THE RESPONSE OF A TALL BUILDING TO WIND LOADING

A thesis submitted for the degree of Doctor of Philosophy in the Faculty of Engineering of the University of London

by

John David Littler

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To Lizzy, Susan and Katherine.
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ABSTRACT

Although previous attempts have been made to make full-scale measurements of the response of tall buildings to wind loading, none of them have obtained data of sufficient accuracy to enable the validity of prediction methods to be assessed. This thesis gives details of the major full-scale tests that have been carried out and the reasons why the data obtained was not of high accuracy.

The dynamic characteristics of Hume Point, the building selected for the long-term testing, were found by forced vibration testing at a number of amplitude levels. The results from this testing were used to calculate suitable recording parameters for the acquisition and analysis of data. Hume Point is 66.9m tall and over 300m away from other buildings of comparable height.

Anemometers mounted on a 20m mast on the roof of the building and up to fourteen accelerometers were used to measure the wind impinging on the building and the building's dynamic response respectively. Two tracking lasers were used to observe the motion of a plumb line suspended in the building and so determine the building's quasi-static response. Thermistors were used to measure the temperature of all four faces of the building.

Over 2,250 hours of data were obtained in almost 8,000 records each 1024 seconds long. Response spectra calculated from records obtained under similar wind conditions were averaged together. Using this selective ensemble averaging technique, the rms acceleration of Hume Point was obtained to an accuracy of ±10% for a range of wind speeds and directions. The response in all three fundamental modes was found to be proportional to the wind speed raised to between the power of 2.7 and the power of 3.3. Allowing for possible variance error, the response of Hume Point is symmetrical about both the west and north faces except for winds blowing directly onto the east or west faces. The response in all three fundamental modes is greatest for winds blowing onto the west face.

Better correlation was obtained for mean temperature difference between opposite faces and mean quasi-static displacement than for mean wind speed and mean quasi-static displacement. In general the correlation between displacement and wind speed within an individual record was better than that for the mean values of records collected under similar wind conditions.

Both wind tunnel tests and a recent calculation method predicted the along-wind response of Hume Point to within ±50% when the correct natural frequency and other building parameters were used in the predictions. The wind tunnel tests predicted the across-wind response to within a factor of 2, whilst the calculation method overestimated the actually measured across-wind response by between 1.6 and 5 times. However, using the actually measured building parameters can give a false idea of the accuracy of prediction methods. For example, using the suggested estimates of the building parameters rather than the actually measured values increased the overestimate of across-wind response predicted by the calculation method to between 5.5 and 12 times.
## CONTENTS:

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abstract</td>
<td>3</td>
</tr>
<tr>
<td>Acknowledgements</td>
<td>7</td>
</tr>
<tr>
<td>List of Figures</td>
<td>8</td>
</tr>
<tr>
<td>List of Tables</td>
<td>13</td>
</tr>
<tr>
<td>Notation</td>
<td>15</td>
</tr>
<tr>
<td>Abbreviations</td>
<td>18</td>
</tr>
</tbody>
</table>

### CHAPTER 1: PREVIOUS WORK

1.1 Introduction
1.2 Wind on tall structures: the situation prior to 1890
1.3 Review of previous work 1890-1940
1.4 Full scale testing 1940-1990
1.5 Other work since 1940

### CHAPTER 2: PROBLEMS IN OBTAINING DATA FROM FULL-SCALE BUILDINGS

2.1 Spectral analysis
2.2 Full-scale data acquisition
2.3 Variance and bias errors
2.4 Stationarity
2.5 Windowing
2.6 Obtaining damping values from spectra
2.7 Digital spectral analysis on data from a tall building
2.8 Selective Ensemble averaging
2.9 Sheffield University Arts Tower

### CHAPTER 3: ASSESSMENT OF FULL-SCALE TESTS CARRIED OUT TO DATE

3.1 Introduction
3.2 Re-assessment of full scale tests
3.3 What has come out of full scale testing to date?
3.4 Requirements for the ideal full-scale building experiment

### CHAPTER 4: DESCRIPTION OF HUME POINT AND FORCED VIBRATION TESTING

4.1 Introduction
4.2 Description of Hume Point
4.3 The terrain surrounding Hume Point
4.4 Ambient vibration tests
4.5 The rationale behind forced vibration testing
4.6 Eccentric mass vibrator system
4.7 General forced vibration test procedure
4.8 Forced vibration tests on Hume Point
4.9 Discussion of results of vibration tests
4.10 Accuracy of results
4.11 Other methods of testing
4.12 How does Hume Point compare with the "ideal building"?
<table>
<thead>
<tr>
<th>CONTENTS: (continued)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CHAPTER 5: INSTRUMENTATION, DATA ACQUISITION AND INITIAL ANALYSIS</td>
</tr>
<tr>
<td>5.1 Introduction 151</td>
</tr>
<tr>
<td>5.2 Instrumentation 151</td>
</tr>
<tr>
<td>5.3 Determination of data acquisition and analysis parameters 178</td>
</tr>
<tr>
<td>5.4 Acquisition of data 178</td>
</tr>
<tr>
<td>5.5 Initial analysis of data 180</td>
</tr>
<tr>
<td>5.6 Wind profile experiment 188</td>
</tr>
<tr>
<td>5.7 Chronology of experiment 193</td>
</tr>
<tr>
<td>CHAPTER 6: THE DYNAMIC RESPONSE OF HUME POINT TO WIND LOADING</td>
</tr>
<tr>
<td>6.1 Distribution of wind records 199</td>
</tr>
<tr>
<td>6.2 Maximum accelerations 201</td>
</tr>
<tr>
<td>6.3 Self-stationarity of individual records 202</td>
</tr>
<tr>
<td>6.4 Selective ensemble averaging procedure 212</td>
</tr>
<tr>
<td>6.5 Stationarity of the ensemble 218</td>
</tr>
<tr>
<td>6.6 Results of selective ensemble averaging 224</td>
</tr>
<tr>
<td>6.7 Assessment of peak accelerations 257</td>
</tr>
<tr>
<td>6.8 Comparison with other full-scale test results 264</td>
</tr>
<tr>
<td>6.9 Comparison of predicted and full-scale results 266</td>
</tr>
<tr>
<td>6.10 Comparison of wind tunnel and full-scale results 278</td>
</tr>
<tr>
<td>6.11 Comparison of predicted, model and full-scale response 286</td>
</tr>
<tr>
<td>6.12 Summary of Chapter Six 288</td>
</tr>
<tr>
<td>CHAPTER 7: THE QUASI-STATIC RESPONSE OF HUME POINT TO WIND LOADING</td>
</tr>
<tr>
<td>7.1 Introduction 291</td>
</tr>
<tr>
<td>7.2 Analysis of mean values for each record 291</td>
</tr>
<tr>
<td>7.3 Analysis of individual records 321</td>
</tr>
<tr>
<td>7.4 Correlation between displacement and wind speed 335</td>
</tr>
<tr>
<td>7.5 Comparison with other full-scale test results 336</td>
</tr>
<tr>
<td>7.6 Summary of Chapter Seven 338</td>
</tr>
<tr>
<td>CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK</td>
</tr>
<tr>
<td>8.1 Experimental and analytical techniques 341</td>
</tr>
<tr>
<td>8.2 Full-scale results from Hume Point 342</td>
</tr>
<tr>
<td>8.3 The Predicted response of Hume Point 345</td>
</tr>
<tr>
<td>8.4 Recommendations for future work 346</td>
</tr>
<tr>
<td>REFERENCES 349</td>
</tr>
<tr>
<td>APPENDIX A: CALCULATING THE OVERALL RMS VALUE OF AN ENSEMBLE 359</td>
</tr>
</tbody>
</table>
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# LIST OF FIGURES

<table>
<thead>
<tr>
<th>FIGURE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 Determination of damping in mode r by the half power bandwidth method</td>
<td>52</td>
</tr>
<tr>
<td>2.2 Illustration of bias error</td>
<td>54</td>
</tr>
<tr>
<td>4.1 Photograph: North and West faces of Hume Point</td>
<td>93</td>
</tr>
<tr>
<td>4.2 Hume Point elevations</td>
<td>95</td>
</tr>
<tr>
<td>4.3 Photograph: South and East faces of Hume Point</td>
<td>97</td>
</tr>
<tr>
<td>4.4 Typical floor layout of Hume Point</td>
<td>96</td>
</tr>
<tr>
<td>4.5 Map of area surrounding Hume Point</td>
<td>100</td>
</tr>
<tr>
<td>4.6 Photograph: Freemasons Estate from the north-east</td>
<td>101</td>
</tr>
<tr>
<td>4.7 Photograph: Freemasons Estate from the south-east</td>
<td>101</td>
</tr>
<tr>
<td>4.8 Photograph: View from Hume Point looking North</td>
<td>103</td>
</tr>
<tr>
<td>4.9 Photograph: View from Hume Point looking North-East</td>
<td>103</td>
</tr>
<tr>
<td>4.10 Photograph: View from Hume Point looking East</td>
<td>105</td>
</tr>
<tr>
<td>4.11 Photograph: View from Hume Point looking South-East</td>
<td>105</td>
</tr>
<tr>
<td>4.12 Photograph: View from Hume Point looking South</td>
<td>107</td>
</tr>
<tr>
<td>4.13 Photograph: View from Hume Point looking South-West</td>
<td>107</td>
</tr>
<tr>
<td>4.14 Photograph: View from Hume Point looking West</td>
<td>109</td>
</tr>
<tr>
<td>4.15 Photograph: View from Hume Point looking North-West</td>
<td>109</td>
</tr>
<tr>
<td>4.16 Photograph: Hume Point from the West showing low rise housing</td>
<td>111</td>
</tr>
<tr>
<td>4.17 Power spectra from ambient vibration test 12/13 February 1986</td>
<td>114</td>
</tr>
<tr>
<td>4.18 Photograph: One of the vibrators and its control equipment</td>
<td>119</td>
</tr>
<tr>
<td>4.19 Photograph: Equipment in the control room at Hume Point</td>
<td>119</td>
</tr>
<tr>
<td>4.20 Hume Point East-West response</td>
<td>127</td>
</tr>
<tr>
<td>4.21 Hume Point North-South response</td>
<td>128</td>
</tr>
<tr>
<td>4.22 Hume Point Torsional response</td>
<td>129</td>
</tr>
<tr>
<td>4.23 Mode shapes for fundamental modes</td>
<td>130</td>
</tr>
<tr>
<td>4.24 Plan mode shapes for fundamental modes</td>
<td>131</td>
</tr>
<tr>
<td>FIGURE</td>
<td>FIGURE DESCRIPTION</td>
</tr>
<tr>
<td>--------</td>
<td>-------------------</td>
</tr>
<tr>
<td>4.25</td>
<td>Mode shapes for second order modes</td>
</tr>
<tr>
<td>4.26</td>
<td>Plan mode shapes for second order modes</td>
</tr>
<tr>
<td>4.27</td>
<td>Mode shapes for third order modes</td>
</tr>
<tr>
<td>4.28</td>
<td>Plan mode shapes for third order modes</td>
</tr>
<tr>
<td>4.29</td>
<td>Curve fitting around the NS1 mode</td>
</tr>
<tr>
<td>4.30</td>
<td>Decay of oscillation for NS1 mode</td>
</tr>
<tr>
<td>5.1</td>
<td>Photograph: 3-component Gill anemometer set</td>
</tr>
<tr>
<td>5.2</td>
<td>Photograph: Calibrating the anemometers in the wind tunnel</td>
</tr>
<tr>
<td>5.3</td>
<td>Velocity profiles measured at model scale</td>
</tr>
<tr>
<td>5.4</td>
<td>Photograph: The mast being hauled over the parapet</td>
</tr>
<tr>
<td>5.5</td>
<td>Photograph: The mast being bolted to the plant room</td>
</tr>
<tr>
<td>5.6</td>
<td>Plan view and elevation of roof</td>
</tr>
<tr>
<td>5.7</td>
<td>Elevation of Hume Point showing position of anemometers and mast</td>
</tr>
<tr>
<td>5.8</td>
<td>Photograph: The anemometers on the mast</td>
</tr>
<tr>
<td>5.9</td>
<td>Photograph: Two accelerometers mounted on blocks</td>
</tr>
<tr>
<td>5.10</td>
<td>Position of 5 accelerometers used with 8 channel data acquisition system</td>
</tr>
<tr>
<td>5.11</td>
<td>Position of 14 accelerometers used with 32 channel data acquisition system</td>
</tr>
<tr>
<td>5.12</td>
<td>Accelerometer signal conditioning system</td>
</tr>
<tr>
<td>5.13</td>
<td>Position of plumb line and thermistors</td>
</tr>
<tr>
<td>5.14</td>
<td>Photograph: The author with the tracking lasers</td>
</tr>
<tr>
<td>5.15</td>
<td>Photograph: Double exposure of the laser beams</td>
</tr>
<tr>
<td>5.16</td>
<td>Photograph: The author holding one of the optical discs</td>
</tr>
<tr>
<td>5.17</td>
<td>Photograph: The control room at Hume Point</td>
</tr>
<tr>
<td>5.18</td>
<td>Time histories of wind speed for record HZ2203.939</td>
</tr>
<tr>
<td>5.19</td>
<td>Time history of acceleration for record HZ2203.939</td>
</tr>
<tr>
<td>FIGURE</td>
<td>DESCRIPTION</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>5.20</td>
<td>Power spectra of acceleration for record HZ2203.939</td>
</tr>
<tr>
<td>5.21</td>
<td>Time history of North-South displacement for record HZ2203.939</td>
</tr>
<tr>
<td>5.22</td>
<td>Velocity profiles</td>
</tr>
<tr>
<td>5.23</td>
<td>Inwind turbulence intensity profiles</td>
</tr>
<tr>
<td>5.24</td>
<td>Crosswind turbulence intensity profiles</td>
</tr>
<tr>
<td>6.1</td>
<td>Run test using 100 samples</td>
</tr>
<tr>
<td>6.2</td>
<td>Run test using 25 samples</td>
</tr>
<tr>
<td>6.3</td>
<td>Acceleration time history before linear correction applied</td>
</tr>
<tr>
<td>6.4</td>
<td>Acceleration time history after linear correction applied</td>
</tr>
<tr>
<td>6.5</td>
<td>Acceleration time history for record HZ0501.962, channel 5</td>
</tr>
<tr>
<td>6.6</td>
<td>Typical ensemble averaged power spectra</td>
</tr>
<tr>
<td>6.7</td>
<td>Curve fitting around the EW1 mode</td>
</tr>
<tr>
<td>6.8</td>
<td>Curve fitting around the θ1 mode</td>
</tr>
<tr>
<td>6.9</td>
<td>Rms acceleration against wind speed (165° to 195°)</td>
</tr>
<tr>
<td>6.10</td>
<td>Rms acceleration against wind speed (165° to 195°) showing response in θ1 mode measured in three positions</td>
</tr>
<tr>
<td>6.11</td>
<td>Rms acceleration against wind speed (165° to 195°) with linear regression using all points</td>
</tr>
<tr>
<td>6.12</td>
<td>Rms acceleration against wind speed (165° to 195°) with linear regression using points &gt; 0.5 x10^-4 m/s^2</td>
</tr>
<tr>
<td>6.13</td>
<td>Rms acceleration against wind speed (195° to 225°)</td>
</tr>
<tr>
<td>6.14</td>
<td>Rms acceleration against wind speed (195° to 225°) with linear regression using points &gt; 0.5 x10^-4 m/s^2</td>
</tr>
<tr>
<td>6.15</td>
<td>Rms acceleration against wind speed (225° to 255°)</td>
</tr>
<tr>
<td>6.16</td>
<td>Rms acceleration against wind speed (225° to 255°) with linear regression using points &gt; 0.5 x10^-4 m/s^2</td>
</tr>
<tr>
<td>6.17</td>
<td>Rms acceleration against wind speed (255° to 285°)</td>
</tr>
<tr>
<td>6.18</td>
<td>Rms acceleration against wind speed (255° to 285°) with linear regression using points &gt; 0.5 x10^-4 m/s^2</td>
</tr>
</tbody>
</table>
LIST OF FIGURES (continued)

<table>
<thead>
<tr>
<th>FIGURE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.19</td>
<td>Rms acceleration in EW1 mode against wind speed</td>
</tr>
<tr>
<td>6.20</td>
<td>Rms acceleration in NS1 mode against wind speed</td>
</tr>
<tr>
<td>6.21</td>
<td>Rms acceleration in θ1 mode against wind speed</td>
</tr>
<tr>
<td>6.22</td>
<td>Rms acceleration against wind direction for all directions</td>
</tr>
<tr>
<td>6.23</td>
<td>Rms acceleration against wind direction (175° to 285°)</td>
</tr>
<tr>
<td>6.24</td>
<td>Plan mode shapes for fundamental mode shapes obtained from selective ensemble averaging</td>
</tr>
<tr>
<td>6.25</td>
<td>Rms acceleration in 2nd order modes against wind speed</td>
</tr>
<tr>
<td>6.26</td>
<td>One minute extract from filtered time history of acceleration, record HZ2604.901, channel 4</td>
</tr>
<tr>
<td>6.27</td>
<td>One minute extract from filtered time history of acceleration, record HZ1903.959, channel 4</td>
</tr>
<tr>
<td>6.28</td>
<td>Comparison of actually measured and predicted along-wind rms acceleration against wind speed for Southerly records</td>
</tr>
<tr>
<td>6.29</td>
<td>Comparison of actually measured and predicted across-wind rms acceleration against wind speed for Southerly records</td>
</tr>
<tr>
<td>6.30</td>
<td>Comparison of actually measured and predicted along-wind rms acceleration against wind speed for Westerly records</td>
</tr>
<tr>
<td>6.31</td>
<td>Comparison of actually measured and predicted across-wind rms acceleration against wind speed for Westerly records</td>
</tr>
<tr>
<td>6.32</td>
<td>Profiles for terrain simulation type (i)</td>
</tr>
<tr>
<td>6.33</td>
<td>Profiles for terrain simulation type (ii)</td>
</tr>
<tr>
<td>6.34</td>
<td>North-South displacements measured in wind tunnel tests</td>
</tr>
<tr>
<td>6.35</td>
<td>East-West displacements measured in wind tunnel tests</td>
</tr>
<tr>
<td>6.36</td>
<td>Torsional displacements measured in wind tunnel tests</td>
</tr>
<tr>
<td>7.1</td>
<td>Temperature against time for 46 hour period</td>
</tr>
<tr>
<td>7.2</td>
<td>NS displacement against NS temperature difference (all records)</td>
</tr>
<tr>
<td>7.3</td>
<td>EW displacement against EW temperature difference (all records)</td>
</tr>
<tr>
<td>FIGURE</td>
<td>DESCRIPTION</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>7.4</td>
<td>EW displacement against EW temperature difference for 29 consecutive records 8/9 March 1989</td>
</tr>
<tr>
<td>7.5</td>
<td>NS displacement against wind speed for Northerly records</td>
</tr>
<tr>
<td>7.6</td>
<td>NS displacement against NS temperature difference for Northerly records</td>
</tr>
<tr>
<td>7.7</td>
<td>NS displacement against wind speed for Southerly records</td>
</tr>
<tr>
<td>7.8</td>
<td>NS displacement against NS temperature difference for Southerly records</td>
</tr>
<tr>
<td>7.9</td>
<td>NS displacement against NS temperature difference for 37 consecutive records 21/22 February 1989</td>
</tr>
<tr>
<td>7.10</td>
<td>NS displacement against EW temperature difference for Southerly winds</td>
</tr>
<tr>
<td>7.11</td>
<td>EW displacement against wind speed for Westerly winds</td>
</tr>
<tr>
<td>7.12</td>
<td>EW displacement against EW temperature difference for Westerly records</td>
</tr>
<tr>
<td>7.13</td>
<td>NS displacement against NS temperature difference for 151 consecutive records 14/16 February 1989</td>
</tr>
<tr>
<td>7.14</td>
<td>NS and EW displacements against wind speed for 59 consecutive records 18/19 February 1989</td>
</tr>
<tr>
<td>7.15</td>
<td>Unfiltered time histories of wind speed and EW displacement</td>
</tr>
<tr>
<td>7.16</td>
<td>One minute extract of time history showing frequency composition of EW displacement</td>
</tr>
<tr>
<td>7.17</td>
<td>Extract of time history for wind speed and EW displacement</td>
</tr>
<tr>
<td>7.18</td>
<td>Filtered time histories of wind speed and EW displacement</td>
</tr>
<tr>
<td>7.19</td>
<td>Filtered EW displacement against filtered wind speed for record HZ2203.939</td>
</tr>
<tr>
<td>7.20</td>
<td>Filtered time histories of wind speed squared and EW displacement</td>
</tr>
<tr>
<td>7.21</td>
<td>Filtered NS displacement against filtered EW displacement for record HZ2203.939</td>
</tr>
</tbody>
</table>
# LIST OF TABLES

<table>
<thead>
<tr>
<th>TABLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>Predicted and measured values of Hume Point natural frequencies</td>
</tr>
<tr>
<td>4.2</td>
<td>Results of forced vibration tests - fundamental modes</td>
</tr>
<tr>
<td>4.3</td>
<td>Results of forced vibration tests - higher order modes</td>
</tr>
<tr>
<td>4.4</td>
<td>Soil-structure interaction measurements at Hume Point</td>
</tr>
<tr>
<td>6.1</td>
<td>Distribution of all records by wind speed and direction</td>
</tr>
<tr>
<td>6.2</td>
<td>Distribution of records by wind speed and direction (32 channel data only)</td>
</tr>
<tr>
<td>6.3</td>
<td>Testing of individual record for self-stationarity</td>
</tr>
<tr>
<td>6.4</td>
<td>Testing of 35 records for self-stationarity</td>
</tr>
<tr>
<td>6.5</td>
<td>Frequency limits used for determining the response in the fundamental modes</td>
</tr>
<tr>
<td>6.6</td>
<td>Comparison of four ensembles, each of 50 records, with ensemble of all 200 records</td>
</tr>
<tr>
<td>6.7</td>
<td>Results of selective ensemble averaging for winds 165° to 195°</td>
</tr>
<tr>
<td>6.8</td>
<td>Results of selective ensemble averaging for winds 195° to 225°</td>
</tr>
<tr>
<td>6.9</td>
<td>Results of selective ensemble averaging for winds 225° to 255°</td>
</tr>
<tr>
<td>6.10</td>
<td>Results of selective ensemble averaging for winds 255° to 285°</td>
</tr>
<tr>
<td>6.11</td>
<td>Exponents for wind velocity</td>
</tr>
<tr>
<td>6.12</td>
<td>Results of selective ensemble averaging for wind speed 9.01 to 10.00 m/s, from all wind directions</td>
</tr>
<tr>
<td>6.13</td>
<td>Results of selective ensemble averaging for wind speed 7.51 to 9.00 m/s, from East and West winds</td>
</tr>
<tr>
<td>6.14</td>
<td>Results of selective ensemble averaging for wind speed 9.01 to 10.00 m/s, wind direction 175° to 285°</td>
</tr>
<tr>
<td>6.15</td>
<td>Results of selective ensemble averaging for all channels - wind speed 9.01 to 10.00 m/s, wind direction 195° to 225°</td>
</tr>
<tr>
<td>6.16</td>
<td>Comparison of average rms accelerations obtained by both time and frequency domain analysis</td>
</tr>
<tr>
<td>6.17</td>
<td>Peak and rms acceleration values for 50 records, channel 4</td>
</tr>
<tr>
<td>6.18</td>
<td>Peak and rms acceleration values for 50 records, channel 5</td>
</tr>
</tbody>
</table>
LIST OF TABLES (continued)

<table>
<thead>
<tr>
<th>TABLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.19</td>
<td>269</td>
</tr>
<tr>
<td>ESDU calculated tip resonant accelerations for a southerly wind of 13.31 m/s</td>
<td></td>
</tr>
<tr>
<td>6.20</td>
<td>274</td>
</tr>
<tr>
<td>ESDU calculated tip resonant accelerations for a westerly wind of 12.24 m/s</td>
<td></td>
</tr>
<tr>
<td>6.21</td>
<td>286</td>
</tr>
<tr>
<td>Comparison of full-scale and model-scale rms accelerations</td>
<td></td>
</tr>
<tr>
<td>7.1</td>
<td>298</td>
</tr>
<tr>
<td>Mean displacements against temperature difference (all records)</td>
<td></td>
</tr>
<tr>
<td>7.2</td>
<td>304</td>
</tr>
<tr>
<td>Mean displacements for Northerly winds</td>
<td></td>
</tr>
<tr>
<td>7.3</td>
<td>308</td>
</tr>
<tr>
<td>Mean displacements for Southerly winds</td>
<td></td>
</tr>
<tr>
<td>7.4</td>
<td>315</td>
</tr>
<tr>
<td>Mean displacements for Westerly winds</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>330</td>
</tr>
<tr>
<td>Linear regression coefficients for wind speed vs. displacement</td>
<td></td>
</tr>
<tr>
<td>7.6</td>
<td>330</td>
</tr>
<tr>
<td>Linear regression coefficients for various different parameters for record HZ2203.939</td>
<td></td>
</tr>
<tr>
<td>7.7</td>
<td>333</td>
</tr>
<tr>
<td>Linear regression coefficients for wind speed squared against displacement</td>
<td></td>
</tr>
</tbody>
</table>
NOTATION

A  Maximum response in a mode
B  (Minimum frequency) resolution bandwidth (1/T) in Hertz
C  Value of the ordinate when the abscissa is zero
E  The base of natural logarithms
E[ ]  Ensemble average
EW1  East-west fundamental mode
Fr  Natural frequency in mode r
F_r  Modal Force applied in mode r
G  Acceleration due to gravity
GR  Peak response factor
Gxx(f)  One-sided autospectral density function
H  Height (above ground), height of a pendulum
H  Overall height of a building
Ir  Modal Inertia of mode r
Iu  Inwind turbulence intensity
Iv  Crosswind turbulence intensity
J  Number of terms in a series
Kr  Modal Stiffness of mode r
L  Overall length of a building
m  Slope of a line
Mr  Modal mass of mode r
NOTATION (continued)

N  Number of sample records
N1  Mode shape exponent
NS1  North-south fundamental mode
$r^2$  Coefficient of regression
$R_{xx}(\tau)$  Autocorrelation function
$S_{xx}(f)$  Autospectral density function
$t_1$  Starting time
T  Time of a single sample record
u(t)  Rectangular time window
V  Wind velocity
$V$  Mean wind velocity
$v(t)$  Unlimited time history
W  Overall width of a building
$X_k$  Fourier component
$\{x_r\}$  Series of discrete values obtained by sampling
a continuous function $x(t)$
$X_r, \dot{X}_r, \ddot{X}_r$  Displacement, velocity and acceleration response in mode $r$
x(t), y(t)  Time history records
z  Height up a building
NOTATION (continued)

$\alpha$ An exponent

$\beta$ Half power bandwidth, in Hertz

$\Delta$ Time interval between successive samples

$\varepsilon_h$ Normalised bias error

$\varepsilon_r$ Normalised variance or random error

$\zeta_r$ The ratio of actual damping to critical damping in mode $r$

$\theta_1$ Torsional fundamental mode

$\mu$ Mean value

$\pi$ The ratio of the circumference of a circle to its diameter

$\sigma$ Standard deviation

$\tau$ Arbitrary time displacement

$\phi$ Mode shape factor, wind azimuth

$\psi$ Time constant

$T$ Total time available for analysis
ABBREVIATIONS

Acc., Accel. Acceleration
Ang. Acc. Angular acceleration
BLWTL Boundary Layer Wind Tunnel Laboratory (at University of Western Ontario)
BRE Building Research Establishment
BRS Building Research Station (part of BRE)
CEBTP Centre Expérimental de Récherches et d'Etudes du Bâtiment et des Travaux Publics
D.C. Direct current
DEWU (Temperature) Difference between thermistors on the East and West Upper (16th) floor
DFT Discrete Fourier transform
DNSU (Temperature) Difference between thermistors on the North and South Upper (16th) floor
ESDU ESDU International (formerly the Engineering Sciences Data Unit)
EW East-West
FFT Fast Fourier transform
HP High pass (filter)
HPB Half power bandwidth
LP Low pass (filter)
mph miles per hour
NS North-South
P-P peak to peak
rms root mean square
TMD Tuned mass damper
UWO University of Western Ontario
WSPEED Wind speed
CHAPTER ONE:

PREVIOUS WORK

1.1 INTRODUCTION

"It is apparent that the practice of tall-building construction is rapidly outrunning our knowledge of the actual behaviour of these structures in practice. We know that there have been no spectacular collapses of high towers, but we know very little else in the way of definite checks on current design practices. The large crop of new theories for wind design indicates that the profession is conscious of the fact that we have outgrown the old approximate methods; but it is not always apparent that we yet realize the extent of our ignorance in regard to the factors which should be taken into account."

Sixty years have passed since David Cushman Coyle wrote the preceding paragraph as an introduction to a paper in Engineering News Record (ref 1). It could well be argued that little has changed in the interim.

The response of a tall building to wind loading is an essential factor in its design. The design engineer must check that the building can withstand the design forces and should check that the movement of the building in strong winds is acceptable to the occupants. In order to carry out these checks, the designer needs to estimate the characteristics of the building as well as the wind forces acting upon it. The known characteristics of an existing structure can be put into the equations used by the designer to determine the response of the building to wind loading, and the result compared with the observed behaviour of the building to known winds. If all the parameters used in these equations are known to a sufficient degree of accuracy, and the observations are also sufficiently accurate, then any differences between predicted and observed behaviour must be due to the method used to calculate the response.
The main objective of the work presented in this thesis was to carry out a full-scale experiment to assess whether the measured response of a tall building was the same as the response predicted by current design methods. The major difficulty in meeting this objective, is to collect experimental data of sufficient accuracy to assess the design methods. If the discrepancy between the experimental and predicted results is greater than the possible errors in the experimental data, then the design methods are in error. The secondary objective of the work was to determine where the design methods were at fault, if such a discrepancy was found.

The rest of this chapter is a description of what work has been carried out to date in this field. Chapter Two discusses the problems encountered when acquiring and analysing full-scale data from tall buildings. In Chapter Three some of the work detailed in the following pages will be re-assessed in the light of the discussion in Chapter Two. Therefore, at this stage, the previous work in the field is described without further comment. Some of the work is described in some detail as particular aspects of it will be referred to in later chapters.

1.2 WIND ON TALL STRUCTURES: THE SITUATION PRIOR TO 1890

Until the advent of steel being used as a construction material for tall buildings, those few tall structures that there were tended to be so massive that their response to wind loading was not a problem. Whilst the great pyramid of Cheops is over 135m high, as it is constructed with stone blocks, its response to wind loading is negligible. Apart from religious buildings, few high-rise structures were constructed until the 19th century. However, wind action caused some spectacular failures of what few high rise structures there were. When these failures occurred, there was a choice of three courses of action to take. Firstly, rebuild on a less ambitious scale or, in the extreme, don’t rebuild at all. Secondly, rebuild to exactly the same design, and hope that a storm of the same severity would not occur again in the structure’s lifetime. Thirdly, rebuild to an improved design. Even when this third option was chosen, the builder would not know, for example, how much additional bracing to include in a revised design, only that the amount used originally was insufficient.
The following examples of storm damage to three English cathedrals illustrate these different approaches to the problem.

In 1362 the wooden spire which surmounted the tower of Norwich cathedral was blown down in a storm, bringing part of the tower down with it. The tower was repaired, and eventually a stone spire was added in about 1490 (ref 2, p 53). In this case, the improved design was a success as the spire is still in place.

Lincoln cathedral was almost completely rebuilt after an earthquake in 1185. At one time the central tower was surmounted by a steeple made of lead sheathed wood. When this was completed in 1311 it was the tallest structure in the world with an overall height of 160m. It survived for over two hundred years but fell in a great storm in 1548. No attempt was made to rebuild it. Similar, but much shorter, spires also surmounted the two smaller west towers, but these were removed in 1807 (ref 2, p 174). It was not until the Washington Monument was completed in 1884 that a taller structure was built, and even then this stone structure was only slightly higher at 169m (ref 3, p90).

Chichester cathedral spire, built of stone in about 1400, fell in a storm in February 1861 which also blew down part of the Crystal Palace. The fall was expected by the cathedral authorities, and, because of their warning, no one was injured. Anderson and Hicks quote from a contemporary account "the spire was seen to incline slightly to the south-west, then to descend perpendicularly into the church, as one telescope tube slides into another, the mass of the tower crumbling beneath it" (ref 4, p49). The architect for the rebuilding (1861-1866) Gilbert Scott, worked from records of the old spire and tower. He did not try to improve on the original (ref 2, p 220) and was able to reconstruct them so that they "fully preserved the intentions of John Mason, the original architect" (ref 4, p 49).

Although many lessons were learnt from these and other similar failures, little was known about the wind loading which caused them until the end of the nineteenth century. The collapse of the Tay Bridge during a severe storm in 1879, just as a train was crossing it, prompted Baker (ref 5) to start wind loading research in the United Kingdom which could be
applied directly to engineered structures. However, at the time, Baker's research was largely ignored.

1.3 REVIEW OF PREVIOUS WORK 1890 - 1940

1.3.1 EXPERIMENTAL WORK PRIOR TO 1931

A century ago Gustave Eiffel carried out some experiments on his 300m high tower in Paris (ref 6). He made observations of the movement of the tower by using a telescope. The telescope, which was sited on the ground, looked vertically upwards at a target on the top level of the tower. The centre of the target represented "equilibrium conditions under uniform temperature". Eiffel observed the elliptical movement of the tower during wind storms. He also measured the wind speed at the top of the tower and showed that the mean wind speed was some 2 to 3 times greater than that measured at the 20m height used as a reference by the Bureau of Meteorology.

In the same decade that Eiffel was carrying out his experiments in Paris, work was being carried out by Stebbings on two buildings in Chicago (ref 7). The Monadnock building of 17 storeys was completed in 1891 and instrumented with both a plumb bob suspended down the central stairwell, and transits at the north and south ends of the building. The transits, like Eiffel's telescope, would be expected to show the dynamic movement of the building at its fundamental natural frequency (approximately 0.5 Hz). In contrast, the plumb bob would not be able to respond at this frequency, and would show the quasi-static or background movement only. "These two observations agreed very closely and show the vibration east and west to be from \( \frac{1}{2} \) inch to \( \frac{1}{4} \) inch" (12 to 6 mm). According to the plumb bob readings, the north-south vibrations were even greater than this. Also "the north end or old portion of the building, which has solid masonry walls, showed a smaller deflection than the south end or new portion, which is typical steel skeleton construction, with curtain walls carried by the framework." In describing these tests, Fleming wrote (ref 8a) "it should be said that the dead weight of the Monadnock Block would now (1940) be called excessive. This was the last great skyscraper built with structural
masonry walls."

Observations made on the 14 storey Pontiac building "agreed quite closely in their general features with those on the Monadnock block" (ref 7). Again, the longitudinal vibrations were greater than the transverse ones, although the amplitude of the vibrations was somewhat less "owing to its sheltered position." It is stated that "the wind registered a velocity of 80 miles per hour (mph) (36m/s) and was from the north-east." It is unclear from the text whether this refers to both tests, implying that readings on both buildings were conducted at the same time, or just one.

Little progress was made with regard to the collection of full-scale data over the next thirty-five years. Then, in the late 1920's, David Cushman Coyle built a portable horizontal-pendulum seismograph and made many recordings of the motion of New York skyscrapers (refs 9 and 1). While not identifying the individual buildings, Coyle identified them as light or heavy construction. Although not saying that it was one of the buildings tested, Coyle mentioned the 241m tall Woolworth Building, erected in 1912-13, as being at the stiff end of the stiffness range of existing New York towers. The light construction buildings were shown to have clear oscillations at the natural frequency, the amplitude of motion varying with gusts of the wind. In contrast, the heavy construction buildings had much smaller amplitudes of response and less well defined oscillatory motion. The observed frequencies of the various buildings ranged from 8 to 40 cycles per minute (0.13 to 0.67 Hz). Displacements of up to ¼ inch (12mm) were noted.

1.3.2 Empire State Building

The most extensive of the early full-scale experiments was that carried out on the Empire State Building over five years by Rathbun (ref 8). The building, which became the tallest structure in the world when it was completed in 1931, has base plan dimensions of 129.5 by 60 m. At the 6th floor level these reduce to 94.5 by 50 m, and then in four further unequal reductions to 46.3 by 33.2 m by the 86th floor. Although a television mast has been added since, the total height of the building at the time of the testing was 381 m.
Rathbun and his colleagues used a 32 blade rotor anemometer mounted 15 feet (4.6 m) above the stubby mast on top of the building to measure wind speed. Significant discoveries about the composition of the boundary layer were made by comparing the wind speed, as measured by this anemometer on 169 occasions, with simultaneous measurements of wind speed at various other locations in New York. On many of these occasions further instrumentation on the Empire State building was used, so that the readings from these instruments could be related to the wind speed and direction pertaining at the time. Pressure was measured by thirty manometers, ten on each of three floors; and twenty-two extensometers were used to measure the strains in columns on the twenty-fourth floor.

Rathbun also placed two instruments (a plumb bob and a collimator) in the fire tower (stairwell) to observe the movement of the building. The plumb bob consisted of a cable suspended from the ceiling of the 86th floor, with the bob 296m below at the sixth floor. Oil baths were used to damp out the swing of the bob, but upward currents of air caused oscillation of the cable, particularly during high winds. A diaphragm was installed at the 54th floor, with holes for the plumb bob and line of sight of the collimator, in order to minimise this problem of upward air currents. However, even after the installation of the diaphragm, the problem was not eliminated.

The position of the bob was read relative to a grid system with an arbitrary origin, Rathbun stating an error of less than one eighth of an inch for these measurements. Skinner (who had participated in at least some of the experimental work) contributed 12 pages of the discussion which followed Rathbun’s paper (ref 8a). This included some data about the plumb bob readings which were not included in Rathbun’s paper. Skinner included two graphs which showed good correlation when the plumb bob readings were plotted against the square of the wind speed, for winds from the same direction. However, these graphs contained only seven and five points on them as “winds of less than 30 mph were omitted, as the force of these winds apparently was not sufficient to overcome the internal friction in the masonry walls.” By extrapolation from these graphs, Skinner estimated that a deflection of 6.5 inches (165mm) would occur when a wind of 80mph (35.8m/s) blew against either of the broad faces.
In order to assess the effects of temperature upon the plumb bob, readings were taken throughout a sunny day when a maximum temperature of 86°F was recorded. There was little wind blowing, the anemometer reading between 25 and 30 mph (11.2 to 13.4 m/s) and the plumb bob readings varied by no more than 0.25 inch (6mm) from 9:15 to 15:00. By comparison, the extreme measured plumb bob deflection of 4.6 inches (117 mm) occurred during a storm when the anemometer read 90 mph (40.2 m/s). Rathbun concluded that "the deflection of the building due to the sun shining on one side of it (and expanding that side) is practically negligible."

The collimator was used during high winds to study the dynamic movement of the building. However, "in strong winds, the motion of the target was irregular in direction, amplitude and time." Problems were also encountered in levelling the instrument, but Rathbun concluded that it enabled the movements of the top of the building to be studied qualitatively. The collimator observations show clearly the oscillatory motion of the building at its fundamental natural frequency (approximately 0.12 Hz). This same natural frequency and that of the orthogonal mode (approximately 0.18 Hz) are evident, to a lesser extent, in the extensometer observations.

In the discussion (ref 8a), Skinner quotes an extreme deflection of the building of 10.1 inches (257mm) for an 80 mph (35.8m/s) wind blowing against either of the broad faces. This comprises 6.5 inches (165mm) of "static" deflection (as mentioned above) plus 3.6 inches (91mm) of "vibration" deflection. This second figure is derived from a table showing six records where the wind blew directly against a broad face of the building. In these, the average deflection, as measured by the collimator, is about 10% higher than that measured by the plumb bob. Thus Skinner calculates that, where the "static" deflection is 6.5 inches (165mm), the "vibration" deflection will be 10% higher than this: 7.2 inches (183mm). However, as this latter figure is the peak to peak deflection, only half of this figure should be added to the "static" deflection, and thus the figure of 10.1 inches (257mm) results for the extreme deflection.
Dryden and Hill tested a 1:250 scale model of the Empire State Building in an aeronautical type of wind tunnel. The results of this wind tunnel test in smooth uniform flow were published in 1933 (ref 10). Rathbun experienced many difficulties in collecting full-scale data on the building; and even at the time questions were raised about the accuracy of it. However, the agreement between the full-scale and model data was so poor that it became obvious that the wind tunnel representation was wrong.

1.3.3 THEORETICAL WORK

A survey in the July 1929 edition of The Skyscraper showed that there were 377 buildings more than 20 storeys high in the United States, and that 10 of these were over 500 feet (152m) tall. The tallest of these, the 241m Woolworth Building, erected in 1912-13 was soon to be dwarfed by buildings half as high again. Although planned as being only 246m tall at this stage (and so just higher than the Woolworth Building) the Chrysler Building was completed in 1930 at a height of 319m. This increase was to ensure that it would be the highest building in the world, as a rival, the Bank of Manhattan Building, also increased its planned height from 255 to 283m over the same period (ref 11, p89). Despite this 30% increase in its planned height, the Chrysler Building only held onto the title of the world's tallest building for a few months. However, the Empire State Building, which took the title, held onto it for almost 40 years.

These great advances in building were unmatched by advances in the theory of wind forces on tall buildings. However, in 1930, both Spurr's treatise "Wind Bracing" (ref 12) and Fleming's book "Wind stresses in buildings" (ref 11) were published. Fleming suggested that the steel frame of a building should be designed to withstand a wind pressure of at least 20 lbf/ft² (958 N/m²) up to 300 feet (90m) high, and then a further 2 lbf/ft² (96 N/m²) for each further 100 feet (30m) in height (ref 11, p97).

Spurr gives in full the calculations he deems necessary to provide a design to resist wind loading in a hypothetical 120 storey (439m) building in New York. He suggests that this, and other tall buildings, should be designed to withstand a wind pressure of 30 lbf/ft² (1437 N/m²) at the top, this reducing linearly to zero at the base. The design would be acceptable
if, under this assumed wind load, the total deflection at the top of the building was not greater than two-thousandths of the height (ref 12, pp 109-110). Eight years after Spurr's book was published, the only advance offered by Coyle was to modify the design wind pressure at the top of the building to \( 0.03 \cdot h \text{lbf/ft}^2 \), where \( h \) is the height in feet, but still tapering linearly to zero at the base (ref 8a).

Although such designs did not include any allowance for the dynamic effects of the wind or response of the building; there was some recognition that such effects did exist. "Again, as a matter of logic, if nothing more, with a correct choice of maximum wind load it is necessary only to limit the total deflection to a proper amount in order to keep the vibration within proper limits, and as a result have a building which will "feel" right. There is, however, another phase of vibration which should be considered. If you hit a slender, upright body a sharp tap you induce a shiver in it without displacing it, if the stability is sufficient to absorb the blow. Something of this kind may take place in gusty winds. The remedy is a rigid panel or panels in the web system to act as a stiffening truss which will resist the quiver" (ref 12, p15).

1.4 FULL-SCALE TESTING 1940 - 1990

1.4.1 The problems of full-scale testing

Several experiments have been conducted in the last fifty years in which the response of a tall building to wind loading has been measured. However, with five notable exceptions, these have either been confined to measurements of pressure on the faces of the building (as opposed to some measure of the building’s motion) or they have only collected data for, at most, a few tens of hours. As will be explained in Chapter Two; whilst this length of data acquisition may well be sufficient to ascertain the response of a building to a particular storm, it is not sufficient to be able to predict the response of the building to a given wind speed, at least not to the accuracy required if the response is to be compared to that predicted by using a theoretical method.
The measurement of pressure on real tall buildings has been popular for over fifty years. This popularity may well be due to researchers being able to measure easily the same variable at full-scale and in a wind tunnel. However, work carried out twenty years ago by the Building Research Establishment (BRE) and others, showed that it was necessary to use large numbers of pressure transducers on a real tall building in order to understand the wind forces impinging on it (e.g. refs 13 and 14). Adjacent transducers must be placed close enough together so that it can be assumed that no pressure fluctuations occur in between them that cannot be deduced by interpolation. However, the measurement of pressure alone, whilst helping to further our understanding of the forces impinging on tall buildings, does not measure the response of the building to those forces, the end result required by the designer at the design stage.

Little full-scale experimental work was carried out in the 25 years following the publication of Rathbun's paper on the Empire State Building. However, three factors in the late 1960s contributed towards a renewed interest in full-scale work. These three factors were the availability of improved instrumentation, the building of a great number of high rise buildings with much greater flexibility than buildings such as the Empire State, and the development of new theoretical wind loading codes.

In 1966, Wiss and Curth (ref 15) conducted a test on 1000 Lakeshore Plaza, a 162m tall, square plan apartment building in Chicago. Recordings of various instruments attached to the building were made over a 30 day period. They found significant deflection of the building due to differential heating. They also laboriously drew out the simultaneous displacement of the building, as measured at half second intervals by two tiltmeters situated on diagonally opposite corners of the top floor of the building. By examining the resulting plots, they determined the maximum response of the building to the wind prevailing at the time. However, Dalgleish and Ward (ref 15a) expressed some concern about the frequency response characteristics of the tiltmeters used by Wiss and Curth, suggesting that in the region of the building's fundamental natural frequency, the tiltmeters may have been underestimating the actual response of the building to a significant degree.
1.4.2 University of Western Ontario Tests

The Boundary Layer Wind Tunnel Laboratory (BLWTL) of the University of Western Ontario (UWO) has conducted many full-scale experiments on tall buildings, usually ones that they have tested previously in a wind tunnel. In this way they sought to verify the results of their wind tunnel studies.

One of the most important early wind tunnel tests conducted at the BLWTL was for the twin towers of the World Trade Centre in New York. The results of this study "affirmed the important role of dynamics and turbulence in tall building response to wind" and "led to the acceptance of analytical approaches to allow for the dynamic effects - the gust response factors" (ref 16). The development of these new approaches to wind loading codes are discussed in section 1.5.4. Accelerometers were installed in the completed building, although these have mainly been used to monitor the effect of the visco-elastic dampers (ref 17) which were included in the design of the tower.

In 1971, Davenport and Hogan (ref 18) published an evaluation of Wiss and Curth's original data from 1000 Lakeshore Plaza. Damping values were obtained for the two fundamental translational modes by calculating the autocorrelation curves from time histories of between two and five minutes duration. The values obtained were 2.2% and 5.3%. They agreed with Dalgliesh and Ward (ref 15a) that the results from the tiltmeters could be explained by distortions between the shear walls and floors causing a reversal of the sense of the overall static tilt of the building.

Modified versions of the tiltmeters used at 1000 Lakeshore Plaza were placed in the 337m tall John Hancock Center, Chicago. The response of the building was recorded for an hour on each of two days in July 1970. Analysis of these records "provided a valuable confirmation of the predominance of the resonant response in the fundamental modes of vibration for a tall building" (ref 19). Damping values of 0.6% and 0.4% critical were obtained from autocorrelations for the fundamental translational modes, which occurred at frequencies of 0.15 Hz and 0.21 Hz respectively (ref 20). These tests showed that the structural damping values of modern steel structures could be much lower than the previously accepted figures.
Dobryn, Isyumov and Masciantonio (ref 21) measured the wind-induced accelerations of a 43 storey office tower in New York City on 11 occasions over a six month period. Three accelerometers were used to measure the response on the 41st floor of the building, their output being recorded on cassette tapes. Thirty-six cassettes, each 45 minutes long, were recorded. Recordings were made in both relatively calm as well as strong wind conditions. The latter category included the passage of Hurricane Gloria on 27 September 1985. Wind conditions were obtained from the local weather station and two airports.

Isyumov and Masciantonio were also involved in measurements of the response of the Citi-Corp building, which is also in New York City. This 300m tall building was erected in 1977 and includes a tuned-mass damper (TMD). The TMD operates whenever the building motion, as measured by accelerometers, exceeds a certain threshold level. During storms, recordings of the data from the accelerometers and the TMD were made. The data from two storms of between 5 and 6 hours' duration are reported in reference 22. Power Spectra of accelerometer records from different parts of the storm show clearly how the natural frequencies of the building change with the amplitude of response.

Full-scale observations were also made on the 555m high CN tower in Toronto, the world's tallest free standing structure. Instrumentation, including anemometers and accelerometers, was monitored over a four year period from early 1979 (ref 23). Mean and rms values for each sensor were calculated for each 10 minute period; about 1000 days of this averaged data being recorded on magnetic tape. In addition, "during storm conditions all data are recorded at a sampling rate of 5 Hz" for a period of between 1½ and 2 hours. Over 100 hours of this "high speed" data was recorded on tape. The tower, which has rotational symmetry every 120°, has a fundamental frequency of approximately 0.127 Hz.

The 296m tall Allied Bank Plaza tower in Houston was monitored for a ten month period from November 1982 as the building neared completion. Whenever a wind storm was forecast, two accelerometers were positioned on the 71st floor of the building, and their response recorded on magnetic tape. However, only two significant events, where the response was greater
than the sensitivity of the accelerometers were recorded (ref 24). The first of these was an extra-tropical storm with gusts up to 56mph (25m/s). The second was Hurricane Alicia which reached Houston on 18 August 1983. All the published data concerning the Allied Bank Plaza tower come from a 94 minute recording made during Hurricane Alicia. Overall accelerations of up to 46 millig (single peak) were recorded, with a maximum peak acceleration of 43 millig in the fundamental mode (which has a natural frequency of 0.130 Hz). Damping in this mode was estimated as 1.6% critical from both auto-correlation and power spectrum methods (ref 25). As no record of the wind speed at the building was made, a reconstructed record of windspeed in downtown Houston had to be used (ref 26). Using this model, the mean wind speed at gradient height during the recording is thought to have increased from 36m/s to 44m/s.

The maximum acceleration in each ten minute segment of the recording was then compared to the results of a wind tunnel test using an aeroelastic model of the building. Because of the uncertainty of the wind record, it was necessary to produce a likely range of accelerations from the wind tunnel data. Eight of the ten 10-minute peak accelerations measured on the building fell within the likely range.

1.4.3 Other full-scale testing

Details of the tests carried out on the Sheffield University Arts Tower by the Building Research Establishment (BRE) will be given in Chapter Two, after the discussion of the accuracy of spectral analysis of full-scale data. This is because the parameters used in the data acquisition at the Arts Tower were dictated by the required accuracy of the spectral analysis used.

Five other notable experiments are discussed in detail in the next three sections. Apart from these six, what little full-scale experimentation that there has been has tended to be along similar lines to the work done by UWO described above, although generally on a much less ambitious scale.
The response of the Sydney Tower, which is similar in shape to the CN Tower but about half the height, was monitored by Kwok over a two year period (ref 27). However, this work was mainly concerned with monitoring the effects of the installation of a secondary damper in the tower. Four years after the end of the initial monitoring exercise a computerized data acquisition system was installed in the tower. Reference 28 gives graphs of wind speed against acceleration for about sixty records, each of 20 minutes duration, recorded on the tower in 1986 and 1987. These graphs show that the along-wind response of the tower is sensitive to the wind direction, although "for the cross-wind acceleration response it is more difficult to detect any well-defined response characteristic" although "it is evident that the cross-wind response increases more rapidly with increase in wind speed at the high speed range" (reference 28).

BRE carried out some work on the response of the 177m tall Post Office Tower (now the Telecom Tower) in London. Initial measurements were of wind pressure and induced strain (ref 29). However, some accelerometers and an electro-optical deflection device were installed in the early 1970s, and two 87 minute recordings were made (ref 30). It should be noted that the CN, Sydney and Post Office Towers are much more slender than any of the other buildings mentioned in this thesis.

Short term measurements of the response of tall buildings to wind loading are relatively common. These tests are usually carried out to see whether the natural frequencies and mode shapes assumed in design have been achieved in practice. The estimation of damping values from these tests is also common; however, as will be seen in Chapter Two, the quoted damping values may be subject to large possible errors. Such ambient vibration tests have been carried out throughout the world with varying degrees of success. Examples include those in Hong Kong and China (refs 31 and 32), Japan (ref 33), U.S.A (ref 34), Yugoslavia (ref 35), and Czechoslovakia (ref 36).
1.4.4 CEBTP Tests

In 1975, Paquet, of the Centre Expérimental de Récherches et d'Etudes du Bâtiment et des Travaux Publics (CEBTP), reported the results of an experiment on a 50m high tower (ref 37). The tower was built of slipformed reinforced concrete and had plan dimensions of 6.25 by 4.15m, and a total height of 54.35m, 5m of which was below ground level. An eccentric mass vibrator was used to conduct a forced vibration test on the tower. (Although this was a less sophisticated vibrator than the one used by BRE, the principles of its operation and methods of testing are similar to the BRE system, which is described in Chapter Four). Two orthogonal natural frequencies were found (at 0.86 and 1.13 Hz), together with their associated damping values, mode shapes and modal stiffness values. Single peak displacements of over 5mm were obtained in the 1.13 Hz mode.

The tower was instrumented with two anemometers, two servo accelerometers, four wind pressure transducers, and three displacement measuring devices. The two anemometers, one standard, and one fast response or gust anemometer, were mounted on a mast 1.65m above the roof of the tower. The two accelerometers were placed orthogonally, but in line with the major axes of the tower, 2.65m below roof height. The pressure transducers were mounted, one on each face, 0.7m below the level of the accelerometers.

The first of the displacement measuring devices was a plumb line, the position of which was read by a contactless capacitive transducer, which, without touching the plumb line, gave two electrical voltages proportional to the X and Y displacements. The position of the plumb line was read just above where the plumb bob was immersed in an oscillation damping tank. A figure in the paper shows the path of the plumb bob on a sunny day and subsequent night. Thermal effects on the walls clearly had an effect, the north-south displacement varying by over 7mm from the start (9:20am) to 2pm, then returning to almost the same north-south deflection by 8:30am the following morning, when readings ceased. The east-west displacement varied by over 3.5mm with the extreme positions being reached at noon and 6pm. It should be noted that the finish position is about 1mm east-north-east of the start position. For this reason, as Paquet notes "it is not possible
to define a starting position or a position of rest, which fact complicates any definition of displacements under wind effects."

The second displacement measuring device was a vertical axis laser, anchored at the base of the tower and aimed at a target 47.7m above (5.15m below roof level). The laser spot was aimed at a discontinuous target made up of multiple photodetectors, the spot being sufficiently large to cover several of these at once. Air turbulence and thermal displacements in the tower caused problems with the stability of the laser, and so the beam was shone through a plastic tube under a partial vacuum.

The third displacement measuring device was a steel wire stretched vertically between two points 47.7m apart. Again a contactless capacitive transducer was used to establish the position of the wire. This transducer was placed only 2.5m above the lower fixing point of the wire, but Paquet says that the error in measuring over only one twentieth of the full height is absolutely negligible. However, he recognises that this device can only be used to measure static deformation and movement in the first mode, and is limited by the fundamental resonance frequency of the tight wire. Paquet also recognised that both these latter two devices, as they are fixed at the base of the structure, are influenced by the possible tilting of the foundation raft and so can only measure bending displacements.

The paper is mainly concerned with describing the instrumentation used in the tower, although examples of the data obtained, and of the analysis of short record lengths are given. However, there is no indication of how much data was recorded over the two years during which measurements were made, and therefore no attempt to look at how different wind speeds or directions affected the response of the tower.

After testing their experimental methods on the 50m tower, Paquet and his colleagues undertook full-scale observations of the 229m tall Maine-Montparnasse Tower in Paris (ref 38). Firstly, a forced vibration test was carried out in June 1975, and this identified the dynamic characteristics of the building. The fundamental natural frequencies for motion parallel to the minor and major axes were 0.20 Hz and 0.34 Hz; the corresponding damping values as measured by the decay of oscillation
technique (see section 4.8.2) being 0.67% and 0.47% respectively.

The building was instrumented with an anemometer, two servo accelerometers, ten pressure transducers, a plumb line, and a taut wire for measuring displacement. The plumb line and the taut wire were larger versions of those used on the 50m tower, the way in which they measured the displacement being identical. The first taut wire made of Kevlar, which was installed in May 1976, had to be replaced by a steel one six months later because several breakages of the first wire had occurred. However, the observed motions of both wires were influenced by the operation of the air conditioning and thermal effects on the cable, although these effects were considered to produce a maximum apparent deflection at the top of the tower of ±5mm. The maximum deflection noted during the tests was 64mm in the minor axis and 15mm in the major axis.

The initial site for the anemometer was on a lamp standard where it suffered from considerable turbulence as it was at the same level as the helicopter pad. From January 1977 an anemometer was installed on a 4.3m mast attached to a guard rail, at a height of 213m above the ground. Full-scale observations had been planned to finish at the end of 1977, but this period was extended until March 1978 because of the lack of strong winds during 1977.

Data from the various sensors were recorded on a sixteen channel digital tape recorder, which seems to have been triggered to record when the wind speed reached a threshold value, and then operated for up to eight hours. A paper potentiometer recorder was used to obtain a continuous record of wind speed, displacement along the minor axis, and four pressures. A vast amount of data from the building was collected. The total recording period is quoted as 30 x10⁶ seconds. This is equivalent to 347 days or over 8300 hours of data. However, the paper only quotes examples of the response of the structure to particular high winds which blew approximately normal to the faces of the building. There is no attempt to average together data collected during similar wind conditions. No results from or examples of the accelerometer data are given.
1.4.5 John Hancock Building, Boston

In 1987, Durgin and Hansen (ref 39) described data obtained from an unnamed 63 storey, 800 foot (244 m) tall building between February 1973 and January 1975. Between 33 and 138 transducers were used to collect this data, and altogether 20,000 transducer hours of data were acquired. Although this would seem to represent between 150 and 600 hours of data per channel, it should be noted that, during the two year period that data was collected, "the structural frame of the building was modified, tuned mass dampers were added, and the building was occupied".

The majority of the transducers on the building were measuring pressure on the facade of the building, the deflection of windows and/or mullions, and the racking of window frames. However, the wind velocity 100 feet (30.5 m) above the building was recorded, as was the output from eight accelerometers. A pair of orthogonal accelerometers was placed at each end of the long axis of the building on both the 35th and 57th floors. Also, "on several occasions" during periods of high winds, lasers were placed at the bottom of two elevator shafts which were at each end of the long axis of the building. The laser beams were directed onto grids placed in the elevator shaft at the 58th floor level. The grids were photographed by two movie cameras slaved together to take simultaneous photographs at 1/2 second intervals. The paper stressed that great emphasis was placed on obtaining good quality data and thus, amongst other things, the transducers were recalibrated at regular intervals. Unfortunately, the paper was confined to a description of the data acquisition, and figures showing short samples of the data collected. Apart from a brief description of these samples, no results are presented, neither is there any discussion of how the data are to be analysed.

At the Sixth U.S. Conference on Wind Engineering, Durgin and his co-authors presented three further papers on this work (refs 40, 41 and 42). From this it emerged that the still un-named building was in Boston, where, according to reference 43 (p133), the only building over 750 feet high is the John Hancock Tower which is listed as being a 790 foot (241m) tall office building, having 60 storeys and being completed in 1973. A review paper published in 1988 (ref 19) confirmed the building's identity.
The first of these three papers presents a few further examples of data collected, but is mainly concerned with advertising that the data were available (presumably for sale). The original data were recorded in analogue form on magnetic tape. More recently this original data have been reduced to give, for each record, the date and time of recording and the location and reading of each active transducer as ASCII characters. The reduced data were then recorded onto new magnetic tapes in digital form.

The other two papers give details of some limited analysis that has been carried out on the wind velocity and pressure figures collected from the data. However, to date, no analysis has been published concerning the measured acceleration and displacement data collected in the building. The majority of the data from the John Hancock Tower was recorded at a sampling rate of 2 samples per second, but on some occasions this rate was increased to 20 samples per second. Thus, as will be seen in Chapter Two, spectral analysis on the data recorded at a sampling rate of 2 samples per second, is only possible up to one Hertz; however this should encompass the natural frequencies of at least all the fundamental modes.

1.4.6 Commerce Court Tower

1.4.6.1 Description of the building and surrounding terrain

The Commerce Court Tower is a 57 storey, 239m tall office building in Toronto, Canada, and was completed in 1973. It has plan dimensions of 70m by 36m. Although symmetrical about the longer north-south axis, the building is not symmetrical about the shorter east-west axis; elevators and services being located towards the south end of the building. The elastic axis of the building is situated some 5m south of the geometric centre (ref 44). The building, which is situated in downtown Toronto, is surrounded by widely varying terrain resulting in different wind effects depending upon the wind direction. Dalgliesh (ref 45) describes its exposure "the second tallest building in downtown Toronto, (it) is partially sheltered from southwest to northwest by buildings from 175 to 285m in height" (from photographs, the closest of these buildings appears to be within 50m of Commerce Court Tower). Dalgliesh continues: "a strip of tall buildings half a kilometre wide extends several kilometres to the north. Surrounding
these areas are several kilometres of relatively low buildings. Lake Ontario lies one kilometre to the south." The 285m high First Canadian Place building, which lies to the north-west of Commerce Court Tower, was completed in 1976.

1.4.6.2 Wind Tunnel Studies

Wind tunnel studies of the building were carried out by the University of Western Ontario (UWO) in 1969 (ref 46), and then again in 1977 by the National Aeronautical Establishment (NAE) of the National Research Council of Canada (NRCC). An aeroelastic model was used in the 1977 tests (ref 47). All comparisons with model-scale data mentioned below refer to the data obtained when this aeroelastic model was used.

1.4.6.3 Instrumentation and data acquisition

A 3-cup anemometer and vane was mounted on a mast 47m above the roof of the building. From early 1973, the building was instrumented with 32 differential pressure transducers, and over 50 pairs of strain gauges. In March 1975, three accelerometers were placed at the 202m level, one measuring north-south motion in the middle of the north face, the other two measuring east-west motion at the north and south faces. Two further accelerometers were placed at the 234m level in July 1977, when a displacement tracking device was also installed. The displacement tracking device was also situated at the 234m level, and used two pairs of photocells to sense the position of a laser beam directed vertically from the foundation level (15m below ground). Outputs from the photocells controlled two servo motors which moved the carriage on which the photocells were mounted to compensate for movements of the top of the building relative to the laser beam.

Data from these various instruments were sampled 20 times per second for five or ten minute periods and summary data consisting of mean, standard deviation, maximum and minimum for each sensor, recorded onto magnetic tape. From 1973 until May 1979 two five minute periods were recorded each hour: the one with the highest reference wind speed and the last one. From May 1979 until September 1980 when the experiment was
terminated, 10 minute periods were used, and all six from each hour were recorded. In addition, a 35 minute time history for all 48 channels was started whenever the wind speed exceeded 18 m/s. However, data for the time series were only sampled twice a second.

1.4.6.4 Analysis of wind pressure data

Two of the published papers on the Commerce Court Tower experiment deal exclusively with the comparison between the model and full-scale wind pressures (refs 48 and 49). There was good agreement between wind tunnel and full-scale mean and rms pressure coefficients where there was sufficient full-scale data at high wind speeds. "Wind speeds of at least 20m/s are required to obtain reliable estimates of pressure coefficients for a particular wind direction at full-scale. Otherwise, the full-scale data have too much random variation to be of use" (ref 49).

1.4.6.5 Analysis of displacement data

A preliminary paper published in 1978 (ref 50) gives the procedure for obtaining the displacement data. A discrete Fourier Transform of each component of the laser was computed, all data above 0.05 Hz rejected, and then inverted back to the time domain. The resulting low-pass filtered quasi-static displacement data corresponded well with pressure data recorded at the same time. The unfiltered displacement data also agreed well with the data from the strain gauges. Good agreement was also found between the observed displacements and those calculated according to the then current Canadian building code, providing that the observed natural frequencies of the building were used in the calculations. Although the authors were aware that base rotation and thermal interference of the laser beam were potential sources of error in the displacement data, they claim that these errors were small.

Further analysis of the displacement data formed a small part of a paper published in 1982 (ref 45) where the full-scale displacements were compared to those found on the aeroelastic model. Reasonable agreement was found for east-west displacements (wind blowing against the broad face) but much poorer agreement for north-south displacements.
1.4.6.6 Estimation of dynamic characteristics

The reduction of the measured natural frequencies of the building with increasing wind speed was noted by the authors in reference 50. Whereas the natural frequencies of the NS1 and EW1 modes were both 0.139 Hz during an 8m/s wind; they reduced to 0.117 Hz and 0.118 Hz respectively for a 31m/s wind. At this time (1978) a tentative damping value of 1% critical was put forward, but it was noted that the widening of resonance peaks because of these changes in frequency was causing problems with the calculation of damping values. At this time, damping values seem to be derived from the auto-correlation function. However, by 1982, damping values for the building, estimated by the half-power bandwidth method, are given as 3 to 4% critical (ref 45). The corresponding damping values for the aeroelastic model are given as 2 to 3% critical. Dalgliesh then adds: "The difference between structural damping alone (1 per cent) and total damping for the model could be partially attributed to aerodynamic damping (less than 1 per cent). For the building, the variation of frequency with amplitude would contribute to an apparent increase in damping." No information is given as to what spectra the half power bandwidth technique was applied, nor is any differentiation between modes made. These figures are repeated in the final Commerce Court Tower paper (ref 44) but without Dalgliesh's comments. In this paper, a damping figure of 3% critical was used in a comparison with code-based predictions. The 3% damping value was used as it was "approximately midway between model and full-scale observations."

In the paper on the design and performance of the aeroelastic model (ref 47) the initial damping value of the model is given as 0.25%. However as this was lower than the 1% value which they quote as being measured on the full-scale building, foam rubber pads were added between adjacent sections of the facade. These had the desired effect, and damping values of 1% ±0.05% were obtained for the EW1, EW2 and NS1 modes. These values being obtained from decays of oscillation after an electrodynamic shaker, exciting the model at the relevant natural frequency, was turned off. (This is the same technique used by BRE in full-scale testing and is explained in detail in Chapter Four, section 4.7)
Additional accelerometers were temporarily installed in August 1980 to obtain the full-scale mode shapes for the building.

1.4.6.7 Analysis of accelerometer data

None of the published papers gives the total amount of data recorded. However, the most recent one (ref 44), which concentrates on data from the accelerometers, mentions that 4000 summary items were recorded whilst the wind speed exceeded 12.5 m/s in the last two years of the experiment. Some 70% of these items were selected to form the main data set. "They give reasonable coverage of 16 out of 36 10-degree wind direction segments." This main data set was then combined with 100 earlier, "and in some cases less reliable" records to form the data set examined in the paper. Earlier records were included "either because they had exceptionally high winds, or because the main data set had no information about a particular direction." It should be noted that the change from 5 to 10 minute averaging periods took place a third of the way through the last two years of the experiment. Therefore the main data set will consist of both 5 and 10 minute averaging periods, although the ratio between the two is not given.

The resulting data set was plotted out as acceleration versus reference pressure. In all 48 plots are given as the data is divided into the sixteen 10 degree wind direction segments where reasonable coverage was obtained; and the results from each of the three accelerometers on the 202m level are plotted separately. The accelerations are the standard deviation for each record (i.e. the rms acceleration once the mean has been subtracted from each value). The results of wind tunnel data and calculated responses using the then current Canadian building code are shown on the same plots, which are therefore somewhat crowded. Whilst the scatter in some of these plots is quite large, others appear to fit the calculated responses fairly well. The authors state that the full-scale accelerations increased proportional to the 3.3 power of roof-height wind velocity, providing that the data was selected so that the correlation coefficient for the fit was at least 0.5.
One of the consequences of the elastic axis not being at the geometric centre of the building was shown in this analysis. According to the authors, east-west accelerations at the north end of the building tend, on average, to be about 1.4 times those at the south end, for roof-height wind velocities of 20 to 30 m/s. This behaviour was also observed in the aeroelastic model tests (ref 51).

1.4.7 72m tall building in Las Vegas

In 1978, Williams presented preliminary results from a 72m tall building in Las Vegas (ref 52). The building, which had plan dimensions of 51m by 32m, was instrumented with an anemometer, two velocity meters, a vertical laser to measure displacements, and six pressure transducers. The output from the instrumentation was recorded on magnetic tape, each tape running continuously for five days. This paper concentrated on the instrumentation, and mentioned the severe problems caused by both radio interference and thermal turbulence in the laser data. Further results from the tests on this building were presented by Mills and Williams in 1981 (ref 53) and by Mills in 1988 (ref 54). The 3-cup anemometer and microvane were positioned near the centre of the building initially 6.5m above the top of the 7m high parapet wall, although this was subsequently altered to 10m above the parapet wall. However, the authors recognised the problems that this caused "ideally this device should have been at least 15m above the wall to reach the undisturbed flow above the building, but this was prohibited by building regulations" (ref 54). Consequently, the reference wind speed was taken as that at an airport 15 km away.

Data was recorded from May 1977 to December 1982, but only five "storms", which all occurred between November 1980 and August 1981, were analysed in detail. However, no displacement data were obtained for two of these storms, so complete analysis was restricted to three storms which had reference wind speeds of 15.6, 15.6 and 11.1 m/s. No indication is given as to the duration of these storms. A "calm" period was identified on each tape within 1.5 hours of the beginning of each of the storms, and this was used to define the zero reference values for the displacement and pressure transducers. The authors say that "displacement variations of the order of 0.1mm can be accurately detected by the system" and that "errors due to
beam axis tilting produced by the deformation of the building foundation have been negligible for the storms recorded to date" (ref 53). The lowest natural frequency of the building was found to be 0.40 Hz from spectral analysis of data from the velocity meters. The displacement data was defined as quasi-static (up to 0.3 Hz) or dynamic (above 0.3 Hz).

Reference 54 includes typical time histories for differential pressure and displacement (separated into quasi-static and dynamic components). These time histories are for 200 seconds and while Mills states that "obvious similarities" exist between them, there are noticeable differences in both magnitude and sign of response too. No correlation analysis between the two is included. However, Mills concluded that the "quasi-static displacement was the dominant alongwind component, ranging from 80% to 95% of the peak displacement".

1.5 OTHER WORK SINCE 1940

1.5.1 Problems with full-scale structures

Even within the last fifty years, full-scale structures have been built which have demonstrated that their designers have not understood fully the actions of the wind on the structure. The Tacoma Narrows bridge collapsed by oscillatory motion in 1940 (ref 55). In 1959, the Severn power-line crossing was subjected to severe transition-galloping oscillations (ref 56); and three cooling towers collapsed at Ferrybridge power station in 1965 (ref 57). Reference 58 discusses all three of these events in some detail. Although none of these events concerned tall buildings, it is evident that the state of knowledge concerning the response of structures to wind loading was deficient; particularly where the characteristics of the structure had a significant effect upon its response. As taller and more flexible buildings were built, the characteristics of the building played an increasingly important part in determining the overall response of the building.
1.5.2 Wind Tunnel Developments

The reason for the poor agreement that had been noted between Rathbun's full-scale data on the Empire State Building and Dryden and Hill's model data was soon recognised by Bailey and Vincent (ref 59). Whereas Dryden and Hill used a smooth flow aeronautical style wind tunnel; Bailey and Vincent appreciated that, in practice, the wind velocity varies with distance from the ground, and that the magnitude of this wind gradient will depend on the length and roughness of the surface on the upstream side of building, or model being investigated. When this new technique was used, Bailey and Vincent obtained much better agreement with Rathbun's full-scale data.

In 1954 Jensen formulated the scaling laws to match models to full-scale (ref 60); and from then onwards boundary layer wind tunnels were developed so that the characteristics of the natural wind could be reproduced at model-scale.

1.5.3 Determination of the wind spectrum

In 1957, van der Hoven looked at the spectrum of the wind at Brookhaven New York, over a wide frequency range (ref 61). Three distinct features were identified. Firstly, the macrometeorological peak centered around a frequency of 0.01 cycles/hour, or 1 cycle every 4 days. This corresponds to the typical transit period of weather systems. Secondly, the micrometeorological peak which ranges from periods of about 10 minutes to 3 seconds. This peak corresponds to the turbulence of the boundary layer. The third feature is the spectral gap which exists between these two peaks. In the spectral gap, which extends from a period of about 2 hours to a period of about 10 minutes, there is very little wind fluctuation. Similar spectra have been compiled since and these have all confirmed that these three features are typical of temperate latitudes.
1.5.4 New approaches to wind loading codes

Until about thirty years ago wind loading codes used the static assessment method where a design mean wind speed is combined with a design static loading coefficient to produce a design load. However, the work set out in the two preceding sections eventually led to a new quasi-static approach to wind loading codes. This can either be in the time or the frequency domain. In the first of these, the structure is assumed to respond to both the mean wind speed and gusts in the same manner, and thus make no contribution to its own response. This equivalent-static-gust method forms the basis of the United Kingdom wind loading code CP3. This approach is adequate where the structure is sufficiently stiff to follow the quasi-static loads, but not where the structure responds dynamically.

In 1961, Davenport advocated a design approach based upon quasi-static assessment in the frequency domain (ref 62). In this approach, a design mean wind speed determines the turbulence spectrum over a given terrain. This turbulence spectrum is modified by the admittance function of the structure (which is determined by the aerodynamic shape of the structure) to produce a quasi-static load spectrum. The frequency response function of the structure (determined by the dynamic characteristics of the structure) modifies the quasi-static load spectrum to produce the resulting dynamic response spectrum. This method is used in the current Canadian, Australian and U.S. codes. The different assessment methods, their uses and limitations are discussed further in Cook (ref 63, pp 82-86).

In 1987, Lee presented a paper at the 7th International Wind Engineering Conference which compared the response of tall buildings as predicted by different wind loading codes from around the world (ref 64). He, and his co-author Ng, calculated the along-wind responses for three buildings using six different codes. Data for the characteristics of the buildings were obtained from forced vibration tests by BRE. One of the most interesting things to come out of this study is not in the paper but is mentioned by Jeary in his report on Lee’s presentation which forms part of his review of the conference (ref 65). "His studies showed that there were significant differences between the various methods and that the inconsistencies were generally largest for the tallest buildings. The ESDU
(76001) response values were 'startlingly high', but of greater significance was that two people going through the same calculation could get differences of 50% in their answer merely through the choice of parameter values. The people involved were experts in the process, and Lee concluded that it was not therefore possible to rely on such calculations for better than 50% precision in calculating the response of structures to wind loading."

It should be remembered that in Lee's comparison, measured values of fundamental natural frequencies, damping and modal mass were used in the calculations. However, there are large possible errors in estimating these properties at the design stage (ref 66). When these two areas of uncertainty are put together, it can be seen that an accurate prediction of the response of a tall building to wind loading is extremely difficult to achieve at the design stage. Whereas Lee's comparison was between different wind loading codes, the idea of the work described in this thesis is to compare predicted response with that actually measured on a tall building, and so, hopefully, see which of the code approaches is most successful in modelling the real world.
2.1 SPECTRAL ANALYSIS

2.1.1 The need for spectral analysis

In modern methods of finding the dynamic response of a tall building to a given wind, the response of the building is calculated in its different modes of vibration, and then these are summed in various different ways to find the overall response. If a comparison to the codes is to be made, then it is necessary to measure the response of a full-scale building in terms of these different modes of vibration. In order to do this, some form of spectral analysis, that is the conversion of a response in the time domain (a time series) to one in the frequency domain (a response spectrum) is required.

If, however, the total response of a building was all in one mode of vibration (at least as perceived by a single transducer) then such a conversion would be unnecessary. Such an approach has two major drawbacks. Firstly, it would only be as a result of spectral analysis that the supposition could be proved. Secondly, extremely useful information regarding the variation of the natural frequency and damping of the mode with amplitude of response would be lost. Even with these drawbacks, the supposition is unlikely to be born out in practice, as a response in both torsional and translational modes is likely to occur. Therefore, some form of spectral analysis is required.

No attempt is made in the following section to present a rigorous derivation of the mathematical basis of the fast Fourier transform (FFT) technique. Such a derivation can be found in numerous text books on the subject (e.g. refs 67-71). However, the basic concepts behind the technique are outlined, and where the use of the technique has particular relevance to the problems of analysing data from tall buildings, the subject is tackled in detail.
2.1.2 Spectral analysis theory

2.1.2.1 Fourier transforms

It has long been recognised that any periodic function can be broken down into its harmonic components, which can be expressed as an infinite trigonometric (Fourier) series. If the function is not periodic then it cannot be analysed into discrete frequency components. However, if the function obeys certain conditions then the Fourier series can be turned into a Fourier integral, and the Fourier coefficients can be turned into continuous functions of frequency known as Fourier transforms. A Fourier integral (or inverse Fourier transform) can be regarded as the limit of a Fourier series as the period tends to infinity. Thus Fourier integrals indicate the frequency composition of an aperiodic function.

Consider a collection of sample records \( x_1(t), x_2(t), x_3(t) \) etc. taken from a random process \( x(t) \). If a large enough number of samples is taken, then by averaging across these samples an ensemble average, \( E[\cdot] \), is found which will approximate to taking an infinite number of samples. The autocorrelation function, \( R_{xx}(\tau) \), of a random process \( x(t) \) is defined as the average value of the product \( x(t)x(\tau+t) \).

The autospectral density of the random process, \( S_{xx}(f) \), is the Fourier transform of the autocorrelation function.

\[
S_{xx}(f) = \int_{-\infty}^{\infty} R_{xx}(\tau)e^{-i2\pi ft} \, d\tau \quad \text{(Eq 2.1)}
\]

It will be noted that the autospectral density function ranges from \(-\infty\) to \(+\infty\). However, in engineering applications it is usual to deal only with the real part of the spectrum which has positive frequencies only. The one-sided autospectral density function \( G_{xx}(f) \), is defined as \( G_{xx}(f) = 2S_{xx}(f) \) for \( 0 \leq f \leq \infty \) but otherwise is zero. In terms of the autocorrelation function it is defined as follows.

\[
G_{xx}(f) = 4 \int_{-\infty}^{\infty} R_{xx}(\tau)\cos2\pi ft \, d\tau \quad 0 \leq f < \infty \quad \text{(Eq 2.2)}
\]
The mean square value of a stationary random process $x(t)$, $E[x^2]$, is given by the area under a graph of spectral density $G_x(f)$ against $f$, as this area is equivalent to the autocorrelation function when $\tau = 0$.

### 2.1.2.2 Discrete Fourier transforms

Most experimental measurements of random processes are carried out digitally by sampling a random process at regular time intervals to create a time series $\{x_r\}$. Prior to the 1960s, the spectral analysis of times series was accomplished by estimating the appropriate correlation function and then calculating the Fourier transform. However, the advent of the fast Fourier transform (FFT) meant that spectral estimates could be obtained directly from the original time series, without the need to calculate the correlation function. Therefore, as well as being much quicker, FFTs allow spectral estimates to be obtained more accurately. The discrete Fourier transform (DFT) of the series $\{x_r\}$, $r = 0, 1, 2, ..., (J-1)$ is:

$$X_k = \frac{1}{J} \sum_{r=0}^{J-1} x_r e^{-i(2\pi kr/J)} \quad k = 0, 1, 2, ..., (J-1) \quad \text{(Eq 2.3)}$$

where $X_k$ are the Fourier components.

The DFT is the basic tool used in all the spectral analysis described in this thesis. The limitations on its use and the accuracy of the results obtained by using it will now be addressed.

### 2.1.2.3 Aliasing

When carrying out discrete Fourier transforms, care must be taken to avoid aliasing, the repetition of a part of the true spectrum at false frequencies. This can be avoided by passing the signal to be analysed through a low pass filter to remove all frequency components higher than $1/2\Delta$ where $\Delta$ is the time interval between successive samples. The frequency $1/2\Delta$ is known as the Nyquist or folding frequency.
2.1.3 Requirements for valid spectral analysis

For normal spectral analysis to be valid, the input data must be stationary and from a linear process. The input data should be random rather than deterministic if the amplitude of any of the frequency components is required. Whilst not a requirement, it is also desirable to assume that the data have a normal or Gaussian distribution. A Gaussian function is totally determined by the mean value $\mu$ and the standard deviation $\sigma$, hence only these two parameters need to be measured to determine the probability function of the data. All linear operations performed on a normally distributed random variable produce a new random variable which is also normally distributed. Therefore the response of a linear system will be a Gaussian process if the excitation is Gaussian (e.g. ref 68, pp33-35; ref 69, pp79-81).

For a lightly damped structure, the response of the structure tends to a Gaussian response as the damping tends to zero. This is because the structure acts as a filter, suppressing any deviations from the Gaussian form that may exist, assuming no deterministic components are present. If the output is not Gaussian then either a non-linear operation has been performed somewhere in the process or the signal is deterministic (ref 68, p35).

It should be noted that no physical phenomena can be represented by a truly Gaussian distribution. For example, physical phenomena have upper and lower amplitude bounds whereas Gaussian distributions have no bounds. However a Gaussian assumption can usually be used except when assessing extreme values, where the assumption would be inappropriate (ref 68, p35). The important point is to make sure that, whatever model or assumed distribution is being used, it is an appropriate one for the data.

2.2 FULL-SCALE DATA ACQUISITION

The first consideration that must be addressed before collecting data to be used for spectral analysis is what quantities are required from the analysis and to what accuracy are they required? Given the problem
outlined above of determining the response of a tall building to wind loading, the following quantities would be required from the spectral analysis: natural frequencies, magnitude of the response in each mode, and the damping in each mode. Given a stationary response, these quantities can be determined for a structure. However, the accuracy to which they are determined is dependent upon the length of the record which is analysed. Before continuing with the problems involved in data acquisition and analysis it is necessary to define a few quantities and procedures.

The minimum frequency resolution bandwidth \( B \) is the frequency interval between two successive lines in a spectrum, and is the reciprocal of \( T \), the time taken to acquire a single sample record. Thus for a single sample record, the product \( BT \) is always one.

If the maximum response in a mode is \( A \), then for a mode with light damping (less than 10% critical), the Half Power Bandwidth (HPB), \( \beta(\text{Hz}) \), of the mode is the width of the mode where the response is \( A/\sqrt{2} \) (see figure 2.1). This can be approximated to \( 2f_r\zeta_r \) where \( f_r \) and \( \zeta_r \) are the natural frequency and critical damping ratio of the mode in question.

### 2.3 VARIANCE AND BIAS ERRORS

#### 2.3.1 Variance Error

When any spectral analysis is carried out two inevitable errors occur. The first is variance or random error \( (\sigma_v) \). Variance error results from the fact that the analysis must be performed on a finite number of sample records \( (N) \) or over a single sample record of finite length \( T \). On an intuitive level, the more times a random process is sampled, and then the results averaged, the greater the accuracy; providing the same process is sampled each time (i.e. it is stationary). Variance error is equal to the reciprocal of the square root of \( N \). It should be noted that, for example, for the variance error to be halved, the number of sample records must be increased fourfold.
HALF POWER BANDWIDTH ($\beta$) = $f_2 - f_1$

DAMPING ($\zeta_r$) = $\frac{f_2 - f_1}{f_1 + f_2}$

As $f_1 + f_2 = 2f_r$, then $\beta = 2f_r \zeta_r$

FIGURE 2.1 DETERMINATION OF DAMPING IN MODE $r$
BY THE HALF POWER BANDWIDTH METHOD
Normalised random error formula for each spectral line in an autospectral density function:

\[ \varepsilon_r[G_{xx}(f)] = \frac{1}{\sqrt{N}} \]  
\[ \text{(Eq 2.4)} \]

2.3.2 Bias Error

The other inevitable consequence of spectral analysis is bias error (\( \varepsilon_b \)). Bias errors occur where there is a rapid change of amplitude with respect to frequency and there are too few ordinates or spectral lines to be able to resolve the variation accurately. Such a situation occurs in the region of a peak in the response spectrum i.e. a natural frequency. The less spectral lines that lie in the HPB of a particular mode the greater the bias error (see figure 2.2). The following formula is given in Bendat and Piersol (ref 68, p 267). It should be noted that this formula is not accurate for bias errors in excess of 30% (i.e. when \( B > \beta \)). However, bias errors of this magnitude are not acceptable.

Normalised bias error formula for an autospectral density function:

\[ \varepsilon_b[G_{xx}(f)] = \frac{-1}{3} \left( \frac{B}{\beta} \right)^2 \]  
\[ \text{(Eq 2.5)} \]

2.3.3 Implications of variance and bias errors

2.3.3.1 Bias versus variance errors

To minimise bias error it is desirable to make the resolution as fine as possible and hence maximise the number of spectral lines. Therefore, making the resolution finer by reducing B (i.e. reducing \( 1/T \)) means increasing T. Increasing T means that, for a given total length of data available, T, less sample records can be obtained, so the variance error increases. Inevitably a compromise has to be reached between these two conflicting requirements.
FIGURE 2.2 ILLUSTRATION OF BIAS ERROR

Example of mode r with natural frequency $f_r = 1.0$ Hz and damping $\zeta_r = 1.0\%$ critical

Half power bandwidth ($\beta$) = $2 f_r \zeta_r = 2 \times 1.0 \times 0.01 = 0.02$ Hz

Bias error ($\epsilon_b$) = $-\frac{1}{3} \left( \frac{B}{\beta} \right)^2$ where B is the resolution bandwidth

When $B = 0.02$ Hz, $\epsilon_b = -33\%$

When $B = 0.005$ Hz, $\epsilon_b = -2\%$
2.3.3.2 Bias error increases for decreasing frequency

There are several interesting observations that can be made about the formulae for bias and variance error. Firstly, consider an autospectrum containing two modes which has been averaged from N samples. The variance error for each mode will be the same, as this depends upon the number of samples and is independent of frequency. However, although the resolution bandwidth $B$ is the same for each mode, they will probably have different HPBs and consequently different bias errors. If the two modes have the same damping but one is only a quarter the frequency of the other, it will have 16 times the bias error.

2.3.3.3 Calculation of bias error

The second point is that bias error can only be calculated once the natural frequency and the damping of the mode in question are known. This seemingly trivial point has caused many problems in full-scale experimentation. Although natural frequencies can be determined relatively easily from the results of spectral analysis, this is not the case with damping.

Consider the case of a structure, such as a long span suspension bridge, where there is considerable doubt as to what the correct damping values for it are. Consider also that an ambient vibration test is conducted on the structure, and values of natural frequency and damping obtained by spectral analysis. It is possible to use these values to calculate the HPB and then the bias error for each mode. If the calculated bias errors are small, the analysis parameters might be assumed to be acceptable. However, such an argument is fallacious as the results of the analysis are being used to justify the analysis parameters that produced the results.

If damping values found by an independent method are used, or found to be in close agreement to those found by spectral analysis, then the circular argument can be broken. Otherwise gross overestimates of damping may be unwittingly accepted as being accurate. It has been proposed (ref 72a) that this may well account for the large damping figures obtained for
the lowest natural frequencies of long span suspension bridges.

If an independent method of estimating damping is not available, then it is suggested that the analysis parameters are varied and the effect on the damping values noted. This approach was adopted by Littler and Ellis in their analysis of data from the Humber Bridge (ref 73). In this case, there was a clear tendency for damping values to decrease as the frequency resolution used in the analysis was reduced, so indicating that the frequency resolution was not fine enough, and that the damping values were to be treated with caution.

2.3.3.4 Bias error is always negative

The third point is that bias errors are always negative. It is important to remember that the spectrum which results from any spectral analysis is in some ways a histogram. The points in the spectrum are in fact merely denoting the centre point of a bar which has a width equal to the resolution bandwidth. Each spectral point represents the average amplitude of all the frequencies within the spectral bandwidth. Thus, if a peak occurs within the bandwidth, the average value will be less than this peak value. The amount by which the value of the peak is reduced depends upon the sharpness of the peak (i.e. the damping). Thus, the amplitude of a peak in a response spectrum is always underestimated. As the peak value is underestimated then the half power points will be too, thus leading to an overestimation of damping.

Although, given the natural frequency and the damping of a mode, bias error can be calculated, the amount by which damping is overestimated cannot be quantified as this depends upon where the real spectral peak lies. The two extreme cases would be where either the real spectral peak coincides with a spectral line, or lies midway between two lines.

2.3.3.5 Bias error effects on frequency and mode shape determination

It was noted above that the damping values obtained from an ambient vibration test on the Humber Bridge (ref 73) were thought to be large overestimates of the true values. However, the mode shapes and frequencies
obtained by the same spectral analysis which produced these erroneous damping values do not suffer from the same inaccuracy. This is because, providing the same analysis parameters are used when analysing data collected simultaneously from two or more measurement positions on the bridge, the bias error for each mode will be identical for the resulting spectra. Therefore, the proportion by which each modal peak is underestimated will be the same, and so the correct mode shape is obtained. Although bias error will lead to an underestimation of a modal peak, the greatest response will still occur on the spectral line which contains the frequency of maximum response. Therefore an accurate estimate of the true natural frequency will be obtained, although this accuracy will be influenced by the resolution used in the analysis. However, non-linear response of the structure (see section 2.6) can also affect the accuracy, if the force to which the structure is responding is not stationary (see next section). It should also be noted that accurate estimates of natural frequencies and mode shapes may not be obtained where modes are close together in frequency.

It should not be assumed from the above that obtaining mode shapes of structures from ambient vibration tests is simple. In addition to the above points, the following should also be considered. Is the exciting force the same over the whole structure? In the case of the Humber Bridge for example, is the same gust of wind acting over the whole length of the bridge or will any discrepancies even themselves out over a long enough period of time? Such an effect could occur on a tall building if it is partially shielded from the wind by other buildings. Finally, incorrect mode shapes will be obtained if electronic noise in the data acquisition system is of the same order as the response of the structure at any of the measurement positions, or if an insufficient number of samples is averaged together to overcome the affects of this noise, even where the response is higher than the background noise level.
2.4 STATIONARITY

2.4.1 Definition

Bendat and Piersol (ref 67, p11) state that a random process \( \{x(t)\} \) is said to be weakly stationary, or stationary in the wide sense, if the mean value is a constant, and the autocorrelation function (the average product of the data values at times \( t_1 \) and \( t_1 + \tau \)) is dependent only on the time displacement \( \tau \). That is \( \mu_x(t_1) = \mu_x \) and \( R_{xx}(t_1, t_1 + \tau) = R_{xx}(\tau) \). A random process is said to be strongly stationary or stationary in the strict sense if all possible moments and joint moments are invariant with time. However, "for many practical applications, verification of weak stationarity will justify an assumption of strong stationarity". (ref 67, pl2).

There is a problem with this definition of stationarity. Given a finite or indeed infinite random process, over what period should the mean be calculated? For example, given a 24 hour period of data; should a test for stationarity be performed by comparing the mean value each hour, each minute, each second, some other time interval, or all possible time intervals? Clearly, if a small enough time interval is chosen, then the data will only have an unchanging mean, and therefore pass this test of stationarity, if all the data points have the same value. Equally clearly, such a data set is not random; therefore some "engineering judgement" is required in order that a sensible number of averages is chosen. Yang uses the same definition as Bendat and Piersol but adds "note that a sample function in a stationary process does change with time; stationary refers to the time independence of its statistical property" (ref 71, p 24). This question will be returned to after a consideration of the ways in which a test for stationarity can be performed, and an alternative definition for stationarity will be given in section 2.8.3.

2.4.2 Testing for stationarity

A random process can be labelled stationary if it contains no underlying trend. One test for stationarity suggested in Bendat and Piersol (ref 67, pp 95-97) is the run test. In this test, the period of
data to be tested is split up into a number of equal divisions. Then the
value of a suitable quantity, e.g. rms acceleration, is calculated for
each of the divisions. These values are then said to be positive or
negative, depending upon whether they lie above or below the median. A
succession of one or more values of the same sign forms a "run". Given the
number of divisions, the likelihood of a random process giving a certain
number of runs can be calculated. A small number of runs will suggest a
time series that has a trend which is increasing or decreasing with time,
whilst a large number of runs relative to the number of divisions suggests
a periodic function. However, it should be noted that this test can only
determine the confidence with which a series of data points can be said to
be from a stationary random process.

For example, given 100 divisions, 90% of all random processes will
have between 42 and 59 runs. Only 2% of random processes will have less
than 38 or more than 63 runs. It cannot be said with absolute certainty
that a process having say, 10 runs is not stationary. However, the
likelihood that a stationary process will only produce 10 runs is so small
that it can be neglected. Therefore this process would be considered to be
non-stationary.

Experience shows that one hundred seems to be about the right order
for the number of divisions to be used for a run test for the following
reasons. If only ten divisions were chosen, the difference between say,
two and three runs, represents an unacceptably large difference in
confidence levels. Conversely, a thousand divisions would require many
more calculations than can be justified for the narrowing of confidence
limits that a change of one run represents. Bendat and Piersol give a
table for various confidence limits for between 10 and 200 divisions (ref
67, p532). However, from the table it is apparent that an acceptable
minimum number of divisions is about 40.

Where a test for stationarity of a single continuous recording is
required, BRE use a run test with between 50 and 200 divisions (e.g. refs
73, 74). However, as will be seen in section 2.8.2, some care must be
taken in choosing the number of divisions and therefore the length of time
that each division represents. Although a run test is a good way of
assessing the stationarity of a single continuous recording, such a test cannot be applied to selective ensemble averaging as described later in this chapter. Therefore some other method of testing for stationarity has to be employed with this method, and this is described in section 2.8.3.

Bendat and Piersol also describe another way of testing for underlying trends, the reverse arrangement test (ref 67, pp 97-99). They say that this test is more powerful than the run test for detecting monotonic trends; but is not powerful for detecting fluctuating trends.

2.5 WINDOWING

The finite Fourier transform of $x(t)$ can be viewed as the Fourier transform of an unlimited time history $v(t)$ multiplied by a rectangular time window $u(t)$. The resulting Fourier transform inevitably has "side lobes" (i.e. a series of spurious spectral peaks on either side of a true spectral peak, the amplitude of which diminish as the distance from the true spectral peak increases) caused by the attempt to analyse data which has an inexact number of periods within the length of the rectangular window, $T$ (ref 67, pp 393-396). To suppress this "leakage" problem, it is common practice to apply a time window which tapers the time history data to eliminate the discontinuities at the beginning and end of the record to be analysed. One of the earliest but still most widely used such windows is the Hanning or full cosine tapering window; although many others have been developed over the last 25 years.

The problem with Hanning and all such windows is that they broaden the spectral peaks in any spectral density estimates. "For Hanning, the increase in the half-power bandwidth of the main lobe is about 60%" (ref 67, p398). Although it is possible to multiply the values of spectral peaks to allow for windowing, "it is not possible to apply correction factors to data that has been subjected to Hanning, Cosine or other windows in order to find out how much they will overestimate damping" (ref 75).
Windowing becomes less necessary as the record length increases. As $T$ increases so the resolution bandwidth becomes finer. The inevitable side lobes are then included in the actual modal response so are not important. Therefore, in order to obtain the best possible estimate of damping, the best approach is to make $T$ as long as possible and not apply any window function. However, inevitably, the best possible estimate of damping from a spectral density function will be the upper bound of the true damping value (ref 75).

2.6 OBTAINING DAMPING VALUES FROM SPECTRA

As has been seen above, obtaining accurate damping values from the results of spectral analysis is not easy. However, as all the sources of error lead to an overestimation of damping, at least an upper bound can be established. Bendat and Piersol (ref 68, p186) give four conditions that should be met before damping values are estimated from spectra. Firstly, the autospectrum of the excitation force must be reasonably uniform over the frequency of the mode in question. Secondly, the modal damping must be small ($\zeta_r < 5\%$). Thirdly, $B < 0.2 \beta$. Fourthly, the mode must not overlap heavily with neighbouring modes.

The first of these conditions will be met for wind-induced excitation of structures if the mode in question is above say, 0.1 Hz, providing that the wind spectrum is not modified by some nearby surface feature. Perhaps the most important condition when trying to obtain accurate damping values, is that the analysis must be conducted so that bias errors are small (section 2.3). However, in order to calculate bias error the damping value must be known, so assessing whether the second and third conditions have been met is not always possible before carrying out any spectral analysis. As well as these four conditions, the use of a window function should be avoided as this will again lead to an overestimation of damping. By keeping bias errors small there is no need for windowing.

There is a third possible source of damping overestimation. Non-linear behaviour, where the natural frequency and damping vary with the amplitude of response, has been noted on a number of large structures (e.g.
ref 76). Consider the case where the natural frequency of a mode varies over the length of a recording to be used for spectral analysis. Now consider a particular frequency, which corresponds to the natural frequency, and therefore the peak in the spectrum, for part of the recording. At other times in the recording, when the natural frequency has a different value, the amplitude at the particular frequency is reduced. As spectral analysis is in effect an averaging process, the overall amplitude will be less than the value obtained when it was the peak value. The same will be true for any other frequency which corresponds to the natural frequency for that part of the recording. This leads to a broader, flatter overall peak. At the same time, the base of the modal response will be broadened as it contains spectral components obtained when the natural frequency was at its highest and lowest extents, even though, obviously, these components correspond to data obtained at different times. Both of these effects will lead to an overestimation of damping. The change of damping with amplitude of response only complicates the matter further.

Although it is not possible to eliminate this source of error completely, it can be minimised by ensuring that the data are stationary. If the amplitude of the response is kept reasonably constant, then the changes in natural frequency and damping will be small. However, if, for example, the response of a building to a wide range of wind speeds is averaged together, then significant overestimation of damping should be anticipated for this reason alone. One of the advantages of a forced vibration test is that non-linear behaviour can be investigated, and so the significance of this behaviour evaluated.

2.7 DIGITAL SPECTRAL ANALYSIS ON DATA FROM A TALL BUILDING

The following procedure is one which it is suggested should be adopted when collecting data from a tall building, and subsequently performing spectral analysis on the data. To give an idea of the quantities involved, data from the Sheffield University Arts Tower (which is described in detail in section 2.9) are used where appropriate.
1. Estimate the maximum frequency of interest, if necessary pass the signals through a low pass filter to remove excessive high frequency components.

2. Choose an appropriate sampling interval $\Delta$ (seconds) so that the Nyquist frequency $1/2\Delta$ (Hertz) is at least twice the maximum frequency of interest.

3. Estimate (or preferably calculate if data are available) the half power bandwidth of all the modes to be included in the analysis. Use the mode with the smallest HPB in the following calculations.
   For Arts Tower, smallest HPB is for NS1 mode (using 1976 forced vibration test data)
   $f = 0.68$ Hz, 0.86% critical damping, so HPB $= 0.0117$ Hz.

4. Decide the spectral resolution to be used in the analysis, $B$. For four points in the HPB (Bias error of -2.1%) resolution would be 0.002925 Hz. $1/B$ is $T$ the time needed to acquire one record to resolution $B$. Therefore, for the above resolution, $T$ would be 342 seconds.
   (The mathematical trick used in the FFT requires the number of points to be highly composite i.e. have many divisors. Therefore, for efficient spectral analysis, the number of points to be transformed should be a power of 2. For convenience, it is usual for there to be an exact number of samples in each second. It is also common for the number of samples in each second to be a power of 2. In these circumstances, for efficient spectral analysis, $T$ should be a power of 2 in seconds.)
   Therefore, if $T$ is to be a power of 2 in seconds, increase $T$ to the next power of 2 (512 seconds in this example). This then gives $B = 0.00195$ Hz, 6 points in the HPB, and a bias error of 0.93%.

5. If possible, $T$ should be chosen so that it lies within the spectral gap (see section 1.5.3) i.e. $T$ from 600 to 7200 seconds, so that each record is self-stationary. In practice, if $T$ is to be a power of 2 in seconds, then $T$ will be 1024 or, rarely, 2048 seconds. However,
T is often taken to be 512 seconds although, strictly speaking, this does not lie within what is usually considered to be the spectral gap.

6. To achieve a variance error of no greater than ±10%, then N must be at least 100. Therefore, 100 x 512 seconds (14.2 hours) of data will be required. However, for the analysis to be valid, the data must be stationary for the whole of this period. In effect this means that the wind must blow at a constant wind speed and direction for the entire 14 hours. This condition is unlikely to be achieved in practice.

7. Even if this is achieved in practice, this only gives the response of the building to one wind condition. To enable the results of such an experiment to be compared with theoretical predictions, it is necessary to obtain the response of the building to a wide range of wind speeds and directions.

Whilst the above gives the suggested procedure for collecting data, the problem of what storage medium to use for the data still remains. Even on the slowest speed, most FM tape recorders will record data for, at most 12-15 hours. Although data could be recorded directly onto computer, this is likely to create storage problems very quickly. Using an analogue tape recorder to record the original time histories does have the advantage that the analysis parameters are not fixed. Indeed, as will be seen in section 2.9, the original analysis parameters for the Arts Tower were considered to give too few samples; so the time for each individual sample was halved and the analysis redone by going back to the original tapes. However, this re-analysis was very time consuming.

The problem of data storage has led some researchers to abandon the storage of raw time histories, and such an approach was adopted at Commerce Court Tower except when exceptionally strong winds blew (see section 1.4.6.3). However, if doubts arise later about the integrity of the data, or some further analysis is required, it is preferable to have the original time histories available.
2.8 SELECTIVE ENSEMBLE AVERAGING

2.8.1 The concept of Selective Ensemble Averaging

As shown in the previous section, collecting sufficient data for analysis from a building such as the Arts Tower is not an easy task. The problem is that, to produce data to the required accuracy, over 14 hours of stationary data are required, but obtaining 14 hours of continuous stationary data is very unlikely. Although it is convenient for the 100 samples used in the above example to be from a continuous recording, there is no requirement for this, providing that the data are stationary. Therefore, the samples could be selected from a number of recordings made over several days, weeks, months or even years, providing that they form a stationary set and that no changes to the forcing function or the way the structure responds have occurred. In practice, when measuring the response caused by the wind, then these criteria will be met as long as no physical changes have occurred to the structure (either by way of modifications or damage), and it is then only necessary to select a stationary set of data.

2.8.2 Self-Stationarity of individual records

Each sample must, of course, be stationary itself. This applies to consecutive samples from a continuous recording as much as it does to those used in selective ensemble averaging. If the length of a single sample is within the spectral gap (see section 1.5.3) then this condition should be met. Although this really should be checked by the use of a run test, carrying this out in practice is far from simple.

Consider a run test with 100 divisions operating on a single record of length 1024 seconds. Consider also that, as suggested in section 2.4.2, the rms acceleration is calculated for each division, that is a period of just over ten seconds. However, the response over each ten second period is strongly dependent on the response over the previous ten seconds. Therefore the process is not random and the record would be expected to fail the run test. However, as the record length of 1024 seconds is in the spectral gap, the record should be stationary and therefore pass the run test. If a lower number of divisions were used in the run test, such that
the response in one period was not dependent upon the response in the previous period, then this quandry could be avoided. For the response in one period to be independent of that in the previous period, the length of each period should be in the spectral gap. This then leads to the situation where there is only one division and therefore no run test can be conducted. Even if a record length of one hour was used, the maximum number of divisions that would each fall in the spectral gap is six, far too few to conduct a run test.

Although it would seem that self-stationarity cannot be assessed in this way, the 1024 second sample used in the example above could be subjected to a series of run tests. If the hypothesis used above is correct, then the dependence of the value of one division upon that of the value of the previous division decreases as the time between divisions increases. This may be the basis of a method for assessing the stationarity of single records of less than one hour's duration. Where, as in the case of the Arts Tower, a continuous 14 hour recording is available, there is no problem in using a run test. In this example, 80 divisions, each of 10.5 minutes duration could be used. However, if the duration of each division is to be within the spectral gap, and, as suggested in section 2.4.2, between 50 and 200 divisions are used; then the total length of data to be subjected to a run test should be between 8 hours 20 minutes and 200 hours.

If a satisfactory method for demonstrating the self-stationarity of individual records can be found, then it should not be necessary to test each record but only a representative selection. However, if the length of a single record does not fall within the spectral gap, then a more detailed examination of the stationarity of individual records is required in order to justify their use.

2.8.3 Stationarity of the ensemble

In practice it is necessary to select data which were recorded during similar wind conditions in order to achieve a stationary set. So, by labelling the response data according to the wind speed and direction pertaining at the time, and then recalling data recorded during similar
wind conditions, a stationary set should be formed. The question is how narrow do selected ranges of wind speed and direction have to be in order to be sure that a stationary set has been used? As these samples are not from a continuous process, and therefore the order of the samples is immaterial; a run test, or even a reverse arrangements test is inappropriate. Therefore some other test for stationarity must be used.

Although not a positive test for stationarity, the following practical test is suggested. Assuming that the data are stationary, then spectral analysis is valid, and the results will be accurate to within the known bias and variance errors. Choose ranges of wind speed and direction. Then take the category of wind speed and direction which contains the most records, say 600. Split the 600 records up at random into 6 sub-categories each containing 100 records, and therefore ±10% variance error. If the response of a particular mode in the resulting spectra with small bias error varies by no more than ±10%, then it is a good indication that the data are indeed stationary. If not, then either the data are non-stationary, or some other factor is causing the unexpected variation. Indeed, Clough and Penzien define a stationary process as one in which all ensemble averages of examples of the process are independent of time t (ref 77, p443).

It may be necessary to choose the sub-categories with some care so that the mean values of wind speed and direction within each sub-category are as close together as possible as this could affect the result. Ideally this exercise should be repeated several times using different ranges and different data to establish how wide the ranges can be whilst still producing stationary sets.

It should be noted that where the damping value is known only from the results of the spectral analysis, then there is some doubt about the true extent of bias error and such an approach should only be employed with caution. Also, it is necessary to be sure that the wind data relate to the response data, and this is usually done by having wind data recorded on the same building whose response is being measured.
2.9 SHEFFIELD UNIVERSITY ARTS TOWER

2.9.1 Description of the building

A BRE experiment on the Sheffield University Arts Tower took place from 1976 to 1982. The Arts Tower is situated within the University site near the city centre. The building is approximately 78m high, 36m wide east-west, and 20m deep north-south. It stands on sloping ground and is exposed to the north and east but is partially sheltered to the south and west by other University buildings of up to eight storeys. Structurally, the Arts Tower consists of a cast insitu reinforced concrete core with deep concrete floor slabs spanning between the core and external reinforced concrete columns.

2.9.2 Forced vibration tests

In March 1976, a forced vibration test of the building was carried out by CEBTP. The fundamental frequencies of the north-south, torsional and east-west modes were 0.68, 0.79 and 0.86 Hz respectively, all having damping values of about 0.9% critical. The mode shapes and stiffnesses for these modes were also obtained. Further forced vibration tests were carried out by BRE in February 1979 and July 1987. The 1987 tests are described in detail in reference 78. The BRE vibrator system (which is described in detail in section 4.6) has a much better accuracy than the one used by CEBTP in 1976. It can also be operated over a wide range of force levels, so gaining information about the non-linear response of the building. However, a fault in the 1979 tests meant that it was not possible to be certain that all four vibrators were locked together in phase. Thus stiffness and hence modal mass values could not be obtained. The fault was soon rectified, but it was not until some eight years later that the final forced vibration test was performed. It should be stressed that the values obtained in this third test do not differ very much from those obtained in the first test. However, they do show how the frequency and damping differ with increasing force levels.
2.9.3 Spectral analysis considerations

Having obtained the natural frequencies and damping values of the fundamental modes from the CEBTP force vibration tests, the HPB for each of these modes could be calculated. The lowest of these had a value of 0.0117 Hz. A standard record length of 1024 seconds (17 minutes) was chosen for the early analysis (refs 79,80,81). This gave almost 12 spectral lines in the HPB and a bias error of less than 0.2%. However, after the experiment was terminated, it was decided to re-analyse all the data using a standard record length of 512 seconds. This 8.5 minute period was chosen as it doubled the number of records whilst, hopefully, kept each record self-stationary, as the record length was almost within the spectral gap. With 6 spectral lines in the HPB this gave a bias error of less than 1% for each mode. Unless specified otherwise, the following comments refer to this later analysis using a record length of 8.5 minutes.

2.9.4 Wind response measurements

To monitor the wind, a cup anemometer was mounted on a 6m mast located on the roof of the building and the wind speed and direction recorded on a chart recorder and also on an FM tape recorder. The response of the building to the wind was monitored using four fixed accelerometers which were placed on the 19th floor of the building, one in the centre of each face. Each measured motion parallel to the face by which it was mounted. The accelerometers and signal conditioning equipment were calibrated on site at the start of the experiment, and thereafter every two years.

The response of the building was monitored between April 1976 and December 1982. Recordings were started manually whenever strong winds were blowing or were forecast, and the recordings were usually of about twelve and a half hours duration. In all over 500 hours of data were recorded.

Some electronic drift was encountered in the tape recordings of the anemometer response. Although this did not effect the recorded response of the accelerometers, this meant that the wind speed had to be extracted from the chart recordings. The mean wind speed during each 512 second record was extracted to the nearest 0.5m/s. A wind tunnel study carried out by
Evans and Lee of Sheffield University (ref 82) showed that the anemometer was within the region where the air flow was affected by the building. The study also suggested correction factors, ranging from 0.875 to 1.29, for each ten degree sector of wind direction. The appropriate factor was applied to each of the wind speeds extracted from the chart recordings.

The wind tunnel study also showed that the directional information from this anemometer was unlikely to be reliable. Therefore an anemometer sited 31m above street level and 130m west-north-west of the Arts Tower was used. However the data from this anemometer was only available in the form of hourly means to the nearest 10 degrees.

2.9.5 Analysis procedure

The autospectral density of each of the four accelerometer channels was calculated and stored for each 512 second record. Each record was labelled with the mean wind speed and direction pertaining when it was recorded. Then, all those records from a particular accelerometer which were recorded when the wind speed and direction were within a chosen range were recalled and averaged to give an ensemble average. Thirty degree sectors and intervals of 5m/s were chosen for the ranges as this offered a good compromise between choosing narrow enough limits to ensure that the data would be stationary, and broad enough limits to give sufficient records in each category to keep the variance error down.

2.9.6 Comparison with code predictions

References 79 and 80 both present rms accelerations (derived from spectral peaks) and the mean square modal force produced by the wind at each of the fundamental frequencies. These figures are presented for fourteen wind speed and direction cases. These comprise six different mean wind directions (ranging from 254 to 350 degrees) and up to four mean wind speeds (ranging from 7.0 to 13.3 m/s) for each direction. The wind speeds have been "reduced" from the anemometer readings at 84m to the standard height of 10m in open level country. Although it is mentioned that they have been derived from over 200 hours of data, there is no indication of how many hours worth of data each one represents. However, a warning is
given that "many of the spectral estimates still have confidence limits as wide as 30 per cent." These accelerations and modal forces are then compared to the figures predicted by the then current ESDU method for winds blowing normal to the faces of the building.

Several conclusions were drawn in these two papers. The modal forces for both the translational modes were a maximum when the wind was blowing onto the corner of the building. In contrast, the modal force for the torsional mode exhibited two maxima: one for wind blowing onto the narrow face and one for wind blowing onto the corner. The response in the torsional mode was of the same order as the north-south mode for all measured wind directions; the north-south modal force being about four times that of the east-west modal force. Lastly, the response predicted by ESDU tended to underestimate the east-west translational modal force when the wind blew onto the narrow face, and overestimate the north-south translational modal force for the wind blowing onto the broad face.

In reference 81, the same figures are repeated but a revised presentation is also included. These give the wind speed at 84m (i.e. as measured by the anemometer but with directional correction applied) and equivalent rms forces. No comparison with code predictions is made, but the results of the full-scale work are compared to results obtained with a model of the building in a wind tunnel. However it was concluded that the scatter in the full-scale results was too great for a proper comparison to be made.

No published work on the Arts Tower appeared for seven years after reference 81 was presented in 1981. The final Arts Tower paper (ref 83) contains no comparisons between full-scale and either model or predicted values. However, the rms accelerations for each of the three fundamental modes are given for 29 categories of wind speed and direction. As in reference 81, the wind speeds are as measured by the anemometer but with directional correction applied. In addition, the number of records which were averaged together to form the spectrum from which the accelerations were taken, is stated. This enables the reader to calculate the variance error for each category. The variability of the measured response was assessed by splitting the wind category which contained the largest number
of records into six, each containing just over 100 records. There was some difference in mean wind speeds between these six, but even when this was taken into account, the accelerations were still only within 40% of that for all 644 records. However, with over 100 records, the variance error should be no more than ±10%.

This paper also contains the results of the 1987 forced vibration tests, thus, given the dynamic characteristics of the building and the measured responses of the building to known wind speeds and directions, the results could be compared to any theoretical predictions. However, the problems in collecting the full-scale data were discussed, and the analysis procedure described in detail. In this way anyone wishing to compare theoretical predictions with the results of the experiment would have some idea of the reliance to be placed on such a comparison.
CHAPTER THREE:

ASSESSMENT OF FULL-SCALE TESTS CARRIED OUT TO DATE

3.1 INTRODUCTION

In this chapter, the major full-scale experiments are re-examined in the light of the discussion in Chapter Two. It might be useful for the reader to re-read the relevant section from Chapter One before reading the assessment for each experiment. At the end of the chapter what has come out of full-scale testing to date is discussed. Finally, the requirements for the ideal full-scale building experiment are set out, bearing in mind where previous experiments have not succeeded, and what such an experiment should be setting out to achieve.

3.2 RE-ASSESSMENT OF FULL-SCALE TESTS

3.2.1 Empire State Building

The details of the experimental work on the Empire State Building are given in section 1.3.2. The wind tunnel tests conducted by Dryden and Hill at the National Bureau of Standards (ref 10) showed that the velocity of the wind at the site of the anemometer was 23% higher than that of the approaching wind. As the anemometer was sited on a mast only 4.6m above the roof of a 381m tall building, it is hardly surprising that the anemometer readings were influenced by the presence of the building; although the effect is reduced to some extent because the Empire State reduces to a small plan area above the 86th floor.

The figure of a 10.1 inch (257mm) deflection of the Empire State Building to an 80mph (35.8m/s) wind has been much quoted in the literature. It is usually quoted as being from Rathbun's paper (ref 8), although it actually comes from Skinner's discussion on Rathbun's paper; Rathbun gives no figure for an "extreme deflection". As this figure has been so widely quoted, its derivation is important; particularly as the accuracy of this
figure is rarely mentioned. As was shown in section 1.3.2., the figure comes from two of Skinner's extrapolations, which are themselves based on sparse data. Rathbun admits that some of these data are useful for qualitative rather than quantitative observations. It should also be noted that this figure is for an indicated wind speed of 80mph (35.8m/s) which, based on the results of the wind tunnel tests, is equivalent to a true wind speed of 65mph (29.1m/s).

Davenport re-assessed the data from the Empire State Building and conducted a new boundary layer wind tunnel study on a model of the building using the base balance technique (ref 16). Davenport calculated the mean deflection of the plumb bob from the mean position under light wind conditions, and then plotted the deflection from this mean position against velocity squared for four similar wind directions. Although the scatter of the data points was large, the plots were fitted with straight lines. The slopes of these lines were then plotted against the wind azimuth. The base moment coefficients thus obtained from the plumb bob data compare well with those obtained from the boundary layer wind tunnel studies.

Davenport also replotted the collimator data but found no obvious variation with wind direction, thus making the relationship with windspeed difficult to interpret. However, by extrapolating from this replotted data, Davenport calculated that dynamic deflections of between 2.5 and 5.5 inches (64 to 140mm) would be obtained at a wind speed of 98mph (50m/s) as indicated by Rathbun's anemometer.

Whilst, for its time, Rathbun's work on the Empire State Building was a considerable achievement, no amount of re-analysis of the data can hide the fact that there are serious doubts about the accuracy of all the data collected. Unfortunately, this even extended to the manometers, as Davenport comments: "An analysis of the individual pressure readings suggests they are chequered by anomalous values. One has the suspicion that water leakage into the manometer tubes may have been a consistent problem." Davenport's re-analysis of the collimator data reinforces Rathbun's own view that this particular data should be treated with caution. Even when assessing the plumb bob data, which has a better accuracy, it should be borne in mind that only 101 observations were
published in Rathbun's paper. When these are split into different wind directions, it leaves very few observations to cover all the wind speeds from a particular direction. Therefore, even if the original data had a high accuracy, its sparsity should mean that any conclusions reached from the analysis of this data were, at best, subject to large possible errors.

3.2.2 University of Western Ontario Experiments

3.2.2.1 43 storey New York office tower

The details of the experimental work on the 43 storey New York office tower are given in section 1.4.2. Dobryn, Isyumov and Masciantonio's paper (ref 21) is unusual amongst paper's dealing with full-scale experimentation, in that the problem of achieving sufficient frequency resolution is discussed. However, only 27 hours of data was obtained, the conditions ranging from "relatively calm" to the passage of Hurricane Gloria. There is no attempt to analyse all 27 hours together as obviously the data would be far from stationary. Consequently only short periods of data can be analysed.

An example of an analysis of a 3 hour long record is given. The lowest frequency mode has a natural frequency of 0.141 Hz. If the relatively high damping value of 3% critical is assumed, then the half power bandwidth for this mode is 0.00846 Hz. Therefore, in order to obtain at least four spectral lines in the HPB, a recording of at least 473 seconds is required. If 10 records are averaged together, 79 minutes of data are required. Seventy-nine minutes of stationary data will yield bias and variance errors of -2.1% and ±32% respectively, providing the damping is at least 3% critical. Both of these error figures are unacceptably high where the estimation of damping is concerned. Acceptable figures would be -1% and ±10%, which would require over 18 hours of stationary data, even assuming that a 3% damping figure was correct, and over 56 hours if a 1% damping figure was assumed. Therefore, the damping values quoted in the paper (3.0 to 3.5%) may well be serious overestimates of the true damping values. It is interesting to note that when comparing the full-scale data with wind tunnel predictions, damping values of both 1% and 3% were used.
The authors recognised that not having a measure of the wind speed and direction in the vicinity of the building was a serious drawback. However, they were able to show that the measured natural frequencies of the building decreased at high wind speeds, and that, if wind direction was ignored, the response of the building varied approximately as the cube of the wind speed.

In 1988 a review paper by Isyumov, Masciantonio and Davenport was published (ref 19). The work on the 43 storey building was mentioned briefly, but the authors state that the previously published figure of around 3% critical damping was now considered to be an overestimate in view of the difficulties in obtaining sufficient frequency resolution.

3.2.2.2 Allied Bank Plaza

In the same review paper (ref 19) the authors discuss the damping values obtained from the Allied Bank Plaza, Houston. These values (of about 1.5% critical) were obtained from a record of only 94 minutes, during a Hurricane, for a building with a natural frequency somewhat lower (0.130 Hz) than that of the 43 storey New York office tower. Apart from the data almost certainly being non-stationary, the estimated wind speed increasing from 36 to 44m/s during the recording, the errors in the analysis are likely to be even greater than those of the 43 storey building.

However, the main comparison in the Allied Bank Plaza studies was between the peak accelerations measured in each 10 minute segment of the recording, and the wind tunnel tests. Eight of the ten 10 minute peak accelerations fell within the likely range. However, at the time of the measured maximum acceleration (46 millig) the likely range was approximately 24 to 49 millig. The two points which did not fall within the likely range were approximately 7 and 25% higher than the maximum predicted acceleration for that time. Although some allowance was made for changes in natural frequency with increasing wind speed when computing the likely range of accelerations, a damping value of 1.5% critical was assumed throughout. In the light of the preceding paragraph, this 1.5% figure is likely to be subject to a large error bound. As explained in Chapter Two,
damping is almost certainly going to be overestimated in spectral analysis, so 1.5% should perhaps be viewed as an upper limit of the true damping value.

In reference 25 the authors acknowledge that "the dynamic wind-induced response of a tall building with particular mass and stiffness properties is inversely proportional to the square root of the effective total damping". Therefore, if the damping were reduced to 0.95% critical, the upper bound for the likely range would be sufficiently high to include all ten peak accelerations. However, because of the uncertainties with the wind speeds, this should not be taken as an indication that the true damping figure is 0.95%, even if one believes that the wind tunnel modelling is able to estimate full-scale values accurately.

3.2.2.3 Other UWO tests

The unusual shape of the CN tower make it a far from ideal building for code comparisons. Indeed, the full-scale experimentation on it has only attempted to verify the dynamic characteristics assumed in design, although some interesting data concerning the nature of the boundary layer has also been obtained (ref 16). Nevertheless, it is worth noting that records of between 35 and 52 minutes duration would be required to produce a single autospectrum which contained four points in the HPB of the fundamental mode of the tower. Which of these two figures is required depends upon which end of the design range of damping values (0.5% to 0.75% critical) is assumed. So, to obtain a variance error of no more than ±10%, between 60 and 87 hours of stationary data would be required. Given that only just over 100 hours of time history data were recorded in total, and that only during storm conditions, it is evident that no accurate damping figures can be derived from this data.

In the case of each of the other buildings tested by UWO (section 1.4.2) relatively short lengths of data were recorded. Whilst these tests can determine the response of the building to the prevailing wind conditions, they are not accurate enough to be used to make any valid comparisons with theoretical predictions.
3.2.3 CEBTP Tests

The details of the experimental work on the 50m tower and the Maine-Montparnasse Tower are given in section 1.4.4. Paquet's papers are extremely interesting ones regarding the instrumentation of tall buildings for a long term study of wind loading. The CEBTP tests are the only ones of the non BRE experiments where a forced vibration test has been used to determine the dynamic characteristics of the structure. However, in the case of the 50m tower, as only preliminary examples of processing are given, no conclusions can be drawn from the results. Detailed analysis of the recordings from the CEBTP Tower may well have been neglected, on the basis that recordings from the Maine-Montparnasse Tower would soon be available instead.

The Maine-Montparnasse Tower experiment is a major piece of work which seems to have received scant attention, at least amongst the English speaking wind engineering community. However, a great deal of reliance seems to have been placed on the data from the taut wire displacement device, even though it was recognised that the data from this device were subject to significant errors. It is unfortunate that no attempt is made to look at the average response of the building to a particular wind condition. Although data were recorded digitally, this would probably have required transfer of the data to computer where the necessary analysis could be performed, but no such transfer of data to computer was undertaken. Instead, the paper indicates that because so much data was collected, a detailed analysis could only be justified for those few events where large displacements occurred. A further factor which may have influenced the decision not to perform any further analysis, is that it may have been realised that the wind measurements were inadequate (an anemometer on top of a 4.3m mast on a 229m building).

3.2.4 John Hancock Building, Boston

The details of the experimental work on this building are given in section 1.4.5. The data collected at the beginning and at the end of the experimental period, but for the same wind conditions, cannot be averaged together, as during the course of this period "the structural frame of the
building was modified, tuned mass dampers were added, and the building was occupied." Therefore, the amount of data which could be used in a long term study is limited to the largest amount collected while the structure of the building remained unaltered.

The pressure and window deflection data may well provide useful results, but the acceleration data is unlikely to provide anything other than a record of the level of acceleration at a particular moment, which can then be related to other data. Although similar criticisms can be made of the laser displacement data, analysis of it may well produce interesting results as this type of measurement is much rarer.

3.2.5 Commerce Court Tower

3.2.5.1 Determination of damping

The details of the experimental work on Commerce Court Tower are given in section 1.4.6. Commerce Court Tower and Sheffield University Arts Tower are by far the most important of the attempts to measure the response of a building to wind loading, and then to compare this with theoretical predictions. However, both of these experiments suffered from various drawbacks. In the case of Commerce Court Tower, the main problem was that no forced vibration test was carried out on the building. Therefore, even though natural frequencies and even mode shapes were obtained fairly easily from even short lengths of wind data, there was no idea of the damping values before the experiment started. As explained in Chapter Two, in order to obtain a specified accuracy in spectral analysis, a known number of spectral lines are required in the half power bandwidth of a particular mode. However, to determine the HPB, the frequency and damping of the mode in question must be known. The only way to get round this is to assume a very low damping value. Preliminary results from any wind study would give a damping value which is too high as damping is always overestimated in spectral analysis.

Even after the experimental work was completed, establishing the correct damping values for Commerce Court Tower seems to have been a major problem. Whereas the 1% figure mentioned in reference 50 seems reasonable,
the 3-4% given in reference 44 seems very high for this type of tall building. No mention is made in the latter paper that a much lower figure was obtained previously. Two expressions (for along and across-wind accelerations) are given in reference 44 based on equations from the then current Canadian building code. In both, the resulting acceleration is inversely proportional to the square root of the damping. Therefore if the original estimate of 1% is used instead of 3%, the calculated acceleration will be $\sqrt{3}$ (1.73) times higher. The size of this possible discrepancy must cast doubt on the validity of any comparisons drawn between the calculated values and those obtained at full-scale.

In any case, the choice of a 3% figure for damping to be used in the comparison with the code predictions, merely because this was the midway between the full-scale and model observations, is surprising. If the authors were confident of the 3-4% figure they quote for the full-scale observations, then a figure of 3.5% would have seemed more appropriate, although this would have decreased the predicted code values of acceleration by 8%. If the authors were far from confident about the damping figure that they used in the comparison, then it is surprising that they did not emphasise the effect that this uncertainty has upon the predicted values.

In Chapter Two (section 2.3.3) it was shown that large overestimates of damping can be obtained when insufficient resolution is used in spectral analysis. This may well account for the high damping figure obtained latterly at Commerce Court Tower, although no details are given as to how this figure was obtained. However, it should not be assumed that the 1% figure is correct either. It is possible that this too is an overestimate. The lack of a forced vibration test is most unfortunate as this would have established the correct damping values.

3.2.5.2 Statistical errors in spectral analysis

The main paper discussing the accelerations recorded at Commerce Court Tower (ref 44) was presented at the 6th International Conference on Wind Engineering, held in Australia in 1983. One of the two questions asked in the discussion period following the presentation was by Jeary (ref 44a).
"I notice that 5 and 10 minute mean data have been used. In conventional spectral analysis rather longer averaging times are necessary to reduce error bands. Could the author say how he has circumvented the statistical error problem." The author (which is not recorded) replied "The statistical error problem was not formally addressed in this paper. The 10 minute duration was ultimately chosen to reduce the impact of non-stationarity on the correlation which we sought to demonstrate between reference dynamic pressure and rms acceleration. It was frequently necessary to assign adjacent 10 minute averages to different 10 degree sections, and sometimes storms of short duration contributed useful data that would otherwise have been 'diluted' had longer averaging times been used. A more rigorous treatment might prove interesting particularly in the case of the long, severe storm, from the south-west direction."

In order to obtain at least four spectral lines in the HPB of the lowest frequency mode of Commerce Court Tower, a recording of 560 seconds is required, if we assume the lowest measured frequency of this mode (0.119 Hz) and 3% damping. Thus, if the true damping figure were the 1% first published, there would be less than 1.5 spectral lines in the HPB for a ten minute analysis period, and a bias error of -16.3%. The earlier 5 minute averaging times would produce even greater errors, and a period of 5 minutes is not usually considered to lie within the spectral gap.

3.2.5.3 Calibration of accelerometers

No mention is made in any of the Commerce Court Tower papers about the calibration of the accelerometers or other instrumentation. Past experience by BRE has shown that it is certainly worthwhile checking the calibration of accelerometers and their accompanying signal analysis instrumentation at regular intervals when they have been left in the field for as long as those at Commerce Court Tower.

3.2.5.4 Modal coupling

One of the other things that a forced vibration test would have established at the start of the experiment was the coupling between the translational and torsional modes. Such coupling may well be present in a
large population of buildings. However, it makes the response of Commerce Court Tower much more complicated and so far from ideal for a study where the response of the block is to be compared with the response predicted by wind loading codes.

3.2.5.5 Influence of surrounding buildings

In a building which is symmetrical about two vertical axes, the response to the wind from a complete quadrant could be argued to be enough for this type of experiment, the remainder being inferred from symmetry. The lack of symmetry about the east-west axis of Commerce Court Tower means that, by the same argument, two complete quadrants are required. Of course, for this argument to be used, the terrain must be identical for any inferring of symmetry to be valid. Sixteen 10-degree segments contained sufficient data to be analysed in reference 44. Half of these are an 80 degree segment from the west and south-west, while two further groups of four cover 40 degree segments about the north and east faces. Unfortunately, the segment containing northerly winds includes the direction for which Commerce Court Tower is directly in the lee of the 285m tall building, and this is quite evident in the plots for this direction. Presumably, any data collected before the completion of this latter building have been omitted from the analysis.

Even if a complete 180 degree segment had been obtained at Commerce Court Tower, it is doubtful whether any assumption regarding the symmetry of the response there could be justified. Indeed, it is unfortunate that the 80 degree segment from which half of the results come runs from south-west and west. Yet Dalgliesh says that the building "is partially sheltered from southwest to northwest by buildings from 175 to 285m in height" (ref 45).

3.2.5.6 Overview

In view of all the comments above, the agreement between the full-scale, model-scale and predicted response of Commerce Court Tower should be treated with caution. Given the length of the discussion above, it should not be assumed that the Commerce Court Tower experiment was
conducted badly. Indeed, it is because it was well carried out that it deserves a thorough assessment, and it certainly cannot be dismissed in a few sentences. It should also be mentioned that the researchers did not choose not to conduct a forced vibration test on the building. Had a suitable system been available, and permission to test the building been granted, they would have conducted such a test (ref 84).

3.2.6 Sheffield University Arts Tower

The details of the experimental work on this building are given in section 2.9. Three problems were identified by Littler et al in their paper on the Arts Tower experiment (ref 83).

Firstly, the anemometer on the building was within the region affected by the airflow over the building. Although wind speeds could be corrected using information from the wind tunnel study (ref 82), it would have been better if a much taller mast had been used. Had reliable wind speed and direction data been recorded on the tape a much more detailed study could have been undertaken. Inevitable inaccuracies occurred because hourly mean wind direction data from another anemometer had to be combined with data extracted from chart recordings, and then matched to a particular period of accelerometer data recorded on tape. However, these inaccuracies are very difficult to quantify.

Secondly, although 500 hours of data were recorded, it was still not enough to enable sufficiently narrow limits of wind speed and direction to be used in the ensemble averaging. The necessary compromise of choosing too wide limits on the wind speeds had a significant effect on the accuracy of the results which were obtained.

Thirdly, although the Arts Tower was a convenient building to use for the study, the local topography and surrounding buildings were far from perfect. The effects of these environmental factors were shown in the wind tunnel study, but the Arts Tower is not the building in flat open countryside which would be ideal for this experiment.
One problem with the Arts Tower experiment which was not explicitly mentioned in this paper was the relatively large torsional response of the building. It was to minimise the torsional response relative to the translational response that the accelerometers were positioned at the mid point of each face rather than at the corners of the building.

3.2.7 72m tall building in Las Vegas

The details of the experimental work on this building are given in section 1.4.7. The published results have concentrated on the dominance of the quasi-static displacement. However, only a 200 second time history of both the displacement and the differential pressure is given in support of this conclusion. Whilst there are similarities between the two time histories, there are also some significant discrepancies in both magnitude and sign. It is unfortunate that no correlation analysis is given between this displacement and either the pressure or wind speed which were also obtained from the building. The authors recognised the problem in having the anemometer so close to the roof of the building, and this may explain their reluctance to correlate the wind speed and quasi-static displacement.

Four other important points should also be remembered when assessing the importance of the results from this building. Firstly, only three storms of, at most, a few hours duration were analysed in full. Secondly, the reference zero was obtained from a "calm" period within 1.5 hours of the beginning of each storm. Whilst this prevents any analysis relating to the absolute displacement of the building to different storms, it also suggests that there were problems with drift in the laser signal. If so, it is possible that this drift might have been significant during the course of a single storm. Thirdly, although the authors say that tilting of the foundation was not a problem, this possibility should not be dismissed lightly. Fourthly, no temperature measurements were made, so the possibility of some of the displacement being caused by differential temperature effects cannot be dismissed.

Despite these drawbacks, the results from the tests on this building are important as they demonstrate that, at least for some buildings, the quasi-static response is at least as important as the dynamic response.
Therefore, any experiment which tries to measure the total response of a tall building to wind loading must consider both components.

3.2.8 Other full-scale testing

All the other tests mentioned in Chapter One fall into the same category as the UWO tests. Relatively short lengths of data were recorded, so these tests can only be used to determine the response of the building to the wind conditions at the time of the testing. However, even these results are likely to be subject to unacceptably high variance and bias errors.

3.3 WHAT HAS COME OUT OF FULL-SCALE TESTING TO DATE?

Few full-scale experiments have been set up with the objective of collecting sufficient data to be able to validate theoretical wind loading codes. What few experiments there have been, have been discussed in some detail in the preceding chapters. It is only by examining them in detail that their problems can be understood, as all have been conducted by competent scientists and engineers. If nothing else, the preceding chapters should give the reader some idea of how difficult it is to conduct such a full-scale experiment.

Whilst the experiments where relatively short lengths of data were collected may appear to be glossed over, they are still to be welcomed as there is such a dearth of full-scale experimentation. Natural frequencies can be obtained easily from these tests and compared with the values used in design, although the damping values obtained in these tests often have large errors associated with them.

What is often forgotten in these short duration tests is that the wind is a random process and variance error will exist in any analysis carried out on the response of a tall building to wind loading. Consider a situation where a ten minute recording is made of a building's response to a steady measured wind. Consider also that the accelerometer measuring the response effectively only measures the response in one mode of vibration so
that no spectral analysis is considered necessary. In most cases, the rms acceleration would be quoted as occurring at specified mean wind speed and direction. Whilst this is quite acceptable, if the same mean wind speed and direction occur in the next or any other ten minute period, it does not mean that the response will be the same. It is only by taking many samples, each having the same mean wind speed and direction, and then averaging them together that an accurate response level can be established.

Perhaps the most important conclusion from Chapter Two is that if the level of response of a particular mode is required to a high degree of accuracy, then very long lengths of stationary data are required. Unfortunately, several of the long term experiments have been conducted on exceptionally tall and therefore prestigious buildings. The taller the building, the lower its fundamental natural frequency is likely to be (ref 85). The Empire State Building, Allied Bank Plaza and Commerce Court Tower all have fundamental natural frequencies of less than 0.15 Hz. It was shown in Chapter Two that, in order to achieve the same bias error in spectral analysis, six times as much data must be acquired for a mode of 0.15 Hz as for one of 0.90 Hz, assuming that they have the same damping. In practice, the damping for the 0.15 Hz mode is unlikely to be more than for the 0.90 Hz mode and may well be less, so six times could well be increased to ten times more data required.

Large amounts of data are required from these tests. Not only is the storage of this data a problem, but also the data has to be analysed at some stage. The more data that is collected, the longer the analysis will take. This is a further argument against the collection of data from pressure transducers, as to make the use of them worthwhile a large number must be used, and they must be sampled many times per second, at say 30 to 40 Hz. This is considerably faster than is required for the other instrumentation, but, for simplicity, it is likely that all the instruments would have to be sampled at the fastest sampling rate.

Data from full-scale experiments have been able to confirm a number of points about the behaviour of real tall buildings. The following are among the most important. Fundamental modes predominate in determining the overall response to wind loading. The response in the fundamental mode
increases in proportion to the wind speed raised to a power of between 2.5 and 3.5. Whilst codes calculate response in terms of a series of pure, unrelated modes, the situation is often much more complicated on a real building, where some form of interaction between modes is common. In percentage terms, large differences in damping, and to a lesser extent frequency, can occur at different levels of excitation.

3.4 REQUIREMENTS FOR THE IDEAL FULL-SCALE BUILDING EXPERIMENT

Ideally, full-scale experiments should be carried out on a large number of buildings to see what results are true in all cases, and so can be applied to any building, and which are specific to a particular building or site. However, if resources only permit a single experiment, then it is important to ensure that any results obtained from the selected building are applicable to as many similar buildings as possible. Therefore, all factors which might be unique to that particular location must be eliminated. For this reason the building should be located on flat ground and well away from any other tall buildings. In this way unique effects which might be produced by the terrain or the wake effects from other buildings are eliminated.

The chosen building should be rectangular in plan as most wind loading codes use this as the simplest shape. There is little point in looking at complicated shapes until the simplest shapes have been verified. The height of the building is also an important factor. The building must be tall enough to be classified as a tall building, and to rise well above the surrounding buildings. On the other hand, the taller the building, the lower its fundamental natural frequency is likely to be (ref 85). As shown in Chapter Two, the lower the product of a building's fundamental natural frequency and damping, the more data is required to produce the same statistical accuracy in a Fourier analysis of its response to wind loading.

The building should be in a position where winds are equally likely from any direction, and it should experience a wide range of wind speeds. If the building is symmetrical then the requirement for winds from any direction can be eased.
The wind impinging on the building must be measured by anemometers that are not in a position where the air flow is affected by the presence of the building. The best way of establishing this is to use an anemometer in a wind tunnel, siting the anemometer precisely where it is proposed the full-scale one should be above a model of the building. If the measured wind speed is the same with and without the model of the building, then the proposed siting of the full-scale anemometer can be considered satisfactory.

The building should have fundamental translational modes which are well separated in frequency from each other and from any torsional modes. The dynamic characteristics of these modes should be measured at several levels of amplitude by a forced vibration test, so that the frequency, damping, mode shape and stiffness of each mode can be determined accurately.

Using the results from the forced vibration testing, the HPB of each mode can be calculated. The spectral resolution, and therefore the time taken to acquire a single record can then be calculated as set out in section 2.7, items 4 to 7; with the objective of aiming for an overall accuracy of better than ±10%.

Installation of suitably calibrated anemometers and response measuring devices can then take place and recording of data begin. The response measuring devices should be capable of measuring both the dynamic and the quasi-static response of the building. In addition, the temperature of each face of the building should be measured, so that displacements due to differential heating can be differentiated from displacements caused by wind loading. Checks should be carried out on all the instrumentation at frequent regular intervals to check that it is functioning correctly.

As a vast amount of data will be required to be recorded and then analysed, it seems sensible for the data to be recorded directly onto the computer which will carry out the analysis. Then, during relatively calm periods, analysis of data recorded previously can take place. Alternatively, at regular intervals, data recorded at the computer on site can be transferred to another computer dedicated to the analysis. The
analysis of at least some of the data should be carried as soon as possible after the experiment has started, so that any necessary adjustments to the data collection procedure can be carried out in the light of problems or interesting points shown up by the initial analysis.
CHAPTER FOUR:

DESCRIPTION OF HUME POINT AND FORCED VIBRATION TESTING

4.1 INTRODUCTION

In this chapter, the building selected for the long term wind loading experiment, Hume Point, is described. Details of the procedure for the forced vibration testing of the building are given. The results from these tests are discussed, with particular emphasis on their accuracy. Finally, a comparison is made between Hume Point and the ideal building for a long term experiment detailed at the end of Chapter Three.

4.2 DESCRIPTION OF HUME POINT

The selected building for the experiment described in the remainder of this thesis, Hume Point (figure 4.1), is a 23 storey unoccupied building one mile north of the River Thames in the London Borough of Newham (British national grid reference TQ410815). The building, like the eight similar blocks that were built on the same (Freemasons) estate, is a large panel structure with concrete load bearing walls and floors and lightweight concrete cladding. The building is of a Larsen-Nielsen type of design, a system initiated in Denmark in 1948, which, by 1968, had been used in 12 countries in Europe and South America (ref 86, p6). However, the nine blocks on the estate were considerably higher than anything else that had been built using the system previously. Consequently the structural proposals for the blocks were referred to Larsen and Nielsen for approval before they were built. Construction of the first of the blocks began in the summer of 1966.

The best known of the nine blocks built on the estate is Ronan Point, which, in May 1968, suffered a progressive collapse of the south-east corner of the building after a gas explosion occurred on the 18th floor (ref 86, pp12-27). A consequence of this explosion was that the joints of the existing buildings (type A) were strengthened to resist an explosion of
equivalent static pressure of 2.5 p.s.i. (16.2 kN/m²) using bolted steel angles to strengthen the connections between load bearing walls and floors. All mains gas supplies were removed from the buildings and Ronan Point was reconstructed using insitu construction.

Not all of the blocks had been constructed when the Ronan Point collapse occurred. Consequently, when three of the blocks were built, they included revised joint details to provide for a static pressure of 5 p.s.i. These blocks, of which Hume Point is one, are known as type B blocks.

Hume Point, like the other eight blocks on the estate, has plan dimensions of 23.7m by 17.9m and an overall height of 66.9m (figure 4.2). The building is founded on a heavy concrete podium which is in turn supported by numerous piles which are driven into alluvial deposits. The ground floor consists of an entrance lobby and various service areas, whilst the first floor has communal rooms for the tenants. Figure 4.2 shows that the entrance to the building is set back from the south face, the ground and first floors being replaced by concrete beams and columns. This can be seen in figure 4.3, which is a photograph of the south and east faces of the building.

Above the podium there are five flats on each floor, three of these being single bedroom flats and the other two having two bedrooms. The effect of this is to give a slightly asymmetric pattern to the distribution of shear walls. Figure 4.4 shows a typical floor plan. The building's principal axes are aligned exactly north-south and east-west, so that a wind blowing directly onto the north face comes from grid north (0 degrees). All subsequent references to direction at Hume Point are relative to grid north, but this coincides with the building axes so a wind direction of 90 degrees represents a wind blowing directly onto the east face etc.
Figure 4.1: North and West Faces of Hume Point
Figure 4.2  Hume Point Elevations
Figure 4.4  Hume Point: Typical floor layout
Figure 4.3: South and East Faces of Hume Point
4.3 THE TERRAIN SURROUNDING HUME POINT

Figure 4.5 shows a map of the surrounding area and indicates the position of the eight other blocks on the estate. The nearest, Bauckham Point is about 320m away from Hume Point, its north end being on a bearing of 263 degrees. Gannon Point is about 470m away from Hume Point, its south end being on a bearing of 237 degrees. All the other blocks on the estate lie between these bearings, two of them less than 400m away. Figures 4.6 and 4.7 are photographs showing the relationship between Hume Point and the other blocks. Figure 4.6 is taken from the north-east of Hume Point (which is the block on the left) and shows the park to the north of the building. Figure 4.7 is taken from the south-east of Hume Point (which is the building on the right). Ronan Point is the block on the left of figure 4.7. At the time this photograph was taken it was being dismantled. This process was completed before the long term wind loading experiment was started.

Figures 4.8 to 4.15 are photographs taken from the roof of Hume Point. They are, respectively, the view looking north, and then every 45 degrees, round to the view looking north-east. Ferrier Point, which is not one of the blocks on the estate, but is of similar height and width, lies about 700m north-east of Hume Point. It can be seen in the centre of figure 4.15 and also in figure 4.7, although it is partially obscured by Hume Point in the latter. The City of London is just visible on the skyline of figure 4.14, which also shows the other seven blocks on the estate and the partly dismantled Ronan Point. Figure 4.16 is a photograph of Hume Point from the west, and shows that, from this direction, the two storey housing continues almost right up to the building.

4.4 AMBIENT VIBRATION TESTS

Ambient vibration tests were carried out on Hume Point and the eight other similar blocks nearby (ref 74). These tests were carried out in order to assess the variations in dynamic characteristics that could occur between nine almost identical buildings. Typically a 16 hour recording was made overnight. A run test was used to assess whether or not each recording
Figure 4.5: Map of area surrounding Hume Point

Reproduced from Ordnance Survey Map with the permission of the Controller of Her Majesty's Stationery Office. Crown Copyright Reserved.
Figure 4.6: Freemasons Estate from the North-East

Figure 4.7: Freemasons Estate from the South-East
Figure 4.8: View from Hume Point looking North

Figure 4.9: View from Hume Point looking North-East
Figure 4.10: View from Hume Point looking East

Figure 4.11: View from Hume Point looking South-East
Figure 4.12: View from Hume Point looking South

Figure 4.13: View from Hume Point looking South-West
Figure 4.14: View from Hume Point looking West

Figure 4.15: View from Hume Point looking North-West
Figure 4.16: Hume Point from the West showing low rise housing
contained stationarity data. None of the recordings gave stationary data, but spectral analysis was carried out on the recorded data to obtain the natural frequencies of the lower frequency modes. Despite the use of non-stationary data, this method of analysis will yield accurate natural frequencies for this type of structure as the changes in frequency with amplitude of response are small. There was an average difference in the fundamental frequencies of over 21% between two of the blocks, although the frequencies of all the blocks were above those calculated using the predictors given below. The results for Hume Point were at the higher end of this spread of frequencies, that is Hume Point (and the other type B buildings) were amongst the stiffest.

The ambient vibration testing of Hume Point was carried out on the 12/13 February 1986. Figure 4.17 shows two of the power spectra obtained from spectral analysis of the recorded data. During the recording the mean hourly wind speed varied between 4.6 and 7.2 m/s (average 6.0 m/s); whilst the mean hourly wind direction varied between 50 and 80 degrees (average 70 degrees). This wind data (ref 87) is from the London Weather Centre which is 10 km west of Hume Point. The results of the test are given in Table 4.1. It should be noted that the frequencies obtained from the ambient vibration studies are higher than those obtained in the forced vibration tests. This is simply a consequence of the non-linear characteristics of the modes; the ambient vibration results correspond to smaller amplitudes of motion.

It should be noted that the ambient test on Hume Point referred to above should not be confused with the long term wind loading experiment on the building which started in 1987 (reference 88) and is the subject of this thesis.
Figure 4.17: Power Spectra from ambient vibration test 12/13 February 1986
TABLE 4.1: PREDICTED AND MEASURED VALUES OF HUME POINT NATURAL FREQUENCIES

<table>
<thead>
<tr>
<th>MODE</th>
<th>PREDICTED VALUES</th>
<th>MEASURED VALUES</th>
</tr>
</thead>
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<tr>
<td></td>
<td>Predictor Correlation Prediction</td>
<td>Forced</td>
</tr>
<tr>
<td></td>
<td>Coefficient   (Hz)</td>
<td>-max (Hz)</td>
</tr>
<tr>
<td>EV1</td>
<td>46/H 0.883 0.69</td>
<td>0.906</td>
</tr>
<tr>
<td>NS1</td>
<td>58/H 0.838 0.87</td>
<td>1.104</td>
</tr>
<tr>
<td>θ1</td>
<td>72/H 0.657 1.08</td>
<td>1.247</td>
</tr>
<tr>
<td>EV2</td>
<td>- - - -</td>
<td>3.928</td>
</tr>
<tr>
<td>NS2</td>
<td>- - - -</td>
<td>4.420</td>
</tr>
<tr>
<td>θ2</td>
<td>- - - -</td>
<td>4.291</td>
</tr>
</tbody>
</table>

Notes: Predictors and Correlation Coefficients are from reference 85. H is height of building in metres (66.9 in the case of Hume Point).

Max and min refer to the maximum and minimum levels of force applied for that particular mode (see Tables 4.2 and 4.3).

Ambient is from a single 16 hour recording carried out on the 12/13 February 1986 (see text for details).

4.5 THE RATIONALE BEHIND FORCED VIBRATION TESTING

In order to assess the response of a tall building to wind loading, the design engineer needs to estimate the characteristics of the building as well as the wind forces acting upon it. The dynamic characteristics of an existing building can be measured by subjecting the building to a forced vibration test. The comparison between the calculated and measured dynamic
characteristics of a number of buildings provides a means to learn more about the dynamic response of tall buildings in general, and thereby improve the methods used to predict dynamic characteristics of new buildings at the design stage. Since 1977, BRE has carried out such tests on fifteen tall buildings. The results from these tests, when compared with the design predictions for the same buildings, have shown that there are considerable inaccuracies in the methods used to determine dynamic characteristics at the design stage (ref 66).

It is assumed that the dynamic response of a structure can be described by the superposition of a series of similar equations, each of which describes the behaviour of a single mode of vibration. This technique forms the basis for most dynamic design methods.

The modal superposition method describes the modal response for mode r, in the following way:

\[ \ddot{X}_r + 4 \pi f_r \zeta_r \dot{X}_r + 4 \pi^2 f_r^2 X_r = \frac{F_r}{M_r} \]  
(Eq 4.1)

where \( f_r \) is the natural frequency, \( \zeta_r \) the modal damping ratio, \( M_r \) the modal mass and \( F_r \) the modal force for mode r. \( X_r \) is the displacement and \( \dot{X}_r \) and \( \ddot{X}_r \) the velocity and acceleration respectively.

If the mode shape \( \phi(x,y,z) \) is normalised to unity at the largest displacement, then Lagrange's equation (ref 77, pp 273-287) can be used to show that:

\[ M_r = \int \int \int m(x,y,z) \phi^2(x,y,z) \, dx,dy,dz \]  
(Eq 4.2)

where \( L, W, \) and \( H \) are the length, width and height of the building and \( m(x,y,z) \) is the mass of that volume defined by \( dx,dy,dz. \)
It is often assumed that $x$ and $y$ are constants, and so the equation reduces to:

$$
H
$$

$$
M_r = \int_0^z m(z) \delta^2(z) \, dz \quad \text{(Eq 4.3)}
$$

where $m(z)$ is the mass per unit height of the building. Therefore, measurements of the modal mass and of the mode shape allow the total mass of the structure to be calculated.

Thus the measurement of natural frequency, damping ratio, and the mode shape define all the necessary dynamic parameters, since the modal mass can be calculated using equation 4.3 once $f_r$, $\zeta_r$ and the response have been measured for a particular value of $F_r$.

4.6 ECCENTRIC MASS VIBRATOR SYSTEM

In 1977, the University of Bristol, under contract to BRE, designed and built the BRE eccentric mass vibrator system (figure 4.18). Several minor alterations have been made to the system over the last ten years, but its principal features remain unchanged. The system consists of four eccentric mass exciters, each with its own slave control unit, which are driven by a master control unit. The required frequency of the system is dialled on the master control unit where a crystal oscillator provides the dialled frequency to an accuracy of one part in a million. Servo-system techniques control the frequency of the exciters to a stability of 0.001 Hz (i.e. the frequency of the exciters is $\pm 0.001$ Hz of the dialled frequency). The system can be operated from 0.1 to 10 Hz in steps of 0.001 Hz.

The exciters each consist essentially of two motor-driven sets of contra rotating weights. The horizontal sinusoidal force produced by the system can be varied by changing the frequency of rotation, or by changing the rotating mass on each exciter. Different weights, which range from 3.7 kg per set to 145 kg per set, can be attached to the exciters. With the
maximum number of weights attached, the system can be operated up to 3.16Hz, and is capable of producing a force of up to 8.4 tonnes peak to peak. As the force produced by the exciters is proportional to the square of the frequency of rotation, the maximum force that can be produced by the system at 1 Hz is 0.84 tonnes peak to peak. The force produced by the exciters has been calibrated using a load cell and is well within the ±3% of the original specification.

The exciters are mounted on a graduated steel ring, which, in turn, is attached to the structure. This arrangement allows the exciters to be turned to apply force to the structure in any horizontal direction. Any number of the four exciters can be switched to have a 180 degree phase difference from the control signal. This phase difference, which can be introduced whilst the exciters are in motion, is required, for example, to excite torsional modes in buildings.

Several safety features have been incorporated in the design of the system. These include trips on the master control which the operator has to make a conscious effort to override in order to go above a certain frequency. Although not shown in figure 4.18, cages are positioned over the exciters when testing is in progress. These cages, which completely cover the exciters, are 1.0m square and 0.6m high, and are primarily to stop people or equipment accidentally coming into contact with the moving parts of the exciter. A video camera is positioned to monitor each exciter. The pictures from all four cameras can be seen by the operator at the control centre. Each slave unit has an emergency stop button so that an exciter can be stopped by someone next to it, as well as by the operator in the control centre.

4.7 GENERAL FORCED VIBRATION TEST PROCEDURE

The following sequence is that followed for a forced vibration test on a tall building with rectangular plan form and was used at Hume Point:
Figure 4.18: One of the vibrators and its control equipment

Figure 4.19: Equipment in the control room at Hume Point
1. The four exciters are located as close as practical to where it is considered they will produce the greatest response per unit force (in both translational and torsional modes). This will normally mean that the exciters will be positioned at the four corners of the top floor.

2. The exciters are aligned parallel to one of the axes of the building, and an accelerometer situated at a suitable reference location is used to monitor the response in that direction. The frequency of excitation is incremented from 0.5 Hz to 10 Hz, and at each increment the amplitude of response is noted.

3. Natural frequencies are identified by peaks in the resulting frequency/response curve. The process of obtaining frequency/response curves is known as frequency sweeping. Once a peak has been identified, a more detailed study of the response around the resonance peak is made and the measurements fed into a computer. A curve fitting program is used to find the fundamental natural frequency, and associated values of modal damping and stiffness.

4. The exciters are then set to operate at the natural frequency. A second accelerometer is taken in turn to various locations throughout the building, and the amplitude of response and phase (relative to the response of the reference accelerometer) is measured.

5. The signals from the reference accelerometer are recorded as the oscillation decays when the exciters are switched off simultaneously. Modal damping values are calculated from the decay.

6. Weights are added or removed and (3), (4) and (5) repeated for the same mode but at different amplitudes of motion.
7. Once the mode has been investigated at all the amplitudes of interest, the exciters are turned through 90 degrees and the whole procedure repeated for the fundamental orthogonal mode.

8. The procedure is then repeated for the fundamental torsional mode. This involves setting two exciters at one end of the building to operate in anti-phase relative to those at the other end of the building, so applying a torque to it. Once the building is being excited at the fundamental torsional natural frequency, the second accelerometer is moved around one of the upper floors to determine the centre of torsion.

9. The exciters are then returned to their original orientation and steps 3 to 8 inclusive are repeated for all other resonance peaks (other than the fundamental modes) identified in the initial frequency/response curves.

Whilst it may appear simpler to find the dynamic characteristics of all the modes in one direction before the orientation of the exciters is changed; it is more important to obtain the dynamic characteristics of all the fundamental modes as a first step, as these dominate the overall response. This procedure is followed so that the information is gathered in order of priority, in case of, for example, illness or mechanical breakdown forcing the premature curtailment of a test.

Where the building does not have a simple rectangular plan shape, the exciters should be positioned initially parallel to an arbitrarily chosen axis and the reference accelerometer aligned parallel to the same axis. A frequency/response curve is then produced and a natural frequency identified from it. The exciters are set to this frequency and the response monitored as the reference accelerometer is incrementally rotated. The direction of minimum response may then be established and the exciters orientated orthogonally to this direction.
The direction of minimum response is used as this is much easier to establish than the direction of maximum response because the response varies as a cosine function. Consequently a deviation of 10° away from the direction of maximum response will lead to a reduction in response of only about 1.5%. However, a deviation of 10° away from the minimum response will increase the response from nothing to about 25% of that at the maximum. The situation can be complicated by the presence of other modes at nearby frequencies, but this method worked well the only time it has been used by BRE (at Leicester University Engineering Tower, ref 76). Once the direction of maximum response has been established, the subsequent procedure is then identical to that set out above for rectangular plan forms.

4.8 FORCED VIBRATION TESTS ON HUME POINT

4.8.1 Presentation of results

The testing described in this paper was performed by a three man team from BRE over thirteen days from 7 to 23 July 1986. The four exciters were located on the 23rd floor of Hume Point, one in each corner of the building. The centre of each vibrator was positioned 1.1m ±15cm from each of the two walls which met in that corner (figure 4.4). A control centre (figure 4.19) where all the exciters and the accelerometers used to measure the response were monitored, was set up on the 23rd floor midway along the western face of the building, in the one flat on this floor which did not contain a vibrator.

The characteristics of the modes found at Hume Point are given in Tables 4.2 and 4.3. All the responses in these tables were measured on the 23rd floor. In every case the response in the torsional modes was measured in the east-west direction. The response in the \( \theta_1 \) mode was measured in the NW corner; the response in the \( \theta_2 \) and \( \theta_3 \) modes was measured at the north end of the corridor. The modal properties have been calculated using the frequency and damping values from the curve fitting and the acceleration at the frequency of maximum response.
## Table 4.2: Results of Forced Vibration Tests at Hume Point - Fundamental Modes

<table>
<thead>
<tr>
<th>Mode</th>
<th>Freq. (Hz)</th>
<th>Force (kN)</th>
<th>Accel. (m/s²)</th>
<th>Disp. (mm)</th>
<th>Freq. (Hz)</th>
<th>Damping (% crit)</th>
<th>Damping (% crit)</th>
<th>Mass (x10^6kg)</th>
<th>Stiffness (MN/m)</th>
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<td>EV1</td>
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<td>2.7</td>
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<th>Freq. (Hz)</th>
<th>Torque (kNm)</th>
<th>Accel. (rad/s²)</th>
<th>Disp. (x10^3rads)</th>
<th>Freq. (Hz)</th>
<th>Damping (% crit)</th>
<th>Damping (% crit)</th>
<th>Inertia (x10^6Kgm²/rad)</th>
<th>Stiffness (GNm/rad)</th>
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<td>1.259</td>
<td>1.50</td>
<td>–</td>
<td>301</td>
</tr>
<tr>
<td></td>
<td>1.26</td>
<td>54.5</td>
<td>0.0064</td>
<td>0.10</td>
<td>1.267</td>
<td>1.43</td>
<td>–</td>
<td>299</td>
</tr>
<tr>
<td></td>
<td>1.27</td>
<td>27.7</td>
<td>0.0037</td>
<td>0.058</td>
<td>1.280</td>
<td>1.26</td>
<td>1.15</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>1.29</td>
<td>14.3</td>
<td>0.0022</td>
<td>0.033</td>
<td>1.299</td>
<td>1.11</td>
<td>1.02</td>
<td>297</td>
</tr>
</tbody>
</table>

(Perpendicular distance from centre of torsion to accelerometer position was 12.44m)
(Perpendicular distance from centre of torsion to average vibrator position was 10.375m)
### TABLE 4.3: RESULTS OF FORCED VIBRATION TESTS AT HUME POINT - HIGHER ORDER MODES

**MAXIMUM RESPONSE MEASUREMENTS (P-P)**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Freq. (Hz)</th>
<th>Force (kN)</th>
<th>Accel. (m/s²)</th>
<th>Disp. (mm)</th>
<th>Freq. Damping (Hz)</th>
<th>Damping (% crit)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EW2</td>
<td>3.93</td>
<td>51.1</td>
<td>0.18</td>
<td>0.29</td>
<td>3.928</td>
<td>3.94</td>
</tr>
<tr>
<td></td>
<td>3.95</td>
<td>25.8</td>
<td>0.088</td>
<td>0.14</td>
<td>3.953</td>
<td>3.77</td>
</tr>
<tr>
<td></td>
<td>3.99</td>
<td>13.2</td>
<td>0.047</td>
<td>0.075</td>
<td>3.976</td>
<td>3.61</td>
</tr>
<tr>
<td></td>
<td>4.00</td>
<td>6.6</td>
<td>0.023</td>
<td>0.037</td>
<td>3.996</td>
<td>3.31</td>
</tr>
<tr>
<td>NS2</td>
<td>4.41</td>
<td>64.3</td>
<td>0.16</td>
<td>0.21</td>
<td>4.420</td>
<td>4.24</td>
</tr>
<tr>
<td></td>
<td>4.44</td>
<td>32.6</td>
<td>0.082</td>
<td>0.11</td>
<td>4.452</td>
<td>4.48</td>
</tr>
<tr>
<td></td>
<td>4.46</td>
<td>16.5</td>
<td>0.042</td>
<td>0.054</td>
<td>4.485</td>
<td>4.60</td>
</tr>
<tr>
<td></td>
<td>4.49</td>
<td>8.3</td>
<td>0.021</td>
<td>0.026</td>
<td>4.516</td>
<td>4.71</td>
</tr>
<tr>
<td>NS3</td>
<td>8.6</td>
<td>61.2</td>
<td>0.059</td>
<td>0.020</td>
<td>8.693</td>
<td>5.65</td>
</tr>
<tr>
<td>EW3</td>
<td>9.0</td>
<td>67.0</td>
<td>0.048</td>
<td>0.015</td>
<td>8.988</td>
<td>7.17</td>
</tr>
</tbody>
</table>

**CURVE FITTING**

<p>| Freq. Torque Accel. Disp. Freq. Damping |</p>
<table>
<thead>
<tr>
<th>(Hz) (kNm) (x10⁻³ rad/s²) (x10⁻⁶ radians) (Hz) (% crit)</th>
</tr>
</thead>
<tbody>
<tr>
<td>02</td>
</tr>
<tr>
<td>4.35</td>
</tr>
<tr>
<td>4.38</td>
</tr>
<tr>
<td>4.41</td>
</tr>
<tr>
<td>03</td>
</tr>
</tbody>
</table>
Modal Mass, $M_r$, has been calculated using the formula:

$$M_r = \frac{F_r \cdot a^2}{2 \bar{a} \zeta_r} \quad \text{(Eq 4.4)}$$

where, for the relevant mode, $F_r$ is the applied force, $\bar{a}$ is the response (expressed as an acceleration), $\zeta_r$ is the ratio of actual damping to critical damping, and $a^2$ is a mode shape factor. (As all the responses were measured on the 23rd floor, if the mode shapes are normalised to the 23rd floor then this factor becomes unity for all modes and therefore disappears from the calculations). When calculating the Modal Inertia, $I_r$, Torque is used instead of force, and angular acceleration instead of linear acceleration.

Modal Stiffness, $K_r$, is derived from the Modal Mass using the formula:

$$K_r = 4 \pi^2 f_r^2 M_r \quad \text{(Eq 4.5)}$$

where $f_r$ is the natural frequency of the relevant mode

(For the torsional mode, the same formula is used but with $I_r$ replacing $M_r$).

Figure 4.20 shows the response of the building in the east-west direction as the frequency of excitation was incremented from 0.5 to 10 Hz. It should be noted that the normalised displacement (displacement per unit force) is plotted on a logarithmic scale and that the peak response in the fundamental mode is over a hundred times that of the second order mode, whilst that of the third order mode is about another order of magnitude further reduced. A similar difference in response levels between modes was also observed in the north-south and torsional modes (figures 4.21 and 4.22 respectively) and is typical of the results obtained by BRE from other buildings. For clarity, some of the points around the second and third order modes have been omitted in figures 4.20 to 4.22. However, all the points were used in all the calculations involving these modes.

Elevation mode shapes and basic plan mode shapes for the fundamental modes are presented in figure 4.23, and detailed plan mode shapes for the fundamental modes in figure 4.24. The elevation and plan mode shapes for
Figure 4.20: Hume Point East-West response
Figure 4.21: Hume Point North-South response

Log displacement per unit force (m/N)
Figure 4.22: Hume Point Torsional response
Figure 4.23: Hume Point mode shapes for fundamental modes.
Figure 4.24  Hume Point plan mode shapes (23rd floor) for fundamental modes

EW1 0.905 Hz

NS1 1.105 Hz

θ1 1.260 Hz
Figure 4.25: Hume Point mode shapes for 2nd order modes

- EW2 3.95 Hz
  - SE corner
  - N end of corridor
  - 23rd floor
  - 11th floor

- NS2 4.30 Hz
  - N end of corridor
  - Average of NW and SE corners - EW direction

- 62.440 Hz
  - Average of NW and SE corners - EW direction
Figure 4.26
Hume Point plan mode shapes (23rd floor) for second order modes

EW2 3.95 Hz

92 4.40 Hz

NS2 4.35 Hz
Figure 4.27  Hume Point mode shapes for 3rd order modes
the second order modes are presented in figures 4.25 and 4.26 whilst those for the third order modes are given in figures 4.27 and 4.28. All the elevation mode shapes have been normalised to the value where the largest response was measured. It should be noted that the response on the roof was measured only for the EW3 and NS3 modes; no measurements were taken on the roof for the other modes. All the detailed plan mode shapes are for the 23rd floor except for the aforementioned EW3 and NS3 modes where the plan mode shape for the roof is given. Although several measurements were made on the 23rd floor away from the external walls when assessing the plan mode shapes, these have been omitted from the detailed plan mode shape for the sake of clarity as in every case they merely confirmed the overall mode shape as measured at the faces of the building.

Figure 4.29 shows the results of fitting a single degree-of-freedom curve to the measured responses obtained around the NS1 mode for the lowest of the five levels of amplitude that were investigated. These particular data are those that were obtained with the lowest force level and demonstrate that, even at these low force levels, there is no evidence of interference from the wind induced vibration of the building. The best fit curve follows the experimental data points very well, thus giving confidence in the values obtained from it.

4.8.2 Damping Values

Figure 4.30 shows the decay of oscillation which occurred when all four vibrators were switched off simultaneously, the vibrators having been exciting the building at resonance in the NS1 mode. The lower part of the figure shows a plot of cycle number against the logarithm of maximum response in that cycle, and the damping can be obtained by fitting a line through these points. As mentioned previously, the damping values obtained from Hume Point, like those from other buildings investigated by BRE, tend to increase with increasing amplitude of motion. Indeed, for some buildings, a plot of cycle number against maximum response during an extended decay of oscillation has shown a distinct change of slope throughout the decay (ref 76). Where large changes in damping values with amplitude are noted within a decay, it is important that the damping value in the first few cycles is quoted, together with the corresponding
Natural frequency 1.121 Hz
Damping 0.97% critical

Figure 4.29 Hume Point curve fitting around the NSI mode
4 vibrators, 1/2 set weights
Figure 4.30  Hume Point decay of oscillation for NS1 mode
4 vibrators, 1/2 set weights
amplitude of motion. However the changes in damping with amplitude are not large at Hume Point; figure 4.30 shows only a slight change of slope at the end of the decay.

The damping value obtained by this decay of oscillation method shown in figure 4.30 is directly comparable to that obtained by the curve fitting method used in figure 4.29. Both are for the NS1 mode being excited by all four vibrators with the same weights, thus producing the same force levels. The values of damping obtained by these two methods for all the fundamental modes are given in Table 4.2. In the case highlighted above, the discrepancy between the damping values obtained by the two methods is 8%. In the majority of the cases the discrepancy is less than this, and in all cases it is less than 14%.

The fact that there is no systematic difference between the damping values obtained from the two methods supports the supposition that the change in damping value within a decay is small. Determining damping values from the relevant mode in the frequency/response curve avoids the problem of changing damping values; but the information obtained from taking decays is slightly different. Of the two methods, damping values taken from decays of fundamental modes are less prone to possible experimental errors by inexperienced researchers. The two methods, though totally separate, are complementary and, where possible, both should be used. Any differences between the results obtained by the two methods which cannot be explained easily as modal interference etc. will show any weaknesses in experimental technique; whereas similar results from the two methods will add to the confidence that can be placed in the results.

4.9 DISCUSSION OF RESULTS OF VIBRATION TESTS

4.9.1 Predicted and Measured values of Natural Frequency

Using the predictor $f = \frac{46}{H}$ (where $f$ is the frequency of the lowest frequency mode and $H$ is the height of the building in metres) the lowest frequency mode for Hume Point would be predicted as being 0.69 Hz. This predictor was proposed by Ellis (ref 85) and was found by correlating the
details of 163 rectangular plan buildings with the actually measured frequencies. Whilst this predictor (correlation coefficient $r = 0.883$) was found to be more accurate than any other, it was warned that large errors could still be obtained by using it. This formula is now included in ESDU 88019 (ref 89) but without the warning. Table 4.1 shows that, as the actual figure for this mode is over 0.9 Hz, Hume Point is, for its height, considerably stiffer than average. In fact, in general, large panel structures tend to be very stiff structures.

Table 4.1 also shows how similar predictors (again from reference 85) underestimate the frequencies of the NS1 and $\theta$1 mode. However, it should be noted that the torsional mode predictor in particular has a much lower correlation coefficient than the predictor for the lowest frequency mode. Whilst these predictors are derived from, and therefore offer only a comparison to, the 163 buildings in the survey, they are a useful tool. Their simplicity has the advantage that it warns the user that an accurate result is not expected, whereas a more complicated formula might give the illusion of greater precision.

4.9.2 Non-linear response

Tables 4.2 and 4.3 show that both the resonance frequency and damping of each mode vary with amplitude of motion. The general trend in all the modes investigated at Hume Point is for the resonance frequency (and hence stiffness) to decrease and for the damping to increase as the amplitude of the exciting force is increased. However, investigating this phenomenon with a vibrator system which only has a stability of one hundredth of a Hertz would be very difficult. In both of the fundamental translational modes, an almost tenfold increase in the applied force produces a reduction in frequency of less than 20 thousandths of a Hertz, as can be seen from Table 4.2.

Similar behaviour has been observed in many other tall buildings (ref 76). In the field of wind engineering, this behaviour may appear to be insignificant, as the changes in frequency and damping are small even at the amplitudes used in design; but it becomes much more important in earthquake engineering, although changes in mechanism may also take place
at higher amplitude levels. However, it is not normally practical, or possible, to conduct a forced vibration test on a tall building at the same amplitude level as that caused by the largest earthquake the building is designed to withstand.

4.9.3 Temporal variations in natural frequency

On one day of the test programme the response in the region of the EW1 mode was investigated several times. This took place over a period of several hours and in sunny, dry but overcast, and cool and drizzly conditions. However, the peak response occurred at the same frequency on each occasion; but this was 5 thousandths of a Hertz lower than the peak response when this mode was investigated initially, some eight days earlier. It is preferable for a mode shape to be measured immediately after the resonance frequency has been identified from the frequency/response curve. However, this is not always possible; and when there is a break between the two it is wise to check the response in the region of the mode to ensure that the mode shape is measured at the natural frequency, even though this might be a slightly different one to the frequency of peak response noted in the initial frequency/response curve. For example, this explains why, although all three mode shapes for the fundamental modes were carried out with the maximum number of weights attached, the frequencies for two of them differ from the frequencies given for the maximum force level in Table 4.2. The frequencies for the NS1 and @1 mode shapes are, respectively, 5 and 20 thousandths of a Hertz higher than the frequencies in Table 4.2. For the higher order modes these differences, where they occur, are of the order of a few hundredths of a Hertz. The maximum variation being the 9 hundredths of a Hertz of the 02 mode. Similarly, the difference in frequency between the plan and elevation mode shapes for the NS2 mode is due to them being carried out on different days.

4.9.4 Mode Shapes

It is interesting to note that the elevation mode shapes for the fundamental modes are not quite straight lines. Whereas the standard mode shapes in ESDU 88019 are either straight lines, convex or concave, the Hume
Point mode shapes are none of these. The bottom third of the mode shape is concave (the response is less than a straight line would give) whilst the top of the NS1 and 01 mode shapes are convex. In the case of the NS1 mode there is a transitional section where the mode shape is approximately a straight line, but no such transition occurs in the 01 mode shape. In the EW1 mode the mode shape above the ninth floor is approximately a straight line without any convex section. It is likely that the insitu concrete podium at the base of Hume Point, which is then surmounted by the precast panels, contributes to this behaviour.

4.9.5 Modal Coupling

Modal analysis, which is also used in most design methods, assumes that the dynamic response of a structure can be described by the superposition of a series of similar equations, each of which describes the behaviour of a single mode of vibration. However, in practice, some degree of coupling between modes is often found. Table 4.2 and figure 4.24 show that the three fundamental modes of Hume Point are fairly well separated although there is some torsional component in the NS1 mode. However, there is a great deal of modal interference in the second order modes as can be seen in figure 4.26. Whilst figure 4.28 shows reasonably pure plan mode shapes for the third order translational modes, the response in these modes is much smaller than that in the 03 mode, as can be seen in figures 4.20 and 4.21. It was for this reason that the NS3 and EW3 plan mode shapes were mapped out on the roof where the response was greater.

4.9.6 Centre of Torsion

The centre of torsion appears to be in a different position for each of the three torsional modes. The following centres of torsion were found on the 23rd floor and are quoted relative to the geometric centre of the building. It should be remembered that the building is not symmetrical about the north-south axis and that the geometric centre of the building is some 700mm to the west of the centre line of the corridor. For the 01 mode the centre of torsion is 0.8m south and 2.2m west of the geometric centre. For the 02 mode the figures are 3.0m south and 0.2m west of the geometric centre; and for the 03 mode 3.6m north and 1.4m east of the geometric
centre. However, the 02 and 03 modes are certainly more complicated than the 01 mode because of the coupling with translational modes. There were two sources of interference in the higher order modes. As well as translational modes having torsional components, where two modes are close together in frequency it is very difficult to excite one mode without exciting the other. If it had been possible to excite the higher order torsional modes without so much modal interference from translational modes, different results might have been obtained.

During the tests carried out by BRE on Ronan Point (ref 90) similar modal coupling occurred between the NS1 and 01 modes. The vibrators were orientated and operated perpendicular to an imaginary line drawn from each one to the estimated centre of torsion. In this way the amount of interference from the translational mode was minimised. Consequently, a different centre of torsion was found to the one which had been obtained with the vibrators in line with the building. This technique was not used at Hume Point as the modal interference was confined to the higher order, and therefore less important modes. The interference was also considerably less than at Ronan Point, which had particularly asymmetric properties because one corner of it was rebuilt of insitu rather than precast panels following the collapse there (refs 90 and 91).

4.9.7 Soil-Structure Interaction

A small amount of motion was noted at the base of the building. The amount of base movement relative to the movement at the top of the building increased with increasing order (or stiffness) of the modes, and this probably represents a significant contribution to the increased damping noted in the higher frequency modes. For the fundamental modes, the amount of movement at the ground floor as a percentage of that at the 23rd floor is between 2% and 4%. For the second order modes the figures are between 7% and 15%; whilst for the third order modes the figures are between 11% and 20%. The effects of this base motion were evaluated for each fundamental mode using the method described in reference 92. In the analysis it was assumed that the vertical mode shapes are straight lines (see figure 4.23) and the mass (and inertia) is uniform and evenly distributed. The fixed base frequencies were derived using the Dunkerley
approximation (ref 92). The results of this analysis on Hume Point are given in Table 4.4. The largest effect noted was that for the NS1 mode, where the base interaction was estimated to have produced a 23.3% reduction in the frequency (relative to the "rigid base" frequency) and accounted for 42.8% of the energy dissipated. The effects of the interaction on the fundamental torsional mode given in table 4.4, are likely to be significant underestimates because a large amount of foundation rocking was noted, and this is not allowed for in the calculations.

Some Rayleigh wave measurements were attempted. However, the response in the far field (more than a few metres away from the building) were small and hence prone to error. The response in the near field was larger, but the interpretation of these measurements so close to the building is far from straightforward. Consequently none of the Rayleigh wave measurements yielded any useable information.

TABLE 4.4: SOIL-STRUCTURE INTERACTION MEASUREMENTS AT HUME POINT

<table>
<thead>
<tr>
<th>Mode</th>
<th>Freq. (Hz)</th>
<th>Damp. (%)</th>
<th>Base Trans. (N/m x 10^4)</th>
<th>Base Rotat. (Nm x 10^12)</th>
<th>Base Tors. (Nm x 10^12)</th>
<th>% Reduction in freq.</th>
<th>% Energy lost in foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>EW1</td>
<td>0.905</td>
<td>0.98</td>
<td>6.83</td>
<td>1.65</td>
<td>-</td>
<td>21.0</td>
<td>39.3</td>
</tr>
<tr>
<td>NS1</td>
<td>1.105</td>
<td>1.20</td>
<td>7.48</td>
<td>2.00</td>
<td>-</td>
<td>23.3</td>
<td>42.8</td>
</tr>
<tr>
<td>01</td>
<td>1.260</td>
<td>1.60</td>
<td>-</td>
<td>-</td>
<td>1.46</td>
<td>0.64</td>
<td>2.9</td>
</tr>
</tbody>
</table>

4.9.8 Higher Order Modes

Although in both the fundamental and second order modes the EW mode has the lowest frequency and the torsional mode the highest frequency; the situation is reversed for the third order modes. Similar changes in modal order have been noted in previous tests by BRE (ref 78).
The ratios between the frequencies of fundamental, second, and third order modes will depend on the relative flexural and shear response of the modes in question. At Hume Point, the \( \Theta_1 \) mode is higher in frequency than either the NS1 or EW1 modes, the \( \Theta_2 \) mode is lower in frequency than the NS2 mode but not the EW2 mode, and the \( \Theta_3 \) mode is lower in frequency than either the NS3 or EW3 modes. In addition, for the fundamental and second order modes, the EW mode is at a lower frequency than the NS mode, but the position is reversed for the third order modes. Thus the ratios between the second and fundamental, and between the third and fundamental natural frequencies, are highest for the east-west modes and lowest for the torsional modes. A uniform cantilever in pure shear has much lower ratios between its natural frequencies than one acting in pure bending. Therefore, the east-west modes can be seen to have the smallest component of shear in them, and the torsional modes the most.

In the NS2 mode it appears that the damping actually decreases with increasing amplitude of motion, but no real significance should be attached to these particular results as there was considerable interference from the \( \Theta_2 \) mode. The errors in calculating the fundamental resonance frequency at the design stage may easily be \( \pm 50\% \), and the errors for calculating the response in this mode may be \( \pm 240\% \) (ref 93). Similar, if not greater errors can be expected for higher order responses. However, as figures 4.20 to 4.22 show, the fundamental mode dominates the overall response of the building. Therefore, at the design stage, using more than just the fundamental modes in the overall response calculation may well give a false idea of the precision of the calculation. For this reason, although the behaviour of Hume Point in the higher modes is of interest, it appears to be of little importance as far as the immediate goal of improving design methods is concerned.

### 4.10 ACCURACY OF RESULTS

The values of modal mass, stiffness and inertia given in Table 4.2 have been calculated using the damping and frequency values from the curve fitting. The variation of these quantities within the same mode but at different amplitudes does not exceed 10\% and is an indication of the
accuracy of the other parameters.

In 1981, Jeary and Ellis (ref 93) suggested that, based on test results on an existing structure where the best possible information is used, the following accuracies would be appropriate: natural frequencies ±0.1%, damping ratios ±10% and modal mass ±20%. The results from Hume Point, at least for the fundamental modes, seem to be of about this accuracy. The apparent increase in the accuracy of estimating the modal mass figures for Hume Point is partly explained by the use of the damping figures from curve fitting rather than decays of oscillation in the calculations. However, Jeary (ref 94) has shown that a vibrator with a stability of only one hundredth of a Hertz, which is exciting a building with a natural frequency of 1 Hz and 1% critical damping, will cause the damping to be overestimated by about 20% due to this factor alone. Therefore, unless a vibrator which is stable to at least about one thousandth of a Hertz is used, the decay method is preferable.

The curve shown in figure 4.29 was calculated by using all the points shown, the weighting given to each point being proportional to its normalised displacement. This method is therefore less susceptible to giving erroneous values than one based on the peak response and calculating the damping from an estimate of the "half-power bandwidth" (the width of the resonance peak where the response is $1/\sqrt{2}$ that of the maximum amplitude). Clearly, the accuracy of this method depends upon obtaining a sufficient number of accurate measurements at discrete frequencies around the mode. In this particular example there are five measurements in the half-power bandwidth. Where there is no interference from other modes, as in this example, a good fit is obtained between the experimental values and the calculated curve. Where the fit is not so good, the problem is usually due to either not having enough data points or modal interference. For this reason, the curve fitting data for the higher order modes is generally less accurate than that for the fundamental modes as many of them are close in frequency. Probably the best test to determine the accuracy of the routine is to see how well the theoretical curve fits the experimental data. Where the damping value from the curve fitting is close to that obtained by the run-down method, more confidence can be placed in the result, but this cannot be done with higher order modes where the run-down
method cannot be used.

4.11 OTHER METHODS OF TESTING

One of the major problems with forced vibration testing of large structures is that it must be carried out whilst the structure is also being subjected to the normal ambient excitation sources. In the case of tall buildings, it must be remembered that the measured response is the sum of that produced by the forced vibrations and the wind. In steady-state testing, where the forced vibration testing is only applied at a single frequency, separating these two sources is not normally a problem. However, extended decays of oscillation, and the investigation of the response of the building to low frequency excitation (where little force is produced by the vibrators) should be conducted when the wind loading is as low as possible.

There are other methods of forcing a building to vibrate, such as suddenly releasing a highly tensioned cable or firing rockets attached to the top of the structure (ref 95). However, such methods are not well-suited to investigating the dynamic characteristics of tall buildings. The ease with which mode shapes can be investigated with steady-state testing is unmatched by any other method. Perhaps the most critical factor in comparing test methods is to consider the long time constants involved in structures, such as Hume Point, with damping of under 1% of critical in modes below 1Hz. Such considerations make steady-state testing the only practical method of forced vibration testing for tall buildings.

The alternative to forced vibration testing is to measure the response of a structure to ambient vibrations. Although this method avoids the rather artificial situation where a structure is excited at only one frequency at a time, there are, as seen in Chapter Two, considerable practical difficulties in using it on large structures. Many hours of stationary data (i.e. constant wind speed and direction) are required to resolve the fundamental modes to a similar degree of accuracy as that shown in figure 4.29. However, almost invariably, recordings of sufficient length will not yield stationary data.
Single non-stationary recordings of say, 16 hours, can yield information about the natural frequencies of a tall building, but damping values found from such analyses are prone to very large errors. The non-linear response of the building cannot be obtained from a single short recording and, as the force of the wind on the building cannot be measured directly, the modal mass and stiffness have to be determined by using a calculated value for the total mass of the building. Ambient vibration testing can be quicker and cheaper than steady-state forced vibration testing, but it cannot provide all the information that the latter can, nor is the information that it does provide as accurate as that provided by a well-conducted forced vibration test on a tall building.

Williams and Tsang used a random wide-band input to a hydraulic exciter to test a fire station drill tower (ref 96). In essence, this is similar to monitoring the response of the structure to wind excitation, but with the major advantage that the input force is known. By measuring the input force and the response, transfer functions can be calculated. However, in common with wind excitation, although they were able to obtain natural frequencies and mode shapes, they realised that the frequency resolution they were able to obtain in their resulting spectra was poor for the length of records they had.

4.12 HOW DOES HUME POINT COMPARE WITH THE "IDEAL BUILDING"?

At this stage, it is interesting to compare Hume Point to the ideal building for a full-scale experiment that was visualised in section 3.4. In plan, Hume Point is not quite the simple rectangle of the ideal building, as there are recesses on both the north and south faces at the ends of the corridor. In addition the east and west faces are not plain as there are balconies on both faces. However, the effect of these deviations from a simple rectangle are thought to be relatively small. The building is almost symmetrical about the east-west axis (at least from the second floor up) but not about the north-south axis. Indeed, the recesses at the ends of the corridor are not in the centre of the north and south faces.
Although they are all over 300m away from Hume Point (almost five building heights) it is unfortunate that the other blocks on the estate lie between the south-west and west of Hume Point, as this is the most common wind direction. At this point the reader may wonder why Abrahams Point was not chosen for the experiment instead, as, by early 1986, all the blocks on the estate were virtually empty and any one of them could have been made available for the experiment. Abrahams Point is 270m away from the site of Ronan Point and 330m away from Merrit Point, about the same distance that Hume Point is from Gannon Point. However, the other blocks lie to between the north-east and east of Abrahams Point, a much less common wind direction.

Four factors decided Hume Point rather than Abrahams Point. Firstly, in the ambient vibration tests, Abrahams Point was found to have much lower natural frequencies than the other eight blocks (about 15% lower than those at Hume Point). Secondly, the block was connected to an adjacent car park at ground floor level. This could have affected its dynamic response in an unusual way. Thirdly, the car park by Abrahams Point was, by early 1986, already suffering from acts of vandalism, whereas Hume Point, being close to a main road and shopping area was thought to be less susceptible to vandalism. The fourth and by far the most important reason was that Abrahams Point is a type A block whereas Hume Point is a type B (see section 4.2 for the difference between the two types). In early 1986, it was thought quite likely that all the type A buildings would be demolished in the near future, although such a fate would be unlikely to happen to the type B blocks.

Apart from the other blocks on the estate, the terrain surrounding Hume Point is flat and with fairly uniform two or three storey housing for some distance in all directions. There are some exceptions, notably the park areas to the north-west and north-east of the building, and the dockside warehouses to the south. However, these are not major drawbacks.

One important aspect of choosing a suitable tall building is obtaining permission from the owner to conduct the experiment. In this respect, Hume Point was ideal as Newham Council were quite willing for one of their blocks to be used. Also, it was thought that, being unoccupied, there was
less chance of instrumentation being disturbed, while cabling and instrumentation could be placed in stairwells and service ducts which could not be used if the building were occupied. Finally, conducting a forced vibration test on an empty building is much easier than conducting one with residents in it, although the latter is perfectly possible.

The forced vibration tests showed that the three fundamental modes were fairly well separated in frequency; although this confirmed what had been apparent since the ambient tests (see figure 4.17). The lowest natural frequency of Hume Point was over 0.9 Hz even though the building is 66.9 m high. This is some 30% higher than would have been predicted using the formula \( f = \frac{46}{H} \) (see section 4.9.1). Therefore Hume Point is a building which is sufficiently tall to have significant dynamic response; but which has high enough natural frequencies to make the acquisition of sufficient data to compare with theoretical codes a practical proposition.

All the other requirements for the ideal experiment refer to the instrumentation of the building rather than the building itself. One last point is that Hume Point is only about one hour's travelling time by road away from BRE. This meant that return trips to Hume Point could be made quite easily within a working day.
CHAPTER FIVE:

INSTRUMENTATION, DATA ACQUISITION AND INITIAL ANALYSIS

5.1 INTRODUCTION

In this chapter, the instrumentation used in the long term test at Hume Point is described. The instrumentation consisted of anemometers, accelerometers to measure the building's dynamic response, and a plumb line and tracking laser system to measure the quasi-static response of the building. In addition, thermistors were used to measure the temperature of each face of the building, to assess whether some of the quasi-static response was caused by differential heating. The chapter continues with details of the data acquisition system and the initial analysis performed on the data. Finally, a chronological history of the experiment is given.

5.2 INSTRUMENTATION

5.2.1 Anemometers

5.2.1.1 Description of anemometers

The wind velocity and turbulence intensity at Hume Point were measured by three Gill propeller anemometers, model 27106 (figure 5.1), manufactured by the R. M. Young Company of Traverse City, Michigan. These anemometers use a 180mm diameter polypropylene helicoid propeller which turns through one revolution for each 294 mm of passing wind. The propeller drives a miniature d. c. tachometer generator, so providing an analogue voltage output which is directly proportional to the wind speed. It will measure both forward and reverse air flows, responding only to that component of the wind which is parallel with its axis of rotation. The propeller response is a function of its orientation to the incident wind velocity vector and closely follows the cosine law. The propeller has a threshold wind speed of 0.2-0.4m/s, and a maximum operating wind speed of 50m/s. A 60mm propeller nose extension was used to reduce the stall angle and
improve the low speed response. With this fitted, the propeller has a stall region of about 2° either side of the incident wind blowing orthogonally to the axis of rotation.

5.2.1.2 Calibration of anemometers

Although it was noted above that the output from the anemometers closely follows a cosine response to off-axis winds, a correction factor has to be applied to allow for the response not having a perfect cosine form. Wind tunnel tests were conducted by Bowen and Teunissen (ref 97) to establish the correction factor for both horizontally and vertically orientated anemometers for wind from any direction. When the optimum set of correction factors is applied, they estimate that mean velocities are accurate to within about ±3%. Greater errors are experienced by the vertical component anemometer as this component normally experiences large incidence angles and continually reverses the direction of rotation through the stall region. Therefore, a modified set of correction factors was derived for the vertical component anemometer. All the tests conducted by Bowen and Teunissen were carried out using the earlier 27103 model anemometer. The basic difference between the two models being that the earlier one used a 75mm propeller extension, rather than the 60mm one used on the later model. Accordingly, it was thought necessary to check the calibration factors derived by Bowen and Teunissen to see whether they were affected by the change in the length of the propeller extension.

Tests were conducted in the boundary layer wind tunnel at BRE to calibrate the anemometers and determine the correct set of correction factors to use with them. A smooth flow was used in all the tests. Firstly, each of the three anemometers to be used at Hume Point was checked to ensure that its output was linear. Tests were conducted with wind speeds of up to 20m/s and with the anemometer at several angles to the direction of the wind flow. All the anemometers performed satisfactorily, and an overall calibration figure (m/s per volt) derived for each one. None of the measured values deviated from the overall calibration figure by more than 1%. 
Figure 5.1: 3-component Gill anemometer set

Figure 5.2: Calibrating the anemometers in the wind tunnel
For the second test, the anemometers were mounted on a bracket so that they measured the component of wind speed in the vertical and two orthogonal horizontal directions. The bracket, which was the one which would be used on top of the mast at Hume Point, was then placed on a turntable in the wind tunnel (fig 5.2). The output from both the horizontally mounted anemometers was then monitored as a steady wind of 15m/s was applied and the turntable was rotated through 360° in steps of 1°. Correction factors for both anemometers for all angles from 0° to 360° were determined. The average of the two values (one from each anemometer) for each angle was then used as the overall correction factor for each angle. By having only one correction factor for the two horizontal anemometers, the existing program which had been written to apply the correction factors could be utilised. Whilst this compromise inevitably introduced some minor errors, when the averaged set of correction factors was applied they were still able to determine true mean velocities to an accuracy of within ±3%.

The correction factors determined in the BRE wind tunnel were not very different to those obtained by Bowen and Teunisson. Unfortunately, there was no simple satisfactory way of checking the correction factors for the vertical anemometer. Therefore, the values obtained by Bowen and Teunisson were used for the vertical anemometer, and the BRE correction factors for the horizontal anemometers.

Towards the end of the experiment at Hume Point it became necessary to replace the anemometers when one suddenly failed. A careful check was made to ensure that none of the data collected before the failure of the anemometer was corrupted, and indeed this was the case. As none of the newer model anemometers were available, three of the older models, which had been tested previously in the BRE wind tunnel to obtain their calibration factors, were used. As these older anemometers had the 75mm propeller extensions, the horizontal correction factors used from then onwards were changed to those obtained by Bowen and Teunisson.
5.2.1.3 Siting of anemometers

A test was carried out by Dr. P. A. Blackmore in the boundary layer wind tunnel at BRE in order to determine how high the anemometers had to be above the roof of Hume Point in order to measure wind velocities that were unaffected by the building. A domestic housing simulation was used in the tunnel and the tunnel run at the scaled down equivalent of the design wind speed (38 m/s at 10m height). Figure 5.3 shows that the mean velocity of the shear layers shed from both the narrow and broad faces of the model converge to within 1.5% of the unobstructed velocity at a height of 20m above the parapet level. This 1.5% difference represents a wind speed of 0.6m/s at the design wind speed. Therefore, 20m above the parapet level was taken to be the minimum acceptable anemometer height in order that good estimates of the incident flow velocity were obtained. At anemometer heights below 20m, the velocity data would be subject to a correction factor.

Accordingly, a 21m telescopic mast (Clark model WT6) was used to site the anemometers on. Although telescopic, the mast is 4m long even when collapsed, too big to be brought up the inside of Hume Point. Consequently the mast had to be pulled up the outside of the building by a team of a dozen volunteers from BRE (figure 5.4). The base of the mast was secured in a specially built cage which was then bolted to the side of the plant room on top of Hume Point (figure 5.5). Figure 5.6 shows a plan and elevation of the roof and the position of the mast. The mast was mounted on the western side of the plant room as this was the side that it was expected that the majority of the wind records would be obtained from. Figure 5.7 shows an elevation of Hume Point and the mast.

The three Gill anemometers were mounted on the top of the mast, and a small compressor used to pump the mast up to its full height. After each section was pumped up clamping collars were attached so that the mast would remain at its full height even when the compressor was switched off and to ensure that no rotation of the mast was possible. Eight guys were attached to the mast; four at the top and four at mid height. The other ends of these guys were attached to anchorage points on the roof.
FIGURE 5.3 VELOCITY PROFILES MEASURED AT MODEL SCALE
WITH AND WITHOUT MODEL OF HUME POINT

- Profile without model
- Profile with model, wind onto broad face
- Profile with model, wind onto narrow face

Height of anemometers

Height of Hume Point to top of parapet

Mean Velocity ($\overline{V}/\overline{V}_{63.1}$)
Figure 5.6: Plan view and elevation of roof
Figure 5.4: The mast being hauled over the parapet

Figure 5.5: The mast being bolted to the plant room
Figure 5.7: Outline elevation of Hume Point showing position of anemometers and mast
Figure 5.8 shows the anemometers on top of the mast. The anemometers were aligned with the building's principal axes so that (from top to bottom) they were facing up, west and south. The heights above the roof of Hume Point were 23.2m, 22.8m and 22.7m respectively. This gave an average height for the two horizontal anemometers of 85.05m above ground level.

The signal from each anemometer was passed through a 10 Hz low pass filter before being fed into the computer. The output voltage from the anemometer was approximately 1 volt at a wind speed of 17.3m/s so no amplification was required, and the low pass filter was the only signal conditioning used.

5.2.2 Accelerometers

All the accelerometers used at Hume Point were ±2g linear servo accelerometers type A223 manufactured by Schaevitz-EM of Slough, Berkshire. These accelerometers use the closed loop torque balance principle and provide a voltage which is proportional to the applied acceleration. The accelerometers have overall dimensions of 66x34x29mm. Because of their small physical size and mass, the accelerometers are mounted on a one kilogram steel cube which is supported on three pointed feet. This assists in aligning the accelerometer parallel to the walls of a building and ensures that no rocking takes place.

The accelerometers were calibrated by attaching them to a clinometer and measuring their output as they were tilted by a small angle. This subjects the accelerometer to an acceleration of g multiplied by the sine of the angle of tilt. The output of each accelerometer was checked at several angles to ensure that the output was linear over the anticipated range of accelerations. Providing that a relatively calm day is chosen, this calibration exercise can be carried out in situ at the top of the building. This is a great advantage as the cabling and signal conditioning to be used for the experiment can be used for the calibration. This also makes a calibration check during the course of an experiment a fairly straightforward task.
Figure 5.8: The anemometers on the mast

Figure 5.9: Two accelerometers mounted on blocks
In all, fourteen accelerometers were used in the experiment at Hume Point. These fourteen were placed as seven orthogonal pairs, one of each pair measuring east-west motion, and the other north-south motion (figure 5.9). However, only five accelerometers were used for the first year of the experiment as only an eight channel data acquisition system was available (the other three channels being used for the three anemometers). All five accelerometers were placed on the 23rd floor. An orthogonal pair was placed in both the north-west and the south-east corners. The fifth accelerometer was placed in the south-west corner to measure north-south motion (see figure 5.10).

When the 32 channel data acquisition system was available, a further nine accelerometers were used. When all fourteen were in place there was an orthogonal pair at each corner of the 23rd floor, a pair at the centre of torsion on the 23rd floor (as determined for the 01 mode in the forced vibration test), and two pairs on the 11th floor, one in the north-west corner and one in the south-east. Figure 5.11 shows the positions of all fourteen accelerometers.

The signal conditioning used for each accelerometer was made by the BRE electronic workshops and is shown diagramatically in figure 5.12. Each signal was passed through a D.C. offset (to place the mean value of the signal at about zero volts), a 10 Hz low pass filter (to remove any high frequency signals which could lead to aliased data in spectra), amplified one hundred times, passed through a second 10 Hz low pass filter and a second D.C. offset before being fed into the computer.

The absolute value of the output from the accelerometers tends to be subject to long term drift, hence there is a potential need to adjust the D.C. offsets every few days. If the drift is all in one direction, there is the danger that if left too long, some or all of the acceleration signal will be beyond the permissible range of input voltages to the computer. At the start of the experiment a modem link was established so that the progress of the experiment could be monitored from BRE without having to visit Hume Point. By using the modem the accelerometer signals were checked about every other day so identifying when it was necessary for someone to go to Hume Point to adjust the D.C. offsets. At the start of
Figure 5.10  Position of 5 accelerometers used with 8 channel data acquisition system
Figure 5.11 Position of 14 accelerometers used with 32 channel data acquisition system
Figure 5.12: Accelerometer signal conditioning diagram
the second year of the experiment the second D.C. offset in the accelerometer signal conditioning was changed from a manual to a digital one. Thereafter, using the modem facility, the accelerometer signals were checked, and, if necessary, adjusted from BRE by telling the controlling computer to alter the digital D.C. offset for a particular channel.

In reference 98 Rainer gives a method for calculating the effect that rotation of the measurement position has on horizontal measurements of acceleration. The effect of rotation, expressed quantitatively as a fraction of the translational signal amplitude, is shown to depend inversely on the effective radius of rotation of the measurement location and the square of the frequency component of the signal. Rainer calculates the necessary correction that should be applied to data for the fundamental mode of Commerce Court Tower as 4.1%. The differences in mode shape and fundamental frequency between Commerce Court Tower and Hume Point result in the correction factor for Hume Point being more than two orders of magnitude less than that for Commerce Court Tower. Consequently, no correction has been applied to data from Hume Point to allow for any effects due to rotation of the measurement positions.

5.2.3 Plumb line and tracking lasers

At the start of the second year of the experiment a plumb line was installed in one of the refuse chutes at Hume Point. The position of the refuse chute is shown in figure 5.13. A 0.0025mm diameter nichrome wire was suspended from a ferrule set in a tapered seating on the 23rd floor, while a 2.1Kg weight was attached to the bottom of the wire 60.42m below. The weight, whose centre was approximately 100mm above ground level, was then placed in a bucket of oil to damp out any vibrations of the wire.

The formula for the calculation of the frequency \( f \) of an undamped pendulum for small angles is:

\[
f = \frac{1}{2\pi} \sqrt{\frac{g}{h}} \tag{Eq 5.1}
\]

where \( g \) is the acceleration due to gravity (9.81m/s\(^2\)) and \( h \) is the height of the pendulum in metres (ref 37).
Figure 5.13 Position of plumb line and thermistors
It can be seen that when, as in this example, $h=60\text{m}$, then $f=0.064\text{Hz}$. The frequency of a damped pendulum of the same height will be slightly less than this. However, the frequency of the pendulum is over an order of magnitude lower than the lowest natural frequency of the building (0.9 Hz) so movement at the natural frequency of the plumb line should not be induced by the dynamic response of the building.

Two lasers, made to order by Photometric Consultants of Dorking, Surrey, were used in the experiment at Hume Point. These lasers, known as photometric "vibrotrak" type 814/1, use a modulated beam of laser light to measure the vibration characteristics of a target. This is achieved by imparting a small conical scan to the beam and aiming at a small round disk of reflective paper or paint. Light reflected from the disk is detected and gives rise to signals enabling the conically scanned laser beam to follow or track the disk. Motion of the disk is therefore obtained by appropriate analysis of the signals.

A small cylinder covered in reflective paper was attached to the plumb line at the second floor level 53.26m below the top of the plumb line and 7.16m above the weight in the oil bath. The two lasers were set up on the second floor to measure the position of this cylinder and hence the plumb line. As access to the refuse shaft was only possible from one side, the lasers were positioned parallel to one another, but one beam was reflected in a mirror placed at 45° to the beam, so that the two beams measured the angular displacement of the cylinder in two orthogonal directions (north-south and east-west). As the lasers measure the angular displacement of the target, it is important to measure the distance from the lasers to the target. For this reason the lasers and the mirror were fixed onto an aluminium plate at BRE (figure 5.14).

Figure 5.15 is a double exposure photograph showing the target at two extreme positions and the two pairs of laser beams which mark its position. This figure also shows the cards used to calibrate the lasers. Each card contains two vertical lines beyond the extreme deflection positions of the plumb line. At the start of each record the laser locks onto each of these lines in turn and the voltage corresponding to this position is noted. As the position of these lines and the position of the laser does not alter,
the laser can be calibrated by knowing that these voltages correspond to known angles. To ensure that the laser locked onto the correct lines within a reasonable time, the laser was told to start searching for the lines from a set voltage which corresponded to the position of each line as found under laboratory conditions. Calibrating the laser in this way at the start of each record was necessary in order to eliminate any drift in the value of the output voltage, which would otherwise be interpreted as a movement of the plumb line.

After the calibration exercise, the laser beam returns to the position that the target was in previously. If the target is not in its previous position, the laser conducts an ever widening search for it until it is found. However, in practice, the plumb line was found almost instantly as it had rarely moved very far from the position it was in at the end of the previous record. Calibrating both lasers in this way and returning the beams until they were locked onto the target took sixteen seconds. The period of data acquisition did not start until the laser calibration had been completed.

Originally the idea had been to have six lasers. As well as the pair at the second floor, it had been planned to put further pairs on the ninth and sixteenth floors, so enabling the quasi-static mode shape to be monitored. However financial constraints, and the deteriorating security situation at Hume Point (see section 5.7) forced the cancellation of the order for the final four lasers.

5.2.4 Thermistors

Eight mini thermistors, type EU-U-V25, manufactured by Grant Instruments of Cambridge were used at Hume Point. These thermistors are specifically made for attaching to a flat surface and are made of epoxy-coated copper, with the sensor on the back of a disc. They are six millimetres in diameter and three millimetres high. Thermistors were used as they have a short response time and provide a larger electrical change for a given temperature than any other type of sensor. Also, the high electrical resistance of the sensor minimises the effect of contact resistance in plugs and sockets and allows long cables to be used without
Figure 5.14: The author with the tracking lasers
Figure 5.15: Double exposure of the laser beams
causing large errors. The maximum deviation for an individual sensor from
the theoretical resistance/temperature characteristics is 0.2°C over the
working range of the thermistor. However, typical deviations are half this
maximum or less. The thermistors are given a period of thermal cycling by
the manufacturer to ensure that they are stable over long periods in spite
of frequent and rapid thermal cycling. A typical drift of ±0.02°C in eight
years at 25°C is quoted by the manufacturers.

A power unit for driving the thermistors was built at BRE, and all
eight thermistors subjected to a calibration test where the output from
each one was monitored at a range of stable temperatures from -19°C to
+33°C. A single calibration was used for all eight thermistors, rather
than eight individual calibrations, one for each thermistor. This was
because the difference between any of the individual calibrations and the
overall one was less than 1% over the whole of the thermistor’s working
range. As temperature differences, rather than absolute temperatures, were
to be used at Hume Point, the extra complication of individual calibrations
was not considered worthwhile. The overall calibration used was that the
temperature, in °C, was 18.025 times the output voltage, less 38.48°C.

The thermistors were used to measure the temperature of each of the
four faces of Hume Point. Eight thermistors were used, four on the 16th
floor and four in identical positions on the 9th floor (figure 5.13). The
thermistors were attached to the interior faces of exterior walls at the
north and south ends of the building. However, as the exterior walls of
the east and west faces are cladding rather than structural panels, the
thermistors were attached to structural panels close to the exterior faces.
All the thermistors were stuck to the walls with Schnelkleboff adhesive,
and a 200x200x75mm block of expanded polystyrene stuck to the wall around
to ensure that the temperature measured was that of the wall rather than
the ambient temperature of the room. The thermistors were installed at the
start of the second year of the experiment when the 32 channel data
acquisition system became available.
5.3 DETERMINATION OF DATA ACQUISITION AND ANALYSIS PARAMETERS

The procedure set out in section 2.7 was used to determine the relevant parameters for the Hume Point experiment. A sampling interval of 0.0625 seconds (16 samples per second) was chosen, giving a Nyquist frequency of 8 Hz. This was high enough to obtain data on both the fundamental and second order modes of vibration but not the third. All the instrumentation was passed through 10Hz low pass filters to remove any high frequency components which might lead to aliased data in spectra.

The HPB of each of the fundamental modes of Hume Point was calculated using the least conservative data from the forced vibration tests. The lowest was found to be the EW1 mode which had a HPB of 0.014 Hz. To obtain twelve points in the HPB (bias error of -0.23%) the required resolution (B) is therefore 0.00117 Hz. This requires a record length (T) of 857 seconds. At 16 samples per second this gives 13,712 samples. This was increased to 16,384 samples, the next power of 2, for efficient spectral analysis. Thus T is 1024 seconds, so B is 0.000977Hz, giving over 14 points in the HPB of the EW1 mode, and a bias error of 0.16%. As the other modes have larger HPBs than that of the EW1 mode, the bias error in these modes will be less than 0.16%.

5.4 ACQUISITION OF DATA

In the preceeding section it was determined that each record would be 1024 seconds in length, and each sensor would be sampled 16 times per second. Whilst it was not necessary to sample the laser and thermistor data 16 times per second, it was simplest for all the sensors to be sampled at the same rate, even though this meant that the vast majority of the data obtained from the eight thermistors was superfluous.

Thus each record would contain 16,384 or $2^{14}$ readings for each channel. With 32 channels this is well over half a million readings for each record. As many thousands of records would have to be obtained in order to limit variance errors, the problem of how to store and process this amount of data was of prime importance.
The chosen storage medium was an optical disc, as a single disc can hold up to 2 gigabytes of information (the equivalent of about two thousand floppy discs) on each side. These discs work on the WORM system (Write Once Read Many times) in that data, once written to disc, cannot be deleted. Figure 5.16 shows the author with one of the optical discs. The Kenda optical disc drive was controlled by a Dyna Five LSI 11/73 computer. The computer ran a suite of FORTRAN programs, mainly written by Dr. B. R. Ellis of BRE to the author's specification, which controlled the data acquisition, storage and initial analysis. Figure 5.17 shows the set up in the control room when data from 32 channels was being collected.

The computer digitized the data by means of a 16-bit analogue-to-digital converter, ±10 volts being the range of permissible input voltages. This gave a resolution level of 0.000305 volts. Data from each instrumented channel was sampled 16 times per second. The channels were sampled sequentially, sampling of 32 channels taking approximately one thousandth of a second, so for all practical purposes sampling of all channels can be considered to be simultaneous. The data were multiplexed into a single file on the optical disc so that storage time could be kept to a minimum. After 1024 seconds of data (16,384 samples from each sensor) had been collected, the exact times of the data acquisition starting and stopping were added, and the file closed.

The name of each file incorporated the date and the record number for that date. All the multiplexed original data files are named HZddmmyn where dd is the day of the month, mm the number of the month, y the last digit of the year and nn the number of the file, thus file HZ2203939 was the 39th file recorded on 22nd March 1989. All files containing data extracted or analysed from this file retain the seven last digits to identify the source file, and merely have different prefixes. Thus HW2203939 contains the results of the wind analysis from the anemometer records for the same record mentioned above, HL... laser analysis, HT... thermistor analysis, H4... to H9... and HA... to HH... the results of the accelerometer analysis.
Collection of data was initiated whenever the mean wind speed exceeded 5m/s. This mean wind speed was calculated every 512 seconds until the 5m/s threshold was exceeded, when collection of data over a 1024 second record was initiated. Further records were then collected until the average wind speed over the record just collected fell below 4.5m/s. Initial analysis of previously collected records (see next section) would begin whenever the mean wind speed fell below 2m/s.

5.5 INITIAL ANALYSIS OF DATA

5.5.1 Data required from initial analysis

The idea of the initial analysis was to calculate the basic quantities which would be needed in the envisaged spectral analysis and associated calculations. Thus a spectrum had to be calculated from each time history of an accelerometer channel. The mean wind speed and direction, position of the plumb line and temperature from each thermistor were also required. As well as calculating each of these means over the entire 1024 second record, their value was calculated for each of sixteen parts obtained by splitting the whole record into consecutive 64 second lengths. In this way any large variation between the sixteen parts would indicate a possible error with a particular gauge, or the presence of electrical interference. Further investigation would then be possible, so ensuring that unreliable data would not be included in subsequent analysis.

5.5.2 Analysis of data from anemometers

Figure 5.18 shows an example of time histories of the response of the three anemometers for one record of 1024 seconds. The first analysis carried out on the anemometer signals was to calculate the mean value of the signal from the two horizontal anemometers over the whole 1024 second record, and so determine initial values of wind speed and direction. Then the appropriate correction factor (as discussed in section 5.2.1.2) was used to determine accurate values for the average wind speed and direction for the whole record. The mean wind speed and turbulence intensity (the root mean squared deviation from the mean) were calculated for each
Figure 5.16: The author holding one of the optical discs

Figure 5.17: The control room at Hume Point
FIGURE 5.18 Time histories of wind speed for record HZ2203.939 (22nd March 1989, record 39, mean wind speed 15.01 m/s, mean wind direction 253.2°)
anemometer channel. Finally, the whole record was split into sixteen parts of 64 seconds each, and the mean wind speed and direction of each part determined using the same procedure as used for the calculation of the overall mean wind speed and direction. All of these values, together with the exact start time of the record, were stored in the relevant HW file.

5.5.3 Analysis of data from accelerometers

Figure 5.19 shows an example of a time history for one of the accelerometer channels. The figure shows the response of the accelerometer measuring east-west motion in the north-west corner of the building, and is for the same record as that for the anemometer time history shown in figure 5.18. The upper part of figure 5.19 shows the response over the whole 1024 second record, while the lower part of the figure shows an expanded part of the time history.

The following initial analysis was performed separately on each accelerometer channel for each 1024 second record. The mean value of acceleration was calculated and stored. This value was then subtracted from the whole record so that the mean value of each accelerometer time history was zero. The whole record was split into sixteen parts of 64 seconds each and the maximum and rms value of the acceleration in each section calculated. A discrete Fourier transform (DFT) of this amended time history was then calculated. The computer subprogram used for this was taken from reference 69, appendix 2. Figure 5.20 shows the resulting power spectrum of the same record shown in figure 5.19. The upper part of figure 5.20 shows the complete spectrum from 0 to 8 Hz, while the lower part of the figure shows an expanded section from 0.7 to 1.7 Hz. The complete spectrum from 0 to 8 Hz (8192 spectral lines) was then written to file as a series of integer values and a conversion factor. Also written to file were the mean value subtracted from the initial time history, and the maximum and rms accelerations from each of the sixteen sections of the amended time history.
FIGURE 5.19  Time history of acceleration for record HZ2203.939  
(East-West motion measured by accelerometer in NW corner of 23rd floor)
FIGURE 5.20  Power spectra of acceleration for record HZ2203.939
(East-West motion measured by accelerometer in NW corner of 23rd floor)
FIGURE 5.21  Time history of North-South displacement for record HZ2203.939
5.5.4 Analysis of data from lasers

Figure 5.21 shows an example of a time history from the one of the two laser channels. Again, this is for the same record as that for the anemometer time history shown in figure 5.18. The upper part of the figure shows the entire time history, while the lower part shows the time history over one minute. Initially, the voltages for each laser when locked onto the two calibration lines and the resulting calibration for each laser were calculated (see section 5.2.3). Then the mean value of the voltage from each laser was calculated for each 64 second period of the record. These angular displacement values (as seen by each laser) were converted into values of linear displacement. The values stored in the HT file were the mean and rms linear displacements in the two orthogonal directions for the whole record, plus the mean linear displacements for each of the sixteen 64 second parts.

5.5.5 Analysis of data from thermistors

The time history for each of the thermistor channels was invariably a straight line so no example has been included. The initial analysis for each thermistor channel was to calculate both the mean and rms temperature values for the whole record, and for each of the sixteen 64 second parts of the record, using the calibration factor mentioned in section 5.2.4. Thus 34 values for each of the eight thermistor channels were stored in the relevant HT file.

5.6 WIND PROFILE EXPERIMENT

During the week beginning Monday 18th January 1988 an experiment was undertaken by the BRE Wind Loading section under Dr P. A. Blackmore to investigate the wind characteristics in the vicinity of Hume Point. A 30m trailer mounted mast was set up in the playground of Rosetta Primary School (see figure 4.5). This was the only suitable location that could be found where measurements would be out of the influence of the building, but close enough for the wind characteristics to be representative of those at the building. The playground lies between 280m and 350m due west of Hume Point.
(4.2 to 5.2 times the height of Hume Point) so was in an ideal position. The recreation park to the north-west of Hume Point was too densely planted with trees to be of use.

Three sets of propeller anemometers, identical to those used on Hume Point, were mounted on the mast at heights of 10m, 20m and 30m above ground. The outputs from all nine of the anemometers on the mast, plus the three on top of Hume Point, were fed into an LSI computer sited in the playground where the data were sampled at 2.048 Hz.

Data from the propeller anemometers was collected on two occasions. The first, on 20th January lasted for just over 100 minutes. Initially the wind direction was southerly, with a mean wind direction, \( \phi \), of 183°. However, after 30 minutes, it veered to south-westerly \( (\phi = 226°) \), this change in wind direction occurring very suddenly within a two minute period. The data from 20th January was therefore regarded as two data sets designated G20A and G20B for the first and second parts of the run respectively. The wind direction on the second occasion (21st January) was fairly constant over the two hour recording period with a mean wind direction of 290°. This data set was designated G21.

Figure 5.22 shows the velocity profiles for the three data sets. All three data sets fit power-law velocity profiles of the form:

\[
V \propto h^\alpha
\]

where \( V \) is the normalised wind velocity, \( h \) the height above ground, and \( \alpha \) an exponent.

Data sets G20A and G21 have exponents \( (\alpha) \) of 0.25 and 0.27 respectively, a typical value for a suburban area. By contrast, data set G20B has an \( \alpha \) value of 0.38. This exponent is typical of city centre terrain and is caused by the other tower blocks on the Freemason's estate being upwind of the playground for this data set (see figure 4.5).
FIGURE 5.22 VELOCITY PROFILES

- G20A  $\alpha = 0.25$  $\phi = 183$
- G20B  $\alpha = 0.38$  $\phi = 226$
- G21   $\alpha = 0.27$  $\phi = 290$
FIGURE 8.23 INWIND TURBULENCE INTENSITY PROFILES

HEIGHT ABOVE GROUND (m)

INWIND TURBULENCE $I_u$

- G20A
- G20B
- G21
FIGURE 5.24 CROSSWIND TURBULENCE INTENSITY PROFILES
Figures 5.23 and 5.24 show respectively the in-wind and across-wind turbulence intensity profiles for all the data sets. The across-wind turbulence intensities for data set G20B are, as expected, much greater than for either of the other sets. Figure 5.23 shows that the in-wind turbulence intensities for data set G20A are greater than those for G21 and approaching those of G20B. This suggests that slight shifts in wind direction occurred during the recording so that some of the wake shed by the tower blocks was affecting the results.

5.7 CHRONOLOGY OF EXPERIMENT

5.7.1 Chronology prior to acquisition of long term data

The following chronology of the experiment is included so that the reader can understand why no long term wind loading data was acquired until eighteen months after the forced vibration test, why there was a seven month gap in the data collection, and why the data collection was terminated only sixteen months after it had been started.

Limited ambient vibration tests were carried out on all nine blocks on the Freemasons estate in January and February 1986. The results of these tests (which are detailed in Section 4.2) are given in reference 74. As the London Borough of Newham (the owners of the blocks) was keen to co-operate with BRE on any required research work, and all the blocks were empty, it was suggested that one of them might be used for the long term wind loading experiment. The required criteria for the building to be used in the experiment had been formulated in 1985, but no particular building had been identified up to that time. The proximity of Hume Point to BRE, which meant that return trips could be made easily within a working day was crucial. The ambient vibration tests had shown that Hume Point had well separated fundamental modes.

As a first step the forced vibration tests were carried out in July 1986 (see Chapter Four), all the equipment required for the long term experiment was ordered and arrangements made for power and telephone lines to be re-installed on the top floor of the block.
During the forced vibration tests it was noticed that a large electronic "spike" appeared on the oscilloscope at regular intervals. It was discovered that these spikes coincided with a click emanating from a radio transmitter which was located in a cupboard on the 23rd floor. Whilst these momentary spikes were little more than an irritant during the forced vibration tests, they occurred often enough to cause a serious problem during any collection of wind induced data as they were at least an order of magnitude higher than the acceleration data. Early in 1987 enquiries were made via Newham Council to establish the ownership of the transmitter and to ask for it to be relocated on another block if at all possible.

In September 1987 the propeller anemometers were calibrated in the BRE Boundary Layer Wind Tunnel. Details of this calibration are given in section 5.2.1.2

On the 15 - 16 October 1987 a severe storm caused widespread damage to buildings and other structures over a wide area of the South-East of England (ref 99). The maximum gust recorded at the London Weather Centre (which is 10 kilometres east of Hume Point) was 42 m/s (ref 87). This should be compared with the basic design wind speed for this area of the United Kingdom which is 38 m/s. (The basic speed is the maximum value of the 3-second gust likely to be exceeded on average once in 50 years measured at a height of 10 m above the ground in open, level country). The wind measured at the London Weather Centre had a return period of 120 years. However, it should be remembered that there is always a likelihood that random variables will be exceeded, and in the case of the once-in-50-year wind, there is a 63% chance that it will be exceeded in any period of 50 years.

The mast to hold the anemometers was hauled up the side of Hume Point by a team of volunteers from BRE in November 1987, and extended to its full 21m height on 1 December. The radio transmitter, which belonged to a local taxi firm, was moved to another block on 7 December 1987.
5.7.2 Acquisition of long term data

The computer, optical disc and accelerometers were installed over the next two weeks, and acquisition of trial data started on 21 December 1987. After examining this data it was decided that the accelerometer data would be better if it were passed through further amplification before being recorded. None of the data recorded prior to this extra amplification being added has been used in any of the analysis. Therefore, for practical purposes, data collection began on 6 January 1988. It is interesting to note that the first two data files recorded on 6 January are the only two wind speed records of over 20m/s collected during the whole of the experiment.

A few problems were encountered with the optical disc system in the next couple of months, which meant that even when the 5m/s threshold was exceeded, data were not always recorded. By 6 May 1988, 1046 hours of data had been collected on four optical discs. However, on arrival at Hume Point on that day, it was discovered that some equipment had been stolen, although no data had been lost. Access had been gained by climbing in through a boarded window on the second floor, and then climbing round between balconies on the 23rd floor in the dark. Although this route seemed perilous, it was decided to remove all the equipment from the building (except the mast and the anemometers) until improved security measures could be put into place. Whilst extra security doors were fitted throughout the building, initial analysis of all the data collected up to that time was carried out over the summer months, when lower wind speeds could be expected in any case.

All the equipment was replaced and eight channel data acquisition recommenced on 22nd December 1988. The lasers, thermistors and additional accelerometers were put into place and tested over the next couple of months. Thirty-two channel data acquisition started on 8 February 1989. One of the anemometers failed on 27 February 1989 but was replaced nine days later. Vandalism of the sub-station controlling the electricity supply to the block meant that no data were collected between 25 March and 19 April. On arrival at Hume Point on 16 May 1989 there was further evidence that unauthorised access to the block had been gained and several
of the security doors broken. Consequently, for the second time, all equipment except the mast and anemometers was removed from the block until such time as it was thought safe to return it.

5.7.3 Chronology after acquisition of long term data

No improvements to the security of the block were made during the rest of 1989 so the equipment was not reinstated. The security of the block may have received a low priority as discussions over the possible sale of Hume Point and the other blocks on the estate continued throughout this period. Initial analysis of the data collected on site took place throughout the summer and autumn of 1989. It should be noted that, whereas the analysis of a record containing eight channel data took approximately as long as it did to acquire on site; analysis of the thirty-two channel data took about two hours per record or about eight times as long as it took to acquire on site. Thus the 731 hours of thirty-two channel data from Hume Point took over 200 days to analyse. Some of this was done on site during periods of light winds, but the majority was carried out after all the equipment had been returned to BRE. However, the acquisition of a second optical disc player meant that analysis of the data from two discs could be carried out simultaneously. In the event, because of further problems with one of the optical disc players, the initial analysis of the data was not completed until the beginning of January 1990.

A further severe storm occurred on the 25 January 1990. This storm caused widespread damage to buildings and other structures over Southern England (ref 100). The maximum gust recorded at the London Weather Centre (which is 10 kilometres east of Hume Point) was 39 m/s (ref 87), giving a return period of a little over 50 years. A visit was made to Hume Point shortly after the storm. The mast on top of the building was undamaged, although some of the guy ropes were loose. However, the bracket which fixed the anemometers to the top of the mast, and two of the three anemometers, were found on the roof of the building. All were broken, although whether or not this was caused by the fall from the top of the mast could not be ascertained. The third anemometer could not be found, despite a search of the roof and the ground around Hume Point. The mast was retracted.
Nationally, "February 1990 stands out as the most unusually windy February since before 1881" and "the 35 day mean wind speed between 24 January and 27 February 1990 was almost unprecedented in recent decades, more particularly over central and Southern Britain" (ref 101). The maximum gust at the London Weather Centre exceeding 15 m/s on 33 of the 37 days from 23 January to the end of February (ref 87). The mean hourly wind speed was at least 12 m/s on 124 occasions at the same location over the same period (ref 87). Storms at the end of February caused further damage to structures weakened by the large storm of the 25th January (ref 100).

The collection of useable wind response data from Hume Point lasted over a sixteen month period from 6 January 1988 to 12 May 1989. It is interesting (though somewhat depressing for the author) to note that this period is contained within the gap of only 27 months between two events when winds with a return period of over 50 years were experienced at Hume Point.

Further problems with optical discs and optical disc drives meant that ensemble averaging did not begin until July 1990 and was not completed until the end of November 1990. The analysis of the quasi-static response was carried out concurrently, and was completed in December 1990.
CHAPTER SIX:

THE DYNAMIC RESPONSE OF HUME POINT TO WIND LOADING

6.1 DISTRIBUTION OF WIND RECORDS

In total, 7922 records (the equivalent of over 2,250 hours of data) were recorded at Hume Point. Table 6.1 shows the distribution of these records by wind speed and direction. It should be remembered that the building's principal axes are aligned exactly north-south and east-west, so that a wind blowing directly onto the north face comes from grid north (0 degrees). It should also be noted that the wind speeds quoted throughout this chapter are those actually measured by the anemometers 85.05m above ground level. The records in Table 6.1 are divided up according to the mean wind speed and direction over the whole 1024 seconds, calculated as described in section 5.5.2. It can be seen that two thirds of the records are in the 120° sector from 165° to 285°, a quarter in the sector from 285° to 45°, whilst only 8% are in the remaining 120° sector. Also only about 3% of the records are for wind speeds in excess of 15 m/s. The only two wind records in excess of 20 m/s were the first two obtained in 1988: 23.4 m/s from a direction of 233° followed immediately by 20.7 m/s from 235°.

Table 6.2 shows the distribution of records obtained whilst data from all 32 channels were being collected. This therefore is the distribution for which data from the plumb line, thermistors and all 14 accelerometer channels is available. The overall distribution is very similar to that shown in Table 6.1 although the maximum mean speed was one of 17.6 m/s from a direction of 263 degrees. However, although the distribution between the three 120° sectors is similar, the distribution within the 165° to 285° sector is not, southerly winds being much rarer than in the overall distribution. With two exceptions, the maximum number of records obtained in any 30° sector was 253. However, this is for all wind speeds and at least 100 records from similar wind speeds are required for accurate spectral analysis. Therefore the only two 30° sectors where spectral analysis of the accelerometer records might be worthwhile are the 195°-225° and 225°-255° sectors, with 676 and 533 records respectively.
### TABLE 6.1 DISTRIBUTION OF ALL RECORDS BY WIND SPEED AND DIRECTION

<table>
<thead>
<tr>
<th>SPEED (m/s)</th>
<th>MIN: 3.01</th>
<th>7.01</th>
<th>10.01</th>
<th>15.01</th>
<th>0.01</th>
</tr>
</thead>
<tbody>
<tr>
<td>DIR(°) MAX:</td>
<td>7.00</td>
<td>10.00</td>
<td>15.00</td>
<td>20.00</td>
<td>24.00</td>
</tr>
</tbody>
</table>

15 - 45  113  48  12  -  173
45 - 75  53  37  19  -  109
75 - 105 151  79  16  -  246
105 - 135 49  13  -  -  62
135 - 165 79  74  59  5  217
165 - 195 414 389 234 17 1054
195 - 225 382 676 632 70 1760
225 - 255 480 617 528 71 1698 (Two >20)
255 - 285 266 390 209 22 888 (One <3)
285 - 315 179 213 148 10 551 (One <3)
315 - 345 178 176 193 49 596
345 - 15 245 263 55 4 568 (One <3)
0 - 360 2589 2975 2105 248 7922

### TABLE 6.2 DISTRIBUTION OF RECORDS BY WIND SPEED AND DIRECTION

(32 channel data only – 8th February 1989 onwards)

<table>
<thead>
<tr>
<th>SPEED (m/s)</th>
<th>MIN: 3.01</th>
<th>7.01</th>
<th>10.01</th>
<th>15.01</th>
<th>3.01</th>
</tr>
</thead>
<tbody>
<tr>
<td>DIR(°) MAX:</td>
<td>7.00</td>
<td>10.00</td>
<td>15.00</td>
<td>18.00</td>
<td>18.00</td>
</tr>
</tbody>
</table>

15 - 45  91  42  12  -  145
45 - 75  23  25  1  -  49
75 - 105 54  17  -  -  71
105 - 135 40  11  -  -  51
135 - 165 25  40  34  -  99
165 - 195 44  83  25  5  157
195 - 225 90  264 298 24 676
225 - 255 149 168 189 27 533
255 - 285 94  88  54  10 246
285 - 315 42  34  67  4  147
315 - 345 75  30  3  -  108
345 - 15 134  94  25  -  253
0 - 360 861  896  708  70 2535
As so much of the data was obtained whilst the wind was blowing from between 165° and 285°, it was decided that the main body of the analysis would concentrate on data obtained whilst the wind was blowing from this sector. Although this was known to be the predominant wind direction, it is unfortunate that this corresponds to the other blocks on the estate being upwind of Hume Point (see section 4.3). However, it was necessary to use this data in order to obtain ensembles with both a narrow range of wind speeds and directions and a sufficient number of records to keep the variance error small.

6.2 MAXIMUM ACCELERATIONS

The maximum single peak acceleration measured during the record with the 23.4 m/s wind speed was 0.019 m/s² in the east-west (EW) direction, and 0.013 m/s² in the north-south (NS) direction. These maxima occurred during a 64 second period when the mean wind speed was 29.3 m/s. It is interesting to compare these (single peak) figures with those obtained during the forced vibration testing (which are given as peak-peak figures in table 4.2). In the EW direction, the acceleration during the record with the highest mean wind speed is about four times that of the lowest level of forced excitation and about half that of the highest level. In the NS direction, the acceleration is about twice that of the lowest level of forced excitation and about a quarter that of the highest level.

It should be noted that these accelerations are not necessarily the highest obtained during the experiment. Larger accelerations may well have occurred in the north-south direction when the wind was blowing more directly onto the north or south faces. Similarly, larger EW accelerations may have been obtained with a westerly wind. However, no attempt has been made to find the maximum peak acceleration for any of the accelerometers. Such information would be of little or no practical use, and so could not be justified by the amount of time and effort required to obtain it.
6.3 SELF-STATIONARITY OF INDIVIDUAL RECORDS

6.3.1 Selection of records and test procedure

In order to test the self-stationarity of individual records, the record which came closest to being recorded during one of thirty-six wind criteria was selected. For each of six wind directions: 90, 180, 210, 240, 270 and 360 degrees, a range of six wind speeds was used: 5, 7, 9, 11, 13 and 15 m/s. As there were no records containing easterly winds in excess of 13.5 m/s this produced thirty five records.

A run test was conducted on time histories from each of the thirty five records using the method set out in section 2.4.2. Run tests conducted on time histories from accelerometers which were aligned in parallel gave similar answers for the same record. Consequently, the run test was only conducted on time histories for the two orthogonal accelerometers in the NW corner of the building (channels 4 and 5). The run test works on the rms acceleration values from the time history over a successive number of points. For example, for 100 samples over the record, 163 successive points were used. The results for a typical record are given in table 6.3.

6.3.2 Results of run test on a typical record

Table 6.3 shows that both channels failed the run test by a wide margin with 200 samples, and both failed with 100 samples. With 50 samples, channel 4 passed but channel 5 only passed at the 99% confidence level. However, with 25 samples both channels passed at the 95% confidence level. In each case in this example the number of runs was less than half the number of samples, so the upper confidence limit has been omitted. Figure 6.1 shows the run test for channel 5 with 100 samples, and figure 6.2 the run test for the same data but with only 25 samples.
TABLE 6.3 TESTING OF INDIVIDUAL RECORD FOR SELF-STATIONARITY

Record HZ2904.873 (wind speed 9.02 m/s wind direction 81.7°)

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>No. points per sample</th>
<th>Time for each sample (secs)</th>
<th>No. runs ch4 (NS)</th>
<th>No. runs ch5 (EW)</th>
<th>99% prob.</th>
<th>95% prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>81</td>
<td>5</td>
<td>47</td>
<td>45</td>
<td>84</td>
<td>88</td>
</tr>
<tr>
<td>100</td>
<td>163</td>
<td>10</td>
<td>37</td>
<td>35</td>
<td>38</td>
<td>42</td>
</tr>
<tr>
<td>50</td>
<td>326</td>
<td>20</td>
<td>21</td>
<td>18</td>
<td>17</td>
<td>19</td>
</tr>
<tr>
<td>25</td>
<td>652</td>
<td>40</td>
<td>11</td>
<td>11</td>
<td>7</td>
<td>9</td>
</tr>
</tbody>
</table>

* 99% probability means that 99% of all random samples will have at least this number of runs.

6.3.3 Results of run test on 35 records

6.3.3.1 Dependence of run test result on number of samples

Table 6.4 shows the results of the run tests conducted on the thirty-five records selected as described above. It should be noted that as both the upper and lower number of runs are used in table 6.4, the confidence limits appear to be smaller than those used in table 6.3. However, the minimum number of runs required to pass the test is identical, so the two are comparable. The table has been divided into the results for the records with wind speeds of about 5 m/s, and those records with higher wind speeds.

All the run tests which failed, did so by having too few runs. All the records tested failed the run test at 98% confidence level with 200 samples (72 runs being the maximum number of runs); but only 3 records (4.3%) failed to pass the run test at the 98% confidence level with only 25 samples. All the records which passed at either the 90% or 98% confidence level for 100 samples passed the test at the 90% confidence level for 50 samples. Similarly, all the records which passed at either the 90% or 98% confidence level for 50 samples passed the test at the 90% confidence level...
FIGURE 6.1  Run test using 100 samples for record HZ2904.873 channel 5

35 runs, median = 0.199 x10^{-3} m/s^2

Sample number

FIGURE 6.2  Run test using 25 samples for record HZ2904.873 channel 5

11 runs, median = 0.197 x10^{-3} m/s^2

Sample number
TABLE 6.4 Tester of 35 RECORDS FOR SELF-STATIONARITY

<table>
<thead>
<tr>
<th>NUMBER OF SAMPLES</th>
<th>200</th>
<th>100</th>
<th>50</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of runs for 90% confidence limits</td>
<td>88-113</td>
<td>42-59</td>
<td>19-32</td>
<td>8-18</td>
</tr>
<tr>
<td>Number of runs for 98% confidence limits</td>
<td>84-117</td>
<td>38-63</td>
<td>17-34</td>
<td>7-19</td>
</tr>
</tbody>
</table>

RECORDS with wind speeds of approximately 5 m/s (6 records, 12 run tests):

| run tests passing at 90% confidence level | 0% | 25% | 50% | 75% |
| run tests passing at 98% but not at 90% confidence level | 0% | 17% | 17% | 17% |
| run tests failing at 98% confidence level | 100% | 58% | 33% | 8% |

ALL OTHER RECORDS (29 records, 58 run tests):

| run tests passing at 90% confidence level | 0% | 0% | 33% | 91% |
| run tests passing at 98% but not at 90% confidence level | 0% | 3% | 19% | 5% |
| run tests failing at 98% confidence level | 100% | 97% | 48% | 3% |
for 25 samples. The maximum number of runs with 100 samples was 42, the maximum number of runs with 50 samples was 27. The maximum number of runs with 25 samples was 17, 22% of the results with 25 samples had at least 13 runs.

The reasons for this result are set out in section 2.8.2. Where 100 samples are used, each sample is only ten seconds in length. The rms value of one sample is then strongly dependent on the level of the response in the previous sample, so the values are not random and the time history fails the run test. However, with only 25 samples, the length of each sample is 40 seconds. Although there will still be some dependence on the level of the response in the previous sample, it will be to a much lesser degree.

It is important to appreciate that there are two reasons for the rms value of one sample being dependent upon the level of the response in the previous sample. Firstly, because of the time constant of the building. The time constant (τ) is defined wrongly in the ISO Handbook on Acoustics, vibration and shock (reference 102, p524) as the time taken by an exponentially decaying quantity to decrease in magnitude by a factor of 1/e. This definition, derived originally from electronics, should be the time taken to decrease in magnitude to a value of 1/e times the initial magnitude. The ISO Handbook does give the correct form for calculating the decay at time τ as:

\[ e^{- (τ/Ψ)} \]  

(Eq 6.1)

where \( e \) is the base of natural logarithms, \( τ \) is time, and \( Ψ \) is the time constant. Reference 103 rewrites this expression in terms of frequency (\( f \)) and damping (\( ζ \)) as:

\[ Ψ = \frac{1}{2 \pi f ζ} \]  

(Eq 6.2)
It can be seen from equation 6.2 that the smaller the product of the natural frequency and damping the longer the time constant. When applied to the vibration of structures, it is apparent that there will be a time constant for each mode of vibration. In the case of Hume Point, the longest time constant is for the EW1 mode and is 20 seconds. Therefore, twenty seconds after the cessation of a steady state excitation in the EW1 mode, the response will have dropped to $1/e$ (about 37%) of the steady state response. It should be noted that not only is the time constant a measure of how long the response of a structure takes to decay, it is also a measure of how long it takes the response to build up to a maximum.

Clearly, the response of Hume Point to wind excitation is not as simple as the building's response to steady state excitation, and the fact that the stationarity test looks at the rms acceleration over a period rather than at discrete points complicates the matter further. However, it is useful to look at how the building's time constant affects its response for each of the four periods used in the stationarity tests. At five and ten seconds after the cessation of steady state excitation in the EW1 mode the response will have dropped to about 78% and 61% respectively of the steady state response. These figures show that any one value is highly dependent upon the previous value. Consequently, it is not surprising that all the run tests failed with the five second period and the majority failed with the ten second period.

The respective figures for the response twenty and forty seconds after the cessation of steady state excitation are about 37% and 14%. Consequently, there is a marked reduction in the number of records which failed the run test as the time for each sample increased. Increasing the time for each sample to 60 or 80 seconds would bring the response figures down to about 5% and 2% respectively. However, although the effects of the time constant of the building are the main reason for the failure of the run tests where the time for each sample is twenty seconds or less, another effect becomes more important at sampling times greater than this.

The second reason for one sample being dependent upon the level of the response in the previous sample is the micrometeorological peak of the wind spectrum (see section 1.5.3) which ranges from periods of about 10 minutes
to 3 seconds and has a peak value at a period of about 45 seconds. It is only by using a sampling time outside the micrometeorological peak, or at least on the edges of it, that the sampling can be considered random, and the run test a true test for stationarity.

Whilst all the above evidence points to individual records being self-stationary, eight still failed the run test at the 90% confidence level when only 25 samples were used. If a general assumption that all the records are self-stationary is to be made, then some explanation for these failures must be forthcoming.

6.3.3.2 Records failing run test because of accelerometer drift

The time histories of all of the records that failed the run test were examined. It was apparent that there was a linear trend on the majority of them, the mean value of the response either increasing or decreasing continuously during the record. Figure 6.3 shows one of these records. This linear trend is typical of that caused by long term drift of the accelerometer signal. At low wind speeds, the overall response level is low, and so the drift will have a bigger proportional effect than the same drift occurring at higher wind speeds where the overall response level will be higher. This is borne out by three of the eight records which failed the run test being ones when the wind speed was only about 5 m/s.

A linear correction was applied to each of the eight records which failed the run test and then the run test reapplied to the corrected data. The mean value of the first and last ten seconds of the time history were calculated and a straight line fitted between them. The value of this straight line at the corresponding point along the time history was then subtracted from the original time history to produce a time history with a linear correction. Seven of the eight records which failed the initial run test now passed it at the 90% confidence level. Figure 6.4 shows the same time history as that in figure 6.3 but with the linear correction applied.

Correcting long term drift is an accepted practice in wind engineering (ref 104) and is valid if the following two criteria are met. Firstly, there must be a valid reason for removing the drift (i.e. the output from
FIGURE 6.3  Time history of acceleration for record HZ2102.917 channel 4 before linear correction applied

FIGURE 6.4  Time history of acceleration for record HZ2102.917 channel 4 after linear correction applied
the relevant transducer must be seen to be prone to drifting). Secondly, it must be reasonable to assume that the mean response at the start and the end of the record are the same. Both these criteria are met in this case, so removing the drift is reasonable.

6.3.3.3 Record without accelerometer drift failing the run test

It will be noted that, even with a linear correction factor applied, one time history still failed the run test. This was the only one of the eight time histories to come from channel five. This then leads to two conclusions. Firstly, that the accelerometer on channel five was less prone to drift than the one on channel four. Secondly, that this particular record represents the only one of the 35 tested for which it appears that the dynamic response of the building to wind loading was not stationary.

However, the failure of all the records to pass the run test is not unexpected. As discussed above, even with 25 samples, there will still be some dependence on the level of the response in the previous sample, so it could be argued that this same record would pass the run test if it were possible to conduct it accurately with fewer samples. Whilst this argument is a tenable one, it is unlikely to be the correct reason for this particular record failing the run test. Figure 6.5 shows the time history for channel five for this record. It is apparent that strong gusts at the beginning and end of the period are separated by a much calmer period in the middle. Although this record failed the run test by having too few runs at the 99% confidence level, it must be remembered that 1% of all records from stationary processes would be expected to fail.

Ignoring, for the time being, the record which still failed the run test, the following conclusion can be drawn from this, if it is accepted that the linear trend in the time histories was caused by long term drift in the accelerometers. The dynamic response of the building to wind loading is stationary, but the measurement of this process with accelerometers may or may not produce a stationary time history. However, the lack of stationarity in some of the time histories does not affect the stationarity of the building's response. Therefore spectral analysis of all the time
histories is valid, although the very low frequency data in the resulting spectra will be corrupted. For this reason data below say, 0.01 Hz, in the spectra from Hume Point should be ignored.

It could be argued that records such as that shown in figure 6.5 should be omitted from the ensemble averaging process. However, this record cannot be said to be non-stationary because of its failure to pass the run test as 1% of stationary records are expected to fail anyway. Therefore it would appear to be best to test to see whether results from a series of nominally identical ensembles produce answers that are within the error limits based purely upon the number of records in the ensemble. The results of such a procedure are given in section 6.5. The alternative to this would be to test all the time histories for stationarity and see whether only about 1% of these failed the run test. Given the problems in assessing the self-stationarity of records of this length, such a time consuming task was not thought to be worthwhile.

6.4 SELECTIVE ENSEMBLE AVERAGING PROCEDURE

6.4.1 Initial analysis procedure

The procedure for performing the initial analysis of the data from the accelerometers is set out in section 5.5.3. Essentially, a power spectrum from 0 to 8 Hz (8192 spectral lines) was produced for each accelerometer channel for every record. Some additional information, including the rms value of the relevant original time history, was stored with each spectrum.

6.4.2 Calibration check on power spectrum calculation

One of the most important properties of a power spectrum is that, as mentioned in section 2.1.2.1, the area under the curve is equal to the total mean square response (ref 69, p 190). Therefore, the square root of this area is equal to the total rms response. As the total rms response of each accelerometer time history was calculated after any mean D.C. offset had been removed, the total rms response has been obtained by two totally separate methods. By checking that the two give the same answer for a
particular record, the procedure used to calculate the power spectrum can be checked.

A comparison of the total rms values obtained by the two different methods was carried out on several different records representing a wide range of wind speeds. In all cases, the rms response obtained from the time history was between 0.2% and 1.7% greater than that obtained from the area of the spectrum. There are two factors which contribute to this discrepancy. Firstly, the accelerometer signal was put through a 10 Hz low pass filter but the spectrum is only from 0 to 8 Hz. Although figures 4.20 to 4.22 show that the response of Hume Point between 8 and 10 Hz is small, there will be data in the time history in this range, none of which will appear in the spectrum. Secondly, in order to aid the production of graphs of the spectra by an autoscaling routine, the amplitude of the spectral line at zero Hertz was set to zero when calculating the power spectra. This second factor appears to be the more significant in producing the discrepancy between the two figures. All the spectra contain some contribution due to the long term drift of the accelerometer signals. The amplitude of this drift is independent of wind speed and can be considered to be fairly constant. Therefore, at low wind speeds, the area under the curve caused by this drift (frequencies up to about 0.01 Hz) represents a larger proportion of the total area than at high wind speeds, where the area under the resonant peaks increases. This is reflected in the results of the comparison exercise where the largest discrepancies occurred at low wind speed, and the smallest discrepancies at high wind speed.

As well as checking the rms values for individual records obtained by the two different methods, a check can also be made on the rms values of ensembles. The method for obtaining averaged spectra is described in the next section. Essentially a selection of say, one hundred records would be made. The relevant one hundred spectra would then be averaged together to produce an output spectrum. The square root of the area under the output spectrum is the total rms for the ensemble. This can then be compared to the overall average rms value obtained from the original time histories. It should be noted that this overall average rms value is the square root of the average of the individual mean square values and not the average of the individual rms values. This latter quantity is always less than the
former except when all the rms values are the same, in which case the two quantities are the same (see Appendix A). However, the difference between these two quantities is a good indication of the spread of the individual rms values in an ensemble.

6.4.3 Obtaining averaged spectra

Once the power spectrum calculation had been confirmed, obtaining averaged spectra could begin. Firstly, the files to be averaged together had to be identified. A file was prepared for each optical disc containing the file name, date, start time, mean wind speed and direction for each record on that disc. A program was written to scan through all nine of these files (nine optical discs having been used over the whole experiment) and find any files which were recorded whilst the wind speed and direction were within selectable limits. The filename, wind speed and direction of any files which were found which met the criteria were written to an output file (a WIND file). As only one optical disc could be looked at at any one time, it was then necessary to separate the WIND file into nine parts, each one relating to the files on a particular optical disc.

A further program was written to extract the relevant spectra from a single optical disc, average them together, and then store the output spectrum. The following inputs were required for the program: channel to be looked at, calibration figure for the relevant accelerometer over the period in question, and the relevant part of the WIND file. A Macro routine was written so that, having looked at one channel, the same WIND file could be used to look at further channels. This Macro routine could be set so that, having looked at all the channels of interest for one WIND file, the process could be repeated for further WIND files. In this way the parts of several ensembles from one disc could be obtained for a number of channels and stored without the need for an operator. The output for the program for each ensemble and for each channel consisted of the averaged spectrum, the number of spectra which were averaged together, the mean wind speed and direction from the relevant records, and the overall rms of the original time history for each record.
Having obtained the averaged spectra for all channels of interest for each of a number of WIND files, the next optical disc was loaded and the process repeated. Eventually, for each original WIND file, there were nine output files containing averaged spectra. These were then averaged together in proportion to the number of spectra in each one to produce the spectrum for the whole ensemble. The upper part of figure 6.6 shows such a spectrum from 0 to 8 Hz, while the lower part shows the part of the spectrum from 0.7 to 1.7 Hz. The smooth spectrum produced by the ensemble averaging process should be compared with figure 5.20 which shows the very ragged spectrum for a single record.

6.4.4 Results from averaged spectra

A number of values were stored along with the spectrum for the whole ensemble. These were the number of spectra in the ensemble, the average of all the mean wind speeds and directions of the records in the WIND file, and some statistics based on the overall rms values of the original time histories: minimum, mean, maximum, variance and overall average. The procedure for calculating the overall average rms value is given in section 6.4.2.

As mentioned previously, the area under the output spectrum is equal to the total rms response. As well as calculating the area under the whole curve, the area under the curve was found between specific frequency limits in order to find the contribution of the individual modes to the overall response. To enable the comparison of the results from one ensemble to another, each of the fundamental modes was defined as extending between two fixed frequencies. When fixing these frequencies, due regard was made to the change of frequency with amplitude of response noted in the forced vibration tests. Although the upper and lower frequency limits should both be reduced as the natural frequency of a mode decreases with increasing response, the contribution of the response close to the frequency limits to that for the whole mode is very small. Therefore, changing the frequency limits by a small amount to account for changes in natural frequency would have a minimal effect on the overall response in that mode, so it was decided to adopt fixed frequency limits for both simplicity and compatibility.
Example ensemble contains 100 records from EW accelerometer in NW corner all with mean wind direction 165°-195° and mean wind speed 9.43 - 10.40 m/s.

FIGURE 6.6  Typical ensemble averaged power spectra
Having decided to use fixed frequency limits, a decision had to be made as to how far in frequency a mode extended from its natural frequency. The lower part of figure 6.6 shows the response in the three fundamental modes for one particular record. It can be seen that although the three modes are well separated, the response does not fall between the modes to the same level as it does well away from any modes. Therefore, the centre mode in frequency (NS1) was deemed to extend from 1.04 Hz to 1.27 Hz (235 spectral lines) while the EV1 mode extended up to the lower of these limits and the torsional mode was at frequencies higher than the upper limit. In order not to decrease the relative importance of the NS1 mode merely because it was between the other two, the difference between the upper and lower frequency limits of the other two modes was also set at 0.23 Hz. It could be argued that, for example, the response due to the EV1 mode extends below 0.81 Hz. However, as the response in this region is very small relative to that at the natural frequency, adding it to that above 0.81 Hz would make little difference to the response level in the mode as a whole.

**TABLE 6.5 FREQUENCY LIMITS USED FOR DETERMINING THE RESPONSE IN THE FUNDAMENTAL MODES.**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>EV1</td>
<td>0.81 - 1.04 Hz</td>
</tr>
<tr>
<td>NS1</td>
<td>1.04 - 1.27 Hz</td>
</tr>
<tr>
<td>01</td>
<td>1.27 - 1.50 Hz</td>
</tr>
</tbody>
</table>

Table 6.5 gives the frequency limits for determining the response in the fundamental modes. Apart from these, the response below 0.01 Hz, the response in all the second order modes (3.50 - 5.00 Hz) and the responses in the areas of the spectrum between these three regions were calculated.
6.4.5 Curve fitting around fundamental modes in averaged spectra

The same single degree of freedom curve fitting procedure used in the forced vibration tests (see section 4.10) was used to determine the natural frequency and damping of the fundamental modes for each ensemble. In every case the curve was fitted between the same frequency limits used to determine the rms acceleration in that mode, i.e. the same frequency limits as shown in table 6.5. Figure 6.7 shows the result of this curve fitting exercise around the EW1 mode on a typical ensemble averaged spectrum. The upper part of the figure shows the relevant part of the ensemble averaged power spectrum, so the ordinate is Acceleration²/Hz, but in the lower part of the figure this has been converted to displacement. The accuracy of the figures for natural frequency and damping determined by this method can be judged by the goodness of fit between the spectral values and the fitted curve. There were some instances, particularly for the torsional mode, where the goodness of fit was not as good as that shown in figure 6.7, but this example is typical of the curve fitting around the EW1 and NS1 modes. However, figure 6.8 shows the curve fitting around the torsional mode from the same ensemble averaged spectrum, and this is typical of the curve fitting around the 01 mode.

6.5 STATIONARITY OF THE ENSEMBLE

6.5.1 Procedure for testing stationarity of nominally identical ensembles

To assess the stationarity of the ensemble, the results of four nominally identically ensembles were compared. The two hundred records which were made whilst the wind speed was between 9.20 and 10.00 m/s, and the wind direction was between 195° and 225° were used for this test. Given the 30 degree sectors used in table 6.1, no other 0.8 m/s range of wind speeds contained more records.

Firstly, all 200 records were used to make ensemble A. Then the 200 records were divided into four groups of 50 records in the following way. The data from Hume Point were written onto nine optical discs and eight of
Example ensemble contains 100 records from EW accelerometer in NW corner all with mean wind direction 165°-195° and mean wind speed 9.43 - 10.40 m/s.
Example ensemble contains 100 records from EW accelerometer in NW corner all with mean wind direction 165°-195° and mean wind speed 9.43 - 10.40 m/s

FIGURE 6.8 Curve fitting around the θ1 mode
these contained at least one record which fell within the criteria given above. The first and eighth discs each contained 25 records, so these were combined to make ensemble B. Similarly, the second, sixth and seventh discs contained 50 records between them, so these were combined to make ensemble C. The first 50 records on the fifth disc made ensemble E, whilst the other 50 records made ensemble D. Whilst this method is not entirely random, the ensemble that a particular record should be placed in has not been decided by any criterion other than the one of convenience given above. It should also be noticed that the records are not arranged in chronological order either.

6.5.2 Results of testing stationarity of ensembles

The rms values for the fundamental modes are given in table 6.6 for each of the five ensembles and for each of the five accelerometer channels that were in use throughout the experiment. The other accelerometer channels could not be used in this comparison as only 76 of the 200 records were made whilst the 32 channel set up was in operation.

It should be noted that, although the results of the four ensembles each containing 50 records are given relative to that for 200 records, the results for this latter ensemble should not be thought of as being the "correct" answer. Also, for the same reasons as set out in appendix A, the values for ensemble A are not the mean of the values for the other ensembles. The values for ensemble A are the square root of the average mean square values of the other ensembles, where all the other ensembles each contain 50 records.

Table 6.6 shows that the largest discrepancy between the rms value of any of the modes in ensemble A and any of the smaller ensembles is less than 7%. The largest spread between values being 10.9%, both of these extremes being for the EW1 mode as measured by accelerometer channel 5. These values should be compared with the ±14.1% variance error for 50 records. As all the discrepancies are considerably less than the maximum expected with this number of records, it can be concluded that the spectral analysis has been conducted on stationary data, as set out in section 2.8.3. Strictly, this result only applies to these particular wind
criteria. However, it gives confidence in the use of a 30 degree sector for wind direction.

6.5.3 Determination of which channels to use in detailed ensemble averaging

Some problems were encountered which affected the accelerometer data recorded on channels 6 and 7. In the case of channel 6, the failure of an accelerometer meant that 100 records were lost, eight of them being amongst the 200 used for the comparison exercise. However, the problem was an intermittent one on channel 7, and was therefore not identified as quickly. Consequently, considerably more than 100 records from channel 7 were lost. In both cases, "bad" records could be identified by the rms value for the original time history being at least an order of magnitude higher than anticipated. These "bad" records have been omitted from table 6.6, leaving some ensembles with less than 50 records. Full details are given in the notes at the bottom of table 6.6. However, despite the fewer records for channels 6 and 7 for some ensembles, and therefore higher variance errors, the results for these channels are still well within the expected range of values.

Table 6.6 also shows that the results for channel 8 are almost identical to those for channel 4. Similar results were also obtained in preliminary ensemble averaging; so, to avoid unnecessary duplication, no detailed selective ensemble averaging was carried out on channel 8. Channel 7 was also omitted from the detailed analysis because of the problems described above. However, some limited analysis was carried out on channel 6, despite ensembles for this channel sometimes containing less records than the corresponding ensembles for channels 4 and 5. Whilst all the records where channel 6 contained "bad" data could have been omitted from any of the analysis, this would have meant discarding 1.3% of the data recorded on channels 4 and 5, even though there was nothing wrong with this data. It can be seen from Table 6.6 that the response from accelerometer channel 6 is less than that for channel 4 for the NS1 mode, whilst it is less than that for channel 5 for the E1 and particularly the EW1 mode. Therefore, it was decided that the selective ensemble averaging would concentrate on channels 4 and 5 (the orthogonal pair of accelerometers in the NW corner), the other channels being used to check that the response in
TABLE 6.6 COMPARISON OF FOUR ENSEMBLES, EACH OF FIFTY RECORDS, WITH ENSEMBLE OF ALL 200 RECORDS

<table>
<thead>
<tr>
<th>Ensemble</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of records</td>
<td>200</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Mean wind speed (m/s)</td>
<td>9.59</td>
<td>9.57</td>
<td>9.63</td>
<td>9.59</td>
<td>9.57</td>
</tr>
<tr>
<td>Mean wind dirn. (°)</td>
<td>208.6</td>
<td>206.7</td>
<td>206.9</td>
<td>210.3</td>
<td>210.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>rms accel. (x10⁻³ m/s²)</th>
<th>Percentage difference from ensemble A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channel 4 EW1: 0.0262</td>
<td>-3.5</td>
</tr>
<tr>
<td>(NW corner, NS1: 0.159</td>
<td>-3.7</td>
</tr>
<tr>
<td>NS dirn. 0.0665</td>
<td>-1.1</td>
</tr>
<tr>
<td>Channel 5 EW1: 0.164</td>
<td>-6.6</td>
</tr>
<tr>
<td>(NW corner, NS1: 0.0457</td>
<td>-0.7</td>
</tr>
<tr>
<td>EW dirn. 0.107</td>
<td>-1.4</td>
</tr>
<tr>
<td>Channel 6 EW1: 0.0170</td>
<td>-2.9</td>
</tr>
<tr>
<td>(SE corner, NS1: 0.113</td>
<td>-4.3</td>
</tr>
<tr>
<td>NS dirn. 0.0912</td>
<td>-2.5</td>
</tr>
<tr>
<td>Channel 7 EW1: 0.191</td>
<td>-5.8</td>
</tr>
<tr>
<td>(SE corner, NS1: 0.0374</td>
<td>-0.6</td>
</tr>
<tr>
<td>EW dirn. 0.0965</td>
<td>-3.1</td>
</tr>
<tr>
<td>Channel 8 EW1: 0.0267</td>
<td>-4.2</td>
</tr>
<tr>
<td>(SW corner, NS1: 0.157</td>
<td>-2.1</td>
</tr>
<tr>
<td>NS dirn. 0.0654</td>
<td>-1.1</td>
</tr>
</tbody>
</table>

Notes:
Ensembles B, C, D and E do not contain any records in common.
Ensemble A contains all the records in the other four ensembles.

All ensembles comprise records obtained when wind speed was between 9.20 and 10.00 m/s, and wind direction was between 195° and 225°.

Channel 6 ensemble D has only 42 records (mean wind 9.58 m/s, 210.2°)
so channel 6 ensemble A has only 192 records (mean wind 9.59 m/s, 208.5°).

Channel 7 ensemble B has only 33 records (mean wind 9.58 m/s, 206.1°)
so channel 7 ensemble A has only 183 records (mean wind 9.58 m/s, 208.7°).

Only the linear component of the response for the torsional mode is given to enable an easier comparison of its size relative to that of the translational modes at each of the measurement positions.
the other positions was as shown in the mode shapes found during the forced vibration testing.

6.6 RESULTS OF SELECTIVE ENSEMBLE AVERAGING

6.6.1 Choosing the wind criteria for the ensembles

One of the aims of the Hume Point experiment, as set out in section 3.4, was to aim for an overall accuracy of ±10% in assessing the response of the building. In section 5.3 it was determined that the bias error in the spectral analysis of the data from Hume Point was no greater than 0.16% for any of the modes. Therefore, if ensembles of at least 100 records (giving a variance error of no greater than ±10%) are used, then the original aim of the experiment should be met. Inevitably, some small errors other than those in the spectral analysis will affect the response levels obtained e.g. calibration of accelerometers and amplifiers, the effects of the records in the ensemble having slightly different wind speeds and directions. On the other hand, Table 6.6 shows that the actual differences between nominally identical ensembles are much less than the maximum expected variance error due to the number of records in the ensemble. Therefore, the variance error alone has been taken as determining the overall accuracy of the results of the spectral analysis. Consequently, in order to achieve the aim of ±10% accuracy, it was decided that all ensembles used in the main analysis should have at least 100 records, and be obtained under similar wind conditions.

In section 6.5.2 it was shown that a stationary ensemble could be obtained with a 30° sector for wind direction, providing that the range of wind speeds in the ensemble was not too great. Therefore the four 30° sectors in Table 6.1 with more than 600 records were used for the main analysis. These four sectors are for mean winds blowing onto the south face (±15°), blowing onto the west face (±15°) and the two in between.

Within each sector the maximum number of ensembles, each comprising 100 records, was obtained. Thus, for the 165° to 195° sector, ten ensembles were obtained from the 1054 records. The mean wind speed (to the
nearest 0.01 m/s) of every record in the sector was noted. A suitable minimum wind speed was chosen for the first ensemble (either 4.51 or 5.01 m/s). The maximum wind speed for this ensemble was the minimum speed necessary in order for there to be at least 100 records in the ensemble. The minimum wind speed for the second ensemble was then 0.01 m/s higher than the maximum for the first ensemble. This process was continued until there were less than 100 records remaining.

Using this method, the records with the highest and lowest wind speeds in each sector were eliminated. One possible alternative would have been to put the 100 records with the highest wind speed for a particular sector into one ensemble, the next 100 highest in the next ensemble etc., so that any records not used in ensembles would be the ones with the lowest wind speeds. However, because of the rarity of records with mean winds in excess of 15 m/s, the first ensemble would contain records with widely differing wind speeds e.g. 12.27 to 17.42 m/s in the case of the 165° to 195° sector. This difference is much greater than that used to demonstrate the stationarity of the ensemble in section 6.5.2. and so could not be assumed to be stationary. This difference is even more important when it is considered that the response does not vary linearly with wind speed but approximately with the cube of the wind speed (see section 6.6.4). Consequently, the method of determining the ensembles which was adopted, was chosen so as to include as much of the data recorded at high wind speeds as possible whilst minimising the problem discussed above.

6.6.2 Mean wind direction 165 to 195 degrees

6.6.2.1 Presentation of results

Table 6.7 gives the results of the selective ensemble averaging for winds blowing onto the south face. The minimum, mean and maximum wind speed and direction are given for each of the ten ensembles. The rms acceleration, natural frequency and damping are given for all three modes for each of the ensembles. Sections 6.4.4 and 6.4.5 give details of how these quantities were obtained.
# TABLE 6.7 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR WIND BLOWING ONTO SOUTH FACE (WIND DIRECTION 165 to 195 DEGREES)

<table>
<thead>
<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>EW1 MODE</th>
<th>NS1 MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min Speed (m/s)</td>
<td>Max Speed (m/s)</td>
<td>No. rec</td>
<td>Mean Speed (m/s)</td>
</tr>
<tr>
<td>4.51</td>
<td>5.50</td>
<td>100</td>
<td>5.08</td>
</tr>
<tr>
<td>5.51</td>
<td>6.05</td>
<td>102</td>
<td>5.78</td>
</tr>
<tr>
<td>6.06</td>
<td>6.65</td>
<td>103</td>
<td>6.35</td>
</tr>
<tr>
<td>6.66</td>
<td>7.14</td>
<td>103</td>
<td>6.86</td>
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<tr>
<td>7.15</td>
<td>7.75</td>
<td>100</td>
<td>7.44</td>
</tr>
<tr>
<td>7.76</td>
<td>8.55</td>
<td>100</td>
<td>8.14</td>
</tr>
<tr>
<td>8.56</td>
<td>9.42</td>
<td>100</td>
<td>8.96</td>
</tr>
<tr>
<td>9.43</td>
<td>10.40</td>
<td>100</td>
<td>9.84</td>
</tr>
<tr>
<td>10.41</td>
<td>12.00</td>
<td>103</td>
<td>11.15</td>
</tr>
<tr>
<td>12.01</td>
<td>15.10</td>
<td>100</td>
<td>13.31</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>Θ1 MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min Speed (m/s)</td>
<td>Max Speed (m/s)</td>
<td>No. rec</td>
</tr>
<tr>
<td>4.51</td>
<td>5.50</td>
<td>100</td>
</tr>
<tr>
<td>5.51</td>
<td>6.05</td>
<td>102</td>
</tr>
<tr>
<td>6.06</td>
<td>6.65</td>
<td>103</td>
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<td>6.66</td>
<td>7.14</td>
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<td>7.15</td>
<td>7.75</td>
<td>100</td>
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<td>7.76</td>
<td>8.55</td>
<td>100</td>
</tr>
<tr>
<td>8.56</td>
<td>9.42</td>
<td>100</td>
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<tr>
<td>9.43</td>
<td>10.40</td>
<td>100</td>
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<tr>
<td>10.41</td>
<td>12.00</td>
<td>103</td>
</tr>
<tr>
<td>12.01</td>
<td>15.10</td>
<td>100</td>
</tr>
</tbody>
</table>

Notes for Tables 6.7 to 6.10 inclusive and tables 6.12 to 6.14 inclusive:

All data are for accelerations measured in the NW corner of the 23rd floor.

Acceleration values are rms figures in m/s² and are the square root of the area under the output spectrum between the frequency limits for that particular mode, as given in Table 6.5.

Frequency and damping figures are for the best fit curve applied to the output spectrum between the same frequency limits.

Data for torsional mode are from accelerometer in the E-W direction (perpendicular distance from centre of torsion to line of this accelerometer is 12.44m). The linear component of the response in the torsional mode is included so that a direct comparison can be made with the response in the translational modes.
6.6.2.2 Reliability of data points for low levels of response

In all three modes there is a marked increase in the rms acceleration as the wind speed increases. This is shown in figure 6.9 where both wind speed and rms acceleration are plotted on logarithmic scales. It can be seen that at wind speeds above about 6.5 m/s there is a change in the slope of the response. In all the ensemble averaged spectra, some of the area under the curve will be due to electronic noise in the accelerometer and the signal conditioning. Thus there always appears to be some response at all frequencies in the spectrum. This noise is usually insignificant in the area of the spectrum around a resonance peak. However, at low wind speeds, and therefore, low levels of response, it can lead to a slight increase in the apparent response around a resonance peak. This phenomenon is thought to be responsible for the change in slope in figure 6.9 mentioned above. Confirmation of this is that the spectra obtained for these low wind speeds show a much greater amount of random scatter on the "shoulders" of the resonance peak. This leads the curve fitting routine to produce a higher level of damping than is probably due to the modal response alone.

6.6.2.3 Response in the 61 mode measured in different positions

Figure 6.9 also shows that the response in the torsional mode is almost exactly parallel to that of the EV1 mode. These two responses were calculated from the time histories obtained from the same accelerometer. Therefore, to ensure that this fact was not influencing the results obtained, the response in the 61 mode was calculated using data from two other accelerometers. The result is shown in figure 6.10. The response in the 61 mode is almost identical for each of the three positions except for a constant offset caused by the three positions being at different perpendicular distances away from the centre of torsion.

6.6.2.4 Linear regression

Figure 6.11 shows the results of applying a linear regression best fit analysis to all the data points shown in figure 6.9. A ±10% error bar has been added to all the points as this is the variance error that results
FIGURE 6.9  rms acceleration against wind speed for records with mean wind direction 165° to 195°
FIGURE 6.10  rms acceleration against wind speed for records with mean wind direction 165° to 195° showing response in torsional mode as measured by three different accelerometers
FIGURE 6.11  rms acceleration against wind speed for records with mean wind direction 165° to 195°
linear regression using all points
±10% error bar included for all points
from using 100 records in the ensembles. The linear regression represents
the best fit for the rms response ($X_r$) being proportional to the wind
velocity ($V$) raised to exponent $\alpha$. It can be seen that the best fit lines
do not go through all the points even if the maximum ±10% error is
included. However, if all the data points which represent rms
accelerations less than $0.5 \times 10^{-4}$ m/s$^2$ are ignored, for the reasons given
above, then the best fit lines go through all the points, as can be seen in
figure 6.12. The exponents obtained from the linear regression are also
given in figures 6.11 and 6.12.

6.6.2.5 Effect of increasing wind speed on natural frequency and damping

Table 6.7 gives the frequency and damping figures for the best fit
curve applied to the relevant portion of the output spectrum. In all three
modes there is a general trend for the natural frequency to decrease as the
wind speed increases. There is also a trend for the damping to increase
with increasing wind speed. Both these trends are in line with what was
observed in the forced vibration test. However, the trend in the damping
is complicated by the reliability of the damping values at low wind speeds
mentioned in section 6.6.2.2. Also, as seen in figures 6.7 and 6.8, the
spectra obtained for the torsional mode are much more ragged than those
obtained for the translational modes.

6.6.3 Mean wind directions 195 to 285 degrees

Tables 6.8, 6.9 and 6.10 show the results of the selective ensemble
averaging for wind records from 195° to 225°, 225° to 255° and 255° to 285°
respectively. Figures 6.13, 6.15 and 6.17 show respectively how the
response increases with wind speed. As in the results obtained for the
165° to 195° sector, the responses obtained at low wind speed are higher
than would be anticipated from an examination of the rest of the results.
Consequently, for the reasons set out in section 6.6.2.2, data points where
the response is less than $0.5 \times 10^{-4}$ m/s$^2$ have been omitted from the linear
regression calculations. The results of the linear regression for these
three wind directions are given in figures 6.14, 6.16 and 6.18.
FIGURE 6.12  

rms acceleration against wind speed for records with mean wind direction 165° to 195° linear regression using points $> 0.5 \times 10^{-4} \text{ m/s}^2$ ±10% error bar included for all points.
### TABLE 6.8 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR WIND DIRECTIONS 195 to 225 DEGREES i.e. 210 DEGREES +/- 15 DEGREES

<table>
<thead>
<tr>
<th>CHosen Limits</th>
<th>Mean Values</th>
<th>EV1 Mode</th>
<th>NS1 Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min (m/s)</td>
<td>Max (m/s)</td>
<td>No. rec</td>
</tr>
<tr>
<td></td>
<td>4.51</td>
<td>5.55</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>5.56</td>
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<td>100</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td>7.64</td>
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<th>Ang. Acc. (rad/s^2)</th>
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</thead>
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<td>Min (m/s)</td>
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<td>No. rec</td>
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<td>4.51</td>
<td>5.55</td>
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<td>9.84</td>
<td>100</td>
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<td>9.85</td>
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<tr>
<td></td>
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<td>16.34</td>
<td>100</td>
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</tbody>
</table>

(See notes at bottom of table 6.7)
FIGURE 6.13  rms acceleration against wind speed for records with mean wind direction 195° to 225°
FIGURE 6.14:  rms acceleration against wind speed for records with mean wind direction 195° to 225°. Linear regression using points > 0.5 x 10^-4 m/s² ± 10% error bar included for all points.
### TABLE 6.9 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR WIND DIRECTIONS 225 to 255 DEGREES i.e. 240 DEGREES +/- 15 DEGREES

<table>
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<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>EV1 MODE</th>
<th>NS1 MODE</th>
</tr>
</thead>
<tbody>
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<td>Max (m/s)</td>
<td>No. rec</td>
<td>Speed (m/s)</td>
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(See notes at bottom of table 6.7)
FIGURE 6.15  rms acceleration against wind speed for records with mean wind direction 225° to 255°
FIGURE 6.16  
RMS acceleration against wind speed for records with mean wind direction 225° to 255°, linear regression using points > 0.5 x 10^{-4} m/s², ±10% error bar included for all points.
### TABLE 6.10 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR WIND BLOWING ONTO WEST FACE (WIND DIRECTION 255 to 285 DEGREES)

<table>
<thead>
<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>EW1 MODE</th>
<th>NS1 MODE</th>
</tr>
</thead>
<tbody>
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<td>Min</td>
<td>Max</td>
<td>No.</td>
</tr>
<tr>
<td>Min</td>
<td>Max</td>
<td>rec</td>
<td>(m/s)</td>
</tr>
<tr>
<td>5.01</td>
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</tr>
<tr>
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<td>6.88</td>
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<tr>
<td>7.62</td>
<td>8.29</td>
<td>100</td>
<td>7.94</td>
</tr>
<tr>
<td>8.30</td>
<td>9.01</td>
<td>101</td>
<td>8.65</td>
</tr>
<tr>
<td>9.02</td>
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</tr>
<tr>
<td>11.04</td>
<td>14.26</td>
<td>100</td>
<td>12.24</td>
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</table>

<table>
<thead>
<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>GW1 MODE</th>
<th>Ang. Acc. (rad/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min</td>
<td>Max</td>
<td>No.</td>
</tr>
<tr>
<td>Min</td>
<td>Max</td>
<td>rec</td>
<td>(m/s)</td>
</tr>
<tr>
<td>5.01</td>
<td>6.14</td>
<td>100</td>
<td>5.64</td>
</tr>
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<td>6.87</td>
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<td>6.88</td>
<td>7.61</td>
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<td>9.94</td>
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</tr>
<tr>
<td>11.04</td>
<td>14.26</td>
<td>100</td>
<td>12.24</td>
</tr>
</tbody>
</table>

(See notes at bottom of table 6.7)
FIGURE 6.17  rms acceleration against wind speed for records with mean wind direction 255° to 285°
FIGURE 6.18  

rms acceleration against wind speed for records with mean wind direction $255^\circ$ to $285^\circ$
linear regression using points $> 0.5 \times 10^{-4}$ m/s$^2$
$\pm 10\%$ error bar included for all points
The remarks made above concerning the results obtained for the 165° to 195° sector can be seen to apply equally well to the results obtained for these three wind directions. However, there is an anomaly in the results for the ensembles where the wind was blowing onto the west face. The response in all three modes in the ensemble with average wind speed of 8.65 m/s, and to a lesser extent the ensemble with average wind speed of 7.94 m/s, is less than would be calculated on the basis of the linear regression shown in figure 6.18. Consequently, the response for all three modes for the former of these ensembles is almost the only instance where a measured response (±10% to allow for variance error) does not coincide with the response obtained by linear regression. The only exceptions to this are some measured responses of less than 0.5 x10^-4 m/s^2 and the response in the NS1 mode for the ensemble with average wind speed of 12.24 m/s in the same 255° to 285° sector. However, the best fit line which just fails to come within 10% of this last value is influenced by the response of the ensemble with average wind speed of 8.65 m/s.

6.6.4 Response in each mode vs. wind direction

Figure 6.19 shows the response in the EW1 mode in each of the four wind sectors examined in detail. Only those points which represent responses of greater than 0.5 x10^-4 m/s^2 have been included. The best fit lines obtained by linear regression are also included. It can be seen that although the response in the 180° and 210° sectors is almost identical, there is a greater response in the 240° sector and an even greater one still in the 270° sector. These differences in response are evident across the whole range of measured wind speeds. However, the exponents for these sectors are greatest for the ones with least response (across-wind excitation) diminishing as the level of response increases, the lowest exponent being for the sector with the greatest response (along-wind excitation).

In contrast, figure 6.20 shows that there is little change with wind direction in the response in the NS1 mode. However, the greatest response is obtained for across-wind excitation in the 270° sector. The response in the θ1 mode with wind direction is shown in figure 6.21. The change in response with wind direction is similar to that for the EW1 mode.
rms acceleration in EW1 mode against wind speed for four different wind directions
linear regression using points > $0.5 \times 10^{-4} \text{ m/s}^2$

FIGURE 6.19
FIGURE 6.20  rms acceleration in NS1 mode against wind speed for four different wind directions linear regression using points $> 0.5 \times 10^{-4} \text{ m/s}^2$
FIGURE 6.21  rms acceleration in \( \theta 1 \) mode against wind speed for four different wind directions linear regression using points \( > 0.5 \times 10^{-4} \text{ m/s}^2 \)
Table 6.11 gives the exponents obtained from the linear regression for all four wind directions and for all three modes. The exponents are given when both all points are included, and when only points representing rms accelerations in excess of $0.5 \times 10^{-4} \text{ m/s}^2$ are included. There is little variation between these two although, for the reasons set out in section 6.6.2.2, the latter are considered to be more reliable. This latter set ranges from 2.77 to 3.32 with an average value of 2.97. It should be noted that although all of these exponents are quoted to three significant figures to show variations between them, quoting them to two significant figures would give a better idea of their accuracy.

### TABLE 6.11 EXPONENTS FOR WIND VELOCITY ($\alpha$)

**A: From regression lines using all points:**

<table>
<thead>
<tr>
<th>MODE</th>
<th>MEAN WIND DIRECTION ($^\circ$)</th>
</tr>
</thead>
<tbody>
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<td>EV1</td>
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<tr>
<td>NS1</td>
<td>2.74</td>
</tr>
<tr>
<td>$\theta$</td>
<td>2.94</td>
</tr>
</tbody>
</table>

**B: From regression lines using only points greater than $0.5 \times 10^{-4} \text{m/s}^2$:**

<table>
<thead>
<tr>
<th>MODE</th>
<th>MEAN WIND DIRECTION ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EW1</td>
<td>3.32</td>
</tr>
<tr>
<td>NS1</td>
<td>3.10</td>
</tr>
<tr>
<td>$\theta$</td>
<td>3.31</td>
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</table>
6.6.5 Response in each mode from all twelve 30° sectors

Table 6.12 gives the results of selective ensemble averaging for all twelve 30° wind sectors for wind speeds from 9.01 to 10.00 m/s. It should be noted that only in the four sectors examined previously were there at least 100 records in the ensembles, and that in five of the sectors there were less than 20 records. Figure 6.22 shows the response in each mode for all of the eleven sectors where records were obtained. Error bars have been added to each point denoting the possible variance error for each ensemble. It should be noted that as the variance error is a percentage of the response, even for the same ensemble, the length of the error bar will be longest for the mode with the greatest response and shortest for the mode with the least response.

Ignoring, for the time being, the response to easterly winds, it can be seen in figure 6.22 that the response in all three modes is symmetrical about both the east-west and north-south axes. The slight deviation from this symmetry is well within the possible excursions due to variance error. However, the response to easterly winds is much less than that due to westerly winds and cannot be explained by variance error. Some of the difference is due to the lower mean wind speed for the ensemble of easterly winds but, even if this is increased assuming that the response varies with the cube of the wind speed, the difference between the two ensembles is still greater than can be ascribed to variance error.

Figures 6.19 to 6.21 show the relative response to westerly winds compared with those of the other three sectors examined in detail. It is evident from them that the particular wind speed range chosen (9.01 to 10.00 m/s) does not represent a special case. However, it should be remembered that there are only 11 records in the ensemble representing easterly winds. Therefore, ensembles for a lower wind speed range were examined to see whether they too showed a lower response to easterly rather than westerly winds.
### TABLE 6.12 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR WIND SPEED 9.01 to 10.00 m/s, FROM ALL WIND DIRECTIONS

<table>
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<th>NS1 MODE</th>
</tr>
</thead>
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<td>(degs)</td>
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<td>Speed (m/s)</td>
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<td>9.70</td>
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<table>
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<th>Ang. Acc. (rad/s²) (x10^-6)</th>
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</thead>
<tbody>
<tr>
<td>(degs)</td>
<td>(degs)</td>
<td>No. rec</td>
<td>Speed (m/s)</td>
</tr>
<tr>
<td>135.0</td>
<td>165.0</td>
<td>17</td>
<td>9.70</td>
</tr>
<tr>
<td>165.0</td>
<td>195.0</td>
<td>108</td>
<td>9.50</td>
</tr>
<tr>
<td>195.0</td>
<td>225.0</td>
<td>247</td>
<td>9.50</td>
</tr>
<tr>
<td>225.0</td>
<td>255.0</td>
<td>166</td>
<td>9.46</td>
</tr>
<tr>
<td>255.0</td>
<td>285.0</td>
<td>110</td>
<td>9.47</td>
</tr>
<tr>
<td>285.0</td>
<td>315.0</td>
<td>45</td>
<td>9.50</td>
</tr>
<tr>
<td>315.0</td>
<td>345.0</td>
<td>56</td>
<td>9.48</td>
</tr>
<tr>
<td>345.0</td>
<td>15.0</td>
<td>61</td>
<td>9.44</td>
</tr>
<tr>
<td>15.0</td>
<td>45.0</td>
<td>5</td>
<td>9.29</td>
</tr>
<tr>
<td>45.0</td>
<td>75.0</td>
<td>14</td>
<td>9.37</td>
</tr>
<tr>
<td>75.0</td>
<td>105.0</td>
<td>11</td>
<td>9.33</td>
</tr>
</tbody>
</table>

**Note:**

There were no records with mean wind speeds 9.01 to 10.00 m/s and mean wind direction 105° to 135°.

(See also notes at bottom of table 6.7)
FIGURE 6.22  rms acceleration against wind direction for wind speed 9.01 to 10.00 m/s variance error bar included for all points
Table 6.13 compares the response to easterly and westerly winds for ensembles where the mean wind speed was between 7.51 and 9.00 m/s. This time there are 51 records in the easterly ensemble so the variance error is only ±14%. For the 9.01 to 10.00 m/s ensembles, the response in each of the three modes to easterly winds was between 53% and 59% of that due to westerly winds. For the 7.51 to 9.00 m/s ensembles the corresponding figures are 75% to 87% and, in this case, the discrepancy between the two ensembles can be explained by variance error for the EW1 and 01 modes, but not quite for the NS1 mode. However, it should be remembered that the westerly ensemble essentially comprises the two ensembles with the lower than expected response (see figures 6.17 and 6.18).

<table>
<thead>
<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>EW1 MODE</th>
<th>NS1 MODE</th>
<th>01 MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min Max No.</td>
<td>Speed Dirn.</td>
<td>Accel. (x10⁻⁴)</td>
<td>Accel. (x10⁻⁴)</td>
<td>Accel. Ang. Acc. (x10⁻⁶)</td>
</tr>
<tr>
<td>(degs) (degs)</td>
<td>m/s (degs)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>75.0 105.0 51</td>
<td>8.28 83.7</td>
<td>1.94</td>
<td>0.882</td>
<td>1.04</td>
</tr>
<tr>
<td>255.0 285.0 217</td>
<td>8.24 270.3</td>
<td>2.30</td>
<td>1.17</td>
<td>1.19</td>
</tr>
</tbody>
</table>

(See notes at bottom of table 6.7)

6.6.6 Response in each mode from 175° to 285° in 10° sectors

The lack of records from an easterly direction precludes a definite conclusion, but the evidence points to the response to easterly winds being less than that due to westerly winds of the same mean velocity. To see whether this difference could be ascribed to the presence of the other blocks on the estate causing greater turbulence and therefore increased
response, the response in each mode was found for each of the eleven 10° sectors from 175° to 285°. The results are given in Table 6.14 and plotted in figure 6.23.

**TABLE 6.14 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR WIND SPEED**

9.01 to 10.00 m/s, WIND DIRECTION 175 to 285 degrees

<table>
<thead>
<tr>
<th>CHOSEN LIMITS</th>
<th>MEAN VALUES</th>
<th>EW1 MODE</th>
<th>NS1 MODE</th>
<th>01 MODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min (degs)</td>
<td>Max (degs)</td>
<td>No. rec</td>
<td>Speed (m/s)</td>
<td>Dirn. (degs)</td>
</tr>
<tr>
<td>175.0</td>
<td>185.0</td>
<td>30</td>
<td>9.45</td>
<td>180.4</td>
</tr>
<tr>
<td>185.0</td>
<td>195.0</td>
<td>66</td>
<td>9.52</td>
<td>190.3</td>
</tr>
<tr>
<td>195.0</td>
<td>205.0</td>
<td>94</td>
<td>9.49</td>
<td>200.5</td>
</tr>
<tr>
<td>205.0</td>
<td>215.0</td>
<td>85</td>
<td>9.47</td>
<td>209.6</td>
</tr>
<tr>
<td>215.0</td>
<td>225.0</td>
<td>68</td>
<td>9.53</td>
<td>219.3</td>
</tr>
<tr>
<td>225.0</td>
<td>235.0</td>
<td>65</td>
<td>9.46</td>
<td>230.0</td>
</tr>
<tr>
<td>235.0</td>
<td>245.0</td>
<td>66</td>
<td>9.43</td>
<td>239.7</td>
</tr>
<tr>
<td>245.0</td>
<td>255.0</td>
<td>35</td>
<td>9.53</td>
<td>249.2</td>
</tr>
<tr>
<td>255.0</td>
<td>265.0</td>
<td>38</td>
<td>9.49</td>
<td>260.4</td>
</tr>
<tr>
<td>265.0</td>
<td>275.0</td>
<td>42</td>
<td>9.45</td>
<td>269.5</td>
</tr>
<tr>
<td>275.0</td>
<td>285.0</td>
<td>30</td>
<td>9.48</td>
<td>279.6</td>
</tr>
</tbody>
</table>

(See notes at bottom of table 6.7)

Figure 6.23 shows that the response in all three modes to a 280° wind is less than that to a 260° wind although this discrepancy could be due to variance error. Other than this, there is little to suggest that the other blocks on the estate (which lie on bearings from 237° to 263°) have a significant affect on the response of Hume Point. However, it should be remembered that the ensemble into which a particular record is put depends upon the mean wind speed and direction during that record. Therefore
FIGURE 6.23  rms acceleration against wind direction for wind speed 9.01 to 10.00 m/s and for wind directions 175° to 285° variance error bar included for all points
records with mean directions well away from the direction of the blocks could, for part of the record, have had wind blowing when the blocks were directly upstream of Hume Point. The further away the mean direction of the record is from the position of the other blocks, the less likely this is to happen. The effect of this will be to smooth out any sudden changes caused by the other blocks being upstream of Hume Point.

The presence of the other blocks on the estate is a possible explanation for the greater response to westerly winds rather than easterly ones. It could also explain why the ensemble shown in figure 6.18 with a mean wind speed of 8.65 m/s has a response which is less than expected. This would be the case if the wind was blowing with the other blocks upstream of Hume Point for a much lower proportion of the 28 hours that this point represents than is the case for the average for the other ensembles for this sector. However, the consequence of accepting this explanation is that it must be accepted that the response to winds for at least two of the three 30° sectors examined in detail is greater than would be the case if the blocks were not there. Against this though, figure 6.22 shows that there is virtually no difference in the response in the two thirty degree sectors centred on 240° and 300°.

6.6.7 Response in all three modes at all 14 measurement positions

Table 6.15 shows the response in all three fundamental modes as measured by all fourteen accelerometers. To ensure that the ensemble for each of the channels contained the same records, only records obtained after the installation of the 32 channel data acquisition system were used. Figure 6.24 shows the plan mode shapes for each of the fundamental modes as measured at the four corners of the 23rd floor. This should be compared with figure 4.24 which shows the same plan mode shapes obtained during the forced vibration testing.
TABLE 6.15 RESULTS OF SELECTIVE ENSEMBLE AVERAGING FOR ALL CHANNELS - WIND SPEED 9.01 to 10.00 m/s, WIND DIRECTION 195 to 225 degrees

<table>
<thead>
<tr>
<th>CHANNEL NUMBER AND POSITION</th>
<th>EV1 MODE Accel. (x10^-4)</th>
<th>NS1 MODE Accel. (x10^-4)</th>
<th>Θ1 MODE Accel. Ang. Acc. (x10^-4) (x10^-6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 (23rd floor, NW corner, NS dirn.)</td>
<td>0.261</td>
<td>1.55</td>
<td>0.657 10.03</td>
</tr>
<tr>
<td>6 (23rd floor, SE corner, NS dirn.)</td>
<td>0.168</td>
<td>1.10</td>
<td>0.901 8.23</td>
</tr>
<tr>
<td>8 (23rd floor, SW corner, NS dirn.)</td>
<td>0.258</td>
<td>1.52</td>
<td>0.629 9.61</td>
</tr>
<tr>
<td>10 (23rd floor, NE corner, NS dirn.)</td>
<td>0.121</td>
<td>0.844</td>
<td>0.632 5.78</td>
</tr>
<tr>
<td>12 (23rd floor, C of T, NS dirn.)</td>
<td>0.196</td>
<td>1.37</td>
<td>0.194</td>
</tr>
<tr>
<td>5 (23rd floor, NW corner, EW dirn.)</td>
<td>1.59</td>
<td>0.448</td>
<td>1.06 8.49</td>
</tr>
<tr>
<td>7 (23rd floor, SE corner, EW dirn.)</td>
<td>1.87</td>
<td>0.358</td>
<td>0.963 8.88</td>
</tr>
<tr>
<td>9 (23rd floor, SW corner, EW dirn.)</td>
<td>1.76</td>
<td>0.329</td>
<td>0.919 8.48</td>
</tr>
<tr>
<td>11 (23rd floor, NE corner, EW dirn.)</td>
<td>1.34</td>
<td>0.357</td>
<td>0.772 6.21</td>
</tr>
<tr>
<td>13 (23rd floor, C of T, EW dirn.)</td>
<td>1.17</td>
<td>0.120</td>
<td>0.0331</td>
</tr>
<tr>
<td>14 (11th floor, NW corner, NS dirn.)</td>
<td>0.143</td>
<td>0.742</td>
<td>0.302 4.62</td>
</tr>
<tr>
<td>16 (11th floor, SE corner, NS dirn.)</td>
<td>0.101</td>
<td>0.549</td>
<td>0.429 3.92</td>
</tr>
<tr>
<td>15 (11th floor, NW corner, EW dirn.)</td>
<td>0.689</td>
<td>0.212</td>
<td>0.493 3.96</td>
</tr>
<tr>
<td>17 (11th floor, SE corner, EW dirn.)</td>
<td>0.853</td>
<td>0.176</td>
<td>0.487 4.49</td>
</tr>
</tbody>
</table>

Notes:

"C of T" stands for centre of torsion (nominal).

The ensemble for each of the above channels (except channel 13) contained the same 99 records and had a mean wind speed of 9.49 m/s and a mean wind direction of 208.0 degrees. The ensemble for channel 13 contained only 98 of these records. It had a mean wind speed of 9.49 m/s and a mean wind direction of 207.8 degrees.

Linear acceleration values are rms figures in m/s^2 and are the square root of the area under the output spectrum between the frequency limits for that particular mode, as given in Table 6.5. The linear component of the response in the torsional mode is included so that a direct comparison can be made with the response in the translational modes.

Angular acceleration values are in radians/s^2.
Figure 6.24 Hume Point plan mode shapes (23rd floor) for fundamental modes obtained from selective ensemble averaging (wind speed 9.01 to 10.00 m/s, wind direction 195° to 225°)
Table 6.15 and figure 6.24 show that the response in both directions in the NE corner is less than anticipated in all three modes. This lower response had been noted during the course of collecting the data, but this had been ascribed to channels 9 to 13 (which used the same signal conditioning box) having a significantly lower calibration figure (volts/g) than the other channels. Further investigation showed that the other three channels using this signal conditioning box (channel 9 and the two at the nominal centre of torsion) also had much lower responses than expected for some of the three months that the 32 channel data acquisition system was working. There seems to have been an intermittent fault in the signal conditioning box which affected the amplification of the accelerometer signals. Therefore response data from channels 9 to 13 inclusive should be treated with a great deal of caution.

Table 6.15 shows that although the nominal and actual centre of torsion for the 61 mode were quite close they did not coincide. The actual centre of torsion seems to lie a little further east (i.e. nearer the geometric centre) than the nominal centre of torsion. However, the inevitable uncertainty in the figures due to variance error and the comment above about the fault on the signal conditioning box, make it unwise to give a precise figure for this distance.

All the responses measured on the 11th floor are in good agreement with what would be expected given the elevation mode shape and the response on the 23rd floor directly above each measurement position. Thus the mode shape data obtained from the ensemble averaging (at least for the nine channels which can be relied upon) all agree with the mode shapes obtained during the forced vibration testing.

6.6.8 Response in the second order modes

Although no figures for the response of the second order modes are given in tables 6.7 to 6.10, this data was extracted from the results of the selective ensemble averaging. The response in the second order modes could not be separated into the three modes (see Chapter Four). Therefore, the area under the output spectrum from the ensemble averaging between 3.50 and 5.00 Hz was regarded as being the response in all three second order
modes. It should be remembered that this 1.5 Hz section of the output spectrum contains more than twice as many spectral lines as the section for all three fundamental modes put together (0.69 Hz). This is because of the much higher damping in the second order modes (see Tables 4.2 and 4.3).

Figure 6.25 shows the response in both the fundamental and 2nd order modes for the 225° to 255° sector. A similar result was obtained for the other three sectors examined in detail. At the lowest wind speed, the response in the second order modes is 3.6 times that in the three fundamental modes combined. However, most of this difference is attributable to there being so many more spectral lines in the part of the spectrum. As mentioned in section 6.6.2.2, electronic noise in the spectrum can produce an enhanced response at low wind speeds, so the corresponding response figures for the second order modes should be treated with caution. The relative importance of the second order modes at moderate wind speeds can be seen in the top part of figure 6.6. In this figure, the response in the second order modes is bigger than that in the Ø1 mode, but not as large as that in the EV1 mode. Figure 6.25 shows that, unlike the fundamental modes, there is no evidence that the response in the second order modes increases with the cube of the wind speed.

6.7 ASSESSMENT OF PEAK ACCELERATIONS

6.7.1 Problem of obtaining peak accelerations

The ensemble averaging process detailed above can assess the rms accelerations that occur for a given wind speed and direction. However, the peak acceleration that is likely to occur for given wind conditions cannot be assessed by this method. The peak accelerations that occurred in a particular record have to be measured from the accelerometer time history. However, whilst finding the maximum acceleration that occurred in a particular time history is a fairly straightforward task, some safeguards have to be applied to ensure that the result obtained is the peak acceleration response of the structure. As mentioned in section 6.3, some drift occurred in the accelerometer signals, therefore this drift has to be eliminated before the maximum acceleration can be found. Also, care must
FIGURE 6.25  rms acceleration against wind speed for records with mean wind direction 225° to 255° showing response in 2nd order modes
be taken to ensure that the peak acceleration is a real response of the structure and is not a rogue electronic spike in the time history. Finally, to enable a comparison with the rms accelerations obtained from the selective ensemble averaging, and therefore wind loading codes, it would be preferable to obtain peak accelerations in each mode.

6.7.2 Method of obtaining peak accelerations

Some filtering of the original accelerometer time histories was required in order to carry out the above safeguards. However, running a digital filtering program on a single time history on the LSI 11/73 took over two hours. Consequently, the relevant time histories were transferred to a VAX 8700 computer where the analysis was carried out. However, the time histories had to be transferred in ASCII format, so only one hundred were transferred. The time histories from channels 4 and 5 (the orthogonal pair of accelerometers in NW corner) for the last fifty records obtained with wind speeds between 9.01 and 10.00 m/s and with wind direction between 195 and 225 degrees were used. These criteria are the same as those used to show the response in all fourteen channels (see Table 6.15).

Once the one hundred time histories had been transferred to the VAX computer, the following operations were carried on each one. Firstly, the relevant calibration figure for the accelerometer was used to convert each point on the time history from volts to m/s². Then the same program detailed in section 6.3.3.2 was used to remove any linear drift in the time history. The rms acceleration for the whole record was then calculated and stored. Having done this, the time history was passed through a digital filtering program. This program allowed all frequency components above 0.7 Hz through but none below 0.35 Hz. The digital filtering program was then applied to the time history obtained. This time the program allowed all frequency components below 1.6 Hz through but none above 2.6 Hz. The result of this band-pass filtering procedure was that all frequency components below 0.35 Hz or above 2.6 Hz were eliminated, leaving a time history where only frequency components between 0.7 and 1.6 Hz (i.e. the frequencies of the fundamental modes) were retained in full. The rms acceleration of the whole of the filtered time history was then calculated and stored. Finally the maximum (positive or negative) acceleration which
occurred during the filtered time history and its position in the record were noted and stored.

Figure 6.26 shows a one minute extract from a typical example of one of the filtered time histories, which includes the maximum acceleration for the whole record \(7.44 \times 10^{-4} \text{ m/s}^2\) which occurred after 427.9 seconds.

### 6.7.3 Comparison of values obtained in time and frequency domain analysis

Table 6.16 compares the overall average rms values obtained by both time and frequency domain analysis for the records described above. In the time domain, this is the square root of the mean of the square of all fifty rms values. This is not the same as the mean of the rms values (see section 6.4.2 and appendix A). The frequency domain figures were obtained by selective ensemble averaging (see section 6.4.4). The two methods produce answers which are very close. The time domain method produces slightly larger answers as both the original and filtered time histories are for a marginally wider frequency range than the corresponding frequency domain analysis.

### 6.7.4 Results of peak acceleration analysis

A table was made of the stored values from all 50 time histories obtained from channel 4 (NS direction) data. As well as the stored values (original rms, filtered rms and peak accelerations) three ratios were calculated for each time history and stored in the table. These were the ratio between the filtered and original rms values, between the peak and original rms values, and between the peak and filtered rms values. Table 6.17 gives the minimum, median, mean and maximum for each of these quantities. Table 6.18 gives the same information for the data from channel 5 (EW direction).

One important point to be noted from Tables 6.17 and 6.18 is the spread of values for both rms and peak accelerations. In both cases the maximum original rms acceleration is over twice that of the minimum, this factor being only slightly reduced for the filtered values. This spread is typical of those obtained in the selective ensemble averaging.
FIGURE 6.26 One minute extract from filtered time history of acceleration for record HZ2604.901 channel 4

FIGURE 6.27 One minute extract from filtered time history of acceleration for record HZ1903.959 channel 4
### TABLE 6.16 COMPARISON OF AVERAGE RMS ACCELERATIONS OBTAINED BY BOTH TIME AND FREQUENCY DOMAIN ANALYSIS

<table>
<thead>
<tr>
<th>CHANNEL 4 (NS direction)</th>
<th>CHANNEL 5 (EW direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Whole Spectrum (x10^{-3} m/s²)</td>
</tr>
<tr>
<td>Time domain analysis</td>
<td>0.249</td>
</tr>
<tr>
<td>Frequency domain analysis</td>
<td>0.249</td>
</tr>
<tr>
<td></td>
<td>Whole Spectrum (x10^{-3} m/s²)</td>
</tr>
<tr>
<td></td>
<td>0.265</td>
</tr>
<tr>
<td></td>
<td>0.264</td>
</tr>
</tbody>
</table>

### TABLE 6.17 PEAK AND rms ACCELERATION VALUES FOR 50 RECORDS, WIND SPEED 9.01 to 10.00 m/s, WIND DIRECTION 195 to 225 degrees, CHANNEL 4 (NW CORNER, NS DIRECTION)

<table>
<thead>
<tr>
<th></th>
<th>Original (x10^{-3})</th>
<th>Filtered (x10^{-3})</th>
<th>Peak acc. (x10^{-3})</th>
<th>Filter/Original rms acc.</th>
<th>Peak/Filtered rms acc.</th>
<th>Peak/Original rms acc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>MIN</td>
<td>0.149</td>
<td>0.124</td>
<td>0.579</td>
<td>0.407</td>
<td>4.00</td>
<td>1.66</td>
</tr>
<tr>
<td>MEDIAN</td>
<td>0.216</td>
<td>0.170</td>
<td>0.814</td>
<td>0.783</td>
<td>4.76</td>
<td>3.57</td>
</tr>
<tr>
<td>MEAN</td>
<td>0.240</td>
<td>0.171</td>
<td>0.829</td>
<td>0.748</td>
<td>4.87</td>
<td>3.65</td>
</tr>
<tr>
<td>MAX</td>
<td>0.387</td>
<td>0.222</td>
<td>1.35</td>
<td>0.967</td>
<td>7.53</td>
<td>6.66</td>
</tr>
</tbody>
</table>

Note: All accelerations are in m/s²

### TABLE 6.18 PEAK AND rms ACCELERATION VALUES FOR 50 RECORDS, WIND SPEED 9.01 to 10.00 m/s, WIND DIRECTION 195 to 225 degrees, CHANNEL 5 (NW CORNER, EW DIRECTION)

<table>
<thead>
<tr>
<th></th>
<th>Original (x10^{-3})</th>
<th>Filtered (x10^{-3})</th>
<th>Peak acc. (x10^{-3})</th>
<th>Filter/Original rms acc.</th>
<th>Peak/Filtered rms acc.</th>
<th>Peak/Original rms acc.</th>
</tr>
</thead>
<tbody>
<tr>
<td>MIN</td>
<td>0.163</td>
<td>0.140</td>
<td>0.604</td>
<td>0.471</td>
<td>4.00</td>
<td>2.17</td>
</tr>
<tr>
<td>MEDIAN</td>
<td>0.258</td>
<td>0.187</td>
<td>0.987</td>
<td>0.827</td>
<td>5.23</td>
<td>4.02</td>
</tr>
<tr>
<td>MEAN</td>
<td>0.259</td>
<td>0.193</td>
<td>1.02</td>
<td>0.770</td>
<td>5.26</td>
<td>4.07</td>
</tr>
<tr>
<td>MAX</td>
<td>0.366</td>
<td>0.300</td>
<td>1.87</td>
<td>0.959</td>
<td>7.25</td>
<td>6.07</td>
</tr>
</tbody>
</table>

Note: All accelerations are in m/s²
Although the maximum peak acceleration for channel 5 was for the same record that had the maximum filtered rms acceleration, the maximum peak acceleration for channel 4 was for a record that had a filtered rms acceleration only 5% higher than the mean, and original rms acceleration 12% lower than the mean. This record produced the maximum factors between peak and rms accelerations (7.53 and 6.66). Figure 6.27 shows a one minute portion of the filtered time history for this record which includes the maximum acceleration ($1.35 \times 10^{-3} \text{ m/s}^2$) which occurred after 868.3 seconds. Figure 6.27 shows that less than 30 seconds before the maximum acceleration occurred, the peak accelerations had only been about one sixth of the value of the maximum. Examination of the original time history for this record confirmed that almost all the response of the building was in the fundamental modes. The difference in rms values for the maximum peak accelerations for the two channels shows one of the difficulties in predicting peak accelerations values.

For both channels the maximum value of the peak to filtered rms acceleration ratio is less than twice that of the minimum. However, there is a fourfold difference in the maximum to minimum value of the peak to original rms acceleration ratio for channel 4, the corresponding value for channel 5 being 2.8. This difference is due to the variability of the presence of frequency components other than those of the fundamental frequencies in the original time history. It indicates that predicting the peak acceleration from a given filtered rms acceleration is likely to be more accurate than predicting it from the overall rms acceleration level.

It should be remembered that, for the reasons set out in Appendix A, the ratio of the filtered to original rms accelerations given in Tables 6.17 and 6.18 is not the same as the percentage of the total response that is in the fundamental modes. Both quantities have to be squared to be compared in this way. Thus, for channel 4, the mean filtered response is 51% of the mean original response, the corresponding figure for channel 5 being 56%. However, the overall average values given in Table 6.16 should be used in preference to the mean values given in Tables 6.17 and 6.18 if such a comparison is required.
6.7.5 Limitations of peak acceleration analysis

Although the above peak acceleration analysis has produced some useful and interesting results, it does have its limitations. The most important limitation is the scope of the analysis. Only two channels of fifty records have been analysed. Further analysis is required in order to see whether there are any significant changes with wind speed or direction. However, the problems of filtering a large number of records prevented this. Whilst it would be possible to carry out some analysis on unfiltered records this would be open to producing spurious results. Although a few electronic spikes will have little effect on the ensemble averaging results, as they do not occur at the fundamental frequencies of the building, they are liable to be interpreted as the peak accelerations in a time history. Thus either filtering or visual inspection of all the time histories would be required in order to eliminate any spurious data.

One other limitation of the analysis is that the filtering produced a time history containing frequency components of all three fundamental modes rather than just one of them. In the case of channel 4, the spectral analysis showed that 83% of the response in all three fundamental modes (as given in table 6.5) was in the NS1 mode. If the peak acceleration is all in the NS1 mode then the peak/filtered rms acceleration figures given in table 6.17 are underestimates of this ratio for the NS1 mode but large overestimates of this ratio for the EW1 and Θ1 modes. In the case of channel 5, the spectral analysis showed that 66% of the response in all three fundamental modes was in the EW1 mode, 29% in the Θ1 mode and 5% in the NS1 mode. Further analysis of the peak to rms acceleration ratio in each fundamental mode would require a structure where the modes were much more separated than is the case at Hume Point.

6.8 COMPARISON WITH OTHER FULL-SCALE TEST RESULTS

Few of the full-scale tests mentioned in Chapters One to Three obtained sufficient data to enable any analysis of the change of dynamic response with wind speed or direction to be carried out. However, subject to the assessments given previously in Chapter Three, some qualitative
results have been obtained from previous full-scale tests.

The selective ensemble averaging technique was used in analysing data from the Sheffield University Arts Tower (see sections 2.9, 3.2.6 and ref 83). The majority of the records obtained were from the predominant wind direction, so preventing analysis of change in response with wind direction. However, the change of response with wind speed was calculated for the thirty degree sector with the greatest number of records. The previously unpublished result was that the along-wind response increased proportional to the 2.6 power of the measured wind speed.

The response of the 43 storey New York office tower tested by UWO (see sections 1.4.2, 3.2.2.1 and ref 21) was found to be approximately proportional to the cube of the wind speed if wind direction was ignored.

In the case of Commerce Court Tower (see sections 1.4.6, 3.2.5, and ref 44) it was found that, on average, both across-wind and along-wind rms accelerations increased in proportion to the 3.3 power of roof height wind velocity. This result (which was in broad agreement with the analytical model they used) was obtained by averaging together the exponents from wind directions where the correlation coefficient for the fit was at least 0.5. No indication is given as to what proportion of the wind directions were used to obtain this result. For some wind directions, the full-scale data showed the marked influence of other buildings. However, it should be noted that the largest influence was that of a building 50m taller than Commerce Court Tower but considerably less than one building height away from it.

Although the Sydney Tower (see section 1.4.3 and ref 28) is a very different type of structure from Hume Point, it is interesting to note that the average peak to (unfiltered) rms acceleration factor was 3.63 for along-wind response and 3.65 for across-wind response. None of the other full-scale tests give any results for peak to rms acceleration factors. The along-wind response of the tower is sensitive to wind direction, although there are important differences in the terrain for different wind directions.
The mean peak to rms acceleration factors obtained for Hume Point (3.65 and 4.07) are in good agreement with those obtained on the Sydney Tower. However, the spread of values obtained at Hume Point shows that using these mean values could lead to peak accelerations being seriously underestimated.

The range of values obtained at Hume Point for the power to which the measured wind speed had to be raised in order to be proportional to the response (2.7 to 3.3) covers the range of values observed on the tests described above. Apart from the Sydney Tower, where there are important differences in terrain for different wind directions, Hume Point is the only building of those mentioned above which is not influenced profoundly by comparably sized buildings close by. This has meant that the symmetry of response obtained at Hume Point is unique amongst these experiments.

6.9 COMPARISON OF PREDICTED AND FULL-SCALE RESULTS

6.9.1 Calculation methods used in the comparison

The present U.K. wind loading code does not consider the dynamic response of tall buildings, so the methods used by ESDU International have been used in the comparison exercise as these are the methods most likely to be used by U.K. engineers. The along-wind response of Hume Point has been calculated in accordance with ESDU Data Item 88019 (ref 89) and its corresponding computer program version ESDU Data Item 88036 (ref 105). The across-wind response of Hume Point has been calculated in accordance with ESDU Data Item 89049 (ref 106) and its corresponding computer version ESDU Data Item 90011 (ref 107).

6.9.2 Comparison of wind speed exponent and peak response factor

Two of the relationships predicted by the ESDU methods can be compared with the full-scale measurements from Hume Point without carrying out the whole of the calculation procedure. The ESDU methods predict that the along-wind rms acceleration increases in proportion to the wind speed raised to the power of 3.09, and the across-wind rms acceleration increases
in proportion to the wind speed raised to the power of 2.63. Table 6.11 (B) shows that the full-scale exponents range from 2.73 to 3.32. However, although the highest exponent for the NS1 mode is for southerly winds, the EW1 mode also has the highest exponent for southerly winds i.e. when the EW1 mode is the across-wind response.

In the ESDU methods the peak response factor for the resonant component of response \((g_R)\) is dependent only upon the relevant natural frequency. For Hume Point, using the values from Table 6.12 of 0.939 Hz for EW1 and 1.152 Hz for NS1, \(g_R\) is 4.17 for the EW1 response and 4.22 for the NS1 response. These values should be compared with those given in Tables 6.17 and 6.18 where it can be seen that the predicted values are slightly higher than the mean or median values in the tables but considerably less than the maximum values.

6.9.3 Comparison of measured and predicted tip rms resonant accelerations

6.9.3.1 Input data for calculation methods

The ESDU methods have been used to compare the predicted response of Hume Point with that measured at full-scale. The rms resonant component of response has been calculated for winds blowing directly onto the west and south faces. All the calculations have been done twice, firstly using measured values as the inputs wherever possible, and secondly using values estimated according to the relevant ESDU method.

Four parameters: mass, natural frequency, structural damping and mode shape have different measured and estimated values. For along-wind response the mass of Hume Point has been estimated using the suggestion in ESDU 88019 that average building density is 340 kg/m³, but for across-wind response the mass has been estimated using 300 kg/m³, the upper limit of the range suggested in ESDU 90011 (150 to 300 kg/m³).

For along-wind response ESDU 88019 suggests the use of Ellis's empirical formulae (see section 4.9.1) but without any indication as to the accuracy of these formulae. Ellis and Littler note in reference 66 that "errors of more than 50% are not uncommon in this type of empirical
prediction." Ellis's original paper (ref 85) is not listed as a derivation or reference by ESDU 88019. It is interesting to note that although ESDU 88019 uses the same formula as Ellis (ref 85) for the lowest frequency translational mode, the two differ in the formula for the frequency of the orthogonal translational mode. Ellis found that the best fit was obtained by using $58/H$ (where $H$ is the height of the structure in metres) but ESDU 88019 gives $52/H$. Unfortunately, ESDU 88019 does not give the source of the formula, so it is not clear whether the formula given is an attempt to improve on Ellis's work or merely a misprint. Whilst this difference is not important given the possible error involved in either formulae, it does have an affect when comparing the measured and predicted response of Hume Point. For the across-vind response ESDU 90011 suggests using $46/H$ for the natural frequency of the fundamental translational mode in both directions, giving 0.688 Hz for both EW1 and NS1.

Although ESDU suggests using a range of damping values, only one estimated value was used in the calculations. Ellis and Littler (ref 66) suggest that a damping value of 0.01 should be used for buildings under 100m tall, and this value has therefore been used in the along-vind calculations. ESDU 90011 suggests using a damping value of 0.005, so this value has been used in the across-vind calculations.

The estimated mode shape is $(z/H)^{1.5}$ i.e. the default mode shape used in the computer program version of the ESDU methods for a uniform vertical cantilever. The measured NS1 mode shape (see figure 4.23) has been used in the calculation of the response although it has been necessary to assume that the mode shape above the 23rd floor increases linearly up to the roof. This has the effect of ensuring that the exponent $N_1$, where the mode shape is $(z/H)^{N_1}$, is the same at a particular height up the building whether the mode shape is normalised to the 23rd floor or the roof. For the measured EW1 mode shape normalised to the 23rd floor (see figure 4.23) $N_1$ varies from 1.0 to 1.15 averaging 1.10, so the mode shape normalised to the roof and used in the calculations has been taken as $(z/H)^{1.1}$.

Apart from the physical dimensions of Hume Point, the other input data for the calculations of predicted response have been taken from other ESDU data items (refs 108, 109, 110). However, some approximations have still
been used. In particular, no account has been made of the fact that the lift housing on the top of Hume Point does not extend across the full width of the building (see figure 5.6). Although ESDU 88019 suggests that the response of the building up to parapet level should be added to that of the lift housing alone, this would involve knowing the mass of these two parts separately. As this is not known, it is preferable to calculate the response as if the building extended only up to parapet level and then up to full height but as if it extended over the full width, the "correct" answer therefore lying in between these two.

6.9.3.2 Response to southerly winds

The response of Hume Point to southerly winds has been evaluated for the ten mean wind speeds used in the selective ensemble averaging and given in Table 6.7. Table 6.19 shows the calculated response to a southerly wind of 13.31 m/s.

**TABLE 6.19 ESDU CALCULATED TIP RMS RESONANT ACCELERATIONS FOR A SOUTHERLY WIND OF 13.31m/s**

<table>
<thead>
<tr>
<th>Along-wind response (NS direction):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actually measured at full-scale:</td>
<td>0.432 x 10^{-3} m/s²</td>
</tr>
<tr>
<td>Calculated using measured parameters:</td>
<td>0.361 x 10^{-3} m/s²</td>
</tr>
<tr>
<td>Calculated using estimated parameters:</td>
<td>0.863 x 10^{-3} m/s²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Across-wind response (EW direction):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actually measured at full-scale:</td>
<td>0.513 x 10^{-3} m/s²</td>
</tr>
<tr>
<td>Calculated using measured parameters:</td>
<td>1.69 x 10^{-3} m/s²</td>
</tr>
<tr>
<td>Calculated using estimated parameters:</td>
<td>4.11 x 10^{-3} m/s²</td>
</tr>
</tbody>
</table>

Note: accelerations are at height of 23rd floor (59.67m above ground)
If it is assumed that the building extends only up to parapet level (63.1 m) then the along-wind response using measured parameters reduces marginally to $0.360 \times 10^{-3} \text{ m/s}^2$. No reduction in mass has been made to allow for the lift housing no longer being considered part of the building.

Changing the mode shape exponent for the NS1 mode to only 0.5 reduces the calculated along-wind response using measured parameters to $0.323 \times 10^{-3} \text{ m/s}^2$. Increasing $N_1$ to 1.5 increases this value to $0.410 \times 10^{-3} \text{ m/s}^2$.

It can be seen from Table 6.19 that the calculated along-wind response using the measured parameters underestimates the actually measured response by 20%. The calculated response using estimated parameters is 2.4 times that obtained using the measured parameters, the majority of this discrepancy being caused by the error in estimating the natural frequency.

Figure 6.28 shows the comparison of actually measured and predicted along-wind rms acceleration against wind speed for southerly winds. It can be seen that the differences observed for the 13.3 m/s case are typical of those obtained at wind speeds down to about 6.5 m/s. Below this speed the actually measured response contains an unacceptably high proportion of electronic noise (see section 6.6.2.2), so these values should be ignored. The actually measured values of response for winds in excess of 6.5 m/s range from 1.15 to 1.25 times that predicted by ESDU 88036 using the measured parameters.

Table 6.19 shows that the calculated EW response using measured parameters is some 3.3 times that actually measured. Figure 6.29 shows the comparison of actually measured and predicted rms EW acceleration against wind speed for southerly winds. It can be seen that the overestimation of the response is least at 13.31 m/s, gradually increasing from 3.3 to 5.1 times the actually measured response, until the effects of electronic noise become evident at a wind speed of about 6.5 m/s. This effect can be attributed to the difference in the wind speed exponent between the predicted and actually measured response, and suggests that a better correlation between the two will be obtained at higher wind speeds. The calculated response using estimated parameters ranges from 2.0 to 2.4 times
FIGURE 6.28 COMPARISON OF ACTUALLY MEASURED AND PREDICTED ALONG-WIND RMS ACCELERATION AGAINST WIND SPEED FOR RECORDS WITH MEAN WIND DIRECTION 165 TO 195 DEGREES
Figure 6.29 Comparison of actually measured and predicted across-wind RMS acceleration against wind speed for records with mean wind direction 165 to 195 degrees.
that obtained using the measured parameters, and is therefore eight to twelve times higher than the actually measured response.

6.9.3.3 Response to westerly winds

For westerly winds there is an upwind roughness change caused by the Cities of London and Westminster which occurs approximately seven kilometres due west of Hume Point. According to ESDU 84030 (ref 108) this leads to an increase in turbulence of approximately 5% at the top of Hume Point which results in a proportional increase in response. If the other blocks on the Freemason's Estate were due west of Hume Point, according to reference 108 there would be an increase of approximately 15% in turbulence compared with that if there were no upwind change of turbulence. Although no evidence of an increase in turbulence at the top of Hume Point was evident in the wind profile experiment, it should be remembered that none of the three data sets were collected when the other blocks were directly upwind of Hume Point.

The response of Hume Point to westerly winds has been evaluated for the eight mean wind speeds used in the selective ensemble averaging and given in Table 6.10. Table 6.20 shows the calculated response to a westerly wind of 12.24 m/s. The values in Table 6.20 have been calculated assuming that there are no upwind changes of turbulence.

If it is assumed that the building extends only up to parapet level (63.1m) then the along-wind response using measured parameters increases slightly to 0.536 x10^{-3} m/s².

Table 6.20 shows that the comparison between actually measured and predicted along-wind response using measured parameters is not as good as that for southerly winds. The calculated response using the measured parameters underestimates the actually measured response by 43%. The calculated response using estimated parameters is 1.8 times that obtained using the measured parameters, the majority of this discrepancy being caused by the error in estimating the natural frequency.
TABLE 6.20 ESDU CALCULATED TIP RMS RESONANT ACCELERATIONS FOR A WESTERLY WIND OF 12.24m/s

<table>
<thead>
<tr>
<th>Along-wind response (EW direction):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actually measured at full-scale:</td>
<td>(0.756 \times 10^{-3} \text{ m/s}^2)</td>
</tr>
<tr>
<td>Calculated using measured parameters:</td>
<td>(0.528 \times 10^{-3} \text{ m/s}^2)</td>
</tr>
<tr>
<td>Calculated using estimated parameters:</td>
<td>(0.954 \times 10^{-3} \text{ m/s}^2)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Across-wind response (NS direction):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Actually measured at full-scale:</td>
<td>(0.431 \times 10^{-3} \text{ m/s}^2)</td>
</tr>
<tr>
<td>Calculated using measured parameters:</td>
<td>(0.707 \times 10^{-3} \text{ m/s}^2)</td>
</tr>
<tr>
<td>Calculated using estimated parameters:</td>
<td>(2.41 \times 10^{-3} \text{ m/s}^2)</td>
</tr>
</tbody>
</table>

Note: accelerations are at height of 23rd floor (59.67m above ground)

Increasing the turbulence intensity by either 5% or 15% increases the response by the corresponding amount. However, even with a 15% increase in turbulence the calculated response using the measured parameters is still 25% less than the actually measured response.

Figure 6.30 shows the comparison of actually measured and predicted along-wind rms acceleration against wind speed for westerly winds. It can be seen that the differences observed for the 12.24 m/s case are typical of those obtained at lower wind speeds. The actually measured values of response range from 1.3 to 1.7 times that predicted by ESDU 88036 using the measured parameters.

It can be seen from Table 6.20 that the calculated across-wind response using measured parameters is about 1.6 times that actually measured. The calculated response using estimated parameters is 3.4 times that obtained using the measured parameters, and is therefore some 5.5
Figure 6.30 Comparison of actually measured and predicted along-wind RMS acceleration against wind speed for records with mean wind direction 255 to 285 degrees.
FIGURE 6.31 COMPARISON OF ACTUALLY MEASURED AND PREDICTED ACROSS-WIND RMS ACCELERATION AGAINST WIND SPEED FOR RECORDS WITH MEAN WIND DIRECTION 255 TO 285 DEGREES
times that of the actually measured response. Figure 6.31 shows the comparison of actually measured and predicted across-wind rms acceleration against wind speed for westerly winds. It can be seen that the differences observed for the 12.24 m/s case are typical of those obtained at lower wind speeds. This reflects the fact that the wind speed exponent for the actually measured response (2.81) is much closer to the predicted one (2.63) than was the case for southerly winds.

6.9.3.4 Comments on comparison with full-scale data

It was shown in section 6.6.5 that the EW response to easterly winds of 7.5 to 9.0 m/s was approximately 85% of that for westerly winds of the same speed. If this relationship was true at all wind speeds, then the response predicted by the ESDU along-wind method would be closer to the actually measured response for easterly winds than westerly ones, but not as close as that for southerly ones.

In section 6.4.4 the response in the three fundamental modes was defined as the total response between specified frequency limits. However, there will be a limited response due to the background component of response at these frequencies. This will be a very small percentage of the total response at these frequencies and can be considered to be negligible. It is certainly not sufficient to account for the differences noted between observed and predicted responses.

In the case of Hume Point the calculated response using the estimated parameters has been greater than that calculated using the measured parameters. This is primarily because the mass and natural frequency have been underestimated by the procedures suggested in the ESDU Data Items. However, it is most important to note that these parameters could just as easily be overestimated in another case, leading to an underestimation of the actual response. On the other hand, the use of a low damping value and mode shape with a high value of N1 will lead to conservative estimates of response.
6.10 COMPARISON OF WIND TUNNEL AND FULL-SCALE RESULTS

6.10.1 Wind tunnel test procedure

In 1989, before the analysis of the full-scale data from Hume Point, a series of experiments were conducted by the BRE Wind Loading Section under Dr. P. A. Blackmore to simulate the dynamic behaviour of Hume Point at model-scale. These tests were conducted in the BRE boundary layer wind tunnel using a rigid 1:200 scale model of Hume Point mounted on the BRE 6-component Kistler force balance (ref 111).

Boundary layer simulations were synthesised using the roughness, barrier and mixing device method (ref 112). Three distinct terrain simulations were used to model the area around Hume Point. Type (i) was for low rise domestic and light industrial buildings (azimuths 10° to 225°). Type (ii) was the same as type (i) but with the superimposed wake flow from the seven upstream tower blocks (azimuths 225° to 265°). Type (iii) was the same as type (i) but with open country extending from Hume Point to a radius of 500m (azimuths 270° to 10°).

Mean velocity, and longitudinal and lateral local turbulence intensity profiles were obtained for all three types of terrain simulation, the results being compared with the values obtained from the full-scale wind profile experiment (see section 5.6). Although the model and full-scale velocity profiles showed excellent agreement, the turbulence intensities measured at full-scale were higher than those that could be obtained at model-scale whilst keeping the good agreement between the model and full-scale velocity profiles (figs 6.32 and 6.33).

To assess the sensitivity of the response of Hume Point to the incident wind force, the model was tested in a standard CP3 category 3 simulation (ref 113) as well as the actual terrain simulation described above. This standard simulation is similar to the type (i) actual terrain simulation. Measurements of the response of the model were made for azimuths 180° to 360° for the actual terrain, and for azimuths 270° to 360° for the standard terrain (with azimuths 180° to 270° being inferred by symmetry) all in 10° increments of azimuth. A time scale factor of 1:200
FIGURE 6.32 PROFILES FOR TERRAIN SIMULATION TYPE (i)
FIGURE 6.33 PROFILES FOR TERRAIN SIMULATION TYPE (ii)
was chosen so that the velocity scale factor was unity i.e. velocities measured in the wind tunnel tests would be the same velocities at full-scale.

6.10.2 Qualitative results from wind tunnel tests

Figure 6.34 shows the equivalent full-scale mean and rms NS displacements obtained for winds of 10, 15 and 20 m/s. Figure 6.35 shows the equivalent measurements obtained for EW displacements, while figure 6.36 shows the torsional response as the equivalent horizontal displacement at the NE corner along a line perpendicular from there to the centre of rotation. All displacements are those predicted at parapet level (63.1m) and assume a linear vertical mode shape. Values of modal mass, natural frequency and damping used in the calculation of these displacements were taken from reference 74.

Figure 6.34 shows that there are some relatively minor differences between the NS displacements obtained for the standard and actual terrain simulations. Much larger differences are apparent in figure 6.35, where both simulations give similar EW displacements for a 10m/s wind, but the standard simulation tends to give higher rms displacements for a 15m/s wind. However, the actual terrain simulation gives larger rms displacements for a 20m/s wind, these being about twice those obtained in the standard terrain for azimuths 230° to 330°. It is particularly interesting to note that this difference extends some 60° further round towards the north than the azimuths where the other blocks are directly upstream of Hume Point. That is the discrepancy between the two terrain simulations is approximately symmetrical about 270° not, as might have been expected, to an azimuth of about 245° (the centre of the sector where the other blocks were upstream). This discrepancy matches the observed difference in the full-scale response to easterly and westerly winds, as well as the response being symmetrical about 270° (see figure 6.22). It is also interesting to note the relative reduction in EW response for a wind speed of 15m/s using the actual terrain simulation, as this mirrors the relative reduction in EW response observed at full-scale, albeit at a wind speed of about 8 to 10 m/s (see figure 6.17).
FIGURE 6.34 North-South displacements measured in wind tunnel tests for both standard and actual terrain simulations
FIGURE 6.35 East-West displacements measured in wind tunnel tests for both standard and actual terrain simulations
FIGURE 6.36  Torsional displacements measured in wind tunnel tests for both standard and actual terrain simulations
Figure 6.36 shows that the mean torsional displacement in the region of the wake flow from the upstream tower blocks is lower for the actual terrain than the standard one, whilst there is a difference of sign between the two terrain simulations for mean displacements for azimuths between 270° and 310°.

Whereas the graphs of rms displacements in figures 6.34 to 6.36, particularly those for the actual terrain simulation, show marked discontinuities between successive increments of azimuth, no such effects are observed at full-scale. This is because the actual wind direction varies throughout a single full-scale record whereas it is constant in a wind tunnel test. Therefore the extremes noted in the wind tunnel testing tend to be smoothed out at full-scale.

6.10.3 Quantitative results from wind tunnel tests

Table 6.21 gives the rms acceleration in each of the three fundamental modes predicted by the wind tunnel testing to southerly and westerly winds of 10m/s using the actual terrain simulation. These accelerations have been calculated by assuming that the rms acceleration is $4 \pi^2 f_r^2$ times the rms displacement measured in the wind tunnel tests. In each case the appropriate value of $f_r$ has been taken from the results of the selective ensemble averaging tests. The torsional displacements from the wind tunnel tests have been reduced by a factor of 0.767 (the ratio of the perpendicular distance between the centre of torsion and the line of the measurement positions) so that a direct comparison can be made with the full-scale data. In addition, all the displacements from the wind tunnel tests have been reduced by a factor of 0.946 ($59.67/63.1$) as they are predicted response at 63.1m high whereas the full-scale data were measured on the 23rd floor ($59.67m$ above ground). Full-scale responses at 10 m/s have been interpolated from data in Tables 6.7 and 6.10 allowing for the exponents of increase in response with wind speed given in Table 6.11(B).
TABLE 6.21 COMPARISON OF FULL-SCALE AND MODEL-SCALE RMS ACCELERATIONS OF HUME POINT

<table>
<thead>
<tr>
<th>Wind Direction</th>
<th>EW accel. ($\times 10^{-3}$)</th>
<th>NS accel. ($\times 10^{-3}$)</th>
<th>$\theta$ accel. ($\times 10^{-3}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>South wind of 10 m/s:</td>
<td>0.180</td>
<td>0.173</td>
<td>0.128</td>
</tr>
<tr>
<td></td>
<td>0.0864</td>
<td>0.225</td>
<td>0.644</td>
</tr>
<tr>
<td>West wind of 10 m/s:</td>
<td>0.417</td>
<td>0.219</td>
<td>0.230</td>
</tr>
<tr>
<td></td>
<td>0.293</td>
<td>0.123</td>
<td>0.141</td>
</tr>
</tbody>
</table>

Note: Torsional acceleration given above is the linear component of response measured at the NW corner in the EW direction.

6.11 COMPARISON OF PREDICTED, MODEL AND FULL-SCALE RESPONSE

It can be seen in Table 6.21 that the model response overestimates the along-wind response for a southerly wind by 30% while underestimating the along-wind response for a westerly wind by about the same amount. These figures are about the same as those for the prediction using ESDU 88019 but the ESDU method underestimates in both cases. The model response underestimates both across-wind responses by about a factor of two, whereas the ESDU method overestimates by between 1.6 and 5 times. The torsional response for westerly winds at model-scale is about 60% of that measured at full-scale. By contrast, the torsional response for southerly winds at
model-scale is about five times that measured at full-scale. However, it can be seen from figure 6.36 that there is a large increase in predicted response at 180° compared with that at 190°. The predicted torsional response of the model at 190° is only 60% greater than that measured at full-scale for a wind from 180°.

From the evidence given above it would appear that the wind tunnel testing gives a better indication of the overall dynamic behaviour of Hume Point than the ESDU calculation methods. However, it must be stressed that both the wind tunnel and calculated responses have been obtained by knowing the correct natural frequencies and damping of the relevant modes plus the modal mass. Although the wind tunnel testing requires an accurate modelling of the actual terrain around the building, this is also required in the ESDU calculation methods to give accurate values for the wind related parameters. The two methods therefore give answers of similar accuracy for the along-wind response but the calculation method would be preferable as it is undoubtedly cheaper and quicker. However, the wind tunnel testing method is clearly superior for estimating across-wind and torsional response although, as the same considerations of time and cost apply, the calculation method could be used provided that a sufficient error bound is applied to the response obtained.

Clearly a great deal of caution should be exercised when applying the above assessment as the Hume Point experiments only represent the results from one building. However, these results highlight the need for further full-scale data of comparable accuracy so that a better assessment of both wind tunnel and calculation methods can be made.
6.12 SUMMARY OF CHAPTER SIX

In total, 9722 records (the equivalent of over 2,250 hours of data) were recorded at Hume Point. As far as can be ascertained, each 1024 second record has been shown to be self-stationary. The procedure for carrying out selective ensemble averaging has been detailed. It has also been shown that, providing sufficiently narrow limits of wind speed and direction are chosen, nominally identical ensembles produce results that differ by less than the maximum expected due to variance error.

The response of Hume Point to four 30 degree sectors of wind direction has been examined in detail. In all modes there is a trend for natural frequency to decrease and damping to increase with increasing wind speed. The response in all three fundamental modes increases with wind speed. A linear regression best fit analysis has been carried out on all three fundamental modes for all four wind directions. Providing data points for low levels of response are omitted, the best fit regression line passes within 10% of the value of each data point, with only one exception. The exponents obtained by the linear regression show that the response in all modes is proportional to the wind speed raised to between the power of 2.7 and the power of 3.3.

Further selective ensemble averaging has shown that, allowing for possible variance error, the response of Hume Point is symmetrical about both the west and north faces except for winds blowing directly onto the east or west faces. The reasons for this discrepancy are not clear. The response in all three fundamental modes is greatest for winds blowing onto the west face.

Peak acceleration analysis was carried out on 50 records recorded during similar wind conditions. For one channel the mean ratio of peak to rms accelerations was 3.7, the corresponding figure being 4.1 for the other channel. However, these ratios ranged from 1.7 to 6.7. The ratio of peak to filtered rms accelerations (rms accelerations from time histories containing only frequency components of the three fundamental modes) ranged from 4.0 to 7.5.
Whilst much of the analysis that has been carried out on the data recorded at Hume Point is unique, where such a comparison can be made, the results from Hume Point are in broad agreement with those obtained in previous full-scale tests.

Even when measured values were used in the ESDU calculation method, the predicted along-wind rms resonant acceleration underestimated the actually measured response by 20% for southerly winds and by 25% to 45% (depending upon the turbulence intensity chosen) for westerly winds.

Putting measured values into the ESDU calculation method for across-wind response, the predicted rms resonant acceleration overestimates the actually measured response by between 1.6 and 5 times. Using estimated values suggested by ESDU methods instead of measured ones increases the overestimate of response to between 5.5 and 12 times.

The ESDU methods give no guidance on the possible errors in estimating natural frequency using the suggested formulae (or any other method) neither is the original source of these formulae referenced.

The ESDU methods predict that the rms resonant component of response should increase in proportion to the wind speed raised to the power 2.63 for across-wind response, and to the power 3.09 for along-wind response. These figures are in reasonable agreement with the full-scale data obtained from Hume Point where exponents of 2.7 to 3.3 were obtained.

A 1:200 scale model of Hume Point was tested in the BRE boundary layer wind tunnel. Three distinct terrain simulations were used to model the actual terrain around Hume Point. Using this simulation, and the measured values of mass, natural frequency and damping, the wind tunnel tests were able to predict the along-wind response of Hume Point to within about 30% of the actually measured values, the across-wind response to within a factor of 2, and the torsional response to within a factor of 5.
CHAPTER SEVEN:

THE QUASI-STATIC RESPONSE OF HUME POINT TO WIND LOADING

7.1 INTRODUCTION

Table 6.2 shows the distribution by wind speed and direction of the 2535 records obtained whilst data from the plumb line and thermistors were being collected. This data was only collected for a three month period (8th February to 12th May 1989) before the instrumentation had to be removed. The analysis of the plumb line data is detailed below in the order in which it was carried out. It should be noted that the wind speeds quoted throughout this chapter are those actually measured by the anemometers 85.05m above ground level.

7.2 ANALYSIS OF MEAN VALUES FOR EACH RECORD

7.2.1 Data used in analysis of mean values

As mentioned in section 5.5.4, the mean position of the plumb line was calculated for each 64 second period of the 1024 second record, as well as for the entire record. Examination of the results of these calculations showed that there appeared to be little change in the mean position of the plumb line during the course of a record. Since the plumb line was measuring the quasi-static response of the building this was not an unexpected result. Consequently, initial analysis was carried out using the mean position of the plumb line during each 1024 second record.

Various quantities were extracted from the analysed data for each record and transferred to a VAX 8700 computer so that the ensemble averaging (detailed in Chapter Six) could be carried out in parallel with the analysis of the plumb line data. The following quantities were extracted for each record: record name, mean wind speed and direction, mean and rms displacements for both lasers, and the mean and rms temperatures for all eight thermistors. At the same time that the data
were transferred, the temperature difference between a pair of thermistors at the same height but on opposite faces of the building was calculated and the resulting four temperature differences stored with the other values for each record.

For the rest of this chapter, the thermistors will be referred to by a two letter code. The first letter being the face of the building and the second the level: Upper (16th floor) or Lower (9th floor) on which the thermistor was positioned. Thus thermistor EU is the thermistor positioned on the East face of the Upper (16th Floor). The differences between thermistors are referred to in a similar way by four letters. The first letter being D (for difference) then the two faces and finally upper or lower floor. Thus DEWU is the temperature Difference between the thermistors on the East and West of the Upper (16th) floor.

7.2.2 Failure of thermistors

Four of the thermistors failed before the removal of instrumentation in May 1989. Replacement thermistors were ordered but were not received until after all the instrumentation was removed from Hume Point. Two thermistors (EL and SL) failed and gave temperature readings that gradually drifted away from the average temperature of the other thermistors until a difference of over 20°C between these and the other thermistors was not uncommon. The exact record when these two thermistors failed was found by an examination of the relevant rms temperature values. The output from a working thermistor rarely showed much deviation from a straight line as temperature changes during a record were small. However, changes of up to two orders of magnitude occurred in the rms values from a failed thermistor so the point of failure could be easily identified. In this way, it was determined that the EL and SL thermistors failed after 1742 and 1820 records respectively had been obtained. Subsequent examination showed that the other two thermistors (EU and WU) had been vandalised between the end of recording on one optical disc (when they were both working well and a total of 1835 records had been obtained) and the start of recording on the next disc.
Where a thermistor had failed, both the mean and rms temperatures were set to a default (-99°C) so that spurious results would not be obtained in subsequent analysis. If one or both of the thermistor values used to calculate the temperature difference was set to the default, then the temperature difference was also set to the default.

7.2.3 Range of thermistor and laser data

Figure 7.1 shows how the mean temperature varied over a 46 hour period from 09:40 on 14th February to 07:50 on 16th February 1989. This period was chosen as it was before any of the thermistors failed, and the wind speed did not drop below 5 m/s, so that recording was essentially continuous. It can be seen that, in general, the temperatures for all the thermistors on the 9th floor follow those obtained on the 16th floor but with a constant offset. The average offsets for each face for the period shown in figure 7.1 are typical of the values obtained throughout the three month recording period. As mentioned in section 5.2.4, a single calibration was used for all eight thermistors. Thus the differences in temperature between thermistors on the same face of the building should be regarded as differences in the calibration offset rather than a real temperature difference. For this reason, the analysis was simplified by using only the output from the upper thermistors. The upper thermistors were chosen as both DNSU and DEWU could be calculated for more records than DNSL and DEWL because the first two thermistors to fail were the SL and EL ones. All subsequent references to the thermistors in this chapter will be therefore to the ones on the 16th floor.

The minimum temperature recorded was 4.71°C on the east face on the 27th February, and the maximum was 18.98°C on the south face on the 5th May. DNSU was defined as the temperature of the south face minus the temperature of the north face and ranged from 0.41°C to 5.01°C. DEWU was defined as the temperature of the west face minus the temperature of the east face and ranged from -0.16°C to 2.82°C. However, because of the possibility of slight calibration offsets between different thermistors, the range of values should be treated as more accurate than the absolute values themselves.
Figure 7.1  Temperature against time for 46 hour period 09:40 on 14 February to 07:50 on 16 February 1989
Throughout the three month period during which data were obtained from the lasers and thermistors, recording was triggered by the wind speed exceeding 5 m/s. Therefore no records were obtained during completely calm periods. It had been planned to carry out such recordings in the summer of 1989, but these plans had to be abandoned when all the instrumentation was removed. Consequently it was decided that the "origin" for the position of the plumb line would be the mean position calculated from records containing both the lowest wind speed and the lowest temperature differences. It should be noted that it is the relative position of the plumb line in two or more records rather than the exact position relative to the origin in a single record that is important. However, the method used to find the "origin" should produce an answer that is fairly close to the true position of rest for the plumb line.

The self-calibration check carried out by the lasers at the start of each record (see section 5.2.3) should ensure that data recorded over the whole three month period can be compared. Relative to the "origin" described above, the mean position of the plumb line ranged from -1.37 mm to +3.32 mm in the NS direction, and -2.57 mm to +1.91 mm in the EW direction. In the NS direction positive displacements are towards north, in the EW direction positive displacements are towards east.

7.2.4 Analysis of records grouped by wind speed and direction

7.2.4.1 Selection of groups of records to be analysed.

The data were analysed in five groups according to wind speed and direction. The first group contained all 2535 records. The second group contained only the 861 records where the wind speed was between 3.01 and 7.00 m/s, irrespective of direction. The other three groups were restricted to records where the mean wind blew from the north, south and west. In each case the same criterion as used in the ensemble averaging was applied e.g. south winds were defined as records where the wind direction was from the sector 15° either side of directly onto the south face (between 165° and 195°). The total number of records in the north, south and west groups were 253, 157 and 246 respectively. The failure of the EU and WU thermistors meant that the number of records containing the
difference in temperature between them (DEWU) for each group was significantly less than the total number of records. Thus the number of records including DEWU was reduced to 1835 in total, 380 of these in the 3.01 to 7.00 m/s group. The numbers of records including DEWU for the north, south and west groups were 72, 140 and 238 respectively. Although analysis could have been restricted to those records for which all the parameters were available, this would have meant discarding almost 30% of all the records, and over 70% of the records for the north group. The discrepancy thus caused in the number of records within a group used in a comparison which does or does not contain DEWU is small for two of groups: those for southerly and westerly winds. No analysis of data recorded during easterly winds was carried out. This was because there were only 71 records in total from this sector, and only eight of these included a valid value for DEWU.

Six graphs were produced to show the relationship between the parameters for each group. The NS displacement was plotted against firstly wind speed, secondly DEWU and thirdly DNSU, and then the EW displacement against the same three parameters.

7.2.4.2 Analysis of records for all wind speeds and directions

Figure 7.2 shows the graph for NS displacement against DNSU. Convenient ranges of DNSU were chosen so that there were approximately the same number of records in each range. Table 7.1 gives details of the ranges chosen, the mean values of DNSU, wind speed and direction, and both NS and EW displacement for each range. Table 7.1 also gives details of the regression analysis used to find the best fit linear relationship between DNSU and both NS and EW displacements. Both figure 7.2 and Table 7.1 show that there is a definite trend for the value of the NS displacement to increase as DNSU increases, irrespective of the wind speed or direction. It should be noted that DNSU was defined as the temperature by which the south face is warmer than the north face. Therefore, it can be stated that, in general, the warmer the south face is than the north face, the more the building leans to the north. However, figure 7.2 shows that although there is a clear relationship between NS displacement and DNSU, the wide discrepancies in the value of the NS displacement when DNSU is any
Figure 7.2  NS displacement against NS temperature difference for all records
### TABLE 7.1 MEAN DISPLACEMENTS AGAINST TEMPERATURE DIFFERENCE FOR ALL RECORDS

**a) Mean displacements against mean north-south temperature difference**
(using all 2535 records)

<table>
<thead>
<tr>
<th>DNSU</th>
<th>Number of records</th>
<th>Mean wind speed and direction</th>
<th>Mean DNSU</th>
<th>Mean displacements</th>
</tr>
</thead>
<tbody>
<tr>
<td>(°C)</td>
<td></td>
<td>(m/s) (degrees)</td>
<td>(°C)</td>
<td>NS (mm)</td>
</tr>
<tr>
<td>0.40 - 0.85</td>
<td>198</td>
<td>9.42 221.0</td>
<td>0.66</td>
<td>-0.34</td>
</tr>
<tr>
<td>0.86 - 1.00</td>
<td>220</td>
<td>10.46 244.2</td>
<td>0.93</td>
<td>0.39</td>
</tr>
<tr>
<td>1.01 - 1.10</td>
<td>221</td>
<td>10.12 204.0</td>
<td>1.07</td>
<td>0.13</td>
</tr>
<tr>
<td>1.11 - 1.20</td>
<td>257</td>
<td>10.33 234.9</td>
<td>1.15</td>
<td>0.26</td>
</tr>
<tr>
<td>1.21 - 1.40</td>
<td>269</td>
<td>9.20 235.2</td>
<td>1.31</td>
<td>0.43</td>
</tr>
<tr>
<td>1.41 - 1.60</td>
<td>215</td>
<td>8.87 228.8</td>
<td>1.50</td>
<td>0.52</td>
</tr>
<tr>
<td>1.61 - 1.90</td>
<td>181</td>
<td>9.54 211.8</td>
<td>1.75</td>
<td>0.64</td>
</tr>
<tr>
<td>1.91 - 2.10</td>
<td>187</td>
<td>6.48 157.1</td>
<td>2.03</td>
<td>0.58</td>
</tr>
<tr>
<td>2.11 - 2.50</td>
<td>195</td>
<td>7.71 185.9</td>
<td>2.28</td>
<td>0.80</td>
</tr>
<tr>
<td>2.51 - 2.80</td>
<td>187</td>
<td>8.04 200.5</td>
<td>2.67</td>
<td>0.99</td>
</tr>
<tr>
<td>2.81 - 3.50</td>
<td>213</td>
<td>7.37 204.1</td>
<td>3.06</td>
<td>1.21</td>
</tr>
<tr>
<td>3.51 - 5.10</td>
<td>191</td>
<td>6.28 155.9</td>
<td>4.11</td>
<td>1.92</td>
</tr>
</tbody>
</table>

\[ \text{NS displacement} = 0.542 \text{ DNSU} - 0.385 \quad (r^2 = 0.466) \]
\[ \text{EW displacement} = -0.278 \text{ DNSU} + 0.588 \quad (r^2 = 0.187) \]

**b) Mean displacements against mean east-west temperature difference**
(using only the 1835 records where DEWU is not set to -99°C)

<table>
<thead>
<tr>
<th>DEWU</th>
<th>Number of records</th>
<th>Mean wind speed and direction</th>
<th>Mean DNSU</th>
<th>Mean DEWU</th>
<th>Mean displacements</th>
</tr>
</thead>
<tbody>
<tr>
<td>(°C)</td>
<td></td>
<td>(m/s) (degrees)</td>
<td>(°C)</td>
<td>(°C)</td>
<td>NS (mm)</td>
</tr>
<tr>
<td>-0.20 - 0.55</td>
<td>272</td>
<td>10.50 228.6</td>
<td>1.46</td>
<td>0.39</td>
<td>0.53</td>
</tr>
<tr>
<td>0.56 - 0.70</td>
<td>239</td>
<td>10.94 230.1</td>
<td>1.28</td>
<td>0.62</td>
<td>0.62</td>
</tr>
<tr>
<td>0.71 - 0.80</td>
<td>172</td>
<td>9.74 246.0</td>
<td>1.80</td>
<td>0.76</td>
<td>0.68</td>
</tr>
<tr>
<td>0.81 - 1.00</td>
<td>262</td>
<td>8.63 238.3</td>
<td>1.39</td>
<td>0.88</td>
<td>0.50</td>
</tr>
<tr>
<td>1.01 - 1.20</td>
<td>213</td>
<td>8.99 241.8</td>
<td>1.34</td>
<td>1.12</td>
<td>0.42</td>
</tr>
<tr>
<td>1.21 - 1.50</td>
<td>254</td>
<td>9.64 220.2</td>
<td>1.74</td>
<td>1.34</td>
<td>0.31</td>
</tr>
<tr>
<td>1.51 - 2.00</td>
<td>187</td>
<td>9.29 190.5</td>
<td>1.59</td>
<td>1.78</td>
<td>0.02</td>
</tr>
<tr>
<td>2.01 - 2.90</td>
<td>235</td>
<td>8.55 190.5</td>
<td>1.13</td>
<td>2.34</td>
<td>-0.17</td>
</tr>
</tbody>
</table>

\[ \text{NS displacement} = -0.403 \text{ DEWU} + 0.821 \quad (r^2 = 0.142) \]
\[ \text{EW displacement} = 0.493 \text{ DEWU} - 0.327 \quad (r^2 = 0.341) \]

\[ \text{DEWU} = -0.067 \text{ DNSU} + 1.230 \quad (r^2 = 0.006) \]
particular value, especially when DNSU is small, show that temperature
difference is not the only factor determining the NS displacement.

Figure 7.2 also contains a number of points separated from the main
body of the data which show a clear increase in NS displacement as DNSU
increases from 3°C to 5°C. However, subsequent analysis showed that the
vast majority of these points were for records with southerly winds so
discussion of them will be in section 7.2.4.4.

Figure 7.3 shows the graph for EW displacement against DEWU. There is
a trend for the value of the EW displacement to increase as DEWU increases.
That is, in general, the warmer the west face is than the east, the more
the building leans to the east. Table 7.1 shows the mean displacements
obtained by analysing ranges of DEWU values in the same way as described
above, and the results of the regression analysis carried out on the 1835
records obtained before the EU and WU thermistors failed. The correlation
between EW displacement and DEWU is not as strong as that between NS
displacement and DNSU. However, the best fit linear regression for both is
approximately for the line where the displacement is 0.5 times the
temperature difference (in degrees Centigrade) minus 0.36 mm.

The result at the bottom of Table 7.1 shows that there is very poor
correlation between DNSU and DEWU, although there is a considerably better
correlation between EW displacement and DNSU and between NS displacement
and DEWU.

There are an interesting number of points in figure 7.3 in the region
where DEWU is close to zero which appear to show the overall trend for EW
displacement to increase as DEWU increases particularly well. Figure 7.4
shows the graph for EW displacement against DEWU for the 29 consecutive
records which were obtained between 16:56 on 8 March and 01:50 on 9 March
1989. The mean EW displacement decreased from -1.5 mm to -0.9 mm as the EW
temperature difference increased from -0.08°C to +0.11°C. Thus the overall
change in displacement is 3.6 times the temperature difference in degrees
Centigrade, considerably more than the average value. Over the same period
the mean wind speed varied in an arbitrary way from 6.8 to 11.7 m/s, and
the mean wind direction from 194° to 220°. At the same time DNSU fell
almost linearly from 2.83 to 2.24 °C.
Figure 7.3  EW displacement against EW temperature difference for all records
FIGURE 7.4  EW displacement against EW temperature difference for 29 consecutive records 8/9 March 1989
There was no correlation between either NS or EV displacement and wind speed. The action of a north wind would be expected to produce an opposite displacement to that of a south wind. Similarly, other winds from opposite directions would be expected to produce opposite displacements, so the lack of a trend is not surprising.

In addition to the six graphs produced for each group, four extra graphs were produced for this particular group. These showed the relationship between absolute temperature (as measured by both the NU and WU thermistors) and displacement (in both the NS and EW directions). As expected, these graphs showed that the displacement in both directions was independent of absolute temperature. Therefore, similar graphs were not produced for the other groups.

7.2.4.3 Analysis of records for northerly winds

Figure 7.5 shows the graph of NS displacement against wind speed for all 253 records from the group with northerly winds. There is a slight trend for NS displacement to decrease as the wind speed increases i.e. for northerly winds in general, the higher the wind speed the more the building leans to the south. Table 7.2 gives the mean displacements obtained by analysing convenient ranges of wind speed values. The results of the regression analysis carried out on all 253 records is also included in Table 7.2. The regression analysis has been carried out using both the wind speed value and the square of the wind speed value.

Figure 7.6 shows the graph of NS displacement against DNSU for all records in this group. There is a definite trend for NS displacement to increase as DNSU increases. Table 7.2 gives the mean displacements obtained by analysing convenient ranges of DNSU values and the results of the associated regression analysis.

It can be seen from Table 7.2 and figures 7.5 and 7.6 that the value of DNSU seems to have a much greater influence on the value of the NS displacement than the wind speed does, at least for northerly winds.
FIGURE 7.5  NS displacement against wind speed for records with mean wind direction 345° to 15°
### TABLE 7.2 MEAN DISPLACEMENTS FOR NORTHERLY WINDS

**a) Mean displacements against mean wind speed**
(using all 253 records)

<table>
<thead>
<tr>
<th>Wind speed (m/s)</th>
<th>Number of records</th>
<th>Mean wind speed and direction (m/s) (degrees)</th>
<th>Mean DNSU (°C)</th>
<th>Mean displacements (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.00 - 5.50</td>
<td>51</td>
<td>5.06 357.5</td>
<td>2.17</td>
<td>0.81 0.18</td>
</tr>
<tr>
<td>5.51 - 6.30</td>
<td>48</td>
<td>5.89 358.0</td>
<td>2.10</td>
<td>0.86 0.23</td>
</tr>
<tr>
<td>6.31 - 7.30</td>
<td>45</td>
<td>6.76 359.5</td>
<td>2.17</td>
<td>0.75 0.18</td>
</tr>
<tr>
<td>7.31 - 8.50</td>
<td>45</td>
<td>7.91 357.4</td>
<td>1.96</td>
<td>0.51 0.53</td>
</tr>
<tr>
<td>8.51 - 10.50</td>
<td>47</td>
<td>9.45 356.2</td>
<td>2.10</td>
<td>0.57 0.49</td>
</tr>
<tr>
<td>10.51 - 13.00</td>
<td>16</td>
<td>11.43 7.2</td>
<td>1.67</td>
<td>0.23 0.71</td>
</tr>
</tbody>
</table>

NS displacement = -0.0051 (WSPEED)² + 0.957 (r² = 0.077)
EW displacement = 0.0054 (WSPEED)² + 0.045 (r² = 0.072)

**b) Mean displacements against mean north-south temperature difference**
(using all 253 records)

<table>
<thead>
<tr>
<th>DNSU (°C)</th>
<th>Number of records</th>
<th>Mean wind speed and direction (m/s) (degrees)</th>
<th>Mean DNSU (°C)</th>
<th>Mean displacements (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50 - 1.25</td>
<td>44</td>
<td>8.67 357.8</td>
<td>1.07</td>
<td>-0.18 1.02</td>
</tr>
<tr>
<td>1.26 - 2.00</td>
<td>56</td>
<td>7.03 357.3</td>
<td>1.44</td>
<td>0.79 0.54</td>
</tr>
<tr>
<td>2.01 - 2.50</td>
<td>59</td>
<td>5.59 1.1</td>
<td>2.07</td>
<td>0.64 0.44</td>
</tr>
<tr>
<td>2.51 - 2.90</td>
<td>46</td>
<td>7.77 356.5</td>
<td>2.73</td>
<td>0.96 -0.21</td>
</tr>
<tr>
<td>2.91 - 4.00</td>
<td>50</td>
<td>7.54 358.3</td>
<td>3.04</td>
<td>1.09 -0.07</td>
</tr>
</tbody>
</table>

NS displacement = 0.489 DNSU - 0.337 (r² = 0.418)
EW displacement = -0.550 DNSU + 1.480 (r² = 0.446)
FIGURE 7.6  NS displacement against NS temperature difference for records with mean wind direction 345° to 15°
7.2.4.4 Analysis of records for southerly winds

Figure 7.7 shows the graph of NS displacement against wind speed for the group with southerly winds. There does not appear to be any trend to this data except that the range of NS displacement values decreases as the wind speed increases. Table 7.3 gives the mean displacements obtained by analysing convenient ranges of wind speed values. Only the 140 records which contained a valid value of DEWU have been included in this analysis of data for this wind group. The loss of 17 records from the analysis in this way does not affect the overall results for the effects that DNSU and wind speed have on displacements. However, it means that analysis of the effect DEWU has on the displacements can be assessed, whilst keeping the same overall data set. Table 7.3 shows that the best fit line obtained from the regression analysis has a small but negative slope. Thus for southerly winds in general, the higher the wind speed, the more the building leans to the south! However, figure 7.7 shows that this is a far from definite trend, as shown by the low coefficients of regression obtained.

Figure 7.8 shows the graph of NS displacement against DNSU for southerly winds. Once again there is a definite trend for NS displacement to increase as DNSU increases. Table 7.3 gives the mean displacements obtained by analysing convenient ranges of DNSU values and the results of the associated regression analysis.

It can be seen from Table 7.3 and figures 7.7 and 7.8 that for the records with southerly winds, even more so than for the records with northerly winds, the value of DNSU has a much greater influence on the value of the NS displacement than the wind speed does.

Figure 7.8 also shows a number of points which show a marked increase in NS displacement as DNSU increases from 3°C to 5°C. Subsequent analysis showed these points were for consecutive records. Figure 7.9 shows the graph for NS displacement against DNSU for the 37 consecutive records which were obtained between 21:57 on 21 February and 09:08 on 22 February 1989. The mean NS displacement decreased from 1.84 mm to 0.22 mm as the NS temperature difference decreased from 4.9°C to 3.2°C. At the same time
FIGURE 7.7 NS displacement against wind speed for records with mean wind direction 165° to 195°
TABLE 7.3 MEAN DISPLACEMENTS FOR SOUTHERLY WINDS

a) Mean displacements against mean wind speed
(using only the 140 records where DEWU is not set to -99°C)

<table>
<thead>
<tr>
<th>Wind speed (m/s)</th>
<th>Number of records</th>
<th>Mean wind speed and direction</th>
<th>Mean</th>
<th>Mean</th>
<th>Mean displacements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m/s) (degrees)</td>
<td>DNSU DEVU NS EV</td>
<td>NS</td>
<td>EW</td>
<td></td>
</tr>
<tr>
<td>3.00 - 6.50</td>
<td>28</td>
<td>5.53 180.4</td>
<td>2.79</td>
<td>1.39</td>
<td>0.65 0.24</td>
</tr>
<tr>
<td>6.51 - 7.80</td>
<td>27</td>
<td>7.15 180.1</td>
<td>3.32</td>
<td>1.18</td>
<td>0.92 0.06</td>
</tr>
<tr>
<td>7.81 - 8.80</td>
<td>28</td>
<td>8.31 182.7</td>
<td>2.24</td>
<td>1.56</td>
<td>0.17 0.23</td>
</tr>
<tr>
<td>8.81 - 10.00</td>
<td>28</td>
<td>9.40 183.5</td>
<td>2.00</td>
<td>1.27</td>
<td>0.25 0.06</td>
</tr>
<tr>
<td>10.01 - 18.00</td>
<td>30</td>
<td>12.08 185.9</td>
<td>2.03</td>
<td>1.39</td>
<td>0.12 0.20</td>
</tr>
</tbody>
</table>

NS displacement = -0.089 WSPEED + 1.180 ($r^2 = 0.068$)
EW displacement = 0.0019 WSPEED + 0.145 ($r^2 = 0.0002$)

b) Mean displacements against mean north-south temperature difference
(using only the 140 records where DEWU is not set to -99°C)

<table>
<thead>
<tr>
<th>DNSU (°C)</th>
<th>Number of records</th>
<th>Mean wind speed and direction</th>
<th>Mean</th>
<th>Mean</th>
<th>Mean displacements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m/s) (degrees)</td>
<td>DNSU DEVU NS EW</td>
<td>NS</td>
<td>EW</td>
<td></td>
</tr>
<tr>
<td>0.00 - 1.00</td>
<td>30</td>
<td>9.76 181.3</td>
<td>0.63</td>
<td>1.46</td>
<td>-0.32 0.14</td>
</tr>
<tr>
<td>1.01 - 2.00</td>
<td>30</td>
<td>8.92 180.0</td>
<td>1.57</td>
<td>1.58</td>
<td>0.00 0.39</td>
</tr>
<tr>
<td>2.01 - 2.50</td>
<td>25</td>
<td>7.40 179.8</td>
<td>2.23</td>
<td>1.68</td>
<td>0.09 0.28</td>
</tr>
<tr>
<td>2.51 - 4.00</td>
<td>25</td>
<td>9.74 189.4</td>
<td>3.24</td>
<td>1.23</td>
<td>0.54 0.03</td>
</tr>
<tr>
<td>4.01 - 5.10</td>
<td>31</td>
<td>7.01 183.1</td>
<td>4.67</td>
<td>0.90</td>
<td>1.68 -0.03</td>
</tr>
</tbody>
</table>

NS displacement = 0.486 DNSU - 0.784 ($r^2 = 0.694$)
EW displacement = -0.071 DNSU + 0.336 ($r^2 = 0.096$)

DEWU = -0.157 DNSU + 1.750 ($r^2 = 0.226$)
FIGURE 7.8 NS displacement against NS temperature difference for records with mean wind direction 165° to 195°
FIGURE 7.9  NS displacement against NS temperature difference for 37 consecutive records 21/22 February 1989
DEWU increased almost linearly from 0.8°C to 1.4°C. Over this same period the mean wind speed gradually increased from 6.2 to 12.0 m/s, and the mean wind direction from 176° to 197°. Thus the overall change in displacement is 0.95 times the temperature difference in degrees Centigrade, twice the average value obtained from the regression analysis. This movement of the building towards the south due to DNSU decreasing occurred despite the southerly wind increasing in speed over the same period.

Figure 7.10 shows the graph of NS displacement against DEWU for southerly winds. There is a strong trend for NS displacement to increase as DEWU decreases. It should be noted that the line of points in figure 7.10 where NS displacement decreased from 1.8 mm to 0.2 mm as DEWU increased from 0.8°C to 1.4°C are the points plotted in figure 7.9. DEWU increased over this particular night as EU fell from 8.7°C to 7.8°C while WU fell from 9.5°C to 9.2°C.

7.2.4.5 Analysis of records for westerly winds

Figure 7.11 shows the graph of EW displacement against wind speed for the group with westerly winds. There appears to be a much better correlation with wind speed than for either the northerly or southerly winds. Figure 7.12 shows the graph of EW displacement against DEWU for westerly winds. Table 7.4 gives the mean displacements obtained by analysing convenient ranges of wind speed and DEWU values. It also gives the results of the regression analysis carried out on the 238 records which contained a valid value of DEWU. Table 7.4 and figure 7.11 show that there is a trend for EW displacement to increase as the wind speed increases i.e. for westerly winds in general, the higher the wind speed the more the building leans to the east. Table 7.4 and figure 7.12 show that there is a trend for EW displacement to increase as DEWU increases.

7.2.4.6 Analysis of records for all wind directions but low wind speeds

Analysis of records in this group was carried out to see whether, by minimising the effect of wind on the building, the effects of temperature differential could be seen more clearly. However, although some of the effects described in sections 7.2.4.2 to 7.2.4.5 were evident, none were
FIGURE 7.10 NS displacement against EW temperature difference for records with mean wind direction 165° to 195°
FIGURE 7.11 EW displacement against wind speed for records with mean wind direction 255° to 285°
FIGURE 7.12  EW displacement against EW temperature difference for records with mean wind direction 255° to 285°
### TABLE 7.4 MEAN DISPLACEMENTS FOR WESTERLY WINDS

#### a) Mean displacements against mean wind speed
(using only the 238 records where DEVU is not set to -99°C)

<table>
<thead>
<tr>
<th>Wind speed (m/s)</th>
<th>Number of records</th>
<th>Mean wind speed and direction (m/s) (degrees)</th>
<th>Mean DNSU (°C)</th>
<th>Mean DEWU (°C)</th>
<th>Mean NS displacements (mm)</th>
<th>Mean EV displacements (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.00 - 6.20</td>
<td>48</td>
<td>5.48 269.0</td>
<td>1.28</td>
<td>0.76</td>
<td>0.62</td>
<td>-0.01</td>
</tr>
<tr>
<td>6.21 - 7.20</td>
<td>52</td>
<td>6.71 269.2</td>
<td>1.30</td>
<td>0.90</td>
<td>0.76</td>
<td>0.04</td>
</tr>
<tr>
<td>7.21 - 8.50</td>
<td>47</td>
<td>7.81 268.7</td>
<td>1.33</td>
<td>1.21</td>
<td>0.57</td>
<td>0.39</td>
</tr>
<tr>
<td>8.51 - 11.00</td>
<td>44</td>
<td>9.68 271.9</td>
<td>1.13</td>
<td>1.29</td>
<td>0.47</td>
<td>0.54</td>
</tr>
<tr>
<td>11.01 - 18.00</td>
<td>48</td>
<td>13.56 269.8</td>
<td>1.11</td>
<td>0.84</td>
<td>1.03</td>
<td>0.82</td>
</tr>
</tbody>
</table>

NS displacement = 0.0425 WSPEED + 0.331 (r² = 0.041)
EW displacement = 0.105 WSPEED - 0.557 (r² = 0.278)

#### b) Mean displacements against mean east-west temperature difference
(using only the 238 records where DEVU is not set to -99°C)

<table>
<thead>
<tr>
<th>DEVU (°C)</th>
<th>Number of records</th>
<th>Mean wind speed and direction (m/s) (degrees)</th>
<th>Mean DNSU (°C)</th>
<th>Mean DEWU (°C)</th>
<th>Mean NS displacements (mm)</th>
<th>Mean EV displacements (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 - 0.60</td>
<td>41</td>
<td>7.69 271.5</td>
<td>1.29</td>
<td>0.46</td>
<td>1.03</td>
<td>-0.32</td>
</tr>
<tr>
<td>0.61 - 0.80</td>
<td>53</td>
<td>9.34 269.4</td>
<td>1.36</td>
<td>0.72</td>
<td>1.12</td>
<td>0.22</td>
</tr>
<tr>
<td>0.81 - 1.10</td>
<td>49</td>
<td>8.73 269.0</td>
<td>1.30</td>
<td>0.94</td>
<td>0.63</td>
<td>0.51</td>
</tr>
<tr>
<td>1.11 - 1.25</td>
<td>45</td>
<td>8.66 270.5</td>
<td>1.09</td>
<td>1.15</td>
<td>0.50</td>
<td>0.60</td>
</tr>
<tr>
<td>1.26 - 2.20</td>
<td>51</td>
<td>8.40 268.4</td>
<td>1.11</td>
<td>1.62</td>
<td>0.22</td>
<td>0.64</td>
</tr>
</tbody>
</table>

EW displacement = -0.301 DNSU + 0.718 (r² = 0.048)
NS displacement = 0.921 DNSU - 0.440 (r² = 0.406)

NS displacement = -0.885 DEVU + 1.570 (r² = 0.387)
EW displacement = 0.643 DEVU - 0.291 (r² = 0.227)

DEVU = -0.408 DNSU + 1.500 (r² = 0.161)
made clearer by analysis of records in this group, and no other notable effects became apparent.

7.2.5 Analysis of consecutive records

Two examples have already been given (figures 7.4 and 7.9) of the change in displacement which occurred over a number of consecutive records. Both these examples are for changes of displacement which occurred overnight. Several other occasions were found when a large number of consecutive records showed a generally linear relationship on a graph of displacement against temperature differential. Almost all of these occasions were for times when the temperatures were decreasing (generally overnight) and so showed a decrease in NS displacement (i.e. the building leaning to the south) as DNSU decreased.

Figure 7.13 is a graph of NS displacement against DNSU for 150 consecutive records. The period covered by figure 7.13 coincides exactly with that of figure 7.1 which shows temperature against time. Figure 7.13 shows two periods (between points labelled B and C, and between points labelled E and F) where NS displacement decreases overnight as DNSU decreases. In the portion of figure 7.13 between points labelled C and D the NS displacement increased. This portion of the figure is unusual in that it represents a period when DNSU decreased because the temperature of the north face increased but that of the south face decreased. However, the building leaned progressively further to the north rather than the south as might have been anticipated.

The final part of figure 7.13 to consider is that portion between the points labelled A and B. It can be seen from figure 7.1 that the temperature of the south face rose by about 2°C over this ten hour period, although the temperature of the north face remained fairly constant. This resulted in a rapid increase of DNSU. However, there is no corresponding large change in NS displacement over this period. The wind speed increased gradually from 7.3 to 10.0 m/s whilst the wind direction changed from 287° to 228°.
FIGURE 7.13 NS displacement against NS temperature difference for 151 consecutive records 14/16 February 1989
In order to minimise the effects of differential temperature on displacement, the largest number of consecutive records where neither DNSU nor DEWU exceeded 0.15°C was located. Figure 7.14 shows graphs of both NS and EW displacement against wind speed for the 59 consecutive records obtained between 17:05 on 18th February 1989 and 10:35 the next day. During this period DNSU was always in the range 1.09 to 1.15°C and DEWU in the range 0.32 to 0.47°C. However, the temperature on each face rose steadily by one degree Centigrade. The wind direction varied between 218° and 244°. Figure 7.14 shows that the EW displacement decreased slightly (i.e. the building leaned to the west) as the south-westerly wind abated, as would be expected. However, the NS displacement increased (i.e. the building leaned to the north) over the same period, which is not what would be expected.

7.2.6 Other criteria affecting the quasi-static displacement of Hume Point

The preceding analysis of mean values has shown that both temperature difference and, to a lesser extent wind speed, have an effect upon the mean displacement of Hume Point. However, there are also significant anomalies between the expected and the actual mean displacement (as shown in figures 7.13 and 7.14). Also, even when either wind speed or temperature differential is minimised, the other parameter is unable to explain fully the mean position of the plumb line. Therefore some other parameters must play a part in determining the mean position of the plumb line for each record.

Although absolute temperature by itself seems to have little effect on displacement, it is possible that there is some secondary effect caused by the temperature of opposite faces rising or falling at the same rate. This would not cause any change in temperature differential, but by raising the temperature of opposite faces which already had a temperature differential, and therefore different thermal strains, a net displacement could occur. However, there are insufficient relevant data from Hume Point to test this hypothesis.
FIGURE 7.14  NS and EW displacements against wind speed for 59 consecutive records 18/19 February 1989
One other parameter which has not been considered in the analysis so far is time. Reference 114 shows that large temperature differences and therefore thermal strains can occur between the heated and unheated sides of concrete beams subjected to diurnal heating cycles. One example was of a 200mm deep beam. After heating had been applied for one hour, the heated side had risen by 8°C but the unheated side by less than 1°C. The temperature profile between them was not a straight line although it had almost become one after eight hours of heating. Reference 115 shows that for a 200mm thick column subjected to a 24 hour heating cycle, the change in temperature on the inside face is only about a quarter of that experienced by the outside face.

The thermistors were placed on the inside rather than the outside of the walls at Hume Point to avoid the large short-term changes that would occur when the sun was or was not obscured by clouds. However, by placing the thermistors on the inside, there is a significant time lag between a change in even long-term external temperature occurring and it being measured. As no measure of the external temperature has been taken, the time of the start of the rise in temperature and therefore the shape of the temperature profile cannot be determined. This means that the rise in temperature measured by the thermistors cannot be directly related to the average rise in temperature across the wall. In practice, even with exterior thermistors as well, finding the average rise in temperature across the wall might prove difficult to achieve. Large sustained rises or falls in temperature are less likely to occur in the U.K. than in the U.S.A. where successful full-scale experimental studies have been conducted (e.g. ref 116).

It was noted earlier that most of the consecutive records which showed a linear relationship between displacement and temperature difference were for instances when the building cooled overnight. The time lag between external and internal temperature rise discussed above means that a linear change in displacement caused by the building heating up would have to be sustained over several hours for the effect to be apparent. The sun shining on the building is by far the most likely cause of this heating. The apparent lack of such data may just be a reflection of the fact that the data were collected between February and May in England, and that unbroken sunshine in these conditions is unusual.
The preceding analysis has tended to treat the displacements in the NS and EW directions as separate quantities. However there must be some interaction between the two. Such interaction could well occur because the building displaces not only in the NS and EW directions but torsionally as well. In this case slightly different NS and EW displacements would be measured depending upon the position of the plumb line. However, as only one plumb line was used at Hume Point, this hypothesis cannot be investigated.

The instances where increasing wind speeds for southerly winds seem to cause the building to lean to the south, albeit with very poor correlation, are perplexing. However, a much larger number of records with a much better correlation would be required before this apparent phenomenon can be treated as more than a statistical fluke. Although the south face of the building differs from the north in that there are open columns below the first floor (see figure 4.2) it is difficult to envisage how this could affect the displacements which occur for southerly winds.

7.3 ANALYSIS OF INDIVIDUAL RECORDS

7.3.1 Reason for carrying out analysis of individual records

At this point in the analysis a closer look was taken at the time histories of individual laser records, particularly for those records with high wind speeds. Figure 5.21 is an example. It can be seen that although the mean response may not vary much throughout the record, there are also large excursions from this mean position. If these excursions were caused by gusts of wind then an analysis within a particular record would be appropriate. The record shown in figure 5.21 (HZ2203.939) was the first to be analysed individually and was chosen as the high mean wind speed (15.01 m/s) should mean that effects caused by the wind loading would be maximised. As temperature changes were slow, the differential heating during a particular record would not be expected to change, so differential heating should not be a factor of importance for an individual record.
7.3.2 Analysis of example record

Figure 7.15 is a time history of both the EW displacement and the westerly component of wind speed over the whole of the record. Figure 7.16 shows a one minute portion of the EW displacement time history. It can be seen that the original time history contains a high frequency component at frequencies which correspond to the fundamental natural frequencies of the building. However, by filtering the time history this dynamic component of the displacement could be removed leaving the "pseudo-static" component.

Therefore, the time history was passed through a digital filter program. This program allowed all frequency components below 0.7 Hz through but none above 0.8 Hz. This low pass filtered time history is also shown in figure 7.16. (For simplicity only the mean value of the applied filter is used in figures 7.16 and 7.17 e.g. the filtering described above is described as 0.75 Hz LP (low pass)). There was also some doubt as to whether some of the displacement measured by the lasers was merely the plumb line responding at its natural frequency. The lower part of figure 5.21 shows a low frequency response of about 16 seconds which is very close to the natural frequency of the plumb line (about 0.064 Hz). Consequently, the time history which had already been passed through the filter program to remove the dynamic response of the building was passed through the filter program a second time. This time the filter program allowed all frequency components above 0.075 Hz through but none below 0.065 Hz. The result of this band-pass filtering is also shown in figure 7.16. The component of the overall response above the natural frequency of the plumb line is shown to be small. Therefore, all frequency components at or above the natural frequency of the plumb line could be removed with little effect on the overall response but with a much greater confidence that the remaining time history was solely the pseudo-static response of the building.

In the upper part of figure 7.17 the original time history is shown along with the same time history once it had been passed through the filtering program so that it allowed all frequency components below 0.05 Hz through but none above 0.06 Hz. Before the filtered displacement time history could be compared with the wind speed, it was necessary to filter
FIGURE 7.15 Unfiltered time histories of westerly component of wind speed and EW displacement for record HZ2203.939
FIGURE 7.16  One minute extract showing frequency composition of time history of EW displacement for record HZ2203.939
FIGURE 7.17 One minute extract of time history for both westerly component of wind speed and EW displacement for record HZ2203.939
the anemometer record in exactly the same way so that any possible effects caused by the filtering would be matched in the two signals and so cancel out. The lower part of figure 7.17 shows the original time history from the west anemometer for the same 60 second period. It also shows the time history after it had passed through the 0.05 Hz/0.06 Hz low pass filter. Almost all of the anemometer signal thus removed was below 0.7 Hz.

Figure 7.18 is a time history of both the filtered EW displacement and the filtered west anemometer over the whole of the record. It should be noted that the magnitudes of both the wind speed and displacement time histories in figure 7.18 are lower at both the start and end of the record than for the original time histories shown in figure 7.15. This is an inevitable consequence of the filter program operating on finite lengths of data. Although this only affects a few seconds of data, care should be taken not to infer anything from the first and last few points of filtered data.

It can be seen in figure 7.18 that, for example, the peaks in the displacement time history at about 230, 440 and 575 seconds appear to correspond to peaks in the wind speed time history. However, as the eye can be deceived, it is best to plot each point in the displacement time history against the corresponding point from the wind speed time history to determine the correlation between these two parameters. This has been done in figure 7.19 and the best fit line, as calculated by linear regression, added. Points on this line represent the expression that the EW displacement equals m times the wind speed (as measured by the west anemometer) plus a constant C. In this case, m is 0.0585 and C is 0.0621 mm. The coefficient of regression \( r^2 \) for this best fit line was 0.546, although for some parts of the time history the response appears to be orthogonal to the best fit line rather than along it.

The preceding analysis compared the response of the building to the wind impinging upon it at the same instant. Clearly, the building cannot respond quasi-statically to instantaneous wind changes as this would be a contradiction in terms. Therefore a time lag was introduced to the displacement time history and, as before, each point in the displacement time history plotted against the (new) corresponding point from the wind
FIGURE 7.18 Filtered time histories of westerly component of wind speed and EW displacement for record HZ2203.939
FIGURE 7.19 Filtered EW displacement against filtered westerly component of wind speed for record HZ2203.939
speed time history. Again the best fit line was calculated by linear regression as was the coefficient of regression. It was found that \( r^2 \) increased gradually to a maximum and then decreased again as the time lag was increased. The maximum value of \( r^2 \) was found to be 0.562 for a time lag of 22 seconds. The corresponding values of \( m \) and \( c \) were 0.0593 and 0.0527 respectively.

### 7.3.3 Analysis of other individual records

Five other records were analysed in exactly the same way as described above. Three of these records were chosen because they were obtained whilst the wind was blowing almost directly onto the south face, the mean wind speeds being approximately 7, 10 and 15 m/s. The other two records were chosen because they were obtained whilst the wind was blowing almost directly onto the west face, the mean wind speeds being approximately 7 and 10 m/s. The results of the regression analysis on all six records are given in Table 7.5. It can be seen that the best fit was obtained for the example record, the fit for some of the others being poor. There does not appear to be any correlation between the goodness of fit and wind speed. The best fit time lag varies from zero to over one hundred seconds.

Having failed to get a good correlation for all six records between wind speed and building displacement, various other parameters were plotted against each other to try to obtain a better correlation. The example record was used in this exercise as it had the best fit of the six records for wind speed against displacement.

Three parameters relating to the wind speed were calculated. The first of these was obtained by squaring the instantaneous wind speed from the time history of the west anemometer. Figure 7.20 is time history of wind speed squared and EW displacement for record HZ2203.939. These two time histories were plotted against each other in the same way as described in section 7.3.2. The result of the regression analysis is given in Table 7.6. It can be seen that for zero time lag the fit is identical to that obtained for wind speed alone. There is a small difference in the best fit time lag between the two which results in a minor difference in the fit for the best time lag.
### TABLE 7.5 LINEAR REGRESSION COEFFICIENTS FOR SELECTED RECORDS IN WIND ANEMOMETER vs. IN LINE DISPLACEMENT

<table>
<thead>
<tr>
<th>File name</th>
<th>Parameters plotted</th>
<th>Regression coefficients</th>
<th>Best fit time lag (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HZ2203.939 (15.01 m/s, 253.2°)</td>
<td>W vs. X</td>
<td>r²</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>0.546</td>
<td>0.0585</td>
<td>0.0621</td>
</tr>
<tr>
<td></td>
<td>0.562</td>
<td>0.0593</td>
<td>0.0527</td>
</tr>
<tr>
<td>HZ1403.964 (15.15 m/s, 192.5°)</td>
<td>S vs. Y</td>
<td>0.206</td>
<td>0.0332</td>
</tr>
<tr>
<td></td>
<td>0.232</td>
<td>0.0355</td>
<td>-0.7739</td>
</tr>
<tr>
<td>HZ2403.966 (9.97 m/s, 265.3°)</td>
<td>W vs. X</td>
<td>0.088</td>
<td>0.0248</td>
</tr>
<tr>
<td></td>
<td>0.162</td>
<td>0.0331</td>
<td>0.7897</td>
</tr>
<tr>
<td>HZ1203.907 (10.01 m/s, 176.7°)</td>
<td>S vs. Y</td>
<td>0.430</td>
<td>0.0544</td>
</tr>
<tr>
<td></td>
<td>0.476</td>
<td>0.0574</td>
<td>-0.1658</td>
</tr>
<tr>
<td>HZ2602.925 (6.99 m/s, 268.1°)</td>
<td>W vs. X</td>
<td>0.146</td>
<td>0.0221</td>
</tr>
<tr>
<td></td>
<td>0.167</td>
<td>0.0216</td>
<td>0.9599</td>
</tr>
<tr>
<td>HZ2102.917 (6.97 m/s, 179.3°)</td>
<td>S vs. Y</td>
<td>0.017</td>
<td>-0.0185</td>
</tr>
<tr>
<td></td>
<td>0.017</td>
<td>-0.0185</td>
<td>1.6630</td>
</tr>
</tbody>
</table>

Notes: W is west anemometer, S is south anemometer, X is EW displacement, Y is NS displacement. Where the best fit time lag is greater than 100 seconds, the regression coefficients for 100 seconds are given.

### TABLE 7.6 LINEAR REGRESSION COEFFICIENTS FOR RECORD HZ2203.939 (15.01 m/s, 253.2°) FOR VARIOUS DIFFERENT PARAMETERS

<table>
<thead>
<tr>
<th>Parameters plotted</th>
<th>Regression coefficients</th>
<th>Best fit time lag (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W vs. X</td>
<td>r²</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>0.546</td>
<td>0.0585</td>
</tr>
<tr>
<td></td>
<td>0.562</td>
<td>0.0593</td>
</tr>
<tr>
<td>W² vs. X</td>
<td>0.546</td>
<td>0.0020</td>
</tr>
<tr>
<td></td>
<td>0.567</td>
<td>0.0022</td>
</tr>
<tr>
<td>(W² + S²)½ vs. (X² + Y²)½</td>
<td>0.343</td>
<td>0.0585</td>
</tr>
<tr>
<td></td>
<td>0.422</td>
<td>0.0651</td>
</tr>
<tr>
<td>W² + S² vs. (X² + Y²)½</td>
<td>0.333</td>
<td>0.0020</td>
</tr>
<tr>
<td></td>
<td>0.415</td>
<td>0.0023</td>
</tr>
</tbody>
</table>

Notes: W is west anemometer, S is south anemometer, X is EW displacement, Y is NS displacement.
FIGURE 7.20 Filtered time histories of westerly component of wind speed squared and EW displacement for record HZ2203.939
Two further parameters relating to the wind speed were calculated. Both involved calculating the instantaneous square of the wind speed for both the south and west anemometers and then adding these two quantities together. This quantity (the square of the overall wind speed) was used in one case, the square root of this quantity (the overall wind speed) was used in the other. Both were plotted against the instantaneous position of the plumb line from the assumed origin. The results of this analysis are also given in Table 7.6. It can be seen that the results from these two cases are similar but the fit is significantly worse than for either of the cases using the data from the west anemometer alone.

As the results for the analysis of the instantaneous square of the time history from the west anemometer were almost identical to those obtained for the original time history, the other five records analysed previously in the same manner to that of the example record described above. The results are given in Table 7.7. Comparison of these results with those in Table 7.5 shows that the fit obtained was very similar for each record, whichever parameters were used. The best fit time lag is almost the same in five of the six cases, the largest discrepancy being for record H22602.925 where best fit time lags of 60 and 48 seconds were obtained for the two original and squared time histories respectively.

7.3.4 Examination of position of plumb line during record

As mentioned above, it can be seen in figure 7.19 that for some periods the time history of displacement against wind speed is orthogonal to the linear best fit between them. Similar graphs were obtained for the other records analysed. Therefore a plan view of the motion of the plumb line throughout the record was plotted and is shown in figure 7.21 which is typical of the graphs obtained for the records analysed. It should be noted that this graph is for filtered time histories, and that the motion of the plumb line cannot be ascertained from a similar plot of unfiltered time histories because of the dynamic response which occurs at the same time. The best fit line between the NS and EW displacements has been added to figure 7.21, but it can be seen that the motion of the plumb line does not approximate to it. Whilst in this example the best fit line is at almost exactly the same angle as the mean wind direction for the record, this
TABLE 7.7 LINEAR REGRESSION COEFFICIENTS FOR SELECTED RECORDS
(SQUARE OF IN WIND ANEMOMETER) vs. IN LINE DISPLACEMENT

<table>
<thead>
<tr>
<th>File name</th>
<th>Parameters plotted</th>
<th>Regression coefficients</th>
<th>Best fit time lag (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HZ2203.939</td>
<td>$W^2$ vs. $X$</td>
<td>$r^2=0.546$, $m=0.0020$, $c=0.7645$</td>
<td>0</td>
</tr>
<tr>
<td>(15.01 m/s, 253.2°)</td>
<td></td>
<td>$r^2=0.567$, $m=0.0022$, $c=0.4309$</td>
<td>24</td>
</tr>
<tr>
<td>HZ1403.964</td>
<td>$S^2$ vs. $Y$</td>
<td>$r^2=0.203$, $m=0.0011$, $c=-0.5083$</td>
<td>0</td>
</tr>
<tr>
<td>(15.15 m/s, 192.5°)</td>
<td></td>
<td>$r^2=0.227$, $m=0.0012$, $c=-0.5237$</td>
<td>58</td>
</tr>
<tr>
<td>HZ2403.966</td>
<td>$W^2$ vs. $X$</td>
<td>$r^2=0.092$, $m=0.0013$, $c=0.9780$</td>
<td>0</td>
</tr>
<tr>
<td>(9.97 m/s, 265.3°)</td>
<td></td>
<td>$r^2=0.158$, $m=0.0017$, $c=0.9422$</td>
<td>&gt;100</td>
</tr>
<tr>
<td>HZ1203.907</td>
<td>$S^2$ vs. $Y$</td>
<td>$r^2=0.428$, $m=0.0030$, $c=0.0935$</td>
<td>0</td>
</tr>
<tr>
<td>(10.01 m/s, 176.7°)</td>
<td></td>
<td>$r^2=0.476$, $m=0.0032$, $c=0.0761$</td>
<td>&gt;100</td>
</tr>
<tr>
<td>HZ2602.925</td>
<td>$W^2$ vs. $X$</td>
<td>$r^2=0.131$, $m=0.0015$, $c=1.0313$</td>
<td>0</td>
</tr>
<tr>
<td>(6.99 m/s, 268.1°)</td>
<td></td>
<td>$r^2=0.142$, $m=0.0015$, $c=1.0339$</td>
<td>48</td>
</tr>
<tr>
<td>HZ2102.917</td>
<td>$S^2$ vs. $Y$</td>
<td>$r^2=0.021$, $m=-0.0015$, $c=1.6091$</td>
<td>0</td>
</tr>
<tr>
<td>(6.97 m/s, 179.3°)</td>
<td></td>
<td>$r^2=0.021$, $m=-0.0015$, $c=1.6091$</td>
<td>0</td>
</tr>
</tbody>
</table>

Notes: $W$ is west anemometer, $S$ is south anemometer
$X$ is EW displacement, $Y$ is NS displacement

Where the best fit time lag is greater than 100 seconds
the regression coefficients for 100 seconds are given
FIGURE 7.21 Filtered NS displacement against filtered EW displacement for record HZ2203.939
should be treated as a coincidence, as it was not the case for the other records.

The motion of the plumb line over the course of the record appears to be chaotic. One possible cause of such motion would be if the plumb line was vibrating because of draughts of air in the refuse chute. Although every effort was made to eliminate such draughts it is difficult to be certain that none were present. However, if these were the primary factor affecting the position of the plumb line it is difficult to see how a number of consecutive records produced such linear relationships between displacement and temperature difference as seen in figures 7.4, 7.9 and 7.13. It should be remembered that the time histories plotted in figure 7.21 have been filtered to remove frequency components above 0.065 Hz. It is therefore difficult to envisage how draughts could cause the very low frequency motion of the plumb line shown in this figure.

If draughts were responsible for the seemingly chaotic motion shown in figure 7.21 then it would seem sensible to carry out analysis using the mean position of the plumb line in each record to eliminate the random motion caused by draughts. However, better correlations have been obtained between displacement and wind speed within individual records than were obtained with mean values. It must therefore be concluded that whilst draughts could cause some movement of the plumb line, they are not the primary factor affecting its position.

7.4 CORRELATION BETWEEN DISPLACEMENT AND WIND SPEED

The correlation between the mean displacement and the mean wind speed has been shown to be reasonable for westerly winds, but very poor for northerly and southerly winds. This poor correlation can be partly explained by the effects of differential heating which have been shown to have at least as much, and usually more, influence than wind speed on the mean displacement of the building. However, better correlations have been obtained between displacement and wind speed within individual records. Allowing for a time lag between the two has slightly improved the correlations but not markedly. If the displacement is viewed as a
quasi-dynamic rather than quasi-static quantity, then perhaps there should be a variable time lag between the two rather than the fixed time lag used above. If the building response is in phase with the applied wind force then there will be a shorter time lag between the two than if the two were antiphase. This would explain why there are different time lags between the peaks in the wind speed and displacement time histories shown in figure 7.18 at the times mentioned previously (approximately 230, 440 and 575 seconds). Perhaps the most important question is how well should the wind speed, as measured by an anemometer 20m above the building, be correlated with the overall displacement of that building?

7.5 COMPARISON WITH OTHER FULL-SCALE TEST RESULTS

No effects caused by differential heating were observed on the Empire State building (see sections 1.3.2, 3.2.1 and ref 8). However, there were major problems with upward currents of air causing the plumb line to oscillate. For similar wind conditions, there appeared to be some correlation between the displacement and the square of the wind speed, at least at wind speeds in excess of about 20 m/s. Davenport confirmed this in his re-assessment of the data from the Empire State (ref 16). However, both these assessments were made on less than ten measurements for each wind direction.

In contrast to the Empire State building, significant deflection due to differential heating was noted on 1000 Lakeshore Plaza (see section 1.4.1 and ref 15). Plots of the plan displacement of the building over time show the same apparently random motion seen in figure 7.21. No reliable correlation between displacement and wind speed was obtained.

Differential heating was also found to play a significant part in determining the deflection of the 50m CEBTP tower (see section 1.4.4 and ref 37). A plot of the plan displacement of the tower measured by a plumb line over a 24 hour period shows large deflections caused by differential heating. Importantly the start and finish positions of the plumb line do not coincide. Paquet therefore concluded that a starting position to study the displacement of the tower due to wind loading could not be defined.
Although a vertical laser was also used to measure the displacement of the tower, it had to be put in a tube under a vacuum to prevent air turbulence and thermal problems. It seems that even this was not entirely successful as a similar device was not installed in the Maine-Montparnasse Tower (see section 1.4.4 and ref 38). A plumb line and taut wire device were installed to measure displacement but the motions of both wires were influenced by the operation of the air conditioning and thermal effects on the cable. No attempt to link displacement with wind speed has been made for either tower.

No mention of differential heating is made in the papers which report on the displacement of Commerce Court Tower (see sections 1.4.6 and refs 44 to 51) indeed no measure of temperature seems to have been made. A vertical laser was used to measure the displacement of the building. Sample time histories of pressure and displacement (both filtered and unfiltered) similar to figures 7.15 and 7.18 show reasonable agreement between these two parameters (ref 50). A plot of the plan displacement of the building over a two minute period (ref 50) is fairly similar to figure 7.21. However, good correlation was obtained between mean along-wind displacements and mean pressures for 68 records of five minutes duration obtained when the wind was approximately perpendicular to one face of the building (ref 50). Good agreement was obtained for most wind directions between full-scale displacements and those obtained on a wind tunnel model (ref 45).

A vertical laser was also used to measure the displacement of the 72m tall building in Las Vegas (see sections 1.4.7 and 3.2.7 and refs 52-54). Again no measurements of temperature were made and no mention of differential heating effects is made in any of the papers. A typical time history is given in reference 54 which shows "obvious similarities" between differential pressure and displacement. However, the correlation appears to be no better than that shown in figure 7.18. No attempt appears to have been made to correlate the pressure and displacement during a single recording. No analysis of mean displacements has been carried out either, presumably because of the limited amount of data available.
It is noticeable that the buildings where differential temperature was significant in determining the displacement (1000 Lakeshore Plaza, the CEBTP 50m tower and Hume Point) are all reinforced concrete buildings. The steel frame buildings (Commerce Court Tower, the Empire State building and the Las Vegas building) do not seem to suffer significantly from differential temperature effects and the former two, which are both over 200m tall, seem to have the best correlation between wind speed or pressure and displacement.

7.6 SUMMARY OF CHAPTER SEVEN

Initial analysis was carried out on the mean values for each record. The records were analysed in four main groups according to wind speed and direction: all records, and records where the mean wind direction was from either the north, south or west. It has been shown that there is some correlation between temperature difference and the mean position of the plumb line. In all four groups described above, the coefficient of regression ($r^2$) was greater than 0.4 for the correlation of NS displacement and DNSU, and was greater than 0.2 for the correlation of EW displacement and DEWU for the three groups where it was calculated. In all these cases the best fit linear regression is approximately for the line where the displacement in mm is 0.5 times the temperature difference (in degrees Centigrade) minus 0.4 mm. However, analysis of the group of westerly winds was the only case of the correlation of wind speed and the mean position of the plumb line having a value of $r^2$ greater than 0.1.

On several occasions a number of consecutive records were found to show a linear relationship between displacement and temperature difference. The majority of these occasions occurred overnight as the building cooled. Even when either wind speed or temperature differential is minimised, the other parameter is unable to explain fully the mean position of the plumb line.

Six individual records were analysed in detail. A displacement time history was found to consist essentially of a low frequency component (under 0.07 Hz) and a higher frequency component at frequencies which
correspond to the fundamental natural frequencies of the building. The displacement time history was filtered to remove the higher frequency component and then compared to the wind speed time history. Although there were points where the two had peaks in common, there were points when one was increasing as the other was decreasing. By plotting one time history against the other the relationship between the two was determined. The correlations obtained had coefficients of regression which ranged from 0.017 to 0.546. In general the correlations obtained were better than those obtained when the mean values in each record were used. A time lag was introduced to the displacement signal but generally had little effect on the results obtained. Using the square of the wind speed rather than the wind speed also had little effect. Examination of the plan view of the plumb line during the course of a record showed a seemingly chaotic motion despite the reasonable correlations obtained.

It is interesting to compare the results obtained at Hume Point with those obtained on other full-scale tests where the quasi-static response of a tall building has been attempted. At Hume Point, like 1000 Lakeshore Plaza and the 50m CEBTP tower, there was significant displacement due to differential heating. These three buildings are all of reinforced concrete construction. Obtaining a relationship between displacement and wind loading on these buildings was difficult. On the other hand, there do not appear to have been significant differential heating effects on three steel frame buildings: Commerce Court Tower, Empire State building and the 72m tall building in Las Vegas. Good correlation was obtained at the first two of these buildings between displacement and either pressure or the square of the wind speed. No such correlation was attempted on the data from the Las Vegas building.
CHAPTER EIGHT:

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

8.1 EXPERIMENTAL AND ANALYTICAL TECHNIQUES

Several full-scale experiments have measured the response of tall buildings to wind loading. However, because the wind is a random process, determining the response of a tall building in a particular mode of vibration to a given wind speed and direction to within, say ±10%, requires very long lengths of stationary data. The lower the natural frequency and the damping, the longer the length of stationary data required. As increasing height tends to result in lower natural frequencies and lower damping, this makes unusually tall buildings unsuitable for full-scale experiments if the response to a given wind is required. None of the previous full-scale experiments on tall buildings have obtained data of sufficient accuracy to enable the validity of prediction methods to be assessed.

The full-scale tests on Hume Point are the first to be carried out where the initial objective has been to determine the response in a particular mode of vibration to within ±10%. This objective has been achieved by the use of selective ensemble averaging where response spectra calculated from records obtained under similar wind conditions are averaged together. The length of the record which is required to produce each spectrum is determined by the natural frequency and damping of the modes to be examined. Therefore, either the natural frequency and damping have to be known before data collection begins or the recording has to be continuous, and the length of each record determined by analysis at the end of data collection.

In total, 9722 records (the equivalent of over 2,250 hours of data) were recorded at Hume Point. As far as can be ascertained, each 1024 second record has been shown to be self-stationary. It has been shown that, providing sufficiently narrow limits of wind speed and direction are chosen, nominally identical ensembles produce results that differ by less
than the maximum expected due to variance error. Selective ensemble averaging has therefore been shown to be a suitable technique for assessing the dynamic response of tall buildings to wind loading to within a calculable accuracy. Whilst sufficient records were obtained at Hume Point to enable detailed analysis of the response to some wind directions, insufficient data were obtained to enable detailed analysis of the response to all wind directions or to any wind speeds in excess of about 16 m/s.

All the instrumentation and signal conditioning used at Hume Point worked well. However, the removal of the instrumentation before any quasi-static displacement data had been collected during the summer months limited the extent of this part of the experiment. Nevertheless, it is apparent from the data that was obtained, that any future full-scale experiment that attempts to measure quasi-static displacement must measure the temperature on each face of the building, so that differential heating effects can be assessed. Optical discs rather than magnetic tapes were used to store the vast amounts of data generated. Their use, and the ability to monitor the data remotely, meant that far fewer visits to the site were required than would have been the case with another storage medium.

8.2 FULL-SCALE RESULTS FROM HUME POINT

8.2.1 Dynamic response

The response of Hume Point to four 30 degree sectors of wind direction has been examined in detail. Ensembles of one hundred records obtained under similar wind conditions were used in the selective ensemble averaging. By using one hundred records in the ensemble, the response data from the selective ensemble averaging should be within 10% of the actual value of the response averaged over infinite time.

The response in all three fundamental modes increases with wind speed. A linear regression best fit analysis has been carried out on all three fundamental modes for all four wind directions. Providing data points for low levels of response are omitted, the best fit regression line passes
within 10% of the value of each data point, with only one exception. The exponents obtained by the linear regression show that the response in all modes is proportional to the wind speed raised to between the power of 2.7 and the power of 3.3.

Further selective ensemble averaging has shown that, allowing for possible variance error, the response of Hume Point is symmetrical about both the west and north faces except for winds blowing directly onto the east or west faces. The response in all three fundamental modes is greatest for winds blowing onto the west face.

In all modes there is a trend for natural frequency to decrease and damping to increase with increasing wind speed. The same trends were observed in forced vibration tests carried out on the building.

Peak acceleration analysis was carried out on 50 records from two orthogonal accelerometers recorded during similar wind conditions. The mean ratio of peak to rms accelerations was found to be 3.7 for the records from one accelerometer and 4.1 for the other, although these ratios ranged from 1.7 to 6.7. The ratio of peak to filtered rms accelerations (rms accelerations from time histories containing only frequency components of the three fundamental modes) ranged from 4.0 to 7.5.

Whilst much of the analysis that has been carried out on the data recorded at Hume Point is unique, where such a comparison can be made, the results from Hume Point are in broad agreement with those obtained in previous full-scale tests.

8.2.2 Quasi-static response

Some correlation was found between mean temperature difference between opposite faces of Hume Point and the mean position of the plumb line for each record. In all four groups of records analysed (all records, and records where the mean wind direction was from either the north, south or west) the coefficient of regression ($r^2$) was greater than 0.4 for the correlation of north-south displacement and the temperature difference between the north and south faces, and was greater than 0.2 for the
correlation of east-west displacement and the temperature difference between the east and west faces, for the three groups where it was calculated. In all these cases the best fit linear regression is approximately for the line where the displacement in mm is 0.5 times the temperature difference (in degrees Centigrade) minus 0.4 mm. However, analysis of the group of westerly winds was the only case of the correlation of wind speed and the mean position of the plumb line having a value of $r^2$ greater than 0.1.

On several occasions a number of consecutive records were found to show a linear relationship between displacement and temperature difference. The majority of these occasions occurred overnight as the building cooled. Even when either wind speed or temperature differential is minimised, the other parameter is unable to explain fully the mean position of the plumb line.

Six individual records were analysed in detail. A displacement time history was found to consist essentially of a low frequency component (under 0.07 Hz) and a higher frequency component at frequencies which correspond to the fundamental natural frequencies of the building. The displacement time history was filtered to remove the higher frequency component and then compared with the wind speed time history. Although there were points where the two had peaks in common, there were points when one was increasing as the other was decreasing. By plotting one time history against the other the relationship between the two was determined. The correlations obtained had coefficients of regression which ranged from 0.017 to 0.546. In general the correlations obtained were better than those obtained when the mean values in each record were used. A time lag was introduced to the displacement signal but generally had little effect on the results obtained. Using the square of the wind speed rather than the wind speed also had little effect. Examination of the plan view of the plumb line during the course of a record showed a seemingly chaotic motion despite the reasonable correlations obtained.

At Hume Point, and at two other full-scale tests to measure the quasi-static response of tall buildings of reinforced concrete construction, there was significant displacement due to differential
heating. Obtaining a relationship between displacement and wind loading on these buildings was difficult. On the other hand, there do not appear to have been significant differential heating effects on three steel frame buildings where similar full-scale tests have been carried out. Good correlation was obtained at two of these steel frame buildings between displacement and either pressure or the square of the wind speed. No such correlation was attempted on the data from the third building.

8.3 THE PREDICTED RESPONSE OF HUME POINT

Even when measured values were used in the ESDU calculation method, the predicted along-wind rms resonant acceleration underestimated the actually measured response by 20% for southerly winds and by 25% to 45% (depending upon the turbulence intensity chosen) for westerly winds.

Putting measured values into the ESDU calculation method for across-wind response, the predicted rms resonant acceleration overestimates the actually measured response by between 1.6 and 5 times. Using estimated values suggested by ESDU methods instead of measured ones increases the overestimate of response to between 5.5 and 12 times.

The ESDU methods predict that the rms resonant component of response should increase in proportion to the wind speed raised to the power 2.63 for across-wind response, and to the power 3.09 for along-wind response. These figures are in reasonable agreement with the full-scale data obtained from Hume Point where exponents of 2.7 to 3.3 were obtained.

The ESDU methods predict that the ratio of peak to rms acceleration for modes with natural frequencies of 0.94 Hz and 1.15 Hz, are 4.17 and 4.22 respectively. These compare with the measured mean values obtained at full-scale of 4.1 and 3.7 respectively, although the range of values obtained at full-scale ranged from 1.7 to 6.7.

The ESDU methods give no guidance on the possible errors in estimating natural frequency using the suggested formulae (or any other method) neither is the original source of these formulae referenced. The formulae
used to estimate natural frequencies are those that gave the best fit to the measured data in the original source. Therefore, these formulae will only produce conservative estimates in approximately half of the cases in which they are applied. Some guidance on the possible errors in estimating natural frequencies would seem to be essential. In addition, some thought should be given to using formulae which produce lower estimates of natural frequency (and therefore higher estimates of response) so that conservative estimates of response will be obtained in a higher proportion of cases.

A 1:200 scale model of Hume Point was tested in the BRE boundary layer wind tunnel. Three distinct terrain simulations were used to model the actual terrain around Hume Point. Using this simulation, and the measured values of mass, natural frequency and damping, the wind tunnel tests were able to predict the along-wind response of Hume Point to within about 30% of the actually measured values, the across-wind response to within a factor of 2, and the torsional response to within a factor of 5. The wind tunnel tests showed a similar difference in response to that obtained at full-scale between easterly and westerly winds.

8.4 RECOMMENDATIONS FOR FUTURE WORK

Clearly, analysis other than that described in this thesis could be carried out on the full-scale data obtained from Hume Point. Until another full-scale test of comparable accuracy is carried out, the data from Hume Point will be the best available to use in comparison exercises between full and model-scale data or measured and predicted responses. In particular, measured parameters should be used as inputs to theoretical methods other than the ESDU ones, and the results compared with the full-scale measured response. In addition, a comparative wind tunnel exercise, similar to that carried out on the low rise "Aylesbury" building (ref 117), but on a model of Hume Point, would be useful.

Another tall building (preferably of steel frame construction) should be tested to ensure that the results obtained at Hume Point are repeatable. The same experimental and analytical techniques should be used with a few minor improvements. A 0.01 Hz High Pass filter in the signal conditioning
of the signal from the accelerometers would be useful in preventing the drift seen in the data from Hume Point. Although the additional filter would prevent the apparent drift, it would not prevent actual drift. Therefore the filter would have to be capable of being switched off and then on again remotely to allow D.C. offsets to be reset on a regular (perhaps daily) basis. The security of equipment must be better than that for Hume Point. However, if an inhabited building is used, great care must be taken to ensure that the occupants cannot disturb the instrumentation in any way. The increased use of mobile telephones is likely to make shielding of instrumentation and signal conditioning to prevent radio interference a necessity. Some form of algorithm to test each record for any such interference would also be very useful, particularly if extensive analysis of peak to rms accelerations is to be carried out.

It has been shown that both wind tunnel and the ESDU calculation methods can predict the along-wind response of Hume Point to within ±50% providing the correct natural frequency and other building parameters are used in the predictions. Improvements in the methods used to estimate the natural frequency, damping and mass of tall buildings are likely to lead to much better overall response predictions than refinements in the method used to calculate response using the present estimates to provide the input parameters for the calculations. Until improvements in the estimation of these building parameters are made, refinements to the prediction methods should be limited to ones which simplify the calculation procedure. The exception to this would be if the results obtained can be shown to be significantly more accurate than the previous method by comparing the two sets of results with full-scale data.
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APPENDIX A: CALCULATING THE OVERALL RMS VALUE OF AN ENSEMBLE.

Suppose there are m records each containing n observations:

\begin{align*}
a_1, a_2, \ldots, a_i, \ldots, a_n \\
b_1, b_2, \ldots, b_i, \ldots, b_n \\
\text{etc. to} \\
r_1, r_2, \ldots, r_i, \ldots, r_n
\end{align*}

Writing \( \Sigma \) as \( \Sigma \) throughout for simplicity,

\[ \sum_{i=1}^{n} \]

The mean square value of record a is \( \frac{\Sigma a_i^2}{n} \) and the rms value is \( \sqrt{\frac{\Sigma a_i^2}{n}} \).

Similarly, the mean square value of record b is \( \frac{\Sigma b_i^2}{n} \) and the rms value is \( \sqrt{\frac{\Sigma b_i^2}{n}} \).

Now consider an ensemble of all m records.

The true mean square value, \( T \), of all the observations is:

\[ T = \frac{1}{mn} \left( \Sigma a_i^2 + \Sigma b_i^2 + \ldots + \Sigma r_i^2 \right) \]

which can be written as

\[ T = \frac{1}{m} \left( \frac{\Sigma a_i^2}{n} + \frac{\Sigma b_i^2}{n} + \ldots + \frac{\Sigma r_i^2}{n} \right) \]  \hspace{1cm} (Eq A1)

Equation A1 is the average of the individual mean square values for each record.

It should be noted that this result is valid only for the case where the records all contain n observations.

Let the false mean square value, \( F \), be defined as the value obtained by squaring the average of the individual rms values for each record.

\[ F = \left[ \frac{1}{m} \left( \sqrt{\frac{\Sigma a_i^2}{n}} + \sqrt{\frac{\Sigma b_i^2}{n}} + \ldots + \sqrt{\frac{\Sigma r_i^2}{n}} \right) \right]^2 \]
If the rms value for each record is identical then

\[ F = \left[ \frac{1}{n} \left( \frac{\sum \sqrt{a_i^2}}{m} \right) \right]^2 = \frac{\sum a_i^2}{n} \]

It can also be seen from equation A1 that when the rms value for each record is identical then

\[ T = \frac{1}{m} \left( \frac{\sum a_i^2}{n} \right) = \frac{\sum a_i^2}{n} \]

So that \( T = F \) when the rms values for each record are identical.

When the values from each record are not identical then \( F \) can be rewritten as

\[ F = \frac{1}{m^2} \left( \sum \frac{a_i^2}{n} + \sum \frac{b_i^2}{n} + \ldots + \sum \frac{r_i^2}{n} \right) + \text{sum of } m(m-1) \text{ c.p.t.l.} \cdot 2 \sqrt{\frac{a_i^2}{n}} \sqrt{\frac{b_i^2}{n}} \]

\( \text{(c.p.t.l. = cross product terms like)} \)

So, using equation A1,

\[ F = \frac{1}{m^2} \left( mT + \frac{m(m-1)}{2} \text{ c.p.t.l.} \cdot 2 \sqrt{\frac{a_i^2}{n}} \sqrt{\frac{b_i^2}{n}} \right) \]

or, \( F = T + \frac{1}{m^2} \left( mT - m^2T + \frac{m(m-1)}{2} \text{ c.p.t.l.} \cdot 2 \sqrt{\frac{a_i^2}{n}} \sqrt{\frac{b_i^2}{n}} \right) \) \hspace{1cm} (Eq A2)

As \( 1/m^2 \) must always be positive, then \( T \) is greater than \( F \) if the expression inside the brackets is negative.

That is \( T > F \) if \( m^2T > mT + \text{cross product terms} \)

or,

\[ m \left( \sum \frac{a_i^2}{n} + \sum \frac{b_i^2}{n} + \ldots \right) > \left( \frac{a_i^2}{n} + \frac{b_i^2}{n} + \ldots \right) + \frac{m(m-1)}{2} \text{ c.p.t.l.} \cdot 2 \sqrt{\frac{a_i^2}{n}} \sqrt{\frac{b_i^2}{n}} \]

multiplying through by \( n \):

\[ m \left( \sum \frac{a_i^2}{n} + \sum \frac{b_i^2}{n} + \ldots \right) > \left( \frac{a_i^2}{n} + \frac{b_i^2}{n} + \ldots \right) + \frac{m(m-1)}{2} \text{ c.p.t.l.} \cdot 2 \sqrt{\frac{a_i^2}{n}} \sqrt{\frac{b_i^2}{n}} \]

or \( (m-1) \left( \sum \frac{a_i^2}{n} + \sum \frac{b_i^2}{n} + \ldots \right) > \frac{m(m-1)}{2} \text{ c.p.t.l.} \cdot 2 \sqrt{\frac{a_i^2}{n}} \sqrt{\frac{b_i^2}{n}} \) \hspace{1cm} (Eq i3)
This requirement can be proved by simple algebra. If a and b are two real, unequal numbers then

\[(a-b)^2\] is always positive

so that \[(a-b)^2 > 0\]

or, \[a^2 + b^2 > 2ab\]

Therefore, for three real numbers a, b and c:

\[a^2 + b^2 > 2ab\]

\[b^2 + c^2 > 2bc\]

\[c^2 + a^2 > 2ac\]

and therefore \[2(a^2 + b^2 + c^2) > 2(ab + bc + ac)\]

For four real numbers a, b, c and d:

\[a^2 + b^2 > 2ab\]

\[a^2 + d^2 > 2ad\]

\[b^2 + c^2 > 2bc\]

\[b^2 + d^2 > 2bd\]

\[a^2 + c^2 > 2ac\]

\[c^2 + d^2 > 2cd\]

and therefore \[3(a^2 + b^2 + c^2 + d^2) > 2(ab + bc + ac + ad + bd + cd)\]

Extending this to a set of m real numbers a to r:

\[(m-1)(a^2 + b^2 + c^2 + \ldots + r^2) > \text{sum of } \frac{m(m-1)}{2} \text{ c.p.t.l. } 2ab\] (Eq A4)

Rewriting equation A4 with \[a = \sqrt{\Sigma a_i^2}\] and \[b = \sqrt{\Sigma b_i^2}\] etc.

\[(m-1) \left(\Sigma a_i^2 + \Sigma b_i^2 + \ldots + \Sigma r_i^2\right) > \text{sum of } \frac{m(m-1)}{2} \text{ c.p.t.l. } 2 \sqrt{\Sigma a_i^2} \sqrt{\Sigma b_i^2}\]

This is the requirement in equation A3 for \(T\) to be greater than \(F\).

Therefore, for m records each containing n observations, the true rms value of the ensemble (the square root of the average of the mean square values) is always greater than the average of the rms values, unless all the rms values are identical, in which case the two quantities are equal.