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# Forensic investigation of a slender high-rise structure subject to dynamic wind conditions

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**Abstract.** The forensic engineering of a collapsed reinforced concrete fin wall structure is presented in this paper. The investigative process considers the assessment between the actual wind load conditions structure during the time of collapse against the actual structural capacity of the structure. Three design conditions are considered in this study; in-situ wind speed during the time of incident, wind speed used by the design consultant and wind speed recommended by Jabatan Meteorologi Malaysia (JMM) in 1995, which was the time of design. The structure is both analyzed from a static analysis and dynamic time history analysis standpoint. It was found that the amplification factor due to the near resonance of wind energies between 0.25-0.30 Hz and the natural frequency of the structure, 0.27Hz. This resulted in an amplification of 1.8 over quasi-static loading conditions as governed by CP3 design code. It was further found that the dynamic wind loading imposed against the structure was 10% less from the actual capacity of the structure determined via nonlinear pushover analysis. This result is at best marginal considering the inelastic performance of the structure and the actual as-built condition of the structure; giving strong indication to the role of resonance in the collapse. The technical interpretation of the design codes leading to the responsibilities of the design consultant are not discussed in this paper.

## 1. Introduction

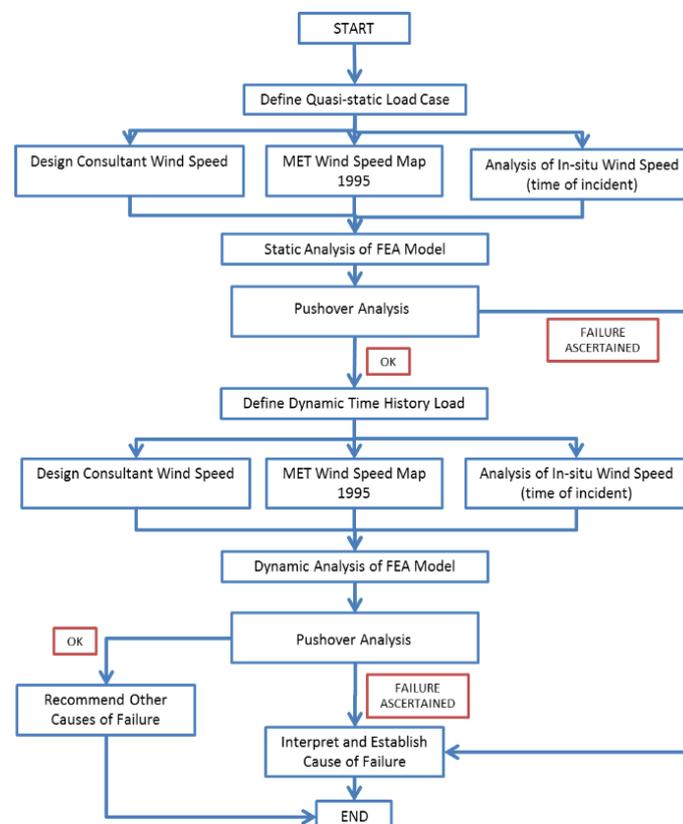
This paper intends to investigate an incident in Malaysia whereby a reinforced concrete fin wall structure coupled with an attached lightning arrestor collapsed and dropped off from about the roof level a high-rise building in Pulau Pinang. It was noted that the collapse coincides with a recorded storm event at the vicinity of the event vis-à-vis recording made by the Jabatan Meteorologi Malaysia (JMM) at Bayan Lepas monitoring station site in Pulau Pinang. The objective of this report is to presents the results that lead to the determination of the cause of the collapse and failure of the reinforced concrete fin wall and the attached lightning arrestor. The forensic investigation of the fin wall structure failure is predicated on the determination of the dynamic loads occurring near the time of event and as to whether the structure had sufficient capacity to withstand the dynamic wind loads on the fin wall. Typically for dynamically sensitive structures due to wind effects, a quasi-static or full dynamic analysis methodologies have to be applied in assessing the ultimate performance of a building structure [1]. This is primarily to assess the possibility of resonant or near resonant wind conditions from occurring on the structure, resulting in destructive amplification of motions [2]. For the general application of most engineering design, a quasi-static design to wind loads is typically utilized. This is usually embodied within design codes such as CP3, MS1553, BS6399 and ASCE7 which applies factors which account for the dynamic effects of wind on a structure[3, 4, 5]; in lieu of a full-fledged dynamic analysis approach. There is however limitations to such considerations as design codes generally do not cater to unusually shaped structures nor structures which exhibit abnormal



dynamic effects to excitation sources. Codes such as ASCE7 also exempt structures which are not dynamically sensitive from such assessment with the threshold being any structure possessing a natural frequency of 1Hz or more [3]. With the above consideration in mind, one can assume that the wind load design of a regularly shaped structure should be sufficiently accounted for by following the design procedures in the aforementioned codes. Conversely, a lightly damped structure operating close to resonant wind conditions and possessing an unusual geometry should be subject to further investigation.

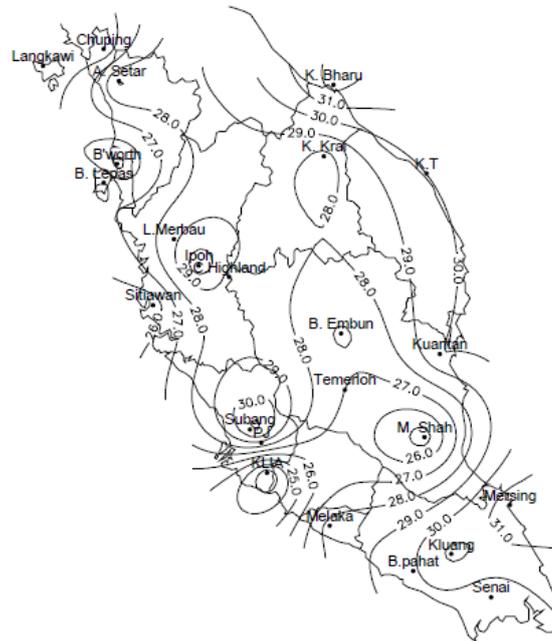
## 2. Methodology

Figure 1 summarizes the methodology undertaken in assessing the collapse of the reinforced concrete fin wall and the lightning arrester attached to the feature wall. In the methodology presented herein, a baseline analysis is done to ascertain the structural response of the feature wall subject to quasi-static load. The results obtained from the baseline analysis are used to ascertain the integrity of the reinforced concrete fin wall and the lightning arrester. Survivability of the reinforced concrete fin wall and the lightning arrester predicates advanced analysis including dynamic analysis on the coupled reinforced concrete fin wall and the lightning arrester.



**Figure 1:** Proposed methodology to determine failure of structure

In understanding the base line case, it is imperative to understand the basic wind speed employed by three entities, namely, a) basic wind speed used by the design consultant, b) basic wind speed based on data made available by JMM up to 1995 (time of design), c) basic wind speed derived from the analysis of wind records presented herein on or about the time when the collapse was documented based on JMM data. For clarity purposes, the JMM data used was based on the wind data provided up to 1995 only, which was the time of the design of the structure. It is the requirement of an engineer to ensure that their design to wind load conforms to the prevailing wind code or local wind maps such as shown in Figure 2. It is also noted that West Peninsular Malaysia is not considered as a high wind region although some areas of Malaysia experienced higher than normal wind speed.



**Figure 2:** Basic wind speed map for 50-year return period also indicating monitoring stations in which wind speeds are interpolated from (MS 1553:2002)

The design wind pressure was determined as per the governing or prevailing wind code at the time when the structure was proposed to be built and designed. This prevailing wind code is based on CP3. Based on the CP3 at that time, the basic wind speed is based on a 3-sec gust averaging time with a mean recurrence interval of 50 years. Table 1 summarizes the various wind speeds under investigation in this paper. The basic wind speeds are compared against the capacity of the structure.

Table 1: Summary of Basic Wind Speeds

Wind speed cases	Basic wind speed (m/s)
Design consultant	22.50
JMM Wind Speed Map 1995	27.37
In-situ condition	17.93

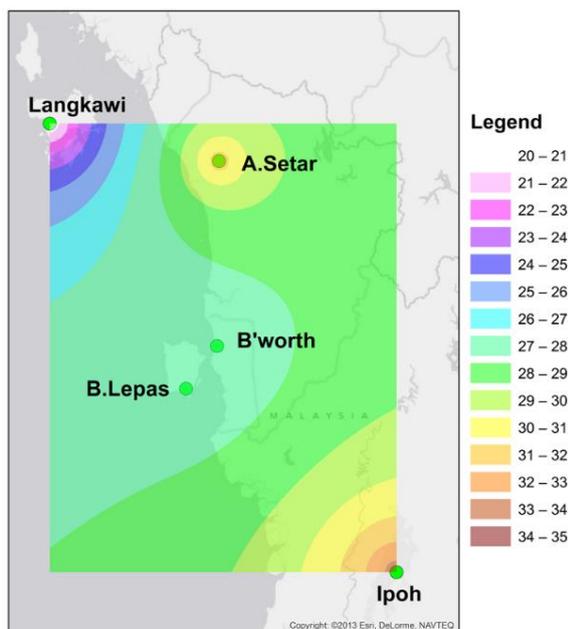
### 3. Wind Design Criteria

Prior to the local wind code, MS 1553 which was introduced in 2002, the recommended basic wind speeds by JMM were site specific based on the official monitoring stations located in most major cities in Malaysia; interpolated wind maps such as in Figure 2 were not available then. As such, the design engineer will need to interpolate the basic wind speed at any locations at minimum without consideration of other factors such as surface roughness and other orographic effects arising from the terrain and its build environment. These governing factors are considered in CP3 by modifying the basic wind speed through factors which will amplify the basic wind speed further in deriving the designed wind speed pressure in the prescribed wind field. Figure 4 shows the basic wind speed contours interpolated based on site-specific historical data provided by JMM up to the year 1995. This involved using five (5) monitored wind speed of stations at Langkawi, Alor Setar, Butterworth, Bayan Lepas and Ipoh and performing and interpolation using the inverse distance weighting technique.

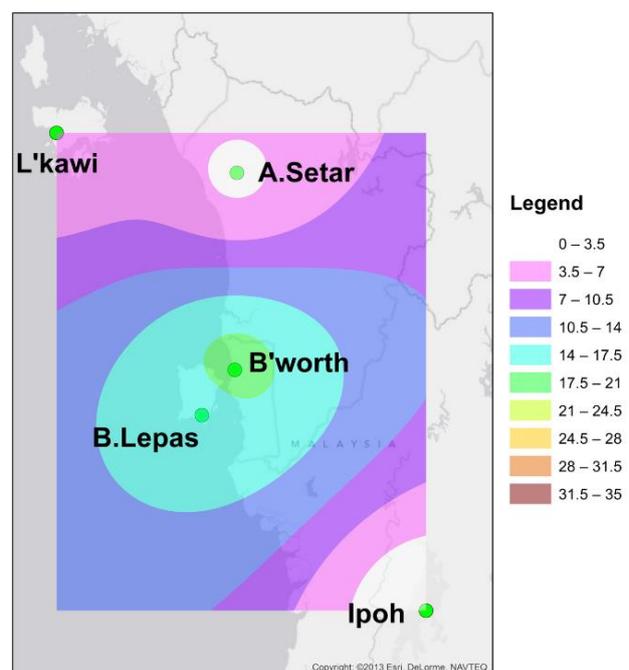
In determining the appropriate basic wind speed for design in 1995, a statistical method based on the extreme value analysis (EVA) is typically deployed. This data was obtained from the JMM which was based on a 3-sec gust averaging time period for wind records between 1966 – 1994, as shown in Figure 3. EVA fundamentally assesses, from a given ordered sample of a given random variable, the probability of events that are more extreme than any previously observed. This reliability based statistical method is widely use in civil engineering applications [1]. In this case, a 50 year recurrence interval was used corresponding with the return period sanctioned by CP3.

Stations	50 Year Return Period ( $\text{ms}^{-1}$ )	100 Year Return Period ( $\text{ms}^{-1}$ )	Period
Alor Setar	31.9	34.2	1966 – 1994
Bayan Lepas	27.4	28.8	1966 – 1994
Butterworth	27.0	28.4	1985 – 1994
Langkawi	19.8	19.9	1987 – 1994
Ipoh	33.8	36.0	1966 – 1994

**Figure 3:** Basic Wind Speed for wind record between 1966 – 1994



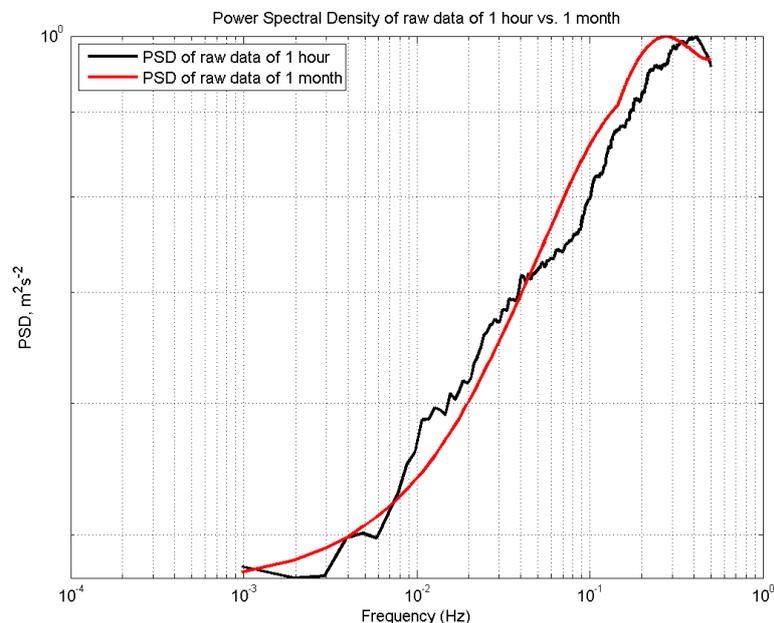
**Figure 4:** Reconstructed Basic Wind Speed Contour Map in 1995 (3-sec gust, 50 years mean recurrence period)



**Figure 5:** Reconstructed Wind Contour Map at the time of incident based on JMM data (3-sec gust)

From Figure 4, the basic wind speed in design consideration for the structure at location  $5.416110^{\circ}\text{N}$ ,  $100.325321^{\circ}\text{E}$  was determined to be  $27.37\text{m/s}$  based on the 50 years of mean recurrence interval. Figure 5 shows the reconstructed basic wind speed contours based on 3 second gust during the time of collapse. This data was similarly obtained from JMM data records. This gave rise to a maximum 3-second gust which occurred at the structure to be  $17.93\text{ m/s}$ . Subsequently, the determination of the design wind pressure was based on the governing CP3 code. This forms the criteria for quasi-static wind loading.

In order to undertake the dynamic analysis of the structure, a singular wind load value derived from CP3 would not be suitable; thus a time history of wind loads need to be applied. This is achieved through the frequency domain analysis or Fourier transform of wind speed records from JMM stations which were recorded at a sampling rate of 1Hz. A limitation of such a methodology would result in an effective representation of wind spectral energy densities of only up to 0.5Hz due to the Nyquist frequency rule. As a general rule of thumb, energy in wind gusts tends to taper off at approximately above 1Hz [1, 6, 7]. Studies have also shown that change in intensity of wind speeds generally do not result in a change in peak frequencies of wind energies [6]. In addition to the above, additional time histories were reconstructed from the inverse Fourier transform of wind spectral energy density records from several nearby offshore locations; while these also do not represent the entire energy spectrum occurring over the particular location, it would be a best case approximation considering the relative proximity of the structure to the shore line; thus topographical factors may not be a large influencing factor in postulating offshore winds on nearshore conditions.

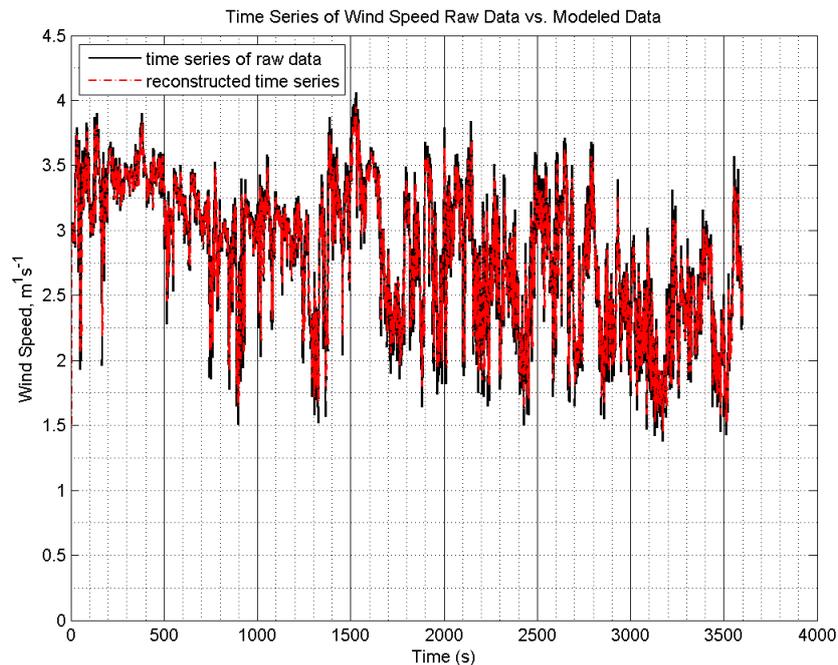


**Figure 6:** Averaged wind power spectral density comparison between 1-month and 1-hour long time histories

In order to achieve statistical reliability and consistency of results, one (1) month of wind records at a sampling rate of 1Hz were analysed. With respect to the frequency content, the statistically parsimonious time history record length required to obtain reliable wind frequencies is one (1) hour' worth of time histories. This is illustrated in

Figure 6 in the form of wind power spectral density plot. The spectral energy or zeroth moments for both time histories, hourly and monthly time series indicate similarity in the frequency components. Wind speeds are seen to possess maximum frequency content in the range of 0.25Hz – 0.4Hz. The reconstructed time history is shown in

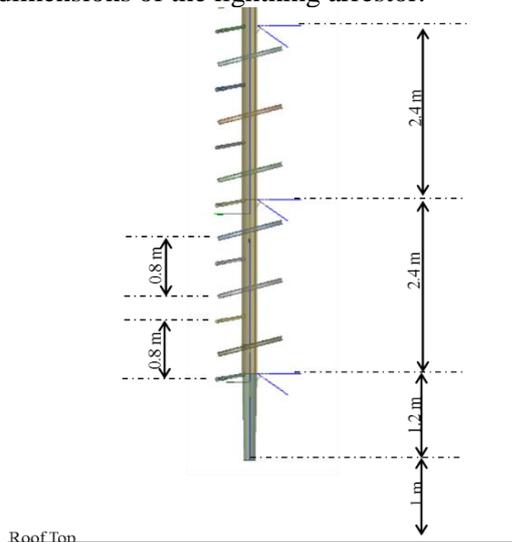
Figure 7 and it can be seen that the reconstructed time history mimicked the actual time histories with a high degree of accuracy.



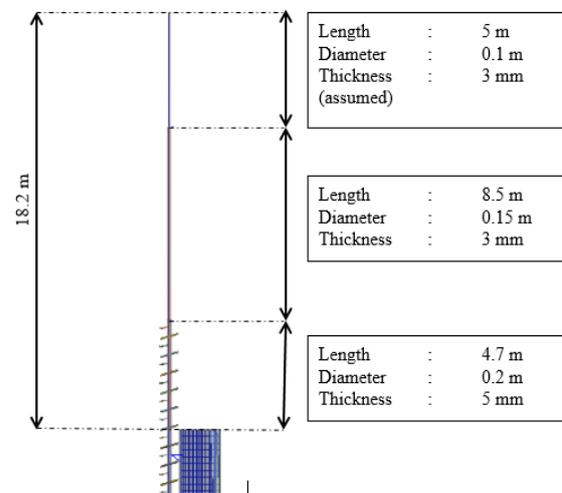
**Figure 7:** Time history of 1-hour time history versus time history reconstructed time series

#### 4. Structural Modelling of Collapsed Structure

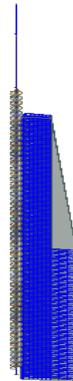
The structure consists of a building in which an irregularly shaped fin wall is constructed at the roof top section. Attached to the fin wall is also an adjoining lightning arrestor structure. The modelled structure representing the final assembly of reinforced concrete fin wall, coupled with the lightning arrestor is shown in Figure 10. The nodes and the base of the concrete and steel reinforcement are constrained in all degrees of freedom, to capture the resulting shear forces and overturning moment at the base due to interaction of the structure with the wind pressure. The self-weight of the reinforced concrete fin wall and the lightning arrestor analysed by ANSYS are found to be 215 tonnes and 4 tonnes, respectively. **Error! Reference source not found.** and Figure 9 respectively presents the dimensions of the lightning arrestor.



**Figure 8:** Bottom section profile of lightning arrestor



**Figure 9:** Profile of lightning arrestor extended beyond top of fin wall



**Figure 10:** Final Finite Element Model of the reinforced concrete fin wall and lightning arrester

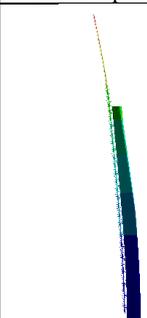
## 5. Results and Discussion

Finite element analysis (FEA) was performed using ANSYS, for both static and dynamic analyses. All three (3) wind load cases as mentioned in the previous sections are analysed, both statically and dynamically (based on wind time histories). In ascertaining the dynamic amplification factors (DAF) related to the dynamic wind pressures, the ratio of structural dynamic responses against static responses were calculated. Table 2 summarizes these dynamic amplification factors and the anticipated bending moments under static and dynamic loads. The bending moments will be used to check the integrity of the reinforced concrete fin wall and the attached lightning arrester as this was the established mode of failure, rather than buckling nor torsional failure. In all cases, closed form solutions from the FEA were obtained to ensure convergence and adequacy of the model. From Table 2, it was found that the DAF generally exceeded unity (static benchmark) by a large margin. The largest DAF occurred at the time of the incident. The DAF for the three (3) cases investigated are equivalent to 1.8 times of the static cases.

**Table 2:** Summary results of static and dynamic analysis using FEA

	Imposed bending moment, kN.m		
	Static Analysis	Dynamic Analysis	Dynamic Amplification Factor, DAF
Time of incident	1817.5	3293.3	1.8
Design consultant	2868	5214.0	1.8
JMM data up to 1995	4246	7744.0	1.8

**Table 3:** Mode shapes and vibration frequencies of reinforced concrete fin wall coupled with lightning arrester

Mode #	Mode Shapes	Frequency, Hz
1		0.270

2		0.678
3		0.684

The quasi-static bending moments were calculated from the support reactions obtained through FEA. It was found that the bending moment arising from the wind speed used by the design consultant (2868 kN.m) is higher than the bending moment arising from the wind speed at the time of the incident (1818 kN.m). The highest bending moment was found to be generated from the 50 year return period with a basic wind speed of 27.37 m/s (4246kN.m). This however only represents the bending moments arising from the quasi-static wind loads determined from CP3 code. The tip of the reinforced concrete fin wall experienced a displacement of 0.23 meters, 0.37 meters and 0.55 meters for basic wind speed of 17.93 m/s, 22.5 m/s and 27.37 m/s wind speed, respectively. The maximum bending moment resulting from the dynamic analysis is 3923.3kN.m, 5214.0kN.m and 7744.0kN.m for the dynamic wind loads arising in-situ, as designed by the consultant and recommended by JMM respectively. The tip of the reinforced concrete fin wall experienced a displacement of 0.45 meters, 0.63 meters and 1.10 meters, respectively under dynamic wind loads. Model analysis was performed on the reinforced concrete coupled fin-wall and lightning arrestor to capture the dynamic response of the assembly and the first three mode shapes with the corresponding frequencies of the structure is presented Table 3. Table 3 exhibits that the fundamental bending mode of the structure has an eigenvalue of approximately 0.27Hz which is within the peak of the wind energy spectrum of approximately 0.25 – 0.30Hz. Along with relatively low assumed damping of concrete structures at approximately 5.0%, the contribution of damping at near resonance conditions are relatively limited, leading to large deformations of the structure. Subsequently, the nonlinear pushover analysis was performed to determine the bending moment capacity of the fin wall based on FEA. This was intended to provide a better approximation of the actual structural capacity. In the pushover analysis, the applied load was incrementally added over a total of 10 steps until the reinforced concrete section fails. The material properties assigned is presented in Table 5.

Table 6 and Figure 11 and Figure 12 shows the results from the pushover analysis and the maximum bending moment capacity of the reinforced concrete fin wall section was determined to be 3453 kN.m.

**Table 4:** Comparison between support reactions from static analysis and dynamic analysis with respective dynamic amplification factor

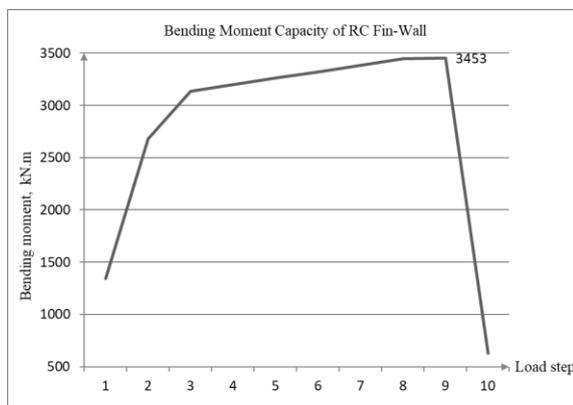
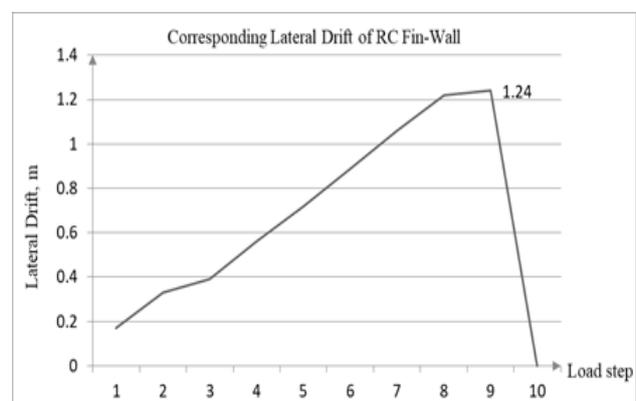
<b>Bending moment, kN.m</b>			
	Static Analysis	Dynamic Analysis	Dynamic Amplification Factor, DAF
Time of incident	1818	3293	1.8
Design consultant	2868	5214	1.8
JMM data up to 1995	4246	7744	1.8

**Table 5.**Material properties for concrete and steel reinforcements

<b>Concrete</b>		<b>Steel Reinforcements</b>	
Compression strength, $f_{cu}$	30 MPa	Yield strength, $f_y$	410 MPa
Density, $\rho$	2400 kg/m <sup>3</sup>	Density, $\rho$	7850 kg/m <sup>3</sup>
Modulus of elasticity, E	30 GPa	Modulus of elasticity, E	200 GPa

**Table 6.** Results from pushover analysis.

<b>Step Number</b>	<b>Bending moment at base, kN.m</b>
1	1341
2	2683
3	3139
4	3200
5	3261
6	3323
7	3384
8	3446
<b>9</b>	<b>3453</b>
10	627

**Figure 11:** Maximum bending moment from pushover analysis**Figure 12:** Max. displacement of idealised fin-wall section (failure, established from pushover analysis)

## 6. Conclusions

- a. The maximum basic wind speed of 17.93 m/s was ascertained at the time of the incident or failure in a storm event
- b. Under quasi-static wind load design as per CP3, the reinforced concrete fin wall will remain intact due to a loading of 1818kN.m
- c. Under dynamic wind loading, a total bending moment of approximately 3293kN.m occurs due to dynamic wind loads. The performance of the structure under this condition is considered marginal when compared against the structural capacity of the structure as determined from the nonlinear pushover analysis
- d. Under dynamic effect from wind loads, the reinforced concrete fin wall will have experienced maximum bending stress and failed if the inelastic behaviour of the reinforced concrete fin wall is considered marginal; other factors such as construction quality and age-related degradation of the structure may have contributed to the failure but is unfortunately unquantifiable from data collected and cannot be considered in this paper
- e. It is highly probable that resonance took place prior to the collapse since the frequency of the natural vibration of the reinforced concrete fin wall matches that of the frequency of the wind speed;

It can also be concluded from the investigation that the quasi-static design of the structure based on the CP3 code had been inadequate in accounting for the dynamic interaction of the wind against the structure, leading to resonant condition. There has been much debate on the adequacy of the CP3 code being used as it is or if the structure had required to undergo further advanced analysis such as a wind tunnel test to preclude any other catastrophic wind loading conditions; that is however not the scope of discussion of this paper. It is noted that in later developments of wind codes beyond the CP3, such as the American ASCE7-98 and Malaysian MS1553, more quantitative definition of the criteria of structures that required further advanced analysis such as dynamic analysis or wind tunnel tests had been defined.

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