given in the last column of Table 5. Clearly for these records NHL is more than adequate.

For comparison, the dynamic component of base shear reached 0.4MN during the strongest recorded winds of 6 April 2000, gusting to 25m.s^{-1} and inducing ±1.5cm dynamic roof sway.

TRANSFER FUNCTIONS, RESPONSE SPECTRA AND FREE FIELD MOTIONS

The very low basement responses during wind excitation (Table 4) suggest that for base excitation the level B1 signals should be reliable indicators of input motion for seismic dsign of buildings. Given the synchronous basement and roof signals that have been recorded for a number of tremors it is possible to draw further inferences about foundation performance and the reliability of the B1 signals as indicators of ground motion. To this end it is necessary to study the relationship or transfer function between the responses at the two levels, starting with the equations of motion for a system subject to excitation at the support point which are

$$\mathbf{M}\ddot{\mathbf{x}}_R + \mathbf{C}\dot{\mathbf{x}}_R + \mathbf{K}\mathbf{x}_R = -a\mathbf{M}\mathbf{g} \qquad (2)$$

with $\mathbf{x}_R = \mathbf{x}_T - x$. (3)

M,**C**,**K** are mass stiffness and damping matrices respectively, **g** is a column vector with ones corresponding to the direction of support motion, *a* and *x* are scalar functions of time-varying support acceleration and displacement respectively and $\mathbf{x}_T$ is absolute or total displacement.

Assuming a linear system with proportional damping, the undamped eigenproblem solves for modes Ψ and eigenvalues $\Lambda = diag\left\{\omega^2\right\}$ satisfying

$$\mathbf{M}^{-1}\mathbf{K}\Psi = \Lambda\Psi \tag{4}$$

where Ψ represents a matrix of eigenvectors ${}^r\psi$ i.e. for mode r

$$\left[\mathbf{K} - \mathbf{M}\omega_r^2\right]{}^r\psi = 0, \tag{5}$$

and modal masses are $m_r = {}^r\psi^T \mathbf{M}\,{}^r\psi$.

In frequency domain, the relationship between acceleration response $\ddot{\mathbf{X}}_T(\omega)$ and support acceleration $A(\omega)$ is

$$\left[\frac{\ddot{\mathbf{X}}_T}{A} - \mathbf{g}\right] \cdot \frac{1}{\omega^2} = \mathbf{HMg} \tag{6}$$

with the frequency response function (FRF) matrix $\mathbf{H}(\omega)$ given by

$$\mathbf{H} = \Psi\mathbf{D}^{-1}\Psi^T \tag{7}$$

where $\mathbf{D} = diag\left\{m_r\left(-\omega^2 + \omega_r^2 + 2i\zeta_r\omega_r\omega\right)\right\}$. (8)

The FRF relating displacement response at point j (or k) to force at point k (or j) represents one element of $\mathbf{H}$ i.e.

$$H_{jk}(\omega) = \sum_r \frac{{}^r\psi_j\,{}^r\psi_k}{m_r} \cdot \frac{1}{-\omega^2 + \omega_r^2 + 2i\zeta_r\omega_r\omega}. \tag{9}$$

For a single response position e.g. k equation (6) leads to the classical transmissibility function for base excitation

$$\frac{\ddot{X}_{Tk}}{A} = \sum_r {}^r\psi_k \Gamma_r \frac{\omega^2}{\omega_r^2} DAF_r + 1 \tag{10}$$

where Γ_r and DAF_r are participation factor and dynamic amplification factor respectively given by

$$\Gamma_r = \frac{{}^r\psi^T\mathbf{Mg}}{m_r} = L_r / m_r \tag{11}$$

and $$DAF_r = 1\Big/\left(1 - \frac{\omega^2}{\omega_r^2} + 2i\zeta_r\frac{\omega}{\omega_r}\right). \tag{12}$$

If support and structure response signals are both available standard modal analysis software can be used to extract frequencies ω_r, damping ratios ζ_r and modal constants ${}^r\psi_k\Gamma_r$.

Taking ${}^r\psi_k \equiv 1$ and the known mass distribution and mode shapes, the participation factors are estimated as:

$\Gamma_{A1} = 1.68$ $\Gamma_{A2} = -0.85$ $\Gamma_{A3} = 0.702$

$\Gamma_{B1} = 1.66$ $\Gamma_{B2} = -0.92$ $\Gamma_{B3} = 0.55$.

In general, where the basement signal is not necessarily true support excitation *a(t)*, the ratio or transfer function of measured signals between say roof (k=65A) and basement (k=B1) is

$$\frac{\ddot{X}_{T65\mathrm{A}}}{\ddot{X}_{T\mathrm{B}1}}=\frac{1+\sum_r {}^r\psi_{65A}\Gamma_r\frac{\omega^2}{\omega_r^2}DAF_r}{1+\sum_r {}^r\psi_{B1}\Gamma_r\frac{\omega^2}{\omega_r^2}DAF_r}\,. \tag{13}$$

If the measured basement response is true support motion and does not contain any element of deformation as part of a vibration mode then the modal ordinate (for mode *r*) at B1 must be zero i.e. ${}^r\psi_{\mathrm{B1}}\equiv 0$ and equation (13) simplifies to equation (10). If basement response is not true support motion but contains some deformation of the structure (which includes the foundation system) the effect is seen in a modification of classical transmissibility function. For example if the soil structure interaction is modelled as a traditional linear spring then the modal ordinate ${}^r\psi_{\mathrm{B1}}$ for the complete soil-structure system will not be zero at the basement.

Fig.12 shows, for the 'medium range' (Aceh) A-direction signals the real and imaginary parts of the transfer function defined in equation (10). Fig. 13 shows the corresponding SDOF circle fit (Nyquist plot) for the second peak, corresponding to mode A2; the diamonds represent the data points. Extracted modal quantities $\Gamma_r=-0.94$, $\zeta_r=0.73\%$, $f_r=0.675\,\mathrm{Hz}$ correspond well with values obtained via ambient vibration survey and subsequent wind induced motions. This suggests that the behaviour of the building is accurately represented by the model with a rigid foundation.

To obtain a more convincing indication that the basement signal is not so much subject to motion relative to the ground, Fig 14 illustrates the sensitivity of recovered modal parameters to non-zero values of ${}^r\psi_{\mathrm{B1}}$. The curves are synthesised from equation (13) including three modes *r*=A1, *r*=A2 and *r*=A3 for the two values of basement modal ordinate, ${}^r\psi_{B1}=0$ and ${}^r\psi_{B1}=0.10$. The latter value is taken as a extreme value for less rigid foundation systems involving significant soil-structure interaction. Using the non-zero mode shape, there are large shifts of apparent modal frequency; for ${}^r\psi_{B1}=0.10$ the shifts are +9.7%, -4.8% and +3.8%. The effect is linear with ${}^r\psi_{\mathrm{B1}}$.

The lack of any discernible frequency shift in the transfer function data for the prototype during the Aceh tremor (Figs 12,13) and other signals shows that the basement level response modal ordinate is negligible. Hence it can be concluded that the B1 signal is reliable, given the absence of excitation from other sources.

EFFECT OF WIND, MACHINERY AND BACKGROUND NOISE

Is the system able to distinguish tremors from low level background noise, for which the daily cycle is evident in Fig. 7? At basement level the signal is close to the accelerometer resolution limit; the dotted and dashed lines in Fig. 11 show, for comparison, response spectra corresponding to basement signals recorded during zero wind (and no tremors) overnight (labelled N noise) and during daytime (labelled D noise). Examination of spectrograms of low level basement signals from the raw 60Hz data shows no systematic noise such as sub-harmonics from rotating machinery until 6Hz. Apart from wind and tremors, a significant source of excitation is the roof-mounted crane used for window-cleaning operations; slewing of the boom induces clear transient response in third and higher vibration modes.

From the inspection of the anemometer signals and the levels of fundamental mode response at the time of the tremors it is clear whether or not there would be any significant influence of wind in the recorded response signals. This would also be seen in the time series as a first mode response prior to the main shaking. Also in a very strong wind the effect is also visible as coherent vibrations with zero phase angle between basement and roof signals rather than the 90° phase angle that is characteristic of ground excitation. For most of the events given in

Table 5 the signals were recorded at times of minimal or zero wind and the basement signal will still be relatively 'clean'.

SIGNAL FROM CONDOMINIUM

Since the purpose of the instrumentation here was to observe wind load and response, no basement instruments were installed, and triggering was dependent on wind speed only. The only tremor information available is for the 6th Oct 1995 (453km) Ms 7.1 event. Fig. 15 shows (on a logarithmic scale) the values of modal RMS acceleration for 15-minute segments between 6th and 11th October 1995. As well as the clear daily cycle, the peak modal RMS value of 2.5mm.s^{-2} in the early hours of October 7th corresponds with the tremor, while the peak at 0.3mm.s^{-2} on 11th October is due to an early morning squall with winds gusting to 20m.s^{-1}.

These two values represent response to relatively strong incidence of two major types of lateral load in the last few decades. The condominium is typical of high rise residential buildings in Singapore and it is clear that the dynamic component of wind-induced response is an order magnitude less than seismic response.

Before 1996 the only tremor data available had been the observed effects at high storey level, such as this single RMS value, or in another case, the ringing of wind chimes during an earlier event felt in Eastern Singapore. In each case[23,24], inverse studies were done to estimate the level of ground motion or base shear involved. Taking into account soil conditions obtained through micro-tremor measurements, peak base shear was estimated[20,24] at 0.5%-0.8% of building weight i.e. roughly half the NHL provision.

DISCUSSION

The original function of the instrumentation shifted from the detection of strong wind data to the capture of ground motion and response signals for distant tremors since these were seen as potentially having the stronger dynamic effect. These 'weak motion' signals recorded so far contribute to better understanding of local ground motions and help identify whether or not it is acceptable to cater for seismic hazard through a code provision for accidental eccentricity.

From the recorded signals, the ground motions are no cause for concern and the effects are within existing provisions for lateral loading in Singapore. Whether ground motions arising from stronger events could be a cause for concern depends on attenuation effects and structural capacity[25]. What is clear is that ground motions are typically strongest in the 0.5Hz-1.0Hz (1second to 2 second) range, roughly coinciding with fundamental natural frequencies of buildings in the 15-30 storey range i.e. predominantly residential structures.

Detection of weak ground motion is a certainty with continuous recording of signals from high specification systems having broad band seismometers installed at well-prepared ground sites. Detection of such signals is also possible using sophisticated seismographs with limited storage capacity and detection strategies. For the weak ground motion resulting from distant tremors, detection against a noisy background is a significant problem and is best approached by looking for characteristic structural response e.g. in a tall building whose fundamental mode response is normally wind-driven.

CONCLUSIONS

Instrumentation of two buildings in Singapore has provided limited but valuable data not only about wind conditions and effects, but more usefully about the type of relatively weak long-distance tremor to which Singapore is frequently an observer. Intimate consideration of the dynamic characteristics of one building have shown that recorded basement signals should be useful for input motion in seismic design of buildings. The limited number of recorded signals have shown that except for the closest tremors (still more than 350km distant), ground motions are strongest in the 0.5Hz to 1.0Hz range and are unlikely to be a problem for very tall buildings even at much higher amplitudes. For tall residential buildings the effects are not so certain and would require specific instrumentation.

ACKNOWLEDGEMENTS

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REFERENCES

1 CP3: Code of basic data for the design of buildings. Chapter V, Loading, Part 2, Wind loads. British Standards Institution, UK (1972).

2 BS6399: British Standard: Loads for buildings. Part 2. Code of practice for wind loads. British Standards Institution, UK (1995).

3 BS8110, Part 1: British Standard: Structural use of concrete: Code of practice for design and construction. British Standards Institution, UK (1985).

4 H. W. Tan (Building Construction Authority Singapore), 'Singapore is not in earthquake belt', Straits Times, Singapore Press Holdings, 4th October, 42. (1999)

5 T. Balendra, T. S. Tan and S. L. Lee, 'An analytical model for far-field response spectra with soil amplification effects', Engineering Structures **12**, 263-268 (1990).

6 F. E. Udwadia and M. D. Trifunac, 'Ambient vibration tests on full-scale structures', Proceedings,. 5th world conference on Earthquake Engineering, Rome **2**, 1430-1439 (1974).

7 T. Tanaka, S. Yoshizawa Y. Osawa and T. Morisishita, 'Period and damping of vibrations in actual buildings during earthquakes', Bulletin of the Earthquake Research Institute **47,** 1073-1092 (1969).

8 P-Y. Bard, H. Afra and A. Argoul, 'Seismic response of buildings during earthquakes: Experimental results from strong motion data', EC-Mexico Workshop on Seismology and Earthquake Engineering, Mexico. Centro Nacional de Prevencion de Desastres, 149-175 (1991).

9 N. Satake and H.Yokota, 'Evaluation of vibration properties of high-rise steel buildings using data of vibration tests and earthquake observations', Journal of Wind engineering and Industrial Aerodynamics **59**, 265-282 (1996).

10 G. L. Fenves, E. Safak and M. Raghavendrachar, 'Examples of structural identification from measured response: Buildings, bridges and dams', Building an International Community of Structural Engineers, ASCE Structures Congress **2**, 659-666 (1996).

11 J. P. Stewart and G. L. Fenves, 'System indentification for evaluating soil-structure interaction effects in buildings from strong motion recordings', Earthquake Engineering and Structural Dynamics **27** , 869-885 (1998).

12 H. Lus, R. Betti and R. W. Longman, 'Identification of linear structural systems using earthquake-induced vibration data', Earthquake Engineering and Structural Dynamics **28** , 1449-1467 (1999).

13 T. C. Pan and J. Sun, 'Historical earthquakes felt in Singapore', Bulletin. Seismological. Society of America **86**, 1173-1178, (1996).

14 J. M. W. Brownjohn and C. K. Ang, 'Full-scale dynamic response of a high rise building subject to lateral loading', ASCE Journal of Performance of Constructed Facilities **12**, No. 1, 33-40 (1998).

15 H. S. Teh, and H. P. Lai, 'Structural aspects of the Republic Plaza', Proceedings, Seminar on Tall Buildings – Design and Construction, The Institution of Engineers Singapore (1995).

16 B. B. Broms and H. P. Lai, 'The Republic Plaza in Singapore –foundation design', Bengt B. Broms symposium in Geotechnical Engineering, Singapore, 13-15, 3-24 (1995)

17 J. M. W. Brownjohn, T. C. Pan and H. K. Cheong, 'Dynamic response of Republic Plaza, Singapore', The Structural Engineer **76** No. 11, 221-226 (1998).

18 K. L. Ngan and Y. M. Khor, 'Investigation and studies of high-rise building subject to wind loading', Final Year Project Report, School of Civil and Structural Engineering, NTU, Singapore (1995).

19 E. G. Ng and C. M. Koh, 'Dynamic performance of a tall building', Final Year Project Report, School of Civil and Structural Engineering, NTU, Singapore (1996).

20 J. M. W. Brownjohn and C. K. Ang, 'Full-scale study of dynamic characteristics of tall buildings in Singapore to investigate lateral loading and response', Applied Research Project RP11/93, School of Civil and Structural Engineering, NTU, Singapore (1996).

21 J. M. W. Brownjohn, T. C. Pan and X. Y. Deng, 'Correlating dynamic characteristics from field measurements and numerical analysis of a high-rise building' , Earthquake Engineering and Structural Dynamics **29** No. 4 , 523-543 (2000).

22 A. Trnkoczy, 'Understanding and setting STA/LTA trigger algorithm parameters for the K2', Application Note 41, Kinemetrics, USA (1998).

23 T. C. Pan, 'When the doorbell rings -a case of building response to a long distance earthquake', Earthquake Engineering and Structural Dynamics **24**, 1343-1353 (1995).

24 N. F. Tao and J. M. W. Brownjohn, 'Estimation of ground motion acceleration and building response to a long distance earthquake' Journal of Earthquake Engineering **2**, No. 3, 477-485 (1998).

25 T. Balendra, K. H. Tan and S. K. Kong, 'Vulnerability of reinforced concrete frames in low seismic region, when designed according to BS8110'. Earthquake Engineering and Structural Dynamics **28**, 1361-1381 (1999).

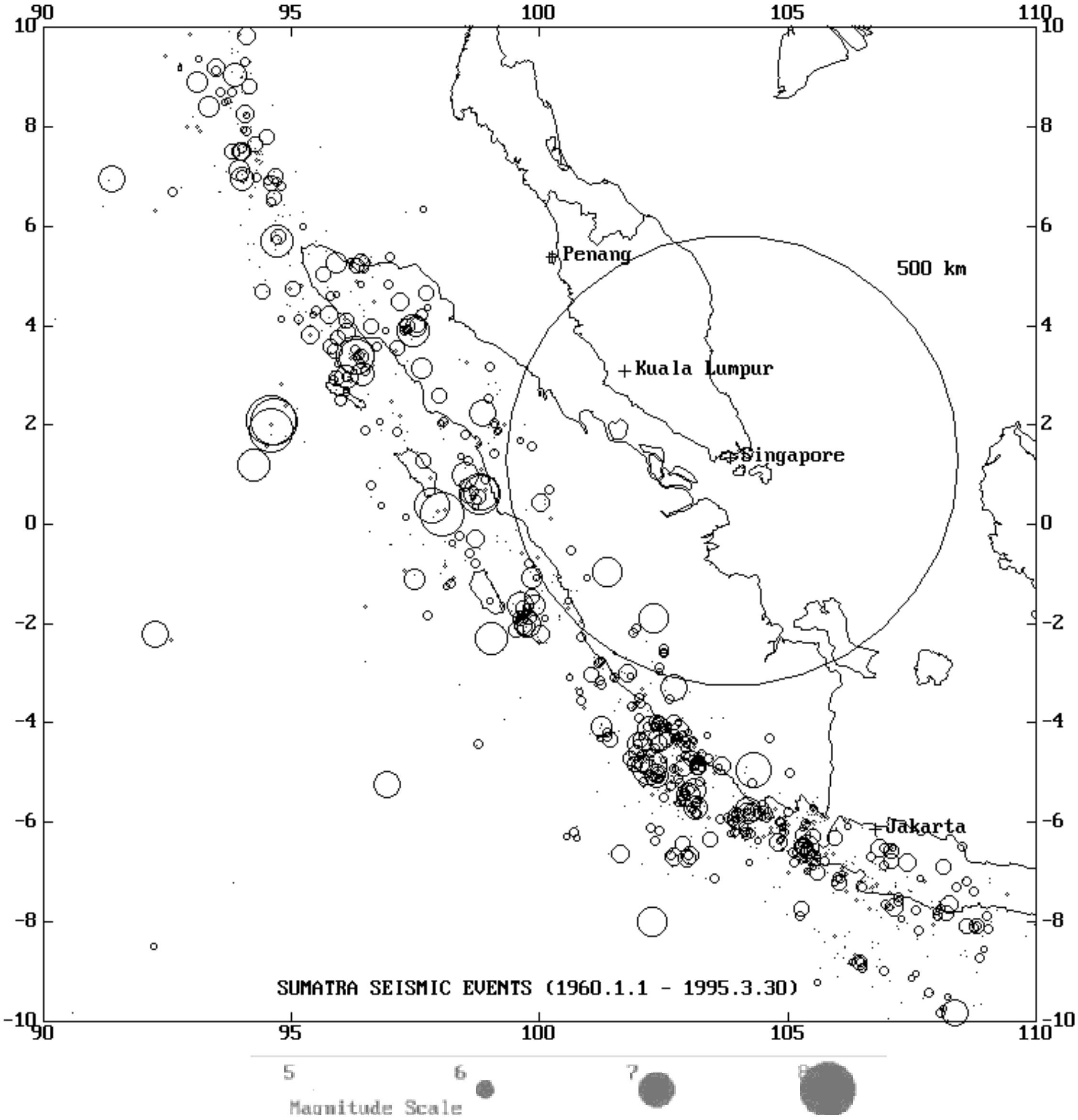

Figure 1 Epicentre locations of earthquakes in Sumatra region (1960.1.1-1995.5.30)

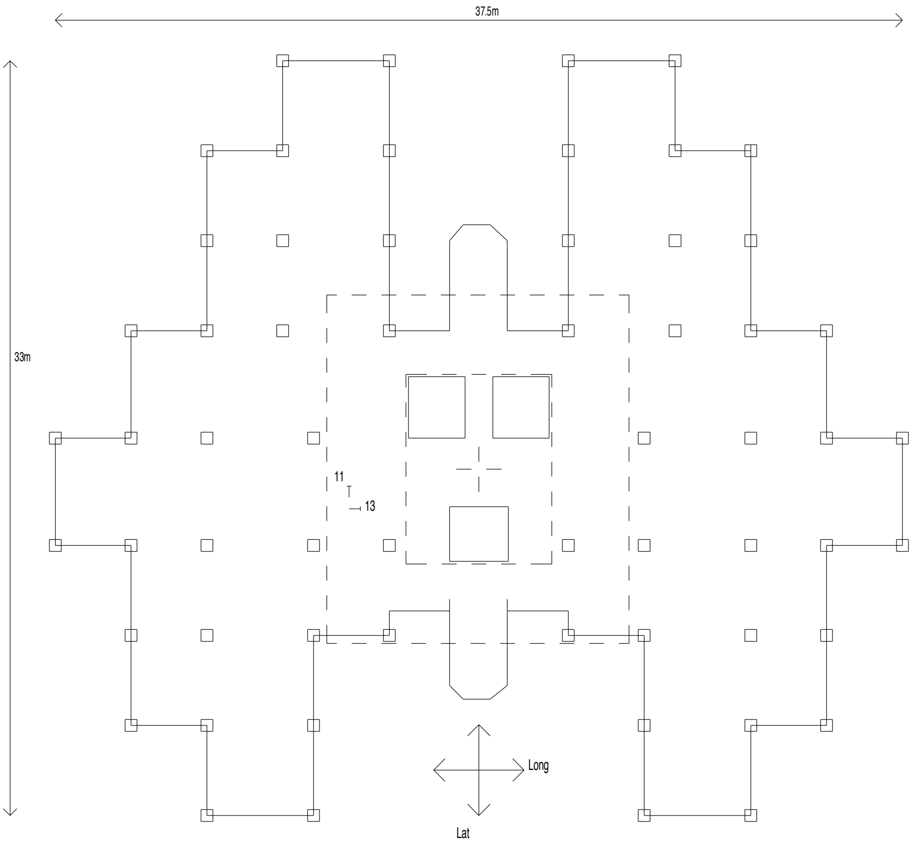

Figure 2 Floor plan for condominium

Figure 3 Republic Plaza

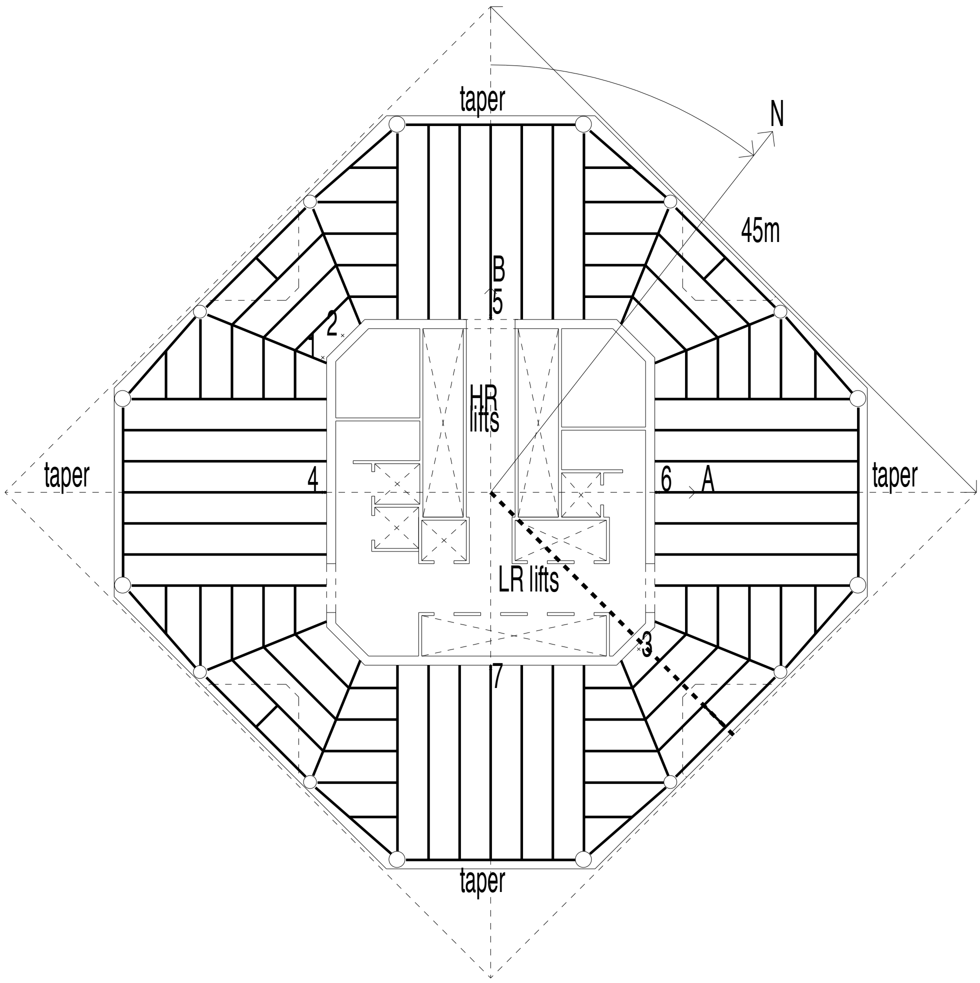

Figure 4 Floor plan for Republic Plaza

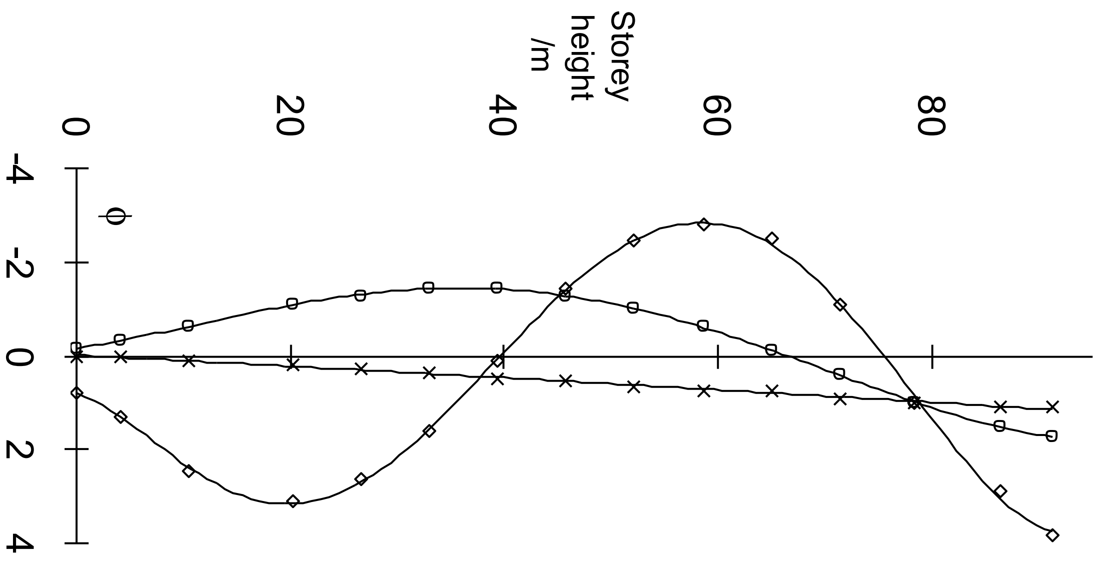

Figure 5 Lowest three Lateral direction translational mode shapes for condominium

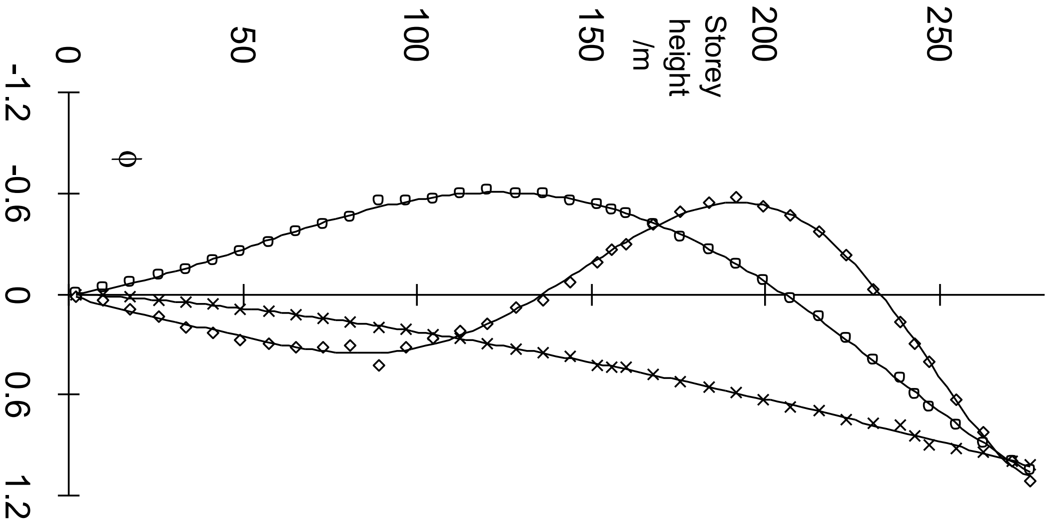

Figure 6 Lowest three A-direction translational mode shapes for Republic Plaza

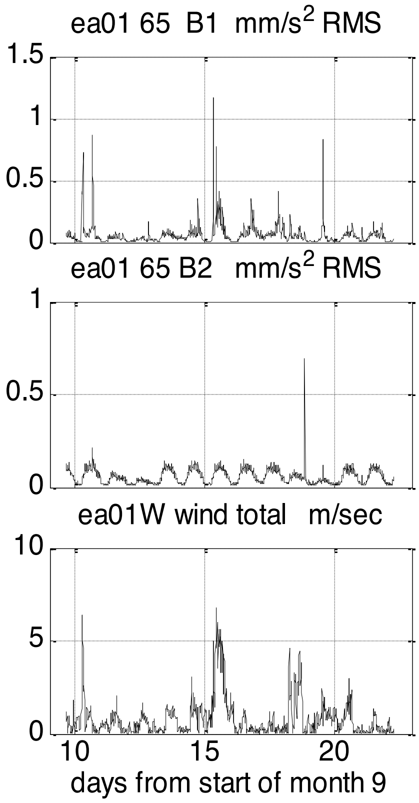

Figure 7 Mode A1 (upper) and mode A2 (lower) RMS acceleration response during September 1999

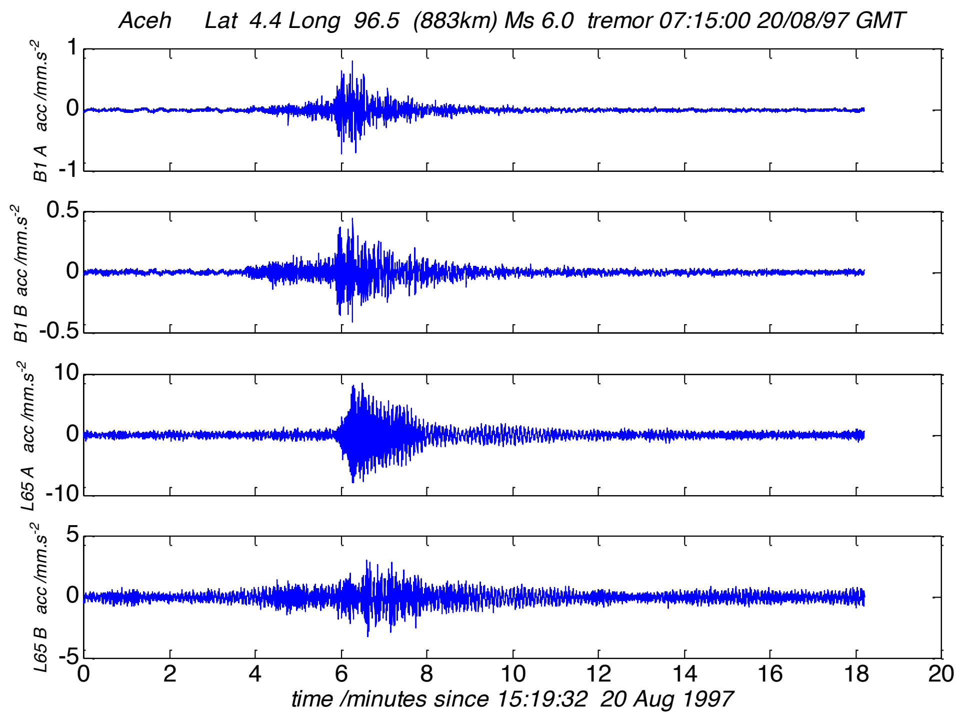

Figure 8 Recorded signals for 'medium range' distant tremor

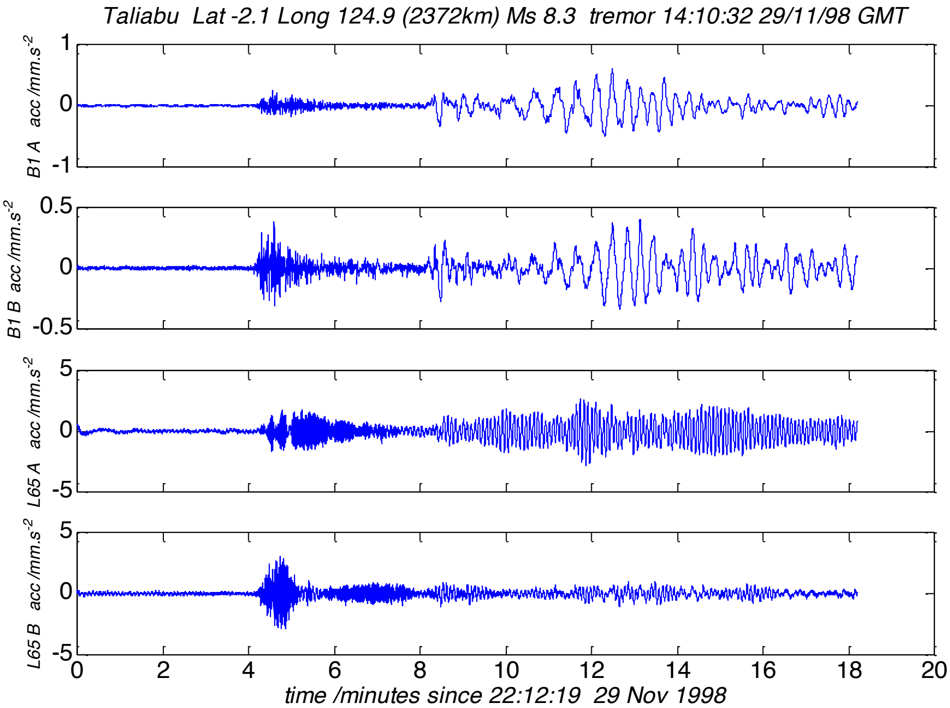

Figure 9 Recorded signals for 'very long range' distant tremor

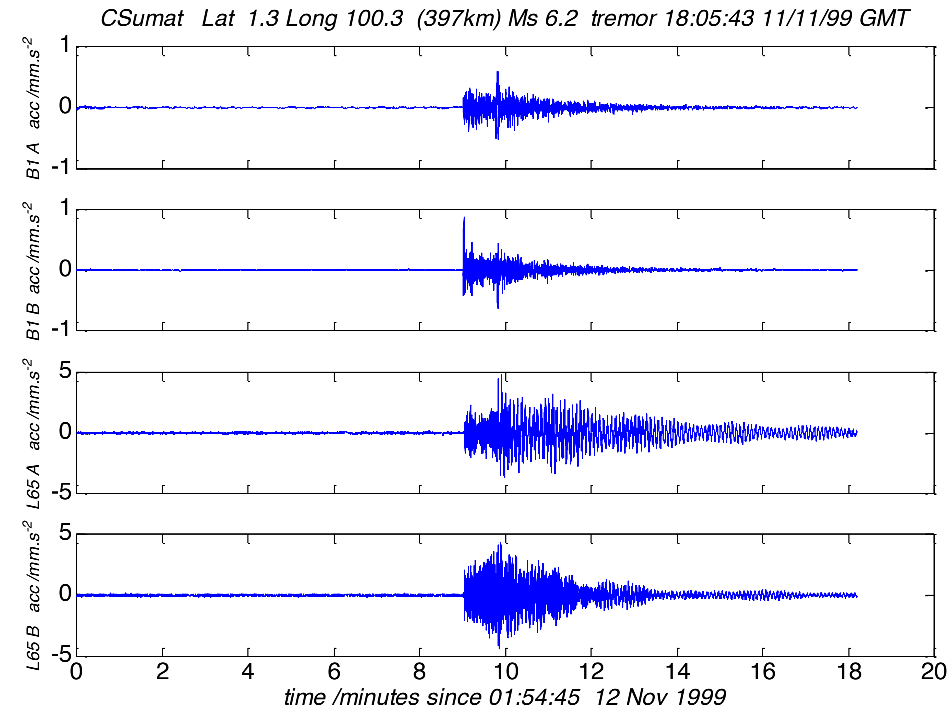

Figure 10 Recorded signals for 'close range' distant tremor

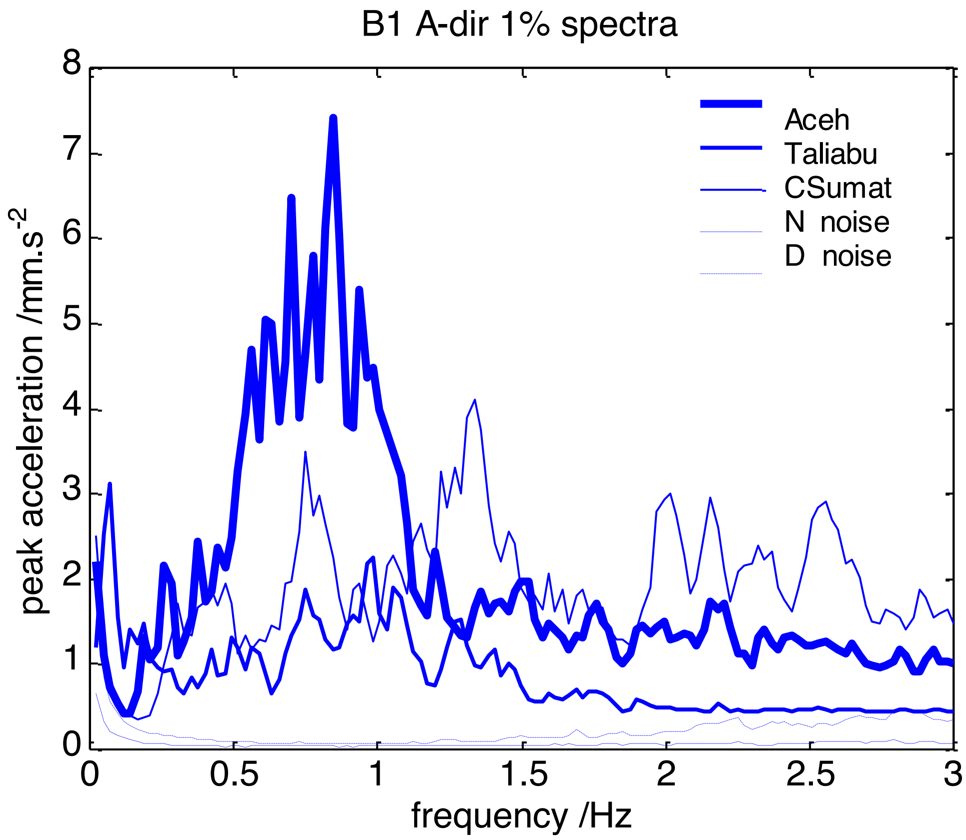

Figure 11 Response spectra for transient (B1) A-direction signals given in Figures 8-10, and for background noise

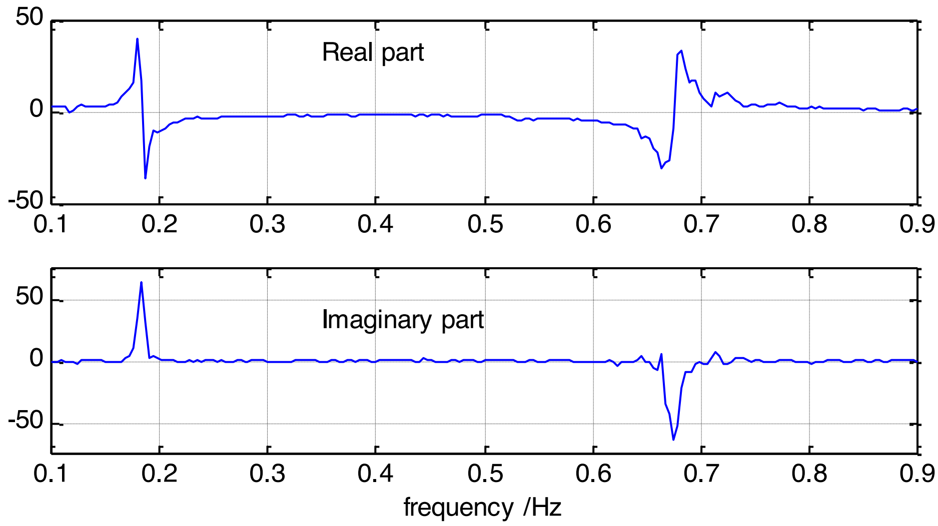

Figure 12 Real and imaginary parts of transfer function for Aceh signal (Figure 8)

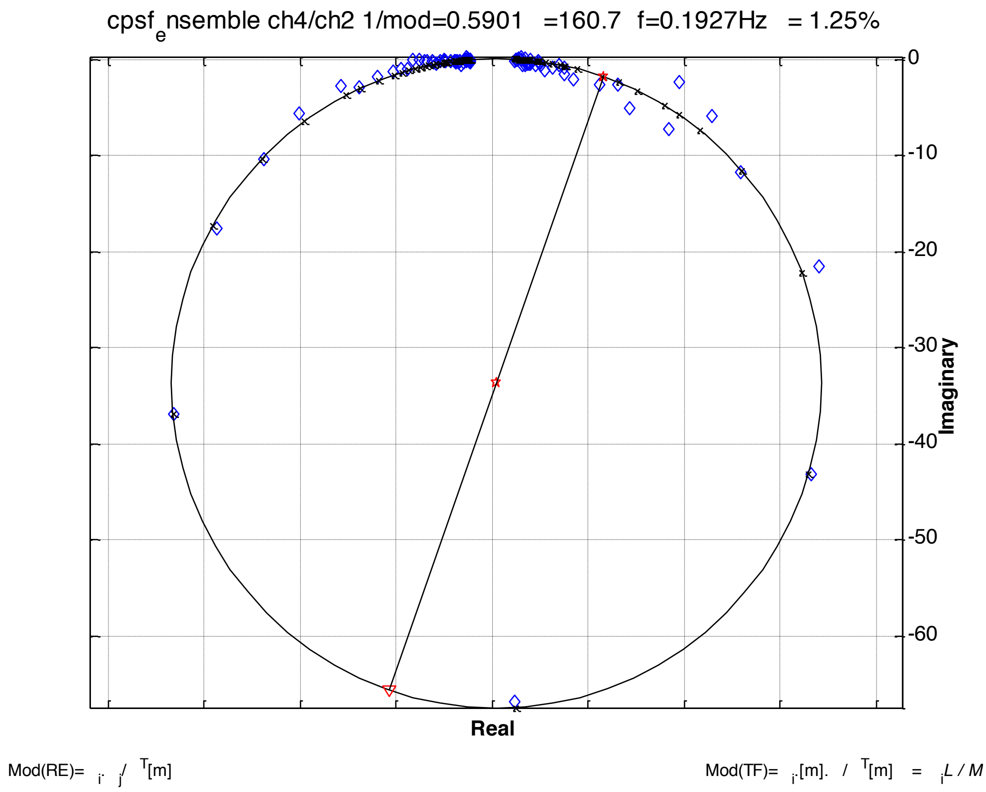

Figure 13 Single degree of freedom circle fit to function of Figure 12

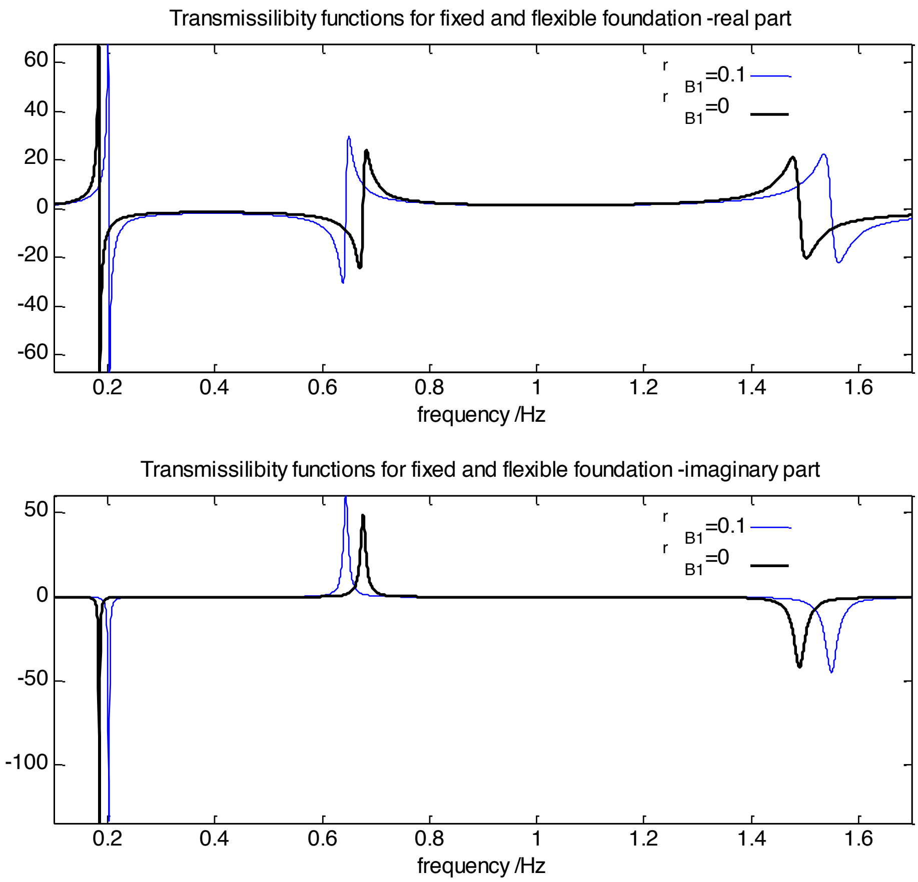

Figure 14 Synthesised transmissibilities from equation (13) for zero and non-zero basement modal ordinates

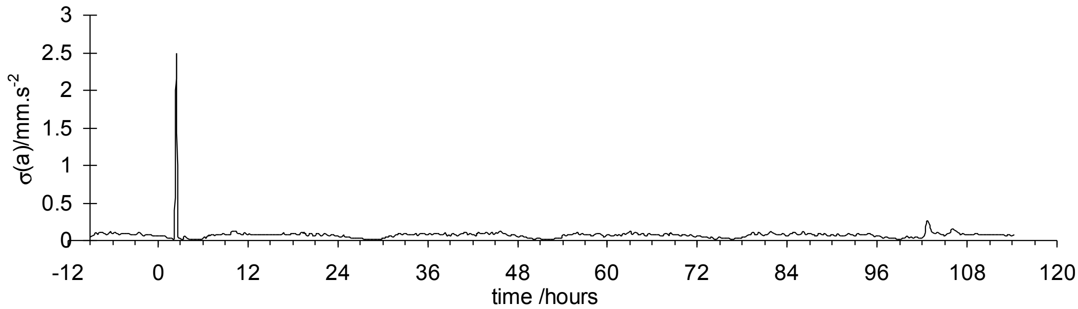

Figure 15 Fundamental mode RMS longitudinal acceleration for condominium during distant tremor (2AM 7 Oct 1996) and strong wind (6AM 11 Oct 1996)