



Coláiste na Tríonóide, Baile Átha Cliath
Trinity College Dublin

Ollscoil Átha Cliath | The University of Dublin

The Wind-Induced Vibration Response of Modular High-Rise Buildings

HOLLIE MOORE



A thesis submitted in partial fulfilment of the requirements for the degree of
Doctor Of Philosophy

in the

Department of Civil, Structural and Environmental Engineering
Trinity College Dublin, the University of Dublin
Ireland

February, 2025

Declaration

I declare that this thesis has not been submitted as an exercise for a degree at this or any other university and it is entirely my own work.

I agree to deposit this thesis in the University's open access institutional repository or allow the Library to do so on my behalf, subject to Irish Copyright Legislation and Trinity College Library conditions of use and acknowledgement.

I consent to the examiner retaining a copy of the thesis beyond the examining period, should they so wish (EU GDPR May 2018).

Signature _____

Hollie Moore
Civil, Struct. & Env. Eng.
Trinity College Dublin

Date _____

27/09/2024

The Wind-Induced Vibration Response of Modular High-Rise Buildings

Hollie Moore

15316957

Abstract

Structural engineering continues to advance and push boundaries enabling taller, more slender and flexible structures to be realised. However, these structures are often more susceptible to significant wind-induced vibrations resulting in a dynamic response which can cause human discomfort. The inherent properties of a structure such as the mass, stiffness and damping ratio are the parameters which have the largest impact on its dynamic response. Quantifying these properties is essential in the design process of tall buildings in order to ensure habitability requirements are satisfied. Volumetric modular construction is a relatively new form of construction in which prefabricated room sized units are constructed in a factory environment and transported to a construction site to be positioned and stacked to create a finished building. Traditionally modular construction has been employed in low to medium rise repetitive structures. However, more recently, this form of construction has been applied in tall structures exceeding 160 m. At present, limited research is available on the inherent properties of modular structures, with no reported estimates of the damping ratio or stiffness of this form of construction available.

This research aims to provide an initial insight into the wind-induced dynamic performance of high-rise modular buildings, develop data-informed surrogate models and to investigate suitable mitigation measures. The acceleration responses of two of the world's tallest volumetric structures, *Ten Degrees* and *College Road*, located in Croydon, London, United Kingdom obtained from full-scale in situ monitoring were studied. OMA techniques have been used to identify the natural frequencies of the structures and to calculate initial estimates of the damping ratios. The identified natural frequencies were used alongside a stepped-beam numerical model to quantify the bending stiffness of the volumetric module - RC core structural system. Using the identified inherent properties, a data-informed MDOF surrogate model of a modular building was developed and calibrated using the results from the in situ monitoring. This MDOF surrogate model was found to reflect the dynamic

response of the case study structures better than FE models. Parametric assessment of the response control of MVDTLs was performed using the MDOF surrogate model, alongside an equivalent TMD model of a TLD. MVDTLs offer a suitable control strategy which can be optimised to suit the form of modular buildings. Hybrid tests using a full-scale TLD and the MDOF surrogate model were also performed, incorporating results from full-scale tests of modular buildings with full-scale testing of control strategies and enabling parametric assessment which would otherwise not be feasible. Both the numerical and experimental results show that multiple distributed TLDs can effectively decrease the wind-induced response of modular buildings, providing an opportunistic control measure capable of ensuring habitability requirements are adhered to.

Summary

Ensuring occupant comfort within structures is an important aspect of structural design. Advances in technology, materials and construction techniques have enabled taller, lighter and more flexible structures to be constructed. These buildings can be sensitive to excessive wind-induced vibrations, causing a number of adverse effects for occupants such as difficulty concentrating and motion sickness. This research investigates the dynamic response of a new generation of high-rise buildings employing modular construction technology and the ability to reduce the dynamic response using supplementary damping devices such as Tuned Liquid Dampers (TLDs).

In modular construction, individual apartment-sized steel-framed volumetric modules are manufactured in a quality controlled factory environment and transported to the building site where they are then assembled to form finished buildings. This achieves higher quality and reliability, and lower environmental and economic costs, but when employed in tall buildings presents additional engineering challenges. Amongst these is providing adequate lateral resistance, which typically relies upon the lateral strength and stability of slender reinforced concrete (RC) cores. The resulting structural system of integrated RC core and modules possesses limited and uncertain global stiffness, rendering tall modular buildings susceptible to increased horizontal deformation during strong winds. Furthermore, there has been limited full-scale monitoring of high-rise modular buildings, with no reported estimates of the damping ratio; an inherent property which has a significant effect on the ability of a structure to dissipate vibrations. The acceptable magnitude of associated wind-induced oscillatory motion is limited by the lowest floor acceleration that humans can perceive. The uncertainty in the inherent properties of this form of construction presents challenges in ensuring that floor accelerations are within these limits and habitability criteria are adhered to during the design process.

To address the lack of research and uncertainty surrounding inherent properties of modular buildings, this research has considered results from full-scale tests of two of the world’s tallest volumetric modular buildings, *Ten Degrees* and *College Road*. Acceleration responses of the two structures recorded at different stages of construction and have been processed using Operational Modal Analysis (OMA) techniques aimed at identifying the natural frequency and damping ratio of the structures. Four different OMA methods have been used including the Bayesian Fast Fourier Transform (BFFT), the Random Decrement Technique (RDT), an improved RDT method including Analytical Mode Decomposition (AMD) to identify closely spaced modes, referred to as the AMD-RDT method, and a bootstrapped AMD-RDT method to allow for quantification of uncertainty in the estimates of inherent properties. This research is the first to report the damping ratio of volumetric modular buildings from full-scale tests. The identified natural frequencies of the structures at different states of construction have been used alongside a numerical stepped beam model to provide an estimate of the global bending stiffness of the combined module-RC core system. Information about these two inherent properties, damping ratio and bending stiffness, enables more accurate prediction of the dynamic response of modular structures.

Using the calculated values of the inherent properties of modular buildings, including the critical damping ratios identified through applying OMA techniques to the measured response of the case study structures, stiffness values calculated using results from the full-scale testing of the modular structures and mass properties estimated from Finite Element (FE) models created in the software ETABs, a data-informed Multi Degree of Freedom (MDOF) surrogate model has been created. The model has been calibrated using the results from the full-scale testing of the modular structures and has been found to more accurately reflect the dynamic behaviour of the structures than FE models. Whilst the wind-induced accelerations of the structures studied are not expected to exceed habitability requirements, there is still a level of uncertainty surrounding their inherent properties and response. As modular construction continues to be employed in structures of exceedingly greater heights, it is likely that a mitigation measure to control wind-induced vibrations will be necessary to provide more confidence in the response and to transcend uncertainty in the inherent properties.

TLDs are a low-maintenance and economical passive damping technology

which can be easily tuned to the natural frequency of a structure. Within this thesis, they are parametrically assessed for their ability to control the dynamic response of modular buildings. In particular, given the off-site manufacture and discrete nature of modular buildings, the concept of Multiple Vertically Distributed Tuned Liquid Dampers (MVDTLDs) has been considered. This is a novel concept, capitalising on the form of modular construction through dispersion of repeatable supplementary damping devices throughout the structure, rather than occupying large areas of the upper floors of a modular building. Parametric analysis of MVDTLDs using the data-informed MDOF surrogate model and an equivalent Tuned Mass Damper (TMD) model of a TLD showed effective response reduction when TLDs were distributed over the top third of a 50 storey modular building. Therefore, through integration of TLDs in the manufacture of individual modules, prime real estate at the top of a structure is not affected and excessive wind-induced accelerations can be controlled.

The sloshing behaviour of liquid within TLDs is highly complex and difficult to reflect through modelling. Therefore, a series of laboratory tests on a full-scale prototype TLD were conducted to first validate the appropriateness of the equivalent TMD model. Real Time Hybrid Testing (RTHT) was also performed using the data-informed MDOF surrogate model and the full-scale prototype TLD. The RTHT combined the results of full-scale tests of the high-rise modular buildings represented in the MDOF surrogate model with the experimental response of a full-scale prototype TLD allowing for a parametric study of the response reduction observed with different mass ratios achieved by using multiple TLDs. In reality, a full-scale parametric analysis using the actual structures would be completely uneconomical and unfeasible. The results from RTHT showed a reduction in the wind-induced response and identified a number of areas for further study.

This research covers a broad range of research methods including acceleration response analysis, field measurements, OMA techniques, numerical modelling and RTHT in order to provide an initial insight into the inherent properties of high-rise volumetric modular buildings, their dynamic response and response reduction using TLDs. The scope of the research is visualised in Figure 1. The work presented in this thesis was completed with the aim of addressing a knowledge gap in the global behaviour of modular structures and to aid Structural Engineers in the future design of modular structures as they continue to reach new heights.

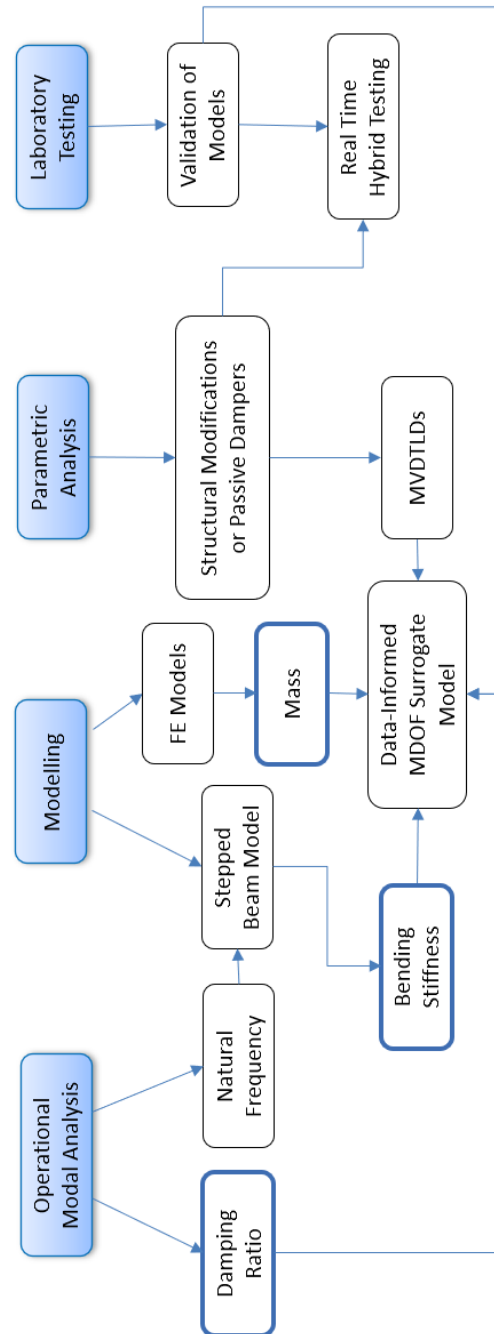


Figure 1: Scope of research presented in this thesis

Acknowledgements

I would not be where I am today without the community of people who have supported me along the way.

Firstly, I would like to thank my supervisor Prof. Brian Broderick. I am incredibly grateful for your continued support, encouragement and calm nature which has been present from the very start of my research journey. I am lucky to have had such an experienced, kind and caring supervisor. I would also like to express many thanks to my co-supervisor Dr. Breiffni Fitzgerald for sharing his knowledge, expertise and always offering sound advice. I am immensely grateful to Prof. Biswajit Basu and Dr. Dermot O'Dwyer for forming my doctoral committee and providing much appreciated feedback throughout my PhD. A special thank you to Dr. John Hickey, who always took the time to answer my endless questions and has gone above and beyond to provide invaluable guidance throughout my research. I would also like to thank Dave McCauley, Robert Fitzpatrick and especially Michael Grimes for all of their help in the lab, being friendly faces, and for putting up with my file naming 'conventions'. My research would not have been possible without all of the staff in the Civil, Structural and Environmental Department at Trinity College Dublin. I could not have asked for a more caring and supportive environment to receive my education.

This endeavour would not have been possible without the generous support from the Irish Research Council and Barrett Mahony Consulting Engineers who jointly funded this research. I'm extremely grateful to Vincent Barrett, who offered a career worth of knowledge and the opportunity to complete this project. Many thanks to Shane Linehan, who graciously provided his expertise in modular construction. I would also like to express my thanks to Tide Construction and MJH Structural Engineers for sharing the information necessary to complete this research.

To everyone in the PhD office, thank you for the camaraderie and for making this process so much more enjoyable. I am especially grateful for James, Shubham and Madhavan, your support and friendship has been the highlight of my PhD. A special shoutout also to James's Mam's caramel squares and to the toastie maker for making the long days a little bit easier (and tastier).

To my friends, thank you for always being there, listening to my complaints and reminding me to have fun. Thank you for rallying around me when I needed you most.

I am incredibly lucky to have inherited a second family by way of my in laws. To Aidan and Marie, the way you have looked after me and loved me like one of your own has meant more to me than you will ever know and will always be a testament to Daniel. Orla, Conor and Aoife, thank you for the sweet treats, the jokes and for never turning down an adventure. Your presence in my life is something I will always cherish.

There are no words to express how grateful I am to my entire family. Thank you to my little sister Lucy for keeping me humble, laughing (mainly at Mam and Dad) and for being you. I am so fortunate to have you as a sister. To my Mam and Dad, I would not be who I am without you. Thank you for all you have done, and continue to do for me. You have given me every possible opportunity in life and your love, belief in me and encouragement has been the greatest gift I could ever ask for.

Lastly, to Daniel. Your love is my greatest strength. Thank you for everything.
Always.

Contents

Declaration	i
Abstract	iv
Summary	vii
Acknowledgements	x
List of Figures	xv
List of Tables	xxiii
Acronyms	xxvii
1 Introduction	1
1.1 Motivation for Research	1
1.2 Statement of Problem	4
1.3 Research Aims and Objectives	5
1.4 Thesis Outline	6
2 Literature Review	9
2.1 Introduction	9
2.2 Modular Construction	9
2.2.1 Form of Modular Construction	11
2.2.2 Advantages of Modular Construction	13
2.2.3 Challenges facing Modular Construction	14
2.3 High-Rise Modular Construction	22
2.3.1 Limitations in Modelling Modular Buildings	24
2.3.2 Stiffness Contribution of Modules to the Completed Structure	25

2.3.3	Global Dynamic Behaviour of Modular Buildings	26
2.4	Damping Values in Structures	27
2.5	Occupant Comfort in High-Rise Structures	31
2.6	Operational Modal Analysis	33
2.6.1	Time Domain OMA Methods	34
2.6.2	Frequency Domain OMA Methods	36
2.7	Mitigation of Wind-Induced Vibration	37
2.7.1	Mitigation of Vibrations Through Structural Design	38
2.7.2	Aerodynamic Modifications for the Mitigation of Vibrations	38
2.7.3	Auxiliary Damping Devices	40
2.8	Real Time Hybrid Testing	52
2.9	Conclusions	56
3	Comparing Structural Modifications to Auxiliary damping for Mitigation of Wind-Induced Acceleration	57
3.1	Introduction	57
3.1.1	Idealised Structural Scheme	58
3.2	Generation of Structural Dataset	61
3.3	Calculation of Building Acceleration Response	64
3.3.1	Calculation Process	64
3.3.2	Calculation Results	66
3.3.3	Required Acceleration Reduction	71
3.4	Modifications to Mitigate Excessive Accelerations	73
3.4.1	Required Increase in Core Dimensions	73
3.4.2	Required Additional Damping	77
3.4.3	Additional Damping Provided by TLD	79
3.5	Maximum Feasible Height of Modular Buildings	84
3.6	Conclusions	87
4	Case Study Structures	89
4.1	Introduction	89
4.2	Ten Degrees, Croydon	89
4.2.1	Structural Design	90
4.2.2	Finite Element Models	95
4.2.3	Wind Tunnel Testing	97
4.3	College Road, Croydon	100

4.3.1	Structural Design	101
4.3.2	Finite Element Models	103
4.3.3	Wind Tunnel Testing	105
4.4	Conclusions	107
5	Operational Modal Analysis of Case Study Structures	111
5.1	Introduction	111
5.1.1	Factors which may Influence Damping Ratio Estimates . . .	112
5.1.2	Scope of Work	114
5.2	Operational Modal Analysis Methods	115
5.2.1	Bayesian Fast Fourier Transform	115
5.2.2	Random Decrement Technique	119
5.2.3	Analytical Mode Decomposition	121
5.2.4	Bootstrapped AMD-RDT	122
5.3	Monitoring	126
5.3.1	Ten Degrees	126
5.3.2	College Road	130
5.4	Processing Measurements from In Situ Monitoring	134
5.4.1	Ten Degrees	134
5.4.2	College Road	154
5.4.3	Damping Ratio of Modular buildings	172
5.5	Conclusions	175
6	Stiffness Identification and Data-Informed Surrogate Models	179
6.1	Introduction	179
6.2	Finite Element Models of Modular Buildings	180
6.2.1	Finite Element Models Reported in Literature	181
6.2.2	Finite Element Models of Case Study Structures	185
6.2.3	Macro Module Finite Element Model	188
6.3	Stepped Beam Model	192
6.3.1	Adomian Decomposition Method	193
6.3.2	Application of the Stepped Beam to Case Study Structures .	196
6.3.3	Identified Bending Stiffness Contribution of Modules	205
6.4	Data-Informed Multi-Degree of Freedom Surrogate Model	208
6.4.1	Calibration of MDOF Surrogate Model	212
6.5	Conclusions	214

7	Assessment, Validation and Testing of Suitable Control Methods	217
7.1	Introduction	217
7.2	Modelling of a Tuned Liquid Damper	219
7.2.1	Equations Governing Behaviour of a TLD	219
7.2.2	Equivalent Tuned Mass Damper Model	224
7.3	Representing MVDTLDS in the Data-Informed MDOF Surrogate Model	225
7.4	Validating the equivalent TMD Model of a TLD	230
7.4.1	Laboratory Setup	230
7.4.2	Archetype Structures	232
7.4.3	Validation using Sinusoidal Loading	232
7.4.4	Validation for Wind-Induced Response	237
7.5	Parametric Study of MVDTLDS	259
7.6	Fragility Curves for Modular High-Rise Buildings with Multiple Ver- tically Distributed Tuned Liquid Dampers under Wind Loading . .	273
7.6.1	Development of Fragility Curves	274
7.7	Conclusions	277
8	Real Time Hybrid Testing	281
8.1	Introduction	281
8.2	RTHT Procedure and Setup	282
8.2.1	RTHT Background	282
8.2.2	RTHT Facilities at Trinity College Dublin	284
8.2.3	RTHT procedure	286
8.2.4	Integration Schemes in RTHT	288
8.2.5	Errors in RTHT	290
8.3	Results from RTHT	292
8.4	Conclusions	299
9	Conclusions	301
9.1	Summary of Completed Work	301
9.2	Main Findings	304
9.3	Suggestions for Future Work	308
Appendix A Validation of Identified Bending Stiffness Contribution of Modules		311
Bibliography		315

List of Figures

1	Scope of research presented in this thesis	viii
1.1	Example of modules being lifted into place in the <i>Ten Degrees</i> project in Croydon, London (Source: Tide Construction [6])	2
2.1	The manufacturing and installation process involved in volumetric modular construction (Source: Liew et al. [30])	10
2.2	Example of elemental, planar and volumetric modules (Source: Lim et al. [31])	11
2.3	Modern Methods of Construction definition framework (Source: Housing, Communities & Local Government Joint Industry Working Group [23])	12
2.4	Example of a corner supported volumetric module (Source: Liew et al. [30])	13
2.5	Intra- and inter-module connections in modular construction (Source: Chen et al. [21])	17
2.6	Various types of intra-module connections used in volumetric modular construction (Source: Chen et al. [21])	17
2.7	Examples of inter-module connections between vertical columns used in volumetric modular construction: (a) vertical rod system [30] and (b) prestressed strands [57] (Source: Liew et al. [30])	18
2.8	Examples of inter-module connections between adjacent beams and columns used in volumetric modular construction: (a) Vectrobloc system [58] (b) corner-joint [59] (c) perimeter joint [60] (d) and (e) interior joint [61, 62] (Source: Liew et al. [30])	19
2.9	Examples of connections between modules and lateral stability systems used in volumetric modular construction (Source: Chen et al. [21])	21

2.10	Example of duplicate members which occur when modules are stacked together	25
2.11	ISO 10137 peak acceleration limits [84]	33
2.12	Possible aerodynamic modifications to corners of a structure (Source: Mooneghi et al. [114])	39
2.13	Possible aerodynamic modifications to the overall shape of a structure (Source: Jafari et al. [7])	39
2.14	Mechanism of an Active Mass Damper (AMD) (Source: Jafari et al. [7])	41
2.15	Example of an Active Bracing system (Source: Jafari et al. [7])	42
2.16	Schematic diagram of Viscoelastic Damper (Source: Vatanshenas et al. [136])	44
2.17	Mechanism of Viscous Fluid Damper (Source: Jafari et al. [7])	45
2.18	TMD system attached to a primary structure (Source: Chen et al. [147])	46
2.19	(a) Rectangular tuned liquid damper and (b) single-degree-of-freedom oscillator with a tuned liquid damper (Source: Malekghasemi et al. [160])	47
2.20	Schematic of MDOF with MTMD model presented by Moon [183]	51
2.21	Example of Real Time Hybrid Testing process (Source: Wang et al. [202])	53
3.1	Typical volumetric modular building with RC core (Source: Tide Construction [6])	59
3.2	Simple representation of archetype building investigated	61
3.3	Relationship between peak acceleration and building mass, shown in both mass per unit volume (Mg/m^3) and module self weight (kN/m^2), for a 3 m storey height) for the 120 m archetypes	68
3.4	Relationship between peak acceleration and core wall thickness for 120 m Archetypes	69
3.5	Impact of building plan dimensions on peak accelerations for 120 m archetypes	70
3.6	Required acceleration reduction for archetypes exceeding ISO acceleration limits	71
3.7	Number of archetypes requiring acceleration reduction by module self weight and building height	72

3.8	Illustration of increasing core outer dimensions (left) and increasing core thickness (right). The method on the left was employed in this study	73
3.9	Incremental increase in core dimensions to reduce response acceleration below ISO limits for two example archetypes	74
3.10	Required increase in core dimensions to achieve percentage reduction in acceleration response	75
3.11	Extent of core increase required to satisfy acceleration limits	75
3.12	Percentage reduction provided by increasing core dimensions for cases where less than a 2000 mm increase is required	76
3.13	Percentage reduction in acceleration provided by increase in damping	78
3.14	Comparison of required increase in core depth and required additional damping for each archetype	78
3.15	Schematic of TLD	79
3.16	Variation in added damping with building core size and total mass for the 120 m and 150 m archetypes	82
3.17	Comparison of required damping and predicted available damping from an optimal TLD for archetypes	83
3.18	Approximated maximum feasible height of single core modular buildings with varying combinations of self weight equipped with a TLD (approximated for a 3 m storey height) and core wall thickness . . .	85
3.19	Approximated maximum feasible height of single core modular buildings with varying combinations of self weight with no supplementary damping mechanism (approximated for a 3 m storey height) and core wall thickness	86
4.1	View of the <i>Ten Degrees</i> Tower and location	90
4.2	Plan view of <i>Ten Degrees</i>	91
4.3	High-Level construction programme for <i>Ten Degrees</i> , <i>Croydon</i> (Source: Tide Construction [216])	91
4.4	Ties between concrete cores during module installation (Source: Tide Construction [6])	92
4.5	Typical volumetric corner post module installed in <i>Ten Degrees</i> . .	93
4.6	Image of the FE model created of <i>Ten Degrees</i> by <i>MJH Structural Engineers</i>	95

4.7	Wind tunnel set up showing scaled model of <i>Ten Degrees</i> in blue and the surrounding conditions	98
4.8	ISO 10137 acceleration limits for occupants comfort compared to top storey accelerations of <i>Ten Degrees</i> estimated through wind tunnel tests and dynamic analysis	100
4.9	<i>Ten Degrees</i> and <i>College Road</i> , Europe's tallest volumetric modular buildings (Source: Tide Construction [25])	101
4.10	Plan view of <i>College Road</i>	102
4.11	FE model of <i>College Road</i>	104
4.12	Wind tunnel set up showing scaled model of <i>College Road</i> in red, <i>Ten Degrees</i> to the left in grey and other surrounding structures	106
4.13	ISO 10137 acceleration limits for occupants comfort compared to top storey accelerations of <i>College Road</i> estimated through wind tunnel tests and dynamic analysis	107
5.1	Conceptualisation of the Random Decrement Technique (Source: Zhou and Li [70])	120
5.2	Illustration of the Bootstrap method as applied to the AMD-RDT method	125
5.3	Picture of accelerometer and data logger used in full-scale monitoring	126
5.4	Location of accelerometers and data logger used in <i>Ten Degrees</i> . .	127
5.5	The <i>Ten Degrees</i> building in Croydon, London, pictured at two points in time during module installation (Source: Tide Construction [216])	128
5.6	Graphical representation of the number of levels of modules installed at <i>Ten Degrees</i> for each set of measurement data	129
5.7	Location of accelerometers during initial 5 months (a) and final 5 months (b) of the monitoring campaign	131
5.8	Graphical representation of the number of levels of modules installed at <i>College Road</i> for each set of measurement data	133
5.9	Average wind speed recorded at <i>Ten Degrees</i>	136
5.10	Acceleration and wind time histories for accelerometer 1 measured at <i>Ten Degrees</i>	138
5.11	Acceleration and wind time histories for accelerometer 2 measured at <i>Ten Degrees</i>	139
5.12	Fast Fourier Transform of Measurement 10 recorded at <i>Ten Degrees</i>	141
5.13	Natural frequencies for each measurement recorded at <i>Ten Degrees</i>	143

5.14	Estimated damping ratios of <i>Ten Degrees</i> calculated using the BFFT method	144
5.15	Estimated damping ratios of <i>Ten Degrees</i> calculated using the RDT and AMD-RDT methods	144
5.16	Decreasing trend identified in AMD-RDT estimate of damping and the natural frequency of <i>Ten Degrees</i>	145
5.17	Comparison of calculated damping ratios from AMD RDT and BFFT methods for <i>Ten Degrees</i>	146
5.18	Probability distribution function of bootstrap samples of measurements from accelerometer 1 in East-West (x) direction from <i>Ten Degrees</i>	148
5.19	Comparison of damping estimates and their possible range using CV values from both the BFFT and bootstrapped AMD-RDT method for <i>Ten Degrees</i>	150
5.20	Estimated damping ratio of <i>Ten Degrees</i> vs estimated natural frequency	151
5.21	Estimated Natural Frequency of <i>Ten Degrees</i> vs RMS acceleration	152
5.22	Estimated damping ratio of <i>Ten Degrees</i> vs RMS acceleration . . .	153
5.23	Acceleration time history for Accelerometer 1 measured from <i>College Road</i>	156
5.24	Acceleration time history for Accelerometer 2 measured from <i>College Road</i>	157
5.25	Fast Fourier Transform of Measurement 10 recorded at <i>College Road</i>	160
5.26	Natural frequencies for each measurement recorded at <i>College Road</i>	162
5.27	Estimated damping ratios of <i>College Road</i> calculated using the BFFT method	163
5.28	Estimated damping ratios of <i>College Road</i> calculated using the RDT and AMD-RDT methods	165
5.29	Comparison of calculated damping ratios from AMD RDT and BFFT methods for <i>College Road</i>	165
5.30	Comparison of damping estimates and their possible range using CV values from both the BFFT and bootstrapped AMD-RDT method for <i>College Road</i>	168
5.31	Estimated damping ratio of <i>College Road</i> vs estimated natural frequency	169
5.32	Estimated Natural Frequency of <i>College Road</i> vs RMS acceleration	170
5.33	Estimated damping ratio of <i>College Road</i> vs RMS acceleration . . .	171

6.1	Macro module element	190
6.2	Coordinate system for the Euler Bernoulli stepped beam elastically restrained at both ends	196
6.3	Partially completed modular structure represented as a stepped beam	204
7.1	(a) Structure-TLD model and (b) linearised structure TLD model (Source: Tait [155])	219
7.2	Liquid sloshing in a TLD	220
7.3	Multiple damping devices distributed vertically over the height of a structure	226
7.4	Full-scale TLD tested in the MTS real-time hybrid test system at Trinity College Dublin, Ireland	231
7.5	Process followed to validate MDOF surrogate model containing an equivalent TMD model of a TLD	233
7.6	Comparison of the natural frequency of the equivalent TMD model and in situ TLD	234
7.7	Theoretical vs Experimental TLD Force Response for sinusoidal loading	236
7.8	Wind forces applied to each archetype building	240
7.9	Comparison of the natural frequency of the equivalent TMD model and in situ TLD under wind loading	242
7.10	Theoretical vs experimental TLD force response of a 40 storey archetype under wind loading	243
7.11	Theoretical vs experimental TLD force response of a 44 storey archetype under wind loading	244
7.12	Theoretical vs experimental TLD force response of a 46 storey archetype under wind loading	245
7.13	Theoretical vs experimental TLD force response of a 50 storey archetype under wind loading	246
7.14	Theoretical vs experimental TLD force response of a 55 storey archetype under wind loading	247
7.15	Theoretical vs experimental TLD force response of a 60 storey archetype under wind loading	248
7.16	RMS force response vs tuning ratio for (a) Theoretical TLD and (b) Experimental TLD	249
7.17	RMS force response vs h/L for (a) Theoretical TLD and (b) Experimental TLD	252

7.18	Theoretical vs experimental Power Spectral Density of Force response of a 40 storey archetype under wind loading	253
7.19	Theoretical vs experimental Power Spectral Density of Force response of a 44 storey archetype under wind loading	254
7.20	Theoretical vs experimental Power Spectral Density of Force response of a 46 storey archetype under wind loading	255
7.21	Theoretical vs experimental Power Spectral Density of Force response of a 50 storey archetype under wind loading	256
7.22	Theoretical vs experimental Power Spectral Density of Force response of a 55 storey archetype under wind loading	257
7.23	Theoretical vs experimental Power Spectral Density of Force response of a 60 storey archetype under wind loading	258
7.24	RMS acceleration and displacement responses from parametric analysis of MVDTLDS	263
7.25	Comparison of MVDTLDS considering evaluation criterion J1 . . .	265
7.26	Comparison of MVDTLDS considering evaluation criterion J2 . . .	266
7.27	Comparison of MVDTLDS considering evaluation criterion J3 . . .	267
7.28	Comparison of MVDTLDS considering evaluation criterion J4 . . .	268
7.29	Comparison of MVDTLDS considering evaluation criterion J7 . . .	269
7.30	Comparison of MVDTLDS considering evaluation criterion J9 . . .	270
7.31	Acceleration and displacement response of 50 storey modular archetype with 960 TLDs installed over the top 15 storeys	272
7.32	Fragility curve for the controlled and uncontrolled 50 storey modular building with varied bending stiffness, EI	276
7.33	Fragility curve for the controlled and uncontrolled 50 storey modular building with varied damping ratio, ξ	277
8.1	RTHT setup at Trinity College Dublin	285
8.2	Simulink model of 50 storey modular building used in RTHT	287
8.3	Acceleration and displacement responses of the uncontrolled and controlled modular building	292
8.4	Performance criteria for each mass ratio investigated	293
8.5	Comparison of numerical model and RTHT force responses of multiple TLDs	295
8.6	PSD of the TLD force calculated using MDOF surrogate model results and RTHT measurements	297

List of Figures

8.7 Comparison of numerical model and RTHT force response of a single TLD	298

List of Tables

2.1	Ten of the tallest modular buildings in the world [5, 25, 24]	23
2.2	Expressions for structural damping in the first mode of vibration reported in literature	29
2.3	Damping ratio estimates in the first mode of vibration from full scale in situ monitoring of structures	31
3.1	Archetype building properties investigated	62
4.1	Natural Frequencies of <i>Ten Degrees</i> identified from FE model	96
4.2	Peak acceleration response of <i>Ten Degrees</i> Tower A and Tower B for 1-year and 10-year return period wind speeds with values exceeding ISO 10137 [84] occupant comfort criteria highlighted in red	99
4.3	Natural Frequencies of <i>College Road</i> identified from FE model . . .	105
4.4	Peak acceleration response of <i>College Road</i> Tower A and Tower B for 1-year and 10-year return period wind speeds	106
5.1	Dates measurement data was recorded <i>Ten Degrees</i> and the number of floors of modules installed	130
5.2	Dates measurement data was recorded at <i>College Road</i> and the number of floors of modules installed	134
5.3	Average and maximum wind speeds for each in situ measurement of response recorded at <i>Ten Degrees</i>	135
5.4	Maximum accelerations for each measurement taken at <i>Ten Degrees</i>	140
5.5	RMS accelerations for each measurement taken at <i>Ten Degrees</i> . . .	140
5.6	Natural frequency calculated using the FFT of each measurement recorded at <i>Ten Degrees</i>	142
5.7	Average identified Damping Ratio of <i>Ten Degrees</i> for Measurements 8-10	146

5.8	CV of BFFT estimates of the damping ratio of <i>Ten Degrees</i>	147
5.9	CV of bootstrapped AMD-RDT estimates of the damping ratio of <i>Ten Degrees</i>	149
5.10	Identified inherent properties of <i>Ten Degrees</i>	153
5.11	Average Hourly wind speeds recorded at Heathrow Airport	154
5.12	Maximum accelerations for each measurement taken at <i>College Road</i>	158
5.13	RMS accelerations for each measurement taken at <i>College Road</i> . .	159
5.14	Natural frequency calculated using the FFT of each measurement recorded at <i>College Road</i>	161
5.15	Average Identified damping ratio of <i>College Road</i> for measurements 8-11	166
5.16	CV of BFFT estimates of the damping ratio of <i>College Road</i>	167
5.17	CV of bootstrapped AMD-RDT estimates of the damping ratio of <i>College Road</i>	167
5.18	Identified inherent properties of <i>College Road</i>	172
5.19	Identified inherent properties of both <i>Ten Degrees</i> and <i>College Road</i>	176
6.1	Natural Frequencies of <i>Ten Degrees</i> identified from FE model com- pared with natural frequencies identified from field measurements .	186
6.2	Natural Frequencies of <i>College Road</i> identified from FE model com- pared with natural frequencies identified from field measurements .	186
6.3	Material properties used in FE model of single module	188
6.4	Properties of single module in the x-direction	190
6.5	Natural frequencies of the full model and macro model of a single module	191
6.6	Natural frequencies of the full model and macro model of a single module	192
6.7	Identified mass per unit length found using the FE models <i>Ten Degrees</i>	206
6.8	Identified natural frequencies found using OMA of in situ acceleration of <i>Ten Degrees</i>	206
6.9	Identified bending stiffness values of <i>Ten Degrees</i> found using the stepped beam numerical model and natural frequencies from modal identification	206
6.10	Identified mass per unit length found using the FE models <i>College Road</i>	207

6.11	Identified natural frequencies found using OMA of in situ acceleration of <i>College Road</i>	207
6.12	Identified bending stiffness values of <i>College Road</i> found using the stepped beam numerical model and natural frequencies from modal identification	208
6.13	Data-informed mass values	209
6.14	Parameters used to predict the natural frequency of <i>Ten Degrees</i> using data-informed MDOF surrogate model	212
6.15	Parameters used to predict the natural frequency of <i>College Road</i> using data-informed MDOF surrogate model	213
6.16	Parameters used to predict the natural frequency of <i>College Road</i> using data-informed MDOF surrogate model	213
6.17	Natural Frequencies of <i>Ten Degrees</i> estimated using different models and in situ testing	214
6.18	Natural Frequencies of <i>College Road</i> estimated using different models and in situ testing	214
7.1	Archetype structures used for validation	232
7.2	Theoretical and experimental TLD natural frequencies	234
7.3	RMS force response of theoretical and experimental TLD	235
7.4	Wind speeds applied to archetype structures	237
7.5	Natural frequencies of the theoretical and experimental TLD	241
7.6	Maximum and RMS values of the theoretical and experimental force response	250
7.7	Range of parameters investigated in assessment of response reduction using MVDTLDS	260
8.1	The number of TLDs required to achieve each mass ratio investigated by the RTHT	288
A.1	Approximated $E_c I_c$ and $E_{cm} I_{cm}$ values for <i>College Road</i>	313

Acronyms

ADM	Adomian Decomposition Method.	193
AMD	Active Mass Damper.	41
AMD	Analytical Mode Decomposition.	vi
AMD-RDT	Analytical Mode Decomposition Random Decrement Techniques.	114
BFFT	Bayesian Fast Fourier Transform.	vi, 36, 114
CFD	Computational Fluid Dynamics.	37
COV-SSI	Covariance-driven SSI.	35
CV	Coefficient of Variation.	147
DATA-SSI	Data-driven SSI.	35
DOF	Degree of Freedom.	209, 210, 228
EFDD	Enhanced Frequency Domain Decomposition.	36
EMA	Experimental Modal Analysis.	33
EMD	Empirical Mode Decomposition.	121
FDD	Frequency Domain Decomposition.	36
FE	Finite Element.	vi, 24, 180
FEM	Finite Element Model.	95
FFT	Fast Fourier Transform.	37
IRF	Impulse Response Function.	35
LETI	London Energy Transformation Initiative.	13
MDOF	Multi Degree of Freedom.	vi, 50, 180, 209
MMC	Modern Method of Construction.	11

Acronyms

- MTMDs** Multiple Tuned Mass Dampers. 49
- MVDTLDs** Multiple Vertically Distributed Tuned Liquid Dampers. vii, 218, 225, 303
- NExT** Natural Excitation Technique. 35
- OMA** Operational Modal Analysis. vi, 6, 33, 108, 112
- PDF** Probability Density Function. 37
- PFC** Parallel Flange Channel. 93
- PP** Peak Picking. 36
- PSD** Power Spectral Density. 34, 36, 249
- PuD** Pseudo-Dynamic. 283
- RC** reinforced concrete. v, 90
- RD** Random Decrement Technique. vi, 34, 114
- RHHF** Rectangular Hollow Hot Formed. 93
- RIBA** Royal Institute of British Architects. 13
- RMS** Root Mean Square. 249
- RTHT** Real Time Hybrid Testing. vii, 5–7, 52, 230, 281, 303
- SCRAMNet** Shared Common Random Access Memory Network. 285
- SHCF** Square Hollow Cold Formed. 93
- SHHF** Square Hollow Hot Formed. 93
- SLS** Serviceability Limit State. 9, 90, 101
- SSI** Stochastic Subspace Identification. 35
- TLD** Tuned Liquid Damper. 6, 42, 47, 218, 219, 228
- TLDs** Tuned Liquid Dampers. v, 9, 43
- TMD** Tuned Mass Damper. vii, 43, 218, 219, 224
- ULS** Ultimate Limit State. 90, 101
- VE** Viscoelastic. 43
- VF** Viscous Fluid. 43

Chapter 1

Introduction

1.1 Motivation for Research

A recent report by the UN Special Rapporteur, Balakrishnan Rajagopal, on adequate housing as a component of the right to an adequate standard of living estimated that 100 million people worldwide are homeless with a staggering 1.6 billion people lacking access to suitable affordable housing [1]. This figure was projected to rise to 3 billion by 2030. The report states that rapid urbanisation and the climate crisis are among the causes of decreased affordability of housing. Among the conclusions of the report, it is expressed that it is the responsibility of the state to address housing affordability by ensuring an ample supply of homes with access to suitable amenities and making the provision of housing a priority. As populations continue to grow and land within cities becomes more scarce, high-rise modular construction can enable the rapid supply of homes whilst minimising the building footprint and costs.

Modular construction is a relatively new form of rapid, lightweight construction. Volumetric modular buildings involve the on site assembly of pre-constructed room sized units manufactured off-site in a factory setting. This form of construction has reportedly less mass than typical construction, requires bespoke connections and has considerably more members than traditional building forms [2, 3, 4, 5]. Whilst modular construction has been traditionally used in temporary, repetitive buildings, it is now used more widely in a range of buildings from residential accommodation to schools and hospitals [2]. However, the use of modular construction in high-rise

structures is still a novel concept with little research or data available on the global dynamic behaviour of this form of construction technology [4, 5].



Figure 1.1: Example of modules being lifted into place in the *Ten Degrees* project in Croydon, London (Source: Tide Construction [6])

Advancements in building technology, computational capacity and software has enabled modern construction to continue pushing structures to increasingly new heights [7, 8]. Modular buildings are no exception. The tall, slender and lightweight nature of any high-rise structure leaves them more susceptible to wind-induced vibrations which may affect occupant comfort [9]. This prevalent phenomenon has led to a significant research into the dynamic response of tall, slender structures and suitable control measures. Such studies have primarily focused on more traditional construction forms such as steel and reinforced concrete, for example [10, 11, 12, 13]. To the authors knowledge, other than the studies reported in this thesis, no previous in situ monitoring of the vibration of modular structures has been carried out.

Typically, the governing design criteria for slender high-rise structures arises in the serviceability limit state and regards peak accelerations under wind-induced loading [8, 14]. The conventional approach to this problem has been to provide the structure with adequate lateral stiffness, taking into account its natural vibration

properties and the characteristics of the design wind loading [15]. A wide variety of structural forms have been developed to this end that employ combinations of tube, core, bracing and out-rigger systems, amongst others. A non-conservative implementation of this approach requires an accurate prediction of the dynamic vibration response of the structure, accounting for highly uncertain wind loading and structural damping characteristics. More recently, supplementary vibration control devices have increasingly been employed to avoid undue design conservatism and associated economic and environmental costs [16].

The factors which influence the susceptibility of a structure to wind-induced dynamic response in excess of the serviceability limit state are its mass, stiffness and damping ratio (energy dissipation) [11]. Whilst the mass of a structure can be accurately approximated using structural design software, the stiffness, especially of a modular building, is more difficult to predict. Finite element based structural modelling software, typically developed for traditional construction techniques, can face challenges when trying to account for the intricacy of connections used in modular buildings [17]. This makes assessing the stiffness of modular structures through modelling arduous and unreliable [18]. While the modules are usually self-supporting under vertical (gravity) loads, a separate structural system is normally required to ensure adequate lateral strength and stiffness [19]. Slender reinforced concrete cores are typically used for this purpose as most other high-rise structural systems (e.g. brace tubes) are incompatible with the modular construction process. However, as the true contribution of the modules to the lateral resistance of the combined module-core system is unknown, the mechanical model employed for structural design is often based on the lateral stiffness of the core alone [20]. This simplification can lead to poor estimates of natural vibration characteristics (frequencies and mode shapes), with implications for dynamic response simulation and the tuning of vibration controllers.

The true damping ratio of a structure can only be obtained through full scale monitoring of the finished structure [10]. Hence, estimates in the design phase rely heavily on previous in situ monitoring of similar structures. Furthermore, choice of form, design and tuning of vibration controllers depend on a required level of auxiliary damping which can only be determined if the true damping ratio of a structure is known [11]. With no previous monitoring campaigns having been performed on modular structures, there exists no specific information on their damping

ratio, leaving an opportunity for gross error in any estimate used [3, 4, 5, 21].

With populations world-wide increasing and housing crises reported in a number of countries, including Ireland and the United Kingdom, rapid, economic and high-quality construction is crucial to ensuring infrastructure and housing requirements are met. Modular construction is primed to fill this role, reporting accelerated construction programmes, superior quality achieved in factory assembly, reduced cost as a result of economy of scale in production and lower environmental impact [2, 22, 23]. However, without substantial research, this new form of construction will suffer from a lack of knowledge about its inherent properties, global dynamic performance and suitable control methods, stunting application in high-rise structures.

1.2 Statement of Problem

Modular construction is becoming an increasingly popular method for the construction of high-rise, slender structures. The Avenue South Residences located in Singapore are 56-storey, 196m tall, residential twin towers built using concrete modules with a shear wall system [24]. Two of the tallest modular buildings using composite steel and concrete volumetric modules are located in Croydon, London, United Kingdom and are 135m and 163m tall respectively [6, 25]. With modular construction continuing to push to new heights, a better understanding of how this form of construction behaves dynamically under uncertain wind loading is imperative to safer, more informed and innovative design.

Modern methods of design and prediction of structural response including structural design codes, standards and software are based on traditional forms of construction. Often, these methods are not fit for purpose in the design of modular structures leading to inaccurate prediction of inherent parameters such as the natural frequency, damping ratio and stiffness contributions of individual elements [4, 5]. This in turn hinders the assessment of habitability requirements and the design of suitable control methods. The lack of research and understanding of volumetric composite steel and concrete modules is a barrier to the development of design codes and modelling software specific to the unique form of modular buildings [4, 22, 26, 27]. Hence, there is a need for in situ testing, modal identification of dynamic inherent properties and suitable modelling techniques to investigate the intrinsic

properties of modular buildings and to enable the application of suitable response control methods.

1.3 Research Aims and Objectives

The overall aim of this research is to perform an initial holistic assessment of the inherent properties of composite volumetric steel and concrete modular buildings and to develop data-informed numerical models for the assessment of wind-induced dynamic performance and suitable control methods to satisfy occupant comfort criteria for a new generation of tall, high-rise modular buildings. The specific objectives that will be fulfilled to meet this aim are to:

1. Identify inherent properties of tall modular buildings by applying modal identification methods to the measured in situ ambient acceleration responses of two case study structures;
2. Determine the lateral stiffness contribution of composite steel and concrete modules to the global stiffness of modular buildings with RC cores through the development of surrogate models calibrated using in situ monitoring results;
3. Develop an improved data-informed structural model for modular tower systems that includes the mechanical properties of modules and core;
4. Incorporate a Tuned Liquid Damper into the data-informed model and validate using laboratory tests of a full-scale prototype TLD;
5. Investigate the optimum vertical distribution of TLDs to maximise wind-induced vibration response reduction through a parametric study of a modular building.
6. Experimentally evaluate the ability of multiple TLDs to control the wind-induced response of modular buildings by performing Real Time Hybrid Testing (RTHT) using a numerical data-informed numerical model and a physical prototype TLD.

1.4 Thesis Outline

Below, a brief description of the layout of the Thesis is presented. In total, there are five main chapters. An overview of each chapter is provided.

Chapter 2 presents a broad discussion of the background theory and information required to understand the topics researched by this thesis. The structural form of modular construction is introduced along with the main advantages and challenges facing its wider adoption. Occupant Comfort Criteria and typical damping ratio values of different forms of construction are discussed. Different methods of Operational Modal Analysis (OMA) are presented as are methods for mitigating wind-induced vibrations. Finally, a brief description of the concept of Real Time Hybrid Testing (RTHT) is provided.

Chapter 3 describes an initial parametric assessment, which was performed using EC 1991-1-4, of the acceleration responses of 6,125 archetype high-rise modular buildings to assess which of two methods are more effective in reducing the wind-induced vibrations of tall modular buildings, namely changing the dimensions of the RC cores or including supplementary damping by way of a Tuned Liquid Damper (TLD).

Chapter 4 introduces the two case study structures which have been studied as part of this thesis; *Ten Degrees* and *College Road*. The structural design of the two high-rise modular buildings is discussed, along with the FE models which were developed for their analysis and the results from wind-tunnel tests which were carried out prior to their construction.

Chapter 5 focuses on the analysis of the measured acceleration responses of the two case study structures obtained from in situ monitoring of the full-scale buildings. The in-situ monitoring of the two buildings is described, the OMA techniques used are evaluated and the identified natural frequencies and damping ratios of the two buildings are identified.

Chapter 6 considers the identified natural frequencies at different stages of construction and their use in identifying the global bending stiffness of the two case-study structures. A stepped-beam numerical model is introduced and used alongside the measured natural frequencies to obtain estimates of the bending stiff-

ness. These estimates, alongside the identified damping ratios and mass values are used to develop a data-informed MDOF surrogate model for the dynamic response of modular buildings of various heights and dimensions. The data-informed MDOF surrogate model is calibrated using the results from Chapter 5.

Chapter 7 builds upon the findings of Chapter 4 and the data-informed MDOF surrogate model developed in Chapter 6 by including the effects of a TLD in the numerical model. Firstly, the modelling of TLD behaviour is discussed, including the equivalent TMD model which is incorporated into the MDOF surrogate model. The equations which are used to model the TLD as an equivalent TMD are provided, alongside the equations used to include MVDTLDS in the MDOF surrogate model. Validation of the equivalent TMD model of a TLD within the MDOF surrogate model is carried out using laboratory tests of a prototype full-scale TLD. A parametric assessment of MVDTLDS is then performed considering a 50 storey modular building and different configurations of MVDTLDS. The effectiveness of the MVDTLDS in controlling the wind-induced response of the building is evaluated using performance criteria. Finally, fragility curves are developed to show the ability of MVDTLDS to overcome uncertainty in the inherent properties of modular structures.

Chapter 8 presents the background, methodology and results of Real Time Hybrid Testing (RHT) carried out to combine the numerical MDOF surrogate model with a physical full-scale TLD; thereby combining the results from physical full-scale monitoring of modular buildings with physical tests of a TLD. The experimental results from the RHT are compared to numerical results from the MDOF surrogate model with TLDs and their differences discussed.

Chapter 9 summarises the work completed within this thesis. The main findings and identified areas for further research are outlined.

Chapter 2

Literature Review

2.1 Introduction

This chapter aims to provide an overview of the basic theory and the current published literature relating to the topics considered by this thesis. In order to give preface to the purpose of this research, the principle of modular construction is described in detail along with associated current state of the art concepts and impediments to its advancement. A brief summary of relevant modelling techniques for dynamic analysis is provided. This description includes the impact of uncontrolled wind-induced accelerations on the comfort of occupants and associated Serviceability Limit State (SLS) guidelines. The focus of this chapter then shifts to in-situ monitoring of structural systems and modal identification techniques. Following on from this, vibration control strategies are discussed to identify the different control methods which are suitable for modular buildings. Finally, the concept of real time hybrid testing and its application in the testing of Tuned Liquid Dampers (TLDs) is presented.

2.2 Modular Construction

Modular construction is a relatively modern concept employing the use of off-site manufacturing of building components under controlled factory conditions which are then transported to site and assembled to create finished structures [2]. Figure 2.1 presents a high level representation of the manufacturing and installation process

for the case of volumetric modular construction. Modular construction has received increased interest in recent years due to the many reported benefits over traditional construction methods including reduced environmental impact, improved quality and accuracy, and speed of construction [4, 19, 28, 29]. The form, advantages and challenges facing modular construction are discussed in more detail below.

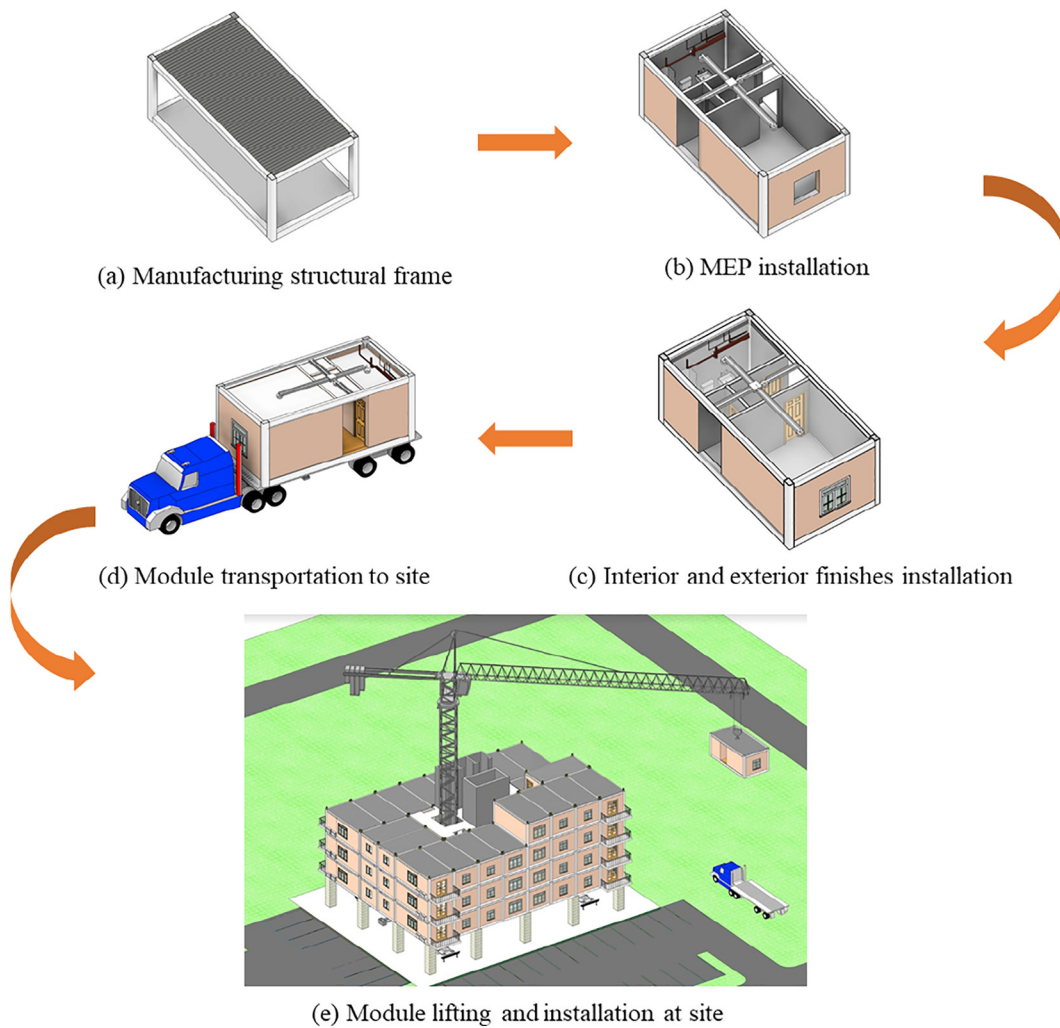


Figure 2.1: The manufacturing and installation process involved in volumetric modular construction (Source: Liew et al. [30])

2.2.1 Form of Modular Construction

Modular construction is a catch all term for the off-site manufacturing of repeatable sections of a construction project and is considered a Modern Method of Construction (MMC). Sections of the completed building which are manufactured off site primarily fall into three categories: elemental, planar and volumetric [5, 31]. Figure 2.2 demonstrates the three different forms. Elemental projects utilise off-site construction of one-dimensional (1D) elements such as beams or columns. Planar projects include two dimensional (2D) diaphragm elements manufactured off-site such as wall panels or floor slabs. Volumetric projects have proven to be superior to 1D and 2D projects and involve the off-site pre-fabrication of entire three dimensional (3D) volumetric pods or room sized units [2, 4, 5].

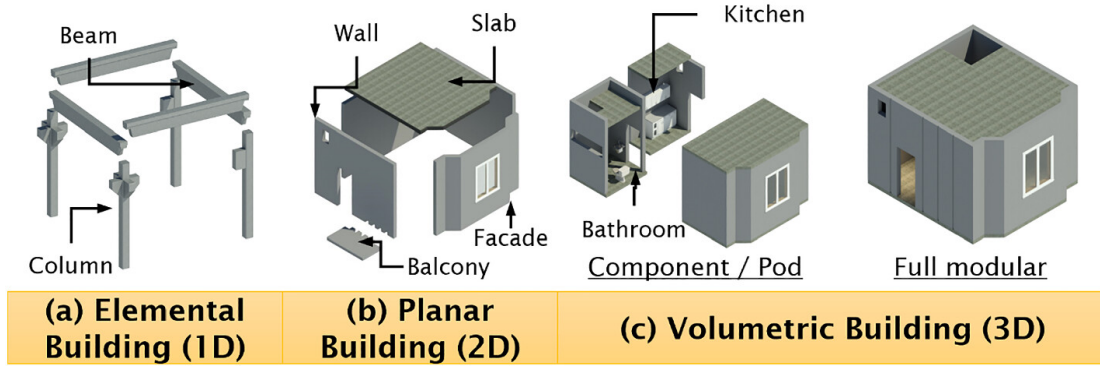


Figure 2.2: Example of elemental, planar and volumetric modules (Source: Lim et al. [31])

In 2019, a cross industry working group established by the Ministry of Housing, Communities & Local Government in the United Kingdom established a framework to define Modern Methods of Construction [23]. The aim of this framework is to establish regularised terminology with regards to innovative construction methods used in home building. The framework defines seven categories spanning off-site manufacturing, site based construction and materials and innovative processes. The seven categories are outlined in Figure 2.3. Of these categories, volumetric modular construction falls into category one if load bearing and category five if non-load bearing, planar modular construction forms part of category two and elemental modular construction is considered category three. The innovative construction methods considered within this thesis relate to category one of this framework.

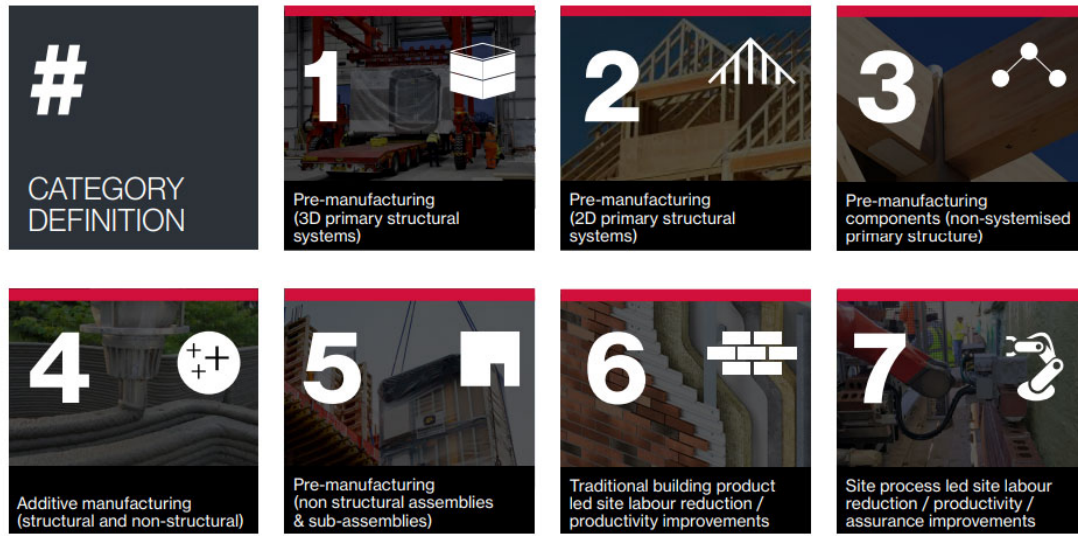


Figure 2.3: Modern Methods of Construction definition framework (Source: Housing, Communities & Local Government Joint Industry Working Group [23])

Volumetric modules come in many forms, including modules with load bearing walls, modules which carry load by supports at the corners of each module, and non-load bearing ‘pods’ used for highly serviced and repeatable units such as kitchens and bathrooms [21, 22, 32]. Often volumetric modules include a concrete floor slab which act to transfer lateral loads to a lateral stability system such as a concrete core. The size of volumetric modules is often limited by transport and lifting restrictions [19, 31, 33]. Materials used in modular construction are similar to those used in more traditional forms of construction and include timber, steel, concrete and composite sections, with steel and timber modules being more lightweight than concrete modules [4]. This research focuses on 3D volumetric modules, in particular corner supported steel frame modules with concrete floor slabs as shown in Figure 2.4.

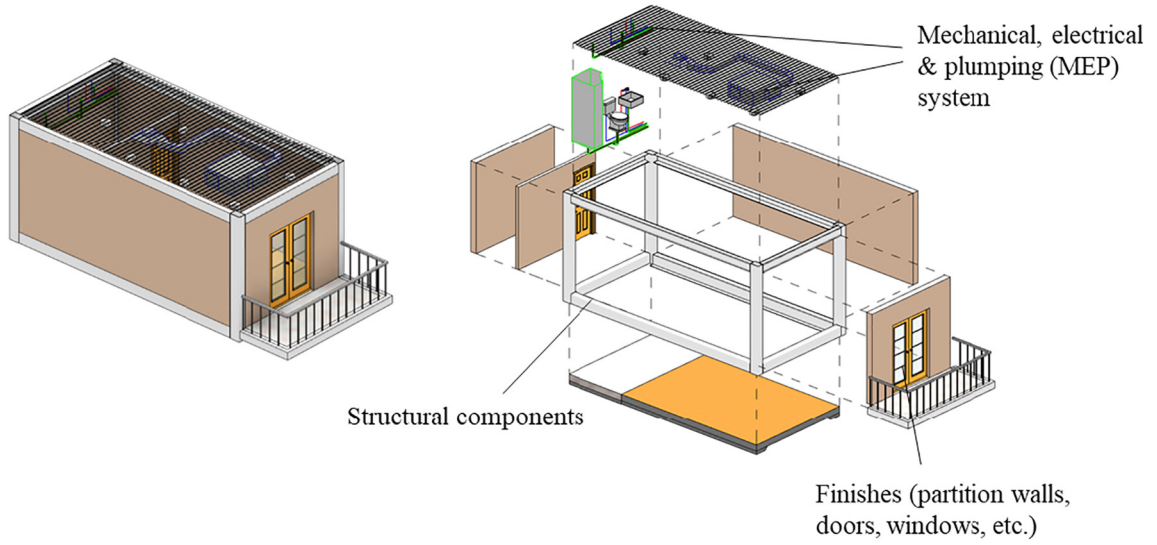


Figure 2.4: Example of a corner supported volumetric module (Source: Liew et al. [30])

2.2.2 Advantages of Modular Construction

Modular construction has many reported advantages over traditional forms of construction. Studies have considered the sustainability of modular construction in comparison with traditional forms of construction and found positive results. Cao et al. [34] found modular construction to reduce resource depletion by 35.82%, health damage by 6.61%, ecosystem damage by 3.47%, timber consumption by 70.93% and water consumption by 22.48% when compared to traditional construction. Similar claims are made by Lawson et al. [29], finding that modular construction can improve the sustainability of structures through both the construction process and the building performance. Among these improvements are reported reductions in construction waste of up to 15%, reduction in delivery vehicles to site of up to 70%, increased air tightness of the finished structure due to the more stringent tolerances achieved in a factory environment, more efficient design and use of light-weight materials with lower embodied energy and improved safety on-site with accidents reduced by over 80% due to less intensive on-site construction.

A study conducted by Forman et al. [35] on behalf of Vision Modular Systems UK Ltd. into the whole life embodied carbon of two of their recently completed residential modular buildings, *The Valentine*, Redbridge, UK and *Ten Degrees*, Croydon, UK, found that the buildings compared well to benchmarks for carbon targets set out by the Royal Institute of British Architects (RIBA) and the London Energy

Transformation Initiative (LETI). It is reported that the values for *The Valentine* and *Ten Degrees* were 40% and 54% lower than the RIBA ‘business as usual’ benchmark and LETI Band E benchmark respectively. This demonstrates the ability of volumetric modular construction to enable the supply of much needed infrastructure whilst meeting sustainability targets.

Whilst modular construction offers many environmental advantages, its primary benefits are found in the reported economic advantages [4]. The economic benefits boasted by modular construction are well documented. Lawson et al. [29] reports that although off-site manufacturing requires an increased investment in the infrastructure required and a regular flow of repeatable units to achieve economies of scale in production, factory processes are more efficient, productive and faster than on-site construction. Liew et al. [30] claims that pioneering projects in Singapore including The Crowne Plaza Hotel Extension, Changi Airport, and NTU North Hill Residence Hall, saw time savings of 6 months and labour savings of 40%. Gunawardena et al. [19] describes how modular construction enabled rapid construction of the Little Hero apartment building in Melbourne, Australia despite narrow access roads. The economic benefits and improved efficiency of modular construction are one of the key drivers behind government incentives encouraging the wider use of modular construction [4]. In 2004, the UK government began an initiative to have 25% of all new social housing constructed using off-site manufacturing [28, 4]. Meanwhile, in Australia it is predicted that by 2030, 5 - 10% of all construction will involve modular construction techniques [4]. Generalova et al. [3] report that modular construction can help to provide affordable housing to relieve housing crises which have been aggravated by rapid urbanisation according to a number of UN studies. Similarly, in Hong-Kong, Singapore and the UK modular construction has been identified by government bodies as a sustainable and economic solution to heightening housing crises [5, 22, 29, 36].

2.2.3 Challenges facing Modular Construction

There are a number of challenges facing more widespread application of modular construction technology throughout the construction industry. These challenges include economic limitations, lack of standardisation and a lack of understanding of the global behaviour of modular buildings owing to their novelty.

Economic limitations have been described by a number of studies. Pan et al. [22] surveyed a number of stakeholders in modular construction projects in Hong Kong to identify the primary constraints. The survey participants included clients, developers, contractors, suppliers and manufacturers, consultants, government institutions and senior university based researchers with the majority stakeholders having more than 10 years of experience. Stakeholders were asked a number of questions relating to economic, technology, supply chain and regulatory constraints. The most significant challenges identified by stakeholders were stringent regulations, limited codes and standards, logistic challenges and limited capable suppliers. Ferdous et al. [5] similarly highlighted economic constraints relating to a lack of experience and expertise in modular construction and the intense initial capital required to establish off-site manufacturing facilities. Despite these identified constraints, the economic argument for modular construction remains strong with benefits such as achieving economies of scale, reduced material requirements and wastage, savings in labour costs and reduced construction timelines [5, 28, 37, 38] with many studies considering minimisation of economic barriers to modular construction [3, 26, 39, 40, 41].

Studies have found that the lack of design codes and standards specific to modular construction are hindering its wider adaptation [4, 5, 22, 31, 38, 41]. Currently, in many countries the design of modular buildings must be completed to design codes and standards developed for traditional steel and concrete design [29]. These codes do not take into account the unique connections, robustness checks or fire safety requirements specific to modular buildings [5]. However, progress is being made, for example, The Building and Construction Authority of Singapore has published a guidebook on design, prefabrication and installation in modular construction [5, 15]. A guidebook was also published in Australia which mentions the Australian design code and the Eurocodes [15]. Furthermore, in the UK, Lawson et al. [2] has outlined extensive design guidelines specific to modular buildings considering the Eurocodes. The continuing development of design guides and standards for modular construction is necessary to enable further application of this form of construction.

One of the most significant challenges facing modular construction is that the global behaviour of modular buildings is little understood [4, 19, 21, 40, 42]. Primary concerns about modular buildings with respect to their global behaviour

include fire safety, disproportionate collapse, stiffness of connections and dynamic behaviour under seismic and wind loading [28, 40, 43]. Fire resistance is a major challenge for modular buildings, with the space between modules offering fire a chance to spread rapidly [40]. However, Lawson et al. [29] showed that through providing adequate fire protection to internal walls and the ceilings of modules, and including fire barriers between modules, the rate at which fire spread could be reduced. Furthermore, including concrete slabs or other composite materials in modules improves the fire resistance and can help to ensure that fire requirements are met [28, 32, 44].

Multiple studies have focused on modelling disproportionate collapse of modular buildings and found that whilst they do require a more detailed design process and more intense analysis than traditional structures, modular buildings meet design standards for robustness and perform well when disproportionate collapse is simulated [5, 45, 46, 47]. In particular the structural design of modular buildings must consider alternative load paths, intra-module connection design and the capacity of corner supporting columns due to load redistribution. Whilst many studies note the need for more research into the robustness of modular buildings [5, 21, 46] the ability of this form of construction to comply with building regulations and to withstand disproportionate collapse has been shown [45, 46, 48, 49].

Connections play a vital role in the structural stability and global behaviour of modular buildings [4, 40]. As such, lateral stiffness has been the primary focus of much of the research conducted on modular buildings [5]. Connections in modular construction fall into three categories (a) intra-module connections, (b) inter-module connections and (c) connections of modules to lateral stability elements such as a concrete core [2, 50]. Figure 2.5 highlights the difference between intra- and inter-module connections. Intra-module connections refer to the connections within the individual module elements and are similar to those used in traditional forms of construction with members welded to a high tolerance level in a factory setting prior to transportation to site [40]. Examples of typical intra-module connections are shown in Figure 2.6. Inter-module connections connect individual modules to one another and create diaphragms within the completed building [4, 29].

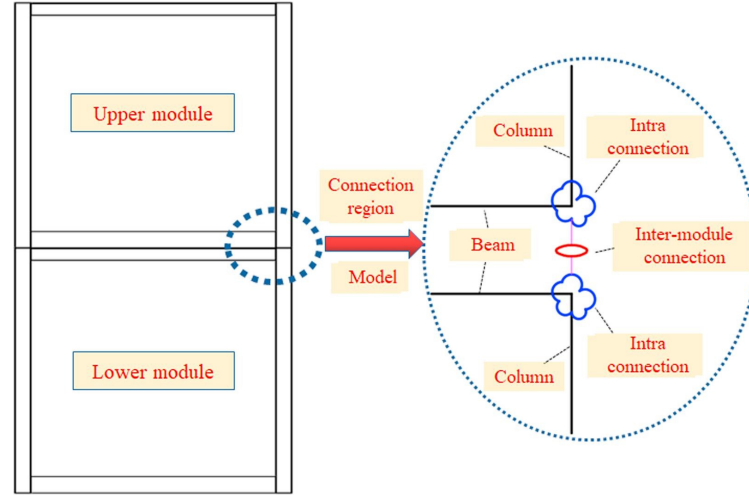


Figure 2.5: Intra- and inter-module connections in modular construction (Source: Chen et al. [21])

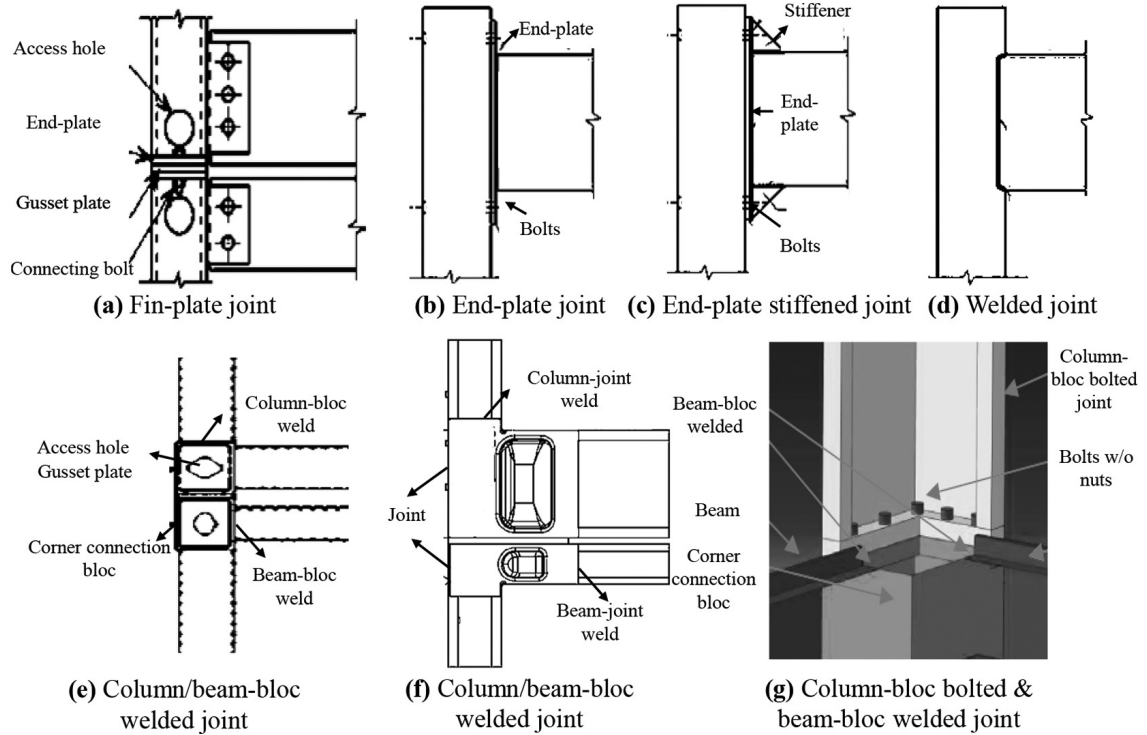


Figure 2.6: Various types of intra-module connections used in volumetric modular construction (Source: Chen et al. [21])

Inter-module connections are unique to modular construction and must be designed to transfer axial, lateral and torsional forces within a completed structure and therefore must have adequate axial, shear and bending stiffness [18, 51, 52]. Consequently, many studies have focused on the robustness, strength and stability of inter-module connections [17, 43, 51, 52, 53, 54, 55, 56], with many varieties of connection identified. Some examples of typical inter-module connections between module columns are provided in Figure 2.7 and between module beams in Figure 2.8.

Srisangeerthanan et al. [52] carried out a review on the performance requirements of inter-module connections and developed theoretical models to predict the axial, shear and bending stiffness of bolted connections. Conclusions were drawn which state that any form of inter-module connection can be designed to meet structural demands. However, this study did highlight that industry-wide standardisation of modular connections has not been achieved and that a standardised approach to modular construction would revolutionise the industry. Other studies also highlighted the lack of standardisation of inter-module connections as a challenge to the development of modular construction [4, 5, 22].

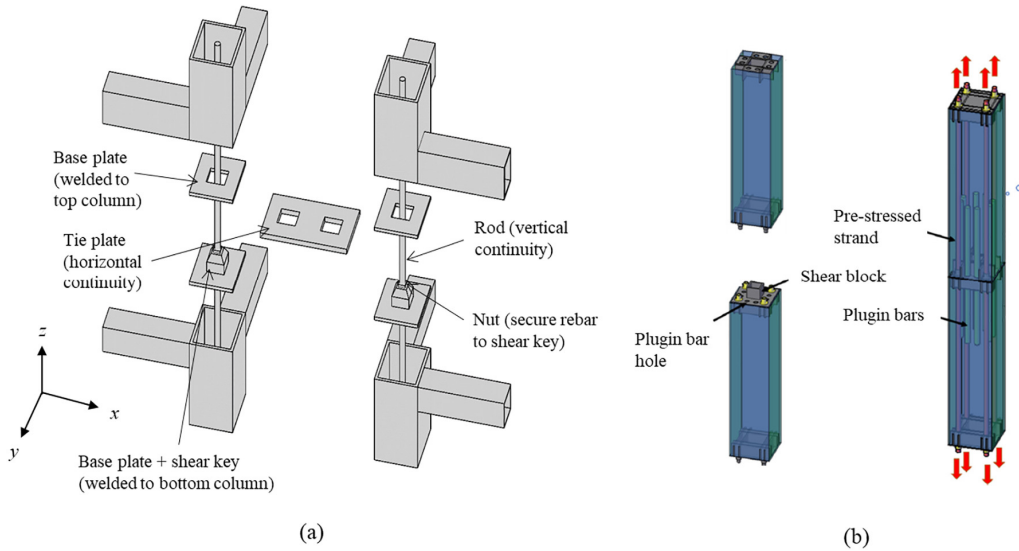


Figure 2.7: Examples of inter-module connections between vertical columns used in volumetric modular construction: (a) vertical rod system [30] and (b) prestressed strands [57] (Source: Liew et al. [30])

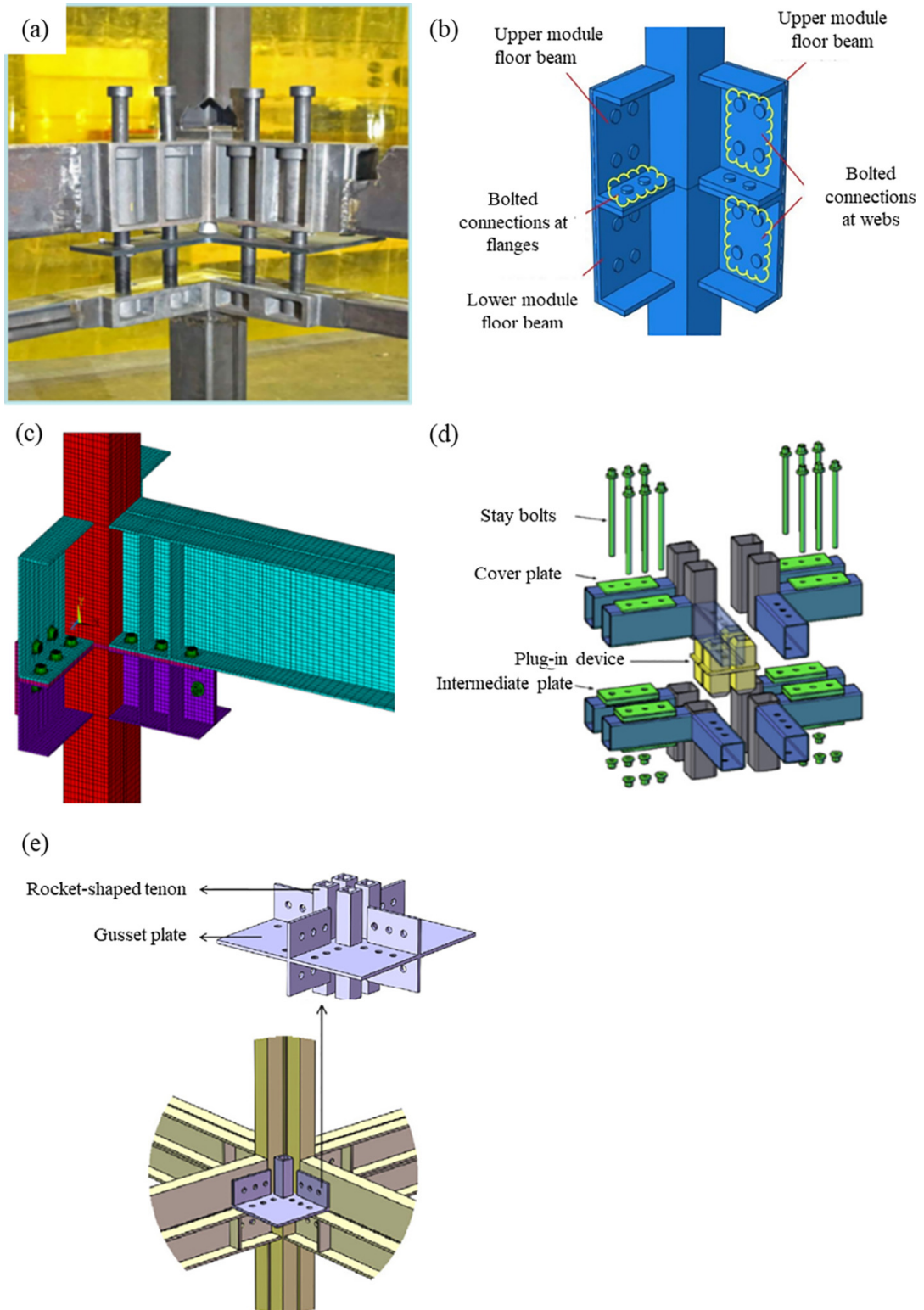
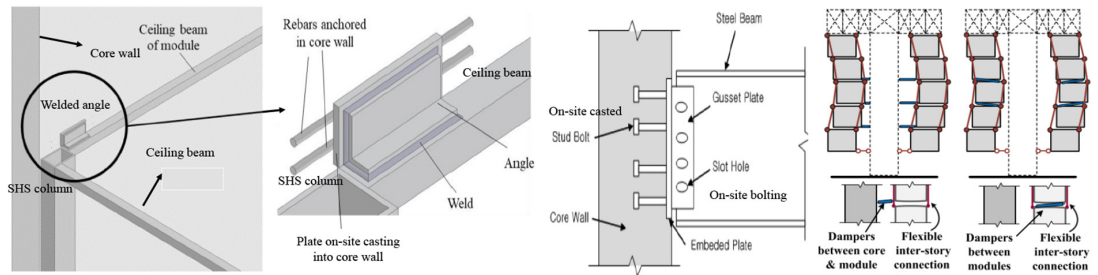


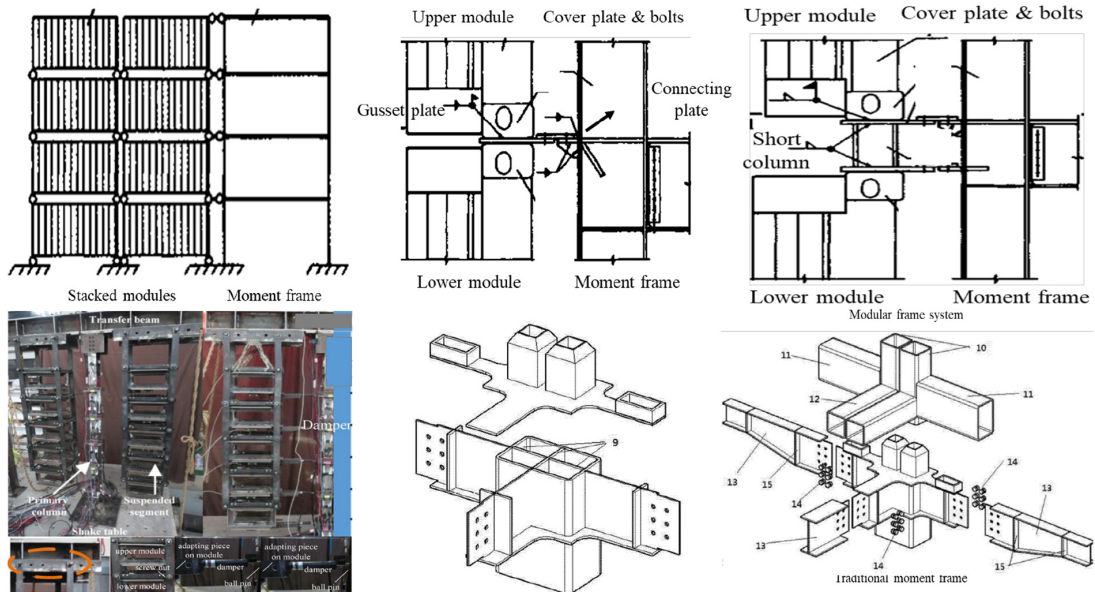
Figure 2.8: Examples of inter-module connections between adjacent beams and columns used in volumetric modular construction: (a) Vectrobloc system [58] (b) corner-joint [59] (c) perimeter joint [60] (d) and (e) interior joint [61, 62] (Source: Liew et al. [30])

The third type of connection in modular construction is that between the modular and non-modular elements, such as concrete cores used for lateral stability. However, some systems have been developed which do not rely on a concrete core for lateral stability. Gunawardena et al. [19] presented a design which uses a purely modular system consisting of bespoke connections and stiff concrete walls to resist lateral loads. This system was found to be adequate for low-medium rise modular buildings. Sharafi et al. [55] proposed an integral interlocking connection which provided ties using fasteners at the corners of steel modules to resist lateral loads and found that this system satisfactorily resisted lateral loads for a three storey structure. Wang et al. [63] also used precast concrete walls within modules to resist lateral loads and tested this system by modelling a 40 storey modular building. The drift of the concrete walls under wind loading was less than the allowable limit set out by Hong Kong building codes [63]. Despite the reported success of such systems, it is widely accepted that for medium-high rise modular construction the inclusion of a concrete core is the most suitable method for resisting lateral forces from wind and earthquakes [2, 5, 32, 43]. Examples of connections between the modules and the lateral stability elements of a volumetric modular buildings are shown in Figure 2.9.

It is evident from Figures 2.8, 2.7 and 2.9 that there are a wide variety of connection systems which can be employed in volumetric modular construction. The variation and complexity of these connections results in difficulty modelling their local behaviour in detail. Given the importance of connections in the performance of a modular building, modelling the behaviour and stiffness of the different connection systems has been the focus of much research [17, 21, 51, 52, 53, 54, 55, 61, 64] and is discussed further in Sections 2.3.1, 2.3.2 and 2.3.3.



a) Stacked modules-core horizontal connection



b) Stacked modules-moment frame horizontal connection

Figure 2.9: Examples of connections between modules and lateral stability systems used in volumetric modular construction (Source: Chen et al. [21])

2.3 High-Rise Modular Construction

As discussed, the benefits of modular construction offer many advantages over more traditional forms of construction and it presents an opportunity for quicker, more efficient and sustainable construction of homes and infrastructure to aid in the problems faced by many rapidly growing communities. In high density cities, high-rise construction can maximise occupiable space while minimising building footprints [22, 39, 41]. Therefore, the research and development of modular construction technology to enable this form of construction to be used in high-rise buildings is crucial.

Traditionally volumetric modular construction has been primarily used in low-rise repeatable structures such as military accommodation, student residences and hospitals due to the repetitive nature and size of the modules [2, 5, 22, 29]. It is only in more recent years that modular construction has been used in medium and high-rise projects with heights of over 200 m now realised [4, 5, 25, 39]. Two of the world's tallest volumetric corner supported modular buildings are *Ten Degrees* and *College Road*, both located in Croydon, London, United Kingdom and standing at 135 m and 163 m respectively [25, 65]. The focus of this research is volumetric corner supported modular buildings, and as such both *Ten Degrees* and *College Road* have formed the basis of a large portion of the research conducted. However, it is worth noting other high-rise modular buildings which have been constructed and the type of prefabrication which was utilised in each case. Table 2.1 shows ten of the world's tallest modular buildings and the type and material used for the prefabricated elements.

Applicability of modular construction to high-rise buildings has been limited by insufficient research into the application of this novel form of construction [21, 40]. As a result of this only 1% of high-rise buildings worldwide are modular buildings [5, 66]. Limitations which become more acute when modular construction technology is employed in high-rise structures include the absence of validated modelling techniques, a lack of understanding of the stiffness contribution of modules to global stiffness, limited research on global dynamic behaviour and methods for ensuring occupants comfort and safety during dynamic loading such as during wind and seismic events [4, 5, 21, 32, 36, 38, 40, 42, 63, 67].

Table 2.1: Ten of the tallest modular buildings in the world [5, 25, 24]

Project	Height	Storey	Year Completed	Location	Modular Type	Module Material
J57 Mini Sky City Tower	204m	57	2015	Changsha, China	2D panel	Steel
Avenue South Residence	192m	56	2023	Singapore	3D volumetric	Concrete
Collins House	190m	60	2019	Melbourne, Australia	2D panel and 3D volumetric	Concrete
College Road	163m	49	2023	London, UK	3D volumetric	Steel and Concrete
Clement Canopy	140m	40	2019	Singapore	3D volumetric	Concrete
Ten Degrees	135m	44	2020	London, UK	3D volumetric	Steel and Concrete
La Trobe Tower	133m	44	2016	Melbourne, Australia	2D panel and 3D volumetric	Concrete
Atira La Trobe Street	131m	44	2018	Melbourne, Australia	2D panel and 3D volumetric	Concrete
T30 Tower	100m	30	2011	Xiangyin, China	2D panel	Steel

2.3.1 Limitations in Modelling Modular Buildings

There are currently no commercial software available specifically for the modelling of modular buildings [5, 32, 40]. Studies which have focused on the modelling of modular buildings have mostly employed simplified models, particularly of intra-module connections in global models [17]. It can be computationally expensive to model both connections and global behaviour of modular buildings in one model and it is more suitable to model connections separately and use a 1D link or spring element in the global model to represent the connection stiffness [5, 40]. Chua et al. [17] considered the ability of simplified connection models proposed by other researchers to capture the transfer of lateral forces through inter-module connections and connections to lateral stability elements. A 40 storey archetype building was modelled using translational spring models to simulate the behaviour of the connection. This mechanical behaviour was then compared to a simplified joint model. Chua et al. [17] reports that the spring model was more appropriate for capturing global behaviour of a modular building, however, it is noted that experimental studies are still required. Lacey et al. [53] compared theoretical models of inter-module connections with the limited available experimental results from testing of this type of connection. The study considered the Force-Displacement and Moment-Rotation behaviour of three different inter-module connections and found that experimental results for shear stiffness, tensile stiffness and moment rotation of the connections were not accurately captured by current simplified models. Lacey et al. [53] reiterates the need for further experimental testing and improved models for modular buildings, particularly under combined actions.

Shan et al. [36] performed a case study of a 31 storey modular building in Hong Kong using piece-wise Finite Element (FE) analysis. To do this, connections were modelled in the FE software ABAQUS so as to identify local behaviour and justify simplified link elements being used in a 3D numerical model of the full structure in ETABs. Analysing the connections separately enabled simplifications that greatly reduced the computational expense which is often more significant for modular buildings [36]. However, it is acknowledged that these results have not been validated experimentally. Further to the computational expense of the intricate connections typical in modular buildings, duplication of elements occurs when modules are stacked, resulting in doubling up of bracing members and regular instances of four columns or four beams needing to be modelled in place of what would be a

single element in traditional buildings as shown in Figure 2.10. This duplication of elements significantly increases the model size and increases computational time [5, 21]. The lack of a modular specific commercial software which captures global dynamic behaviour is one challenge impeding the further development of high-rise modular construction [5, 32].

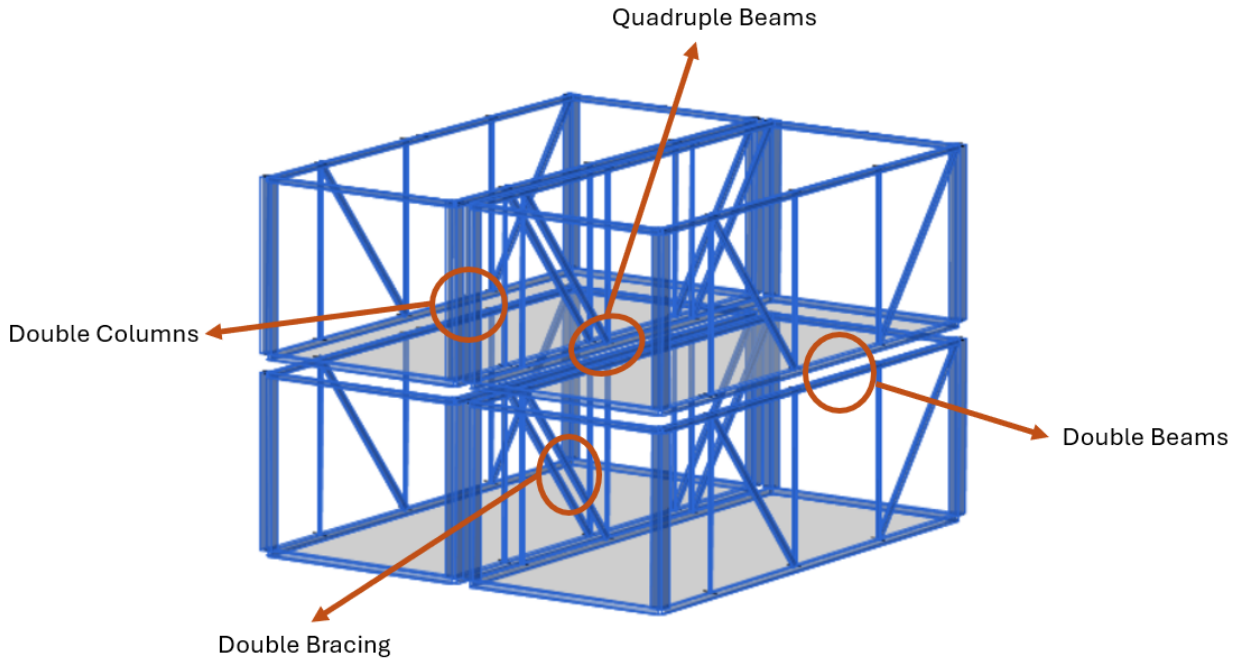


Figure 2.10: Example of duplicate members which occur when modules are stacked together

2.3.2 Stiffness Contribution of Modules to the Completed Structure

In modular construction, the stiffness contribution of individual modules to the overall global stiffness of the finished structure is little understood [32, 40, 63, 66]. Of the ten modular structures listed in Table 2.1, eight rely on concrete shear walls or a concrete core for lateral stability. The remaining two, J57 Mini Sky City Tower and T30 Tower employ moment resisting frames with large steel cross-sections. Typically in high-rise modular construction modules are designed only to resist vertical loads and all lateral loading is transferred to the concrete core or lateral

stability systems [5, 17, 21, 40, 43]. Consequently, understanding the behaviour of the modules acting together to transfer lateral loads and the behaviour of inter-module connections becomes increasingly significant [5, 21, 42].

Chua et al. [17] studied connection stiffness and diaphragm discontinuity on the global sway behaviour of a number of different case study modular buildings. The first case study was a 10 storey modular building with no concrete core. The 10 storey structure was studied with and without bracing in the steel framed modules with concrete floor slabs. Additionally, a 40 storey modular building with a concrete core and no bracing in the steel modules was investigated. It was found by Chua et al. [17] that the global dynamic behaviour of modular buildings is not sensitive to the stiffness of inter-module connections, likely due to the stiffness contribution of the concrete core being more significant than the stiffness of the inter-module connections. Due to the rigidity of the concrete floor slabs of the modules and the presence of core walls, a semi-rigid diaphragm was deemed suitable for modelling the global behaviour of the structure rather than modelling each modules individual rigid diaphragm. Furthermore, Chua et al. [17] found that typical modelling methodology used in practice for modelling modular buildings was not conservative and failed to accurately capture the global behaviour of a modular building. Chen et al. [21] provides a comprehensive state-of-the-art review of the global response of modular buildings, providing a summary of recent developments in structural systems, connection techniques and overall structural response assessment. This study concludes that there has been no reliable link drawn to date between the behaviour of concrete cores and steel modules in high-rise modular construction and it is an area requiring significant research.

2.3.3 Global Dynamic Behaviour of Modular Buildings

The global dynamic response of any structure under wind or seismic loading is a result of the inherent dynamic properties of the structure [11]. The principal dynamic properties which influence the response are:

1. The mass of the structure.
2. The natural frequency of the structure.
3. The damping ratio of the structure.

Of these three properties, the first two can be predicted by FE models during the design process. The mass of a structure is simply the sum of the mass of all the individual components and is usually estimated with a high degree of accuracy [13]. The natural frequency of a structure can be calculated from FE models using the modal mass and modal stiffness of the structure. However, it can be difficult to accurately predict the stiffness of a structure, particularly for modular buildings where there are a high number of connections and discrete diaphragms [17].

The damping ratio of a structure does not relate to a unique physical characteristic in the same manner that mass and stiffness do [68]. Therefore, estimates of damping during the design process of a structure rely solely on empirical results from in-situ monitoring of buildings constructed with similar materials [11]. However, to the best of the author's knowledge, there have been no previous in situ monitoring campaigns of modular buildings which have identified a damping ratio of the structure. From studying literature only one instance of in-situ monitoring of a high-rise volumetric modular building was identified. Bi et al. [69] performed in-situ monitoring of an 18-storey high-rise volumetric modular building to identify the natural frequency of the modular building and verify a FE model. No damping ratio was reported.

The lack of full-scale studies on the dynamic behaviour of modular structures is a major challenge to widespread adoption of modular construction as a viable option for high-rise buildings [5, 21, 32, 36, 40]. Without better understanding of the dynamic response of modular buildings, it is difficult to design modular high-rise buildings which satisfy serviceability limit state design requirements and ensure the comfort of occupants. Moreover, the design and inclusion of supplemental damping devices to control excessive vibrations becomes uncertain.

2.4 Damping Values in Structures

As mentioned previously, there is no theoretical calculation process which can be performed to determine damping values for an individual structure [10, 70]. Damping estimation relies on empirical data from previous full scale monitoring of buildings with many design codes offering values based on type of construction and building materials [68, 70]. For this reason, despite advances in technology allowing for better modelling and scaled wind tunnel tests, the only accurate method for

obtaining the damping ratio of a structure is from full scale monitoring of the completed structure [13, 71, 72]. To the best of the author's knowledge, damping ratios of tall modular buildings have not been previously reported in literature, meaning designers are forced to use estimates based on other forms of construction. For example, the European standard design code, Eurocode 1991-1-4 [73], recommends a value of 0.08 for the logarithmic decrement of damping in the first mode of vibration, equivalent to 1.27% ratio of actual damping to critical damping, for mixed concrete and steel structures. For comparison, the Eurocode 1991-1-4 recommended values for critical damping in the first mode of vibration for steel and concrete buildings are 0.8% and 1.59% respectively. Furthermore, the American standard design code, ASCE 7-10 [74], recommends a critical damping ratio value of 1% for steel structures and 1.5% for concrete structures in the first mode of vibration. Similarly, for structures more than 80m tall, in the first mode, the International Organisation for Standardisation, ISO 4354:2009 [75], recommends values of 1% for steel structures but recommends a lower value than Eurocode 1991-1-4 [73] and ASCE 7-10 [74] of 1.2% for concrete structures.

Tamura et al. [10] proposed the equations presented in Table 2.2 for estimating the critical damping ratio of office buildings in the first mode of vibration based on the Japanese Damping Database for structures with a maximum ratio of tip displacement (x_H) to height (H), $\frac{x_H}{H}$, of 2×10^{-5} .

Similarly, Smith et al. [11] proposed equations for the values of damping ratios derived from a database given by Satake et al. [12] which contains the intrinsic damping in the fundamental mode obtained from full scale monitoring of a number of tall buildings. These expressions are also presented in Table 2.2, where the predicted damping values in the first mode of vibration are compared for a 95 m tall building. It is apparent that the damping ratios estimated using both sets of equations for reinforced concrete are higher than those estimated for steel. However, the estimates calculated using equations developed by Tamura et al. [10] are quite similar for the two materials, whereas estimates calculated using equations developed by Smith et al. [11] differ by 0.21%. Furthermore, Smith et al. provides an equation for estimating the fundamental damping ratio of steel and concrete composite structures. For a 95 m high building, the damping ratio is calculated as 1.58% in the first mode; a higher value than for steel or concrete structures and higher than the Eurocode 1991-1-4 recommended value of 1.27% for composite buildings.

This suggests that structures which are constructed using both steel and reinforced concrete dissipate vibrations more effectively than steel or concrete buildings.

Table 2.2: Expressions for structural damping in the first mode of vibration reported in literature

Building Type	Expression	95m Building	Reference
Reinforced Concrete (10m < H < 100m)	$\zeta = \frac{0.75}{H} + 380\frac{x_H}{H} - 0.0014$	$\zeta = 1.41\%$	Tamura et. al [10]
Steel (30m < H < 200m)	$\zeta = \frac{0.52}{H} + 320\frac{x_H}{H} + 0.0023$	$\zeta = 1.40\%$	Tamura et. al [10]
Steel	$\zeta = 7.20H^{-0.3887}$	$\zeta = 1.23\%$	Smith et. al [11]
Steel and Reinforced Concrete	$\zeta = 21.2H^{0.5708}$	$\zeta = 1.58\%$	Smith et. al [11]
Reinforced Concrete	$\zeta = 28.25H^{-0.6393}$	$\zeta = 1.54\%$	Smith et. al [11]

Studies which have provided damping estimates from full scale in situ monitoring include Ohkuma et. al [76], which reported the full-scale monitoring of a 68m tall predominately steel structure and found the actual damping ratios of the structure to be 0.95% in the first along-wind mode. This value falls within the range of values recommended by Eurocode 1991-1-4, ASCE 7-10 and ISO 4354:2009 and the estimated values given by the equations in Table 2.2. Similarly, Gonzalez-Fernandez et. al [77] performed full-scale monitoring of a 150 m tall concrete structure and found the actual damping ratio to be 0.80% in the first mode, smaller than the aforementioned recommended values.

One of the largest in-situ monitoring campaigns was the Chicago Full-Scale Monitoring Programme which investigated the natural frequencies and damping ratios of three tall buildings in Chicago, Illinois, United States of America and compared these findings to results from FE models and wind tunnel tests of the three structures [13]. Data from the Chicago Full-Scale Monitoring Programme was analysed by Kijewski-Correa et al. [13] who used the Random Decrement Technique (RDT) to estimate the damping ratio of the in-situ structures. In total

three buildings were instrumented for full-scale monitoring, all over 60 stories tall. The first building comprised a steel tube system with exterior columns and stiffening elements and was found to have a damping ratio of 0.87% in the first mode. The second building was a reinforced concrete building with shear walls and was found to have a damping ratio of 1.42% in the first mode. The third building was a steel moment-connected, framed tubular system and was found to have a damping ratio of 1.04% in the first mode.

The results from the aforementioned full-scale monitoring campaigns of in-situ steel and concrete structures are summarised in Table 2.3. These values show large variability in the actual critical damping ratios displayed by full-scale structures. In many cases the damping ratios for the first mode recommended by Eurocode 1991-1-4 [73], ASCE 7-10 [74] and ISO 4354:2009 [75] are higher than the observed results from full-scale monitoring as described above. Similarly, there is variation in the predicted damping ratios using equations derived by Tamura et al. [10] and Smith et al. [11]. Broadly speaking, from the literature studied, the damping ratios of reinforced concrete structures ranges from 0.8 - 1.6% and the damping ratios of steel structures ranges from 0.8 - 1.4 % in the first mode of vibration.

The values presented in Table 2.3 show that there is significant uncertainty in the damping ratio of structures and whilst design codes and equations provide estimates of critical damping ratios based on empirical data, the variability between these estimates is evident. Given the novelty of modular construction for high-rise buildings, it is important to investigate the damping ratio of this new form of construction. There are a much larger number of movement joints in modular structures than in conventional steel or concrete buildings. Therefore, it is not unreasonable to hypothesise that there is much greater potential for frictional energy dissipation and modular structures may have higher damping ratios than their more conventional counterparts [5, 21, 55]. However, in situ monitoring of modular buildings will be required to test this hypothesis.

2.5. Occupant Comfort in High-Rise Structures

Table 2.3: Damping ratio estimates in the first mode of vibration from full scale in situ monitoring of structures

Study	Building Type	Height	Damping Ratio
Ohkuma et. al [76]	Steel	68m	0.95%
Gonzalez-Fernandez et. al [77]	Reinforced Concrete	150 m	0.80%
Kijewski-Correa et al. [13]	Steel	Not Publicly Available	0.87%
Kijewski-Correa et al. [13]	Reinforced Concrete	Not Publicly Available	1.42%
Kijewski-Correa et al. [13]	Steel	Not Publicly Available	1.04%

2.5 Occupant Comfort in High-Rise Structures

All buildings are subject to environmental loads such as wind loading and seismic events. As such, they will experience a resulting dynamic response, with horizontal accelerations and displacements. The dynamic responses of structures under lateral loading increases with structural height and flexibility [78]. In high-rise structures, it is not uncommon for dynamic responses to cause issues with the serviceability of the structure and give rise to habitability issues associated with motion sickness in occupants [8, 79]. A number of studies have considered the impact of wind-induced motion on occupants and at what level vibrations become perceptible to humans [8, 14, 79].

Lamb et al. [14] surveyed 1014 central business district workers in Wellington, New Zealand to capture experiences of wind-induced building motion, susceptibility to motion sickness, reported compensatory behaviours, and complaints about building motion. Lamb et al. [14] reported that over 40% of respondents claimed to have felt wind-induced motion, 83% of these respondents had felt this motion in an office building and the results showed that motion perception was more common with

increasing storey level. Respondents on storey 20 or above were four times more likely to report perceiving motion than those on storeys 10 - 19. Twenty percent of all respondents reported experiencing wind-induced motion on a monthly basis. The most reported symptom was difficulty concentrating, followed by feeling dizzy. The most popular compensatory behaviours were standing up and walking around, and leaving the building. Symptoms of motion sickness such as nausea and difficulty concentrating reduce work performance and compensatory behaviours such as taking more breaks reduces productive time. Therefore, whilst wind-induced motion is not likely to cause issues in the safety of a structure designed appropriately, the significance of its impact on the occupants' comfort and usability of a structure is evident.

Kwok et al [8] provides a detailed summary of various studies that have considered human perception of vibration and tolerance thresholds for wind-induced tall building vibrations. The studies reviewed cover a broad range of methodologies including field experiments and surveys of building occupants conducted in wind-excited tall buildings; motion simulator and shake table experiments testing human participants; and field experiments conducted in artificially excited buildings. The data collected by these numerous studies primarily cover structures in the 0.1 - 1 Hz frequency range, a range where structures are more susceptible to wind-induced motions. Whilst Kwok et al. [8] highlights that due to the subjectivity of tolerance to vibrations, there are no universally accepted design criteria for assessing occupant comfort or serviceability criteria for acceptable limits on wind-induced vibrations in tall buildings. However, most studies have identified frequency dependent criteria for assessing acceptable limits on accelerations [8, 79, 80].

Various codes and standards offer guidance on acceptable levels of wind induced acceleration. The first code to do so was the 1975 National Building Code of Canada (NBCC) [81], while The International Standards Organization (ISO) (1984) [82] and the Architectural Institute of Japan (AIJ) (1991) [83] also produced guidelines. ISO 10137 (2007) [84] is an updated version of the ISO 6897 (1984) and the most widely applied standard in Europe. The acceleration limits suggested by ISO 10137 are shown in Figure 2.11. This standard imposes limits on peak acceleration for a 1 in 1 year wind event for both residential and commercial buildings. These criteria are based on accelerations which are disruptive to approximately 2% of the people occupying the upper third of the building.

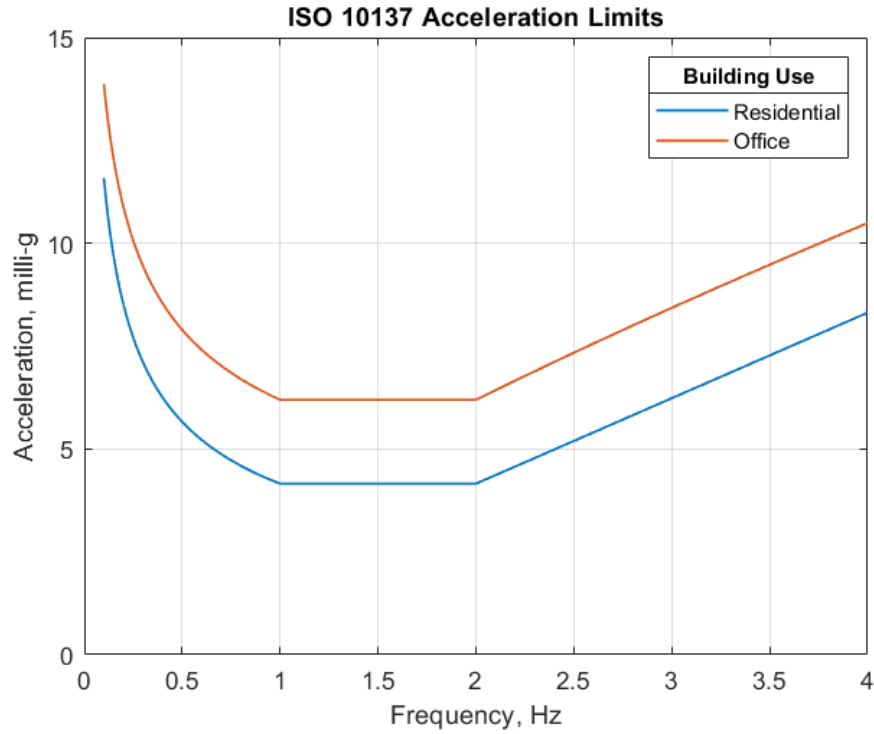


Figure 2.11: ISO 10137 peak acceleration limits [84]

2.6 Operational Modal Analysis

It is well accepted that full-scale in situ testing of structures is the most reliable method for assessing the dynamic response and validating models of a building [10, 11, 13]. There are two types of in situ testing: Experimental Modal Analysis (EMA) and Operational Modal Analysis (OMA).

EMA involves the structure being excited manually with a known force or excitation applied at various locations using an impact hammer or a shaker [85]. The response of the structure is captured by a network of sensors distributed at a number of locations across the structure. The recorded force input and response output signals can then be processed using various modal identification techniques in order to identify the inherent dynamic properties of the structure [85]. Whilst EMA is a well established technique applied in a number of industries including Civil, Aerospace and Mechanical Engineering to study a variety of structures with a broad range of damping ratios and stiffness values, it is not without limitations. Firstly, for large structures with low frequencies, the artificial force required to cause excitation is significant. This requires expensive and substantial equipment which

can become unfeasible [85, 86, 87, 88]. Furthermore, gaining the access required to the structure to perform this form of testing becomes difficult due to the space requirements and the disruptions to the operation of the structure whilst testing is ongoing [87, 88]. OMA was developed to overcome the logistic difficulties of EMA by enabling the measurement and processing of the response of a structure without the need for artificial excitation.

OMA is a method of in-situ testing which takes advantage of the natural excitation of structures due to wind and environmental conditions, negating the requirement of any artificial excitation [85, 86, 87, 88]. As such, OMA is an output only method which captures the wind or earthquake induced dynamic response of the structure using a distributed network of sensors and does not rely on knowledge of the exciting forces. However, in some cases the environmental conditions during monitoring are also recorded such as ambient temperature, wind speed and direction [13, 77, 89]. Given the uncontrolled nature of the excitation of the structure, it is often recommended that structures are instrumented for prolonged periods of time in order to capture responses experienced under a variety of ambient loading conditions, from low excitation to significant excitation [13, 70, 90, 91]. As the only instrumentation required to perform OMA is a network of sensors and data-loggers, this form of testing is discreet and can be performed whilst the structure is fully operational [86, 87, 88].

The measured output signals captured during OMA can be processed using output-only modal identification techniques [88]. These modal identification techniques fall into two categories: frequency domain modal identification and time domain modal identification [87, 88, 92]. Time domain methods focus on analysis of the response time histories and are often based on correlation functions. Frequency domain methods are based on the correlation between input and output Power Spectral Density (PSD).

2.6.1 Time Domain OMA Methods

One of the fundamental methods in Time Domain OMA is the Random Decrement Technique (RDT). The RDT was first developed by Cole [93] in 1973 for use in the quantification of damping in aerospace structures using output only vibrations. The RDT involves identifying segments of a response which exceed a triggering

condition. The signal consists of the response due to the initial displacement of the structure, the initial velocity of the structure and the white-noise input force. When a number of segments exceeding the triggering condition are averaged, the response components due to both the initial velocity and the white-noise force become zero, leaving only the response due to the initial displacement. A decay curve can be fitted to the displacement response and the damping of the structure identified [13, 70, 93, 94, 95]. The RDT has many advantages including its simplicity, versatility and negating the need for lengthy stationary signals required by many frequency domain techniques [94]. It has far reaching application and has been applied in many fields including Civil, Aerospace and Mechanical Engineering [70, 93]. However, it is not without limitations including the effects of noise in the response signals, ability to identify only one value for damping without a margin of error, difficulty identifying damping in closely spaced modes and the assumption that the structure remains linear [70, 71, 94, 96]. For this reason, many studies have focused on modified versions of the RDT including applying filters for noise and statistical methods for obtaining margins of error of the damping estimates [70, 71, 97, 98, 99, 100].

Other popular time domain methods include the Natural Excitation Technique (NExT) and Stochastic Subspace Identification (SSI). NExT involves the computation of the correlation function from the time histories of the measured response signal. The correlation function is then expressed as a sum of decaying sinusoidal waves with each wave having a frequency and damping ratio associated with a mode of the structure which was measured. Therefore, the correlation function can be used as the Impulse Response Function (IRF) for the estimation of modal parameters [87, 88]. NExT has been proven to be reliable, however it is a deterministic method and the nature of the data used in OMA is stochastic [87, 101]. Furthermore, it can be difficult to identify closely spaced modes using NExT [102].

SSI is one of the most prominent OMA methods. It is a time domain method which uses the statistical properties of the measured response signal to build a state-space model of the structure. Mathematical analysis of the state-space model provides the natural frequencies and damping ratios of the structure [103, 104]. Variations of SSI are widely used including Covariance-driven SSI (COV-SSI) which utilises covariance matrices and is robust to noisy signals, and Data-driven SSI (DATA-SSI) which utilises QR decomposition of the data [87, 88]. COV-SSI is more accurate and robust than DATA-SSI and is also reported to be less computationally

expensive [88, 105]. SSI has limitations including difficulty identifying closely spaced modes [99, 106] and the need for subjective identification of frequencies particularly when signals are noisy [105, 107]. Furthermore SSI is best suited to long time histories of data observed at a number of points along the height of the structure due to its basis on the principal of creating a state-space model [104].

2.6.2 Frequency Domain OMA Methods

One of the least complex OMA techniques is Peak Picking (PP). This method involves identifying peak values from the power spectrum of the response in order to determine the natural frequencies of the structure [87, 88]. This OMA technique works well when the damping is low and the modes are well separated. Although simple, this method is not suited to many modern structures where modes are closely spaced [87, 92]. Furthermore, this method cannot be used to identify the damping ratios of a structure [92]. To overcome the limitations of PP, another frequency domain method known as Frequency Domain Decomposition (FDD) was developed [87, 88].

FDD is an extension of the PP method which implements singular value decomposition of the power spectral density matrix in order to detect coupled modes [88]. FDD is capable of identifying the natural frequencies of closely spaced modes [87, 88, 92]. However, FDD cannot identify damping ratios and an extension of this method known as Enhanced Frequency Domain Decomposition (EFDD) was developed which enables identification of frequencies, damping ratios and mode shapes. This is achieved through applying an inverse discrete Fourier Transform to transform the Power Spectral Density (PSD) into the time domain at a peak of resonance. The resonant frequency can then be identified by determining zero-crossing times of the transformed PSD. The logarithmic decrement of the normalised auto-correlation function can be used to identify the damping ratio of specific modes [88]. This method is not demanding and has been proven to be accurate for closely spaced modes but can give biased estimates of the damping ratio [87, 88].

Many OMA techniques focus on identifying a singular optimal value of the natural frequency and damping ratio for each mode [88, 108]. Identification of uncertainty in modal parameter estimates is significant in model updating and assessing the wind induced response of a structure [108]. Bayesian Fast Fourier Transform

(BFFT) is a frequency domain OMA technique developed by Yuen [108, 109] which uses Bayesian methods and the Fast Fourier Transform (FFT) of an ambient structural response to identify not only optimal modal parameters, but their associated uncertainties too. The BFFT method enables the direct calculation of the posterior Probability Density Function (PDF) of the modal properties using multi-variate Gaussian Distribution. Calculation of the mean and covariance matrix of this distribution provides an estimate of the optimal values of the modal parameters and the associated uncertainties. In addition, only one measurement of the response time history is required by the BFFT to obtain accurate results [108, 110]. The BFFT is believed to be superior to SSI as fewer data points are needed and virtually all information in the time histories are used in identifying modal parameters [110]. Furthermore, it is reported to be a suitable method for OMA where modes are well separated [111].

2.7 Mitigation of Wind-Induced Vibration

As previously mentioned, advances in technology and construction techniques have led to increasingly taller, slender and light-weight structures being constructed. These buildings are particularly susceptible to excessive wind-induced vibrations which can lead to occupant discomfort and hinder the operation of the structure. Controlling the wind-induced vibrations of high-rise structures has been a significant area of research enabled by an improved understanding of structural behaviour as a result of advances in experimental testing and facilities, in-situ monitoring and computational power [112]. Larger experimental facilities have seen improved wind tunnel simulators with more accurate simulating capabilities, full-scale testing of damping technology and real time hybrid testing approaches. Improvements in sensing technology, data-acquisition and OMA techniques has enabled in-situ monitoring of structures and real-time assessment of performance. Increased computational power has given rise to enhanced modelling capability, Computational Fluid Dynamics (CFD), processing of larger time-histories with improved OMA methods and hybrid testing. These advances have enabled enhanced understanding of how structures behave in the wind and under other significant environmental conditions such as typhoons and earthquakes and the different mechanisms which can be employed to mitigate excessive vibrations [112].

Generally speaking, research has investigated three fundamental ways to reduce the dynamic response of a structure subjected to wind loading [7]:

1. Modifications to the structural design
2. Aerodynamic modifications
3. Addition of auxiliary damping systems

2.7.1 Mitigation of Vibrations Through Structural Design

Through modification of the structural design of a building, the mass and stiffness can be altered to improve dynamic performance [7, 15, 16]. Changes to the mass and stiffness of the structure can be achieved by increasing the cross-sections of steel and concrete members, using thicker concrete slabs and walls, the use of outriggers, increasing modal mass in upper floors and bracing systems [113]. Increases to the stiffness will increase the natural frequency and in turn reduce accelerations and displacements of the structure [15]. Furthermore, the increased stiffness and natural frequency can decrease the risk of resonance and large amplitude vibrations occurring, reducing the likelihood of human discomfort. Considering Figure 2.11, it can be seen that the limits for acceptable accelerations reduce as the natural frequency approaches 1 Hz due to increased human perception of accelerations. Therefore, increases in natural frequency will not always improve human perception of the decreased accelerations [9]. Furthermore, structural changes to the mass and stiffness results in increased cross-sections and construction costs which may not always be economically feasible [7, 9, 15, 79]. The effect of structural design on reducing the dynamic response of modular structures is examined in Chapter 3.

2.7.2 Aerodynamic Modifications for the Mitigation of Vibrations

Aerodynamic modifications involve modifications to the shape of the structure to reduce wind-induced motion. These modifications can include modifications to the corners of a structure as shown in Figure 2.12. They may also include more significant modifications to the overall shape of the structure such as tapering, improving porosity and including openings to allow wind flow through the structure as shown

in Figure 2.13. The aim of these modifications is to improve the bluff-body aerodynamics of the structure and reduce vortex shedding [7, 15, 16, 114].

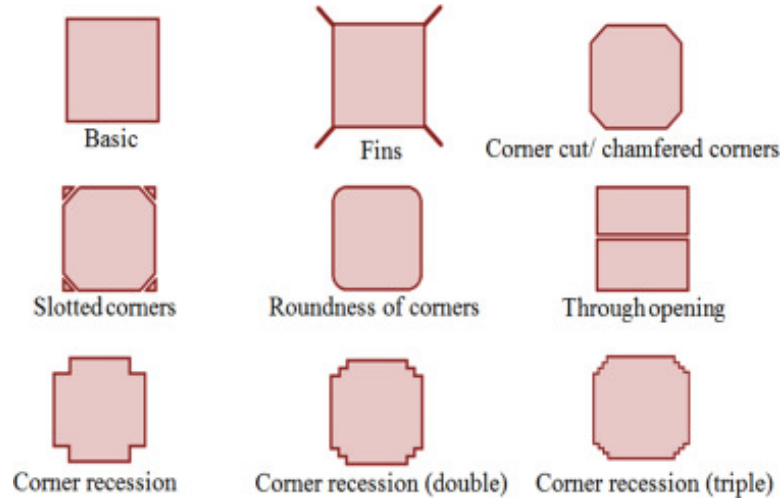


Figure 2.12: Possible aerodynamic modifications to corners of a structure (Source: Mooneghi et al. [114])

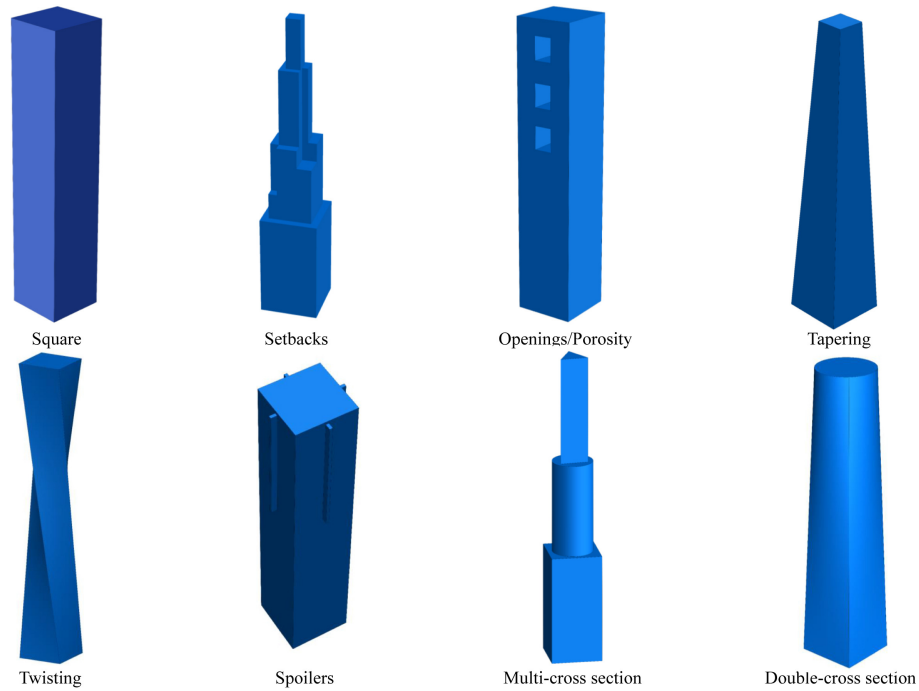


Figure 2.13: Possible aerodynamic modifications to the overall shape of a structure (Source: Jafari et al. [7])

Wind tunnel testing and CFD have enabled advanced studies and improved aerodynamic design of structures to reduce the wind-induced vibrations [7]. However, whilst self-induced vortex shedding of structures can be limited with aerodynamic design, buffeting winds from adjacent structures are harder to control with this form of mitigation strategy [79].

2.7.3 Auxiliary Damping Devices

The role of auxiliary damping devices is to absorb and dissipate energy caused by environmental and operational loading in order to reduce the dynamic response of the structure [115]. The addition of auxiliary damping devices is recognised as a reliable method for reducing wind induced motion [15, 16, 79, 116, 117, 118, 119, 120]. Tamura [10] discusses the importance of supplementing inherent damping with auxiliary devices. It is emphasised that where uncertainty surrounds the quantity of inherent damping, the addition of an auxiliary damping device results in the possible range in the estimate of the inherent damping becoming a smaller fraction of the total damping of the system. This in turn reduces the uncertainty in the acceleration response. For example, if a structure is estimated to have inherent damping of 2% +/- 1.4%, this leaves a range of possible values of the damping ratio between 0.6% and 3.4% , with the upper end of the range being 5.7 times larger than the lower ($3.4/0.6 = 5.7$). However, if the damping is supplemented with 4% damping from an auxiliary damping source, the range of total damping of the system becomes 4.6% to 7.4%, with the upper end being only 1.6 times greater than the lower. Therefore, where there is uncertainty in the inherent damping ratio of a structure, such as for modular buildings, supplementing the damping with an auxiliary damping system provides assurance that the wind-induced response of the structure will meet occupant comfort criteria.

Auxiliary damping devices can be classified by their external power requirements as passive, active, or semi-active dampers [115]. Passive dampers are those which require no external power supply, with properties based on pre-determined design criteria which do not change with respect to the dynamic response of the structure [117]. Passive dampers are tuned to or close to the natural frequency of a structure in order to reduce the dynamic response during resonance [121]. Passive dampers are discussed further in Section 2.7.3.3.

2.7.3.1 Active Damping Devices

Active dampers require a substantial power source (in the range of kilowatts and megawatts) to operate hydraulic or electro-mechanical actuators to supply control forces to the structure and modify the damping and stiffness of the system in order to efficiently dissipate energy [16]. A control strategy must be developed for active dampers in order to activate the damper and apply the correct control force in response to the current dynamic response of the structure. The ability to change the parameters of an active control system allows for control under loads from a wide range of frequencies in a varying external environment [7, 115, 122].

Two of the most common active control mechanisms include Active Mass Damper (AMD) and Active Bracing. AMD's were first developed in the 1980's and include an additional mass attached to the structure with an actuator [123]. A typical diagram of the mechanism is presented in Figure 2.14 with the actuator shown in blue. The dynamic response of the structure is measured by a network of sensors and processed on the control computer by a control algorithm. The algorithm determines a suitable control force to be applied by the actuator, which is then activated and updated as the response of the structure changes [123].

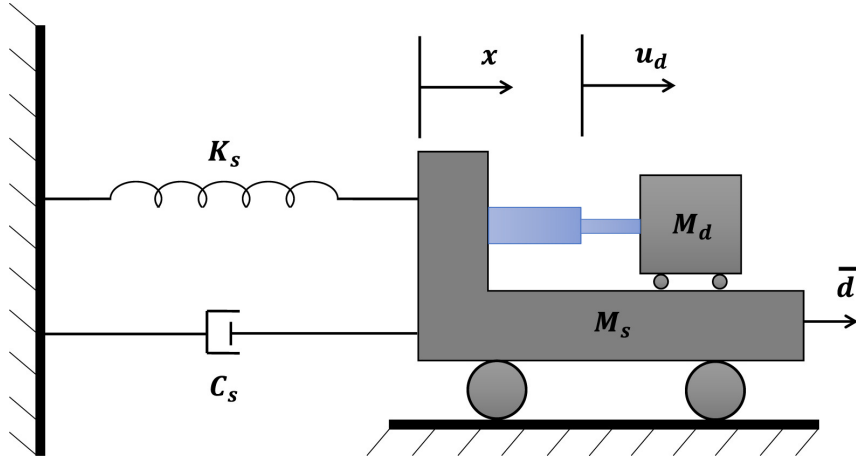


Figure 2.14: Mechanism of an Active Mass Damper (AMD) (Source: Jafari et al. [7])

Active bracing is achieved by placing an actuator between existing bracing and the adjacent floor [7]. Similar to AMDs, the control algorithm processes the signals obtained from a network of sensors across the structure. The necessary control signal is then sent from the control computer to the actuator to modify the bracing [115]. An example of an Active Bracing system is shown in Figure 2.15.

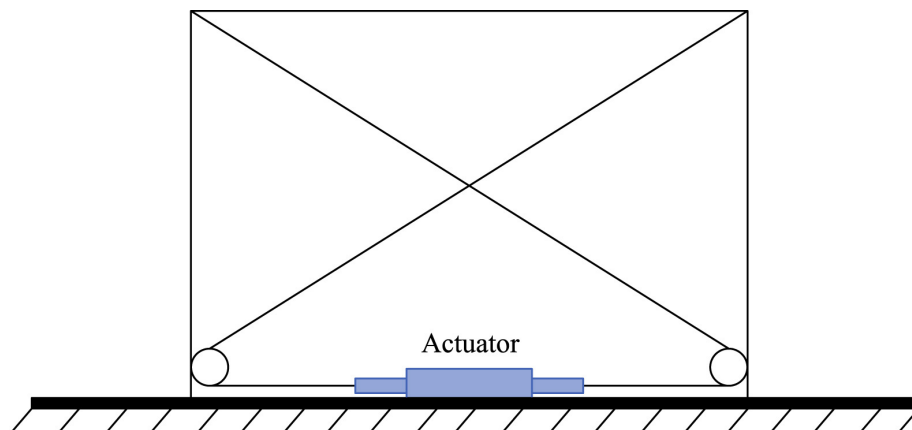


Figure 2.15: Example of an Active Bracing system (Source: Jafari et al. [7])

Whilst active damping mechanisms are advantageous in that they can respond and adapt their damping effects to the induced lateral loading, they have significant power requirements, require complex sensor and signal processing systems, will not work if power to the system is lost (as often occurs during strong earthquakes) and rapid updating of parameters can cause the control strategy to destabilise. Moreover the cost of the equipment and monitoring of the system can result in active dampers not being economically feasible [7, 123].

2.7.3.2 Semi-Active Damping Devices

Semi-active dampers are a natural progression from passive systems. They include adaptive features enabling the reduction of vibrations over a range of natural frequencies and are often employed where the power requirements of an active damping system are deemed too significant [7, 16]. Semi-active dampers combine the reliability of passive dampers with the adaptability of active control systems, providing a promising alternative solution for dynamic systems [16]. Similar to active damper systems, a network of sensors capture the dynamic response of a structure implemented with the semi-active damper. This response is processed by the control computer which then sends a signal to the control actuator to alter the mechanical properties of a passive damper in order to change the frequency of the damper to better match that of the wind-induced motion of the structure [122, 124, 125]. Semi-active dampers exist in a number of different forms. For example, the mechanism of a semi-active Tuned Liquid Damper (TLD) is founded on a passive TLD with mechanical properties which can be altered using a small power input, such

as changing the placement of baffles or the sizes of orifices in screens [7]. Further examples of semi-active systems include changing the resistance of flow within a hydraulic fluid damper or through the use of variable friction dampers by changing the force applied to the components [16]. Semi-active systems require sensor networks, control computers and control algorithms, however, they do not utilise the same power as active systems and are less likely to cause destabilisation in the system [7, 16, 122, 124, 125]. Due to the lower power requirements of semi-active systems, they can often operate using battery power, ensuring control during intense seismic and wind events where power cuts are likely [16]. Kavyashree et al. [16], Koutsoloukas et al. [123] and Casciati et al. [125] provide further reviews of Active and Semi Active systems.

In addition to the three classifications of dampers, combined or hybrid control systems also exist. Hybrid systems are combinations of passive and active or passive and semi-active control systems. They are not within the scope of this body of research but further reading is provided by Stanikzai Mohammad, Elias, and Chae [126], Fisco and Adeli [127], and Arvind and Santhi [128].

2.7.3.3 Passive Control Systems

Passive control systems dissipate energy from an excited structure using mechanical systems with predetermined constant mass, stiffness, natural frequencies and damping ratios [129]. This form of control strategy can be defined as a system which does not require an external power source for operation, instead the motion of the structure is utilised to develop forces which control the structure [124]. They do not require external power, a network of sensors, control algorithms or control computers to operate and as such do not adapt in real time to the variability of the excitation of the structure [121]. Whilst the range of frequencies of excitation which can be optimally controlled by passive damping mechanisms is less than those that active and semi-active systems can control, their advantage lies in that they require less power, will not destabilise and are often more cost effective [117, 121, 123, 129]. Other advantages of passive systems over active and semi-active controllers include easier installation, generally lower costs, lower maintenance and consistent performance [130]. Examples of passive control devices include Viscoelastic (VE) materials, Viscous Fluid (VF) Dampers, Friction Dampers, Tuned Mass Damper (TMD) and Tuned Liquid Dampers (TLDs).

Viscoelastic material dampers are one of the earliest forms of damper developed [15]. This form of damper consists of steel plates with layers of viscoelastic (rubber like) material sandwiched tightly between them as shown in Figure 2.16. The VE material deforms in shear as the structure vibrates providing energy dissipation and a restoring force. They operate best under dynamic loading consisting of high frequencies and low-medium vibrations, such as high winds or moderate earthquakes [7]. VE dampers are typically employed in diagonal bracing members of structures, where vibrational energy can be dissipated by the shearing of the material. However, the performance of VE materials is dependent on the frequency of the excitation and on the ambient temperature. Previous studies have shown that VE dampers not only provide dissipation of vibrations within a structure, but also increase the stiffness and hence the natural frequency of the structure [131]. Further studies of VE dampers have also shown their effectiveness in controlling the dynamic response of structures [132, 133, 134, 135, 136].

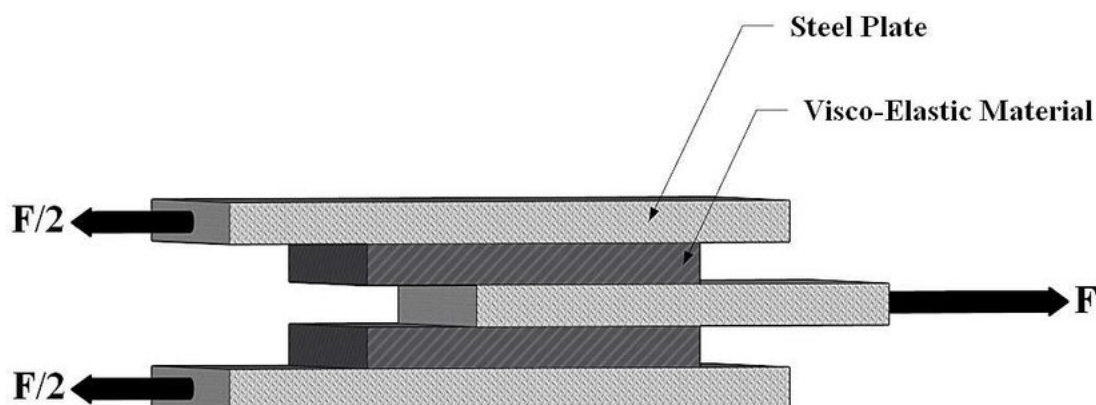


Figure 2.16: Schematic diagram of Viscoelastic Damper (Source: Vatanshenas et al. [136])

Viscous Fluid Dampers control vibration responses through the generation of a resistance force as a piston travels a finite distance through a viscous fluid contained in a chamber [137]. Orifices for the fluid to flow through are designed to achieve the desired level of damping. The restoring force generated by the VF damper is directly proportional to the velocity of the piston [7, 15]. Figure 2.17 shows the mechanism of a VF damper. VF dampers can also take the form of damping walls in which a steel plate moves through viscous fluid contained in the wall of a structure to provide auxiliary damping [138]. This form of damper has been shown to provide damping at the fundamental mode and at higher modes, and can increase damping of the structure to 20-50% of critical damping. Taylor and

Constantinou [139] showed that VF dampers can reduce deflections of structures under ambient dynamic loads by a factor of 2 to 3, improving occupant comfort without the need for large, stiff cross-sections. VF dampers can be space efficient due to large restoring forces resulting from the pressure generated by using viscous fluids. Other advantages of VF dampers include low cost, easy installation and low maintenance. Aydin et al. [117], McNamara and Taylor [137], Hou [138] and Rama Raju et al. [140] have also studied the design and effectiveness of VF dampers and found promising results.

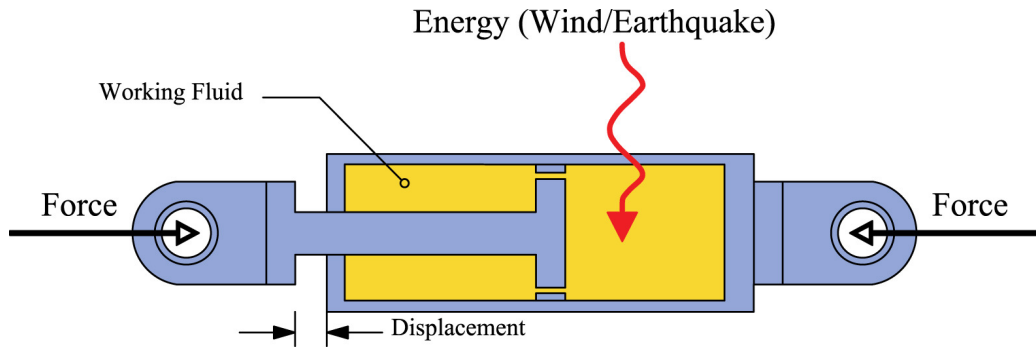


Figure 2.17: Mechanism of Viscous Fluid Damper (Source: Jafari et al. [7])

Friction dampers have been applied in structures for vibration control since 1980 [121]. They aim to mitigate excessive vibrations by dissipating kinetic energy through friction forces generated as components slide over one another [15]. The sliding force is controlled and exhibits a large initial stiffness and rectangular hysteretic behaviour [7]. This form of system ensures that the structure remains elastic whilst allowing for plastic behaviour by providing non-linearity [15]. Friction Dampers are traditionally classified as either rigid frame friction dampers or braced frame friction dampers. Rigid frame friction dampers involve the provision of real plastic hinges which are replaced after yielding in earthquakes or significant wind events [121]. Braced friction dampers involve the use of diagonal bracing members with joints that allow for sliding of members at a designed stress value. Friction dampers are a relatively simple concept, are low cost and reliable [7, 15, 121]. They are not significantly influenced by loading amplitude or frequency type or by ambient temperatures [121]. Furthermore, the rectangular hysteretic loop allows for maximum dissipation of energy. Further studies of friction dampers have been carried out by Bhaskararao and Jangid [141], Lopez et al. [142], López, Busturia, and Nijmeijer [143] and Caudana Quintana and Petkovski [144].

Tuned Mass Dampers (TMDs) are one of the most common forms of passive auxiliary damping devices deployed to control the dynamic response of structures [7]. They typically consist of a mass connected to the primary structure by a spring to provide stiffness and a dashpot which provides viscous damping as shown in Figure 2.18. The mechanism of a TMD is to transfer inertial force to the structure in order to reduce vibrations [145]. The effectiveness of TMDs is determined by its dynamic characteristics, stroke and the amount of additional mass added to the system. TMDs are tuned to have a natural frequency at or close to the natural frequency of the dominant mode of the primary structure and can reduce the dynamic response significantly around this natural frequency [119]. The amount of additional damping supplemented by a TMD is also dependent on the ratio of the mass of the TMD to that of the effective mass of the structure in the mode of interest [119]. TMDs are reported to be efficient vibration controllers, robust, relatively low cost compared to active and semi-active systems and are not affected by changing temperatures [146].

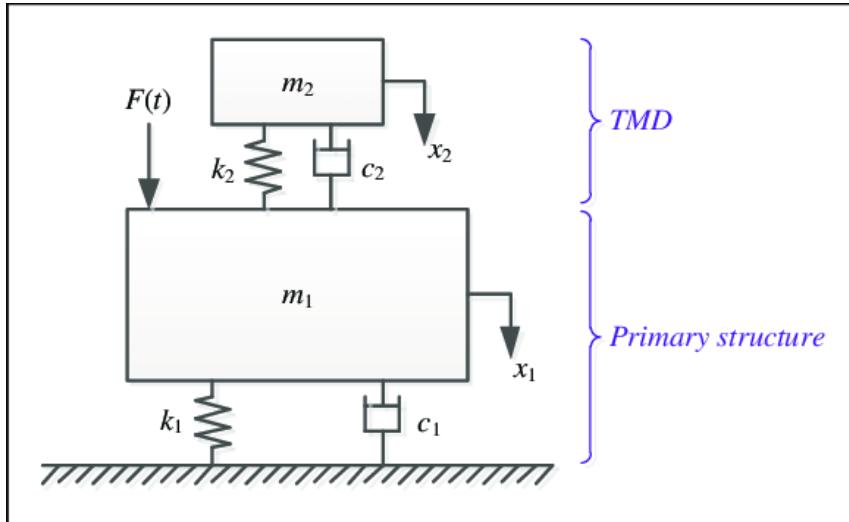


Figure 2.18: TMD system attached to a primary structure (Source: Chen et al. [147])

Many studies have focused on the efficiency and optimum design of TMDs and produced valuable results including Elias and Matsagar [146], Warburton [148], Farghaly and Salem Ahmed [149], Lee et al. [150] and Gerges and Vickery [151]. TMDs are one of the earliest and most researched control methods with the concept first patented by Frahm in 1909 [152] and design methods provided by Den Hartog in 1947 [153]. Due to continued research and their many advantages, TMDs have seen many real-world applications and have been implemented in tall buildings and

other structures globally to control vibrations. The largest TMD is located in the Taipei 101 Tower in Taipei, Singapore, which includes two TMDs to provide control in different modes of vibration of the structure [154]. Other structures with TMDs include the John Hancock Tower in Chicago, Illinois, USA, the CN Tower in Toronto, Canada and Millennium Bridge in London, United Kingdom [7, 146]. However, TMDs are not without limitations, the primary one arising from the requirement of suitable space for the solid mass to move freely back and forth. This drawback is overcome by another passive inertia based controller known as the Tuned Liquid Damper (TLD).

TLDs are cost effective passive auxiliary damping devices that have been well researched and are becoming increasingly popular for controlling the dynamic response of tall buildings [155, 156, 157, 158, 159]. TLDs typically consist of liquid filled tanks which dissipate energy through the sloshing of the liquid induced by oscillations of an excited structure. Figure 2.19 shows the typical mechanism of a TLD, where h is the depth of the liquid within a TLD, η is the sloshing depth above the liquid depth and a is half the tank length. The dimensions of a TLD and the depth of the liquid within the TLD can be tuned to have the same, or close to the same, natural frequency of a structure. Therefore, when a structure begins to vibrate, the fluid within the TLD will begin to slosh, absorbing the vibrational energy from the structure and transforming it into kinetic and potential energy of the sloshing fluid. The sloshing energy is dissipated by the fluids viscosity and friction between the wall of the container and the fluid.

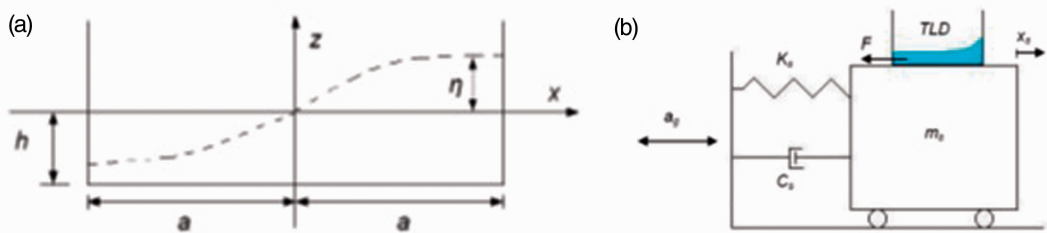


Figure 2.19: (a) Rectangular tuned liquid damper and (b) single-degree-of-freedom oscillator with a tuned liquid damper (Source: Malekghasemi et al. [160])

The advantages of TLDs are well reported and include their simple configuration, low installation cost, and low maintenance costs [161, 162, 163, 164, 165]. Furthermore, when compared to TMDs, they require less space, do not have a threshold external excitation for activation and can be tuned in-situ with ease by changing the depth of liquid [7, 16, 156, 166, 167]. It has also been found that TLDs

can be incorporated in the fire suppression system of a structure and still control vibrations effectively [168].

Some of the earliest applications of TLDs were proposed in the late 19th Century for use in controlling rolling motions of ships. The free-surface of a liquid in an anti-rolling tank was used to control rolling motion [169]. One of the first instances of TLDs being explored for use in Civil Engineering was in the 1970's by Vandiver and Mitome [170] who investigated the ability of liquid filled tanks to control the motion of off-shore platforms. Research into TLDs has progressed over the past 50 years and many variations of TLDs have since been studied including TLDs with different geometries [158, 171, 172, 173], TLDs with screens, baffles or floating objects to alter the sloshing of the liquid and hence the mechanical properties of the TLD [155, 174, 175, 176], Tuned Liquid Column Dampers [177, 178, 179, 180, 181] and pendulum type TLDs [182, 183].

TLDs have been implemented in a number of tall structures globally to control acceleration responses and ensure occupants comfort. The first real-life installation of a TLD was in the Nagasaki Airport Tower located in Japan in 1987 [9]. Other structures equipped with TLDs include One Rincon Hill located in San Francisco, USA, Comcast Centre in Philadelphia, USA, the Hyatt Hotel in Osaka, Japan and Higashi-Kobe Bridge located in Kobe, Japan. A more extensive list is presented by Konar and Ghosh [166].

The most fundamental form of a TLD involves a tank-like container holding a specified level of liquid. Energy dissipation is achieved through the viscous boundary layer near the tank walls and the free surface contamination which occurs when sloshing in the liquid is induced by external excitation [166]. The fundamental frequency of the liquid sloshing is tuned to that of the dominant mode of the primary structure. This is achieved through changing the dimensions of the tank and the depth of the fluid. TLDs can be categorised as shallow, intermediate or deep with shallow TLDs having a depth to breadth, h/L , ratio of < 0.1 , intermediate having values lying in the range $0.1 < h/L < 0.5$ and deep tanks having a $h/L > 0.5$. In shallow TLDs oscillations induce travelling waves that experience wave breaking. Deep TLDs experience standing sloshing waves with no wave motion in the bottom layer of liquid. Intermediate TLDs generate transitional waves. The inherent damping of a TLD is directly related to h/L , and as this value increases the supplementary damping provided by the tank decreases [155, 164, 166]. Generally

h/L values are designed to be below 0.3 in order to achieve maximum free-surface response of the liquid and to enable easier in-situ tuning of TLDs [155, 164, 184, 185]. However, low h/L ratios can result in non-linear behaviour of the liquid [164, 184].

The sloshing motion of the liquid within a TLD is formulated using potential flow theory and is dependent on the maximum amplitude of the sloshing waves [155]. The mathematical model of the motion of the liquid becomes complex and the solution involves solving a set of Navier-Stokes equations. For this reason, software for computational fluid dynamics, experimental tests and equivalent models are often employed in the design of TLDs. one of the first equivalent models for a TLD was presented by Sun et al. [186] and involved modelling a TLD as an equivalent TMD. This method was further developed by Tait [155] and has become a prominent method for providing estimates of the mechanical properties of the TLD for preliminary design [187, 188, 189]. This equivalent model is discussed further in Chapter 7.

2.7.3.4 Multiple and Distributed Passive Damping Systems

As research has progressed, many studies have shown that Multiple Tuned Mass Dampers (MTMDs) can be more effective in controlling the dynamic response of a structure than a single TMD [115, 190, 191, 192, 193, 194]. MTMDs enable a greater range of modes of a structure to be controlled as the individual TMDs can be tuned to the different natural frequencies of the structure [192]. Furthermore, TMDs can be vertically distributed throughout the height of a building, with consideration given to the mode shapes of the structure when the optimal TMD locations and tuning frequencies are designed [183]. Igusa and Xu [191] considered the effectiveness of MTMDs attached in parallel to an SDOF system with the frequencies of each TMD spaced closely together. The performance of the horizontally distributed MTMDs was compared to that of a single TMD. It was observed that the individual optimum frequencies of each TMD did not match that of the primary structure and that maximum vibration attenuation was achieved with ‘mis-tuned’ conditions, concluding that the MTMDs were more effective than a single TMD which can only be tuned to optimally control one unique frequency.

Similarly, Kareem and Kline [192] studied MTMDs tuned to a range of frequencies and attached in parallel to a structure. It was found that the MTMDs

could flatten the frequency response of the excited structure in some frequency ranges and increase the auxiliary damping. Furthermore, the MTMDs system with the same mass ratio as a single TMD performed better. Kareem and Kline [192] conclude that the distribution of multiple smaller TMDs, rather than a single large TMD, improves constructability and manageability providing easier installation and presenting an opportunity to implement MTMDs in more favourable spatial configurations. The ability to distribute MTMDs can negate the requirement to occupy large, and often valuable, real estate space at the top of a structure [195].

More recently, Moon [183] studied the vertical distribution of MTMDs in a 60 storey building under dynamic wind loads. Equations were presented for a Multi Degree of Freedom (MDOF) model with multiple vertically distributed TMDs, allowing for a range of parameters to be studied with respect to the MTMDs. The fundamental representation of a MDOF and MTMD system is shown in Figure 2.20, where a series of TMDs are represented by a mass, spring and dashpot and are attached to the each DOF of a system. This system can be modified to only include TMDs at specified DOFs and can be used to derive equations to analyse the effectiveness of MTMDs systems using the mass, stiffness and damping values of the MDOF system and the mass, stiffness and damping values of the TMDs. These equations are discussed further in Chapter 7. Moon [183] used the equations and methods presented to study control of the first two modes using MTMDs tuned to the first mode in the upper third of the 60 storey structure and MTMDs tuned to the second mode in the upper 6 storeys and middle 12 storeys. This configuration of MTMDs corresponds to the mode shapes of the structure. Moon concluded that vertical distribution did not reduce the effectiveness of the MTMDs in controlling the dynamic response. Moreover, the easier installation of MTMDs, reduced spatial requirements in upper storeys and ability to control multiple modes offers a range of potential future applications.

Elias and Matsagar [193, 196, 197] have studied the vertical distribution of MTMDs and presented a similar methodology to that proposed by Moon [183] for a MDOF MTMD system. In these studies, the control of a 76 storey benchmark building with vertically distributed MTMDs was investigated. It was found that a single TMD at the top storey reduced the peak displacement response by up to 30%, MTMDs horizontally distributed at the top storey saw a 40% reduction while vertically distributed MTMDs resulted in a 50% reduction. However, it was

noted that in some instances the top floor acceleration was marginally larger for the MTMD systems when compared to the single TMD.

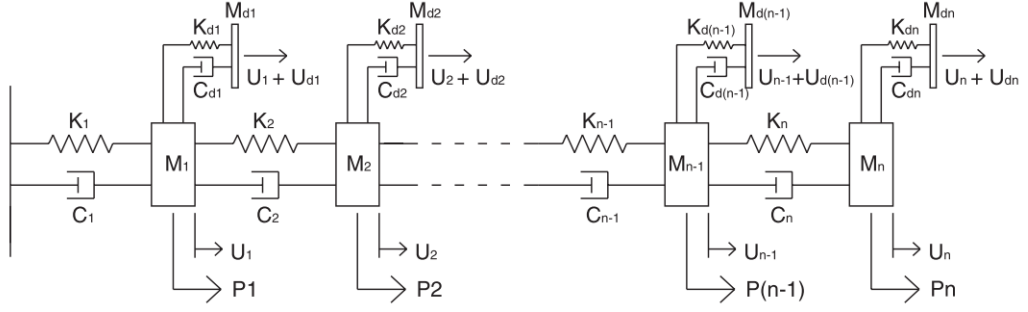


Figure 2.20: Schematic of MDOF with MTMD model presented by Moon [183]

Despite the numerous aforementioned advantages of TLDs over TMDs, there have been limited studies on multiple distributed TLDs. This is likely due to the most conventional method for modelling TLDs being an equivalent TMD model proposed by Sun et al. [186] and further developed by Tait [155]. As discussed, the ratio of the depth of liquid in TLDs to the length of the TLD is generally kept to less than 0.3. This is because as this ratio increases, damping decreases [155]. Therefore to achieve significant damping and mass ratios whilst still tuning TLDs to the fundamental frequency of the primary structure, it is more advantageous to use multiple smaller TLDs than one large TLD [198]. Hence, distributing multiple small TLDs throughout a structure is a practical control strategy which capitalises on the optimum design of TLDs. However, studies have mainly focused on multiple TLDs located at the top storey of structures rather than distributed vertically.

Fujino and Sun [190] considered multiple TLDs at the top storey and investigated the performance of this control method parametrically using both numerical models and shake table tests. It was found that multiple TLDs were more effective than a singular TLD and are a very attractive method for controlling the dynamic response of structures. Similarly, Love and Tait [189] studied the ability of one, two and three TLDs at one location to control a SDOF system and found that the multiple TLDs showed larger reductions in oscillations than a singular TLD when excited at levels typical of wind loading. The robustness of the multiple TLD system was also highlighted as an advantage of multiple TLDs as they are less sensitive to detuning.

The lower space requirements of TLDs compared to TMDs and the ability to distribute the TLDs throughout a structure offers potential practical application of this control method in modular construction where there are repetitive units in which TLDs could easily be installed.

2.8 Real Time Hybrid Testing

Experimental testing plays an important role in validation of numerical models and theoretical findings. However, testing full-scale buildings equipped with auxiliary damping devices subject to dynamic loads is difficult to facilitate in the controlled environment of a laboratory without using scaled models. For this reason, a novel testing method, known as Real Time Hybrid Testing (RTHT) was first proposed by Takanashi et al. [199] in 1975 to enable real-time physical testing of one aspect of a complex model subject to seismic loading whilst the remainder of the system is represented by a numerical model.

RTHT, also referred to as Hybrid in Loop, is a state-of-the-art testing method which enables physical real time testing and model updating of large, complex systems [195, 200, 201]. RTHT is an efficient method to facilitate the real time interaction between a physical component of a system and a numerical model. The concept of RTHT relies on separating a system into two subsystems: a numerical substructure and a physical substructure. The numerical substructure can be simulated by a numerical model on a computer, whereas the physical substructure often exhibits complex dynamic behaviour making it difficult to model efficiently using a numerical model [200]. Therefore, a physical, often full scale, model is built and tested using dynamic testing equipment such as shake tables or actuators [181]. The physical model then interacts with the system which is numerically modelled in real time [198]. During a RTHT, the dynamic response to loading of a structure is calculated numerically on a computer using time step-integration of the equations of motion, the calculated displacement response is then passed to the physical component via actuators and the force response of the physical component is captured and passed back to the numerical model for the next time step where the process is repeated. Figure 2.21 shows the process involved in a RTHT.

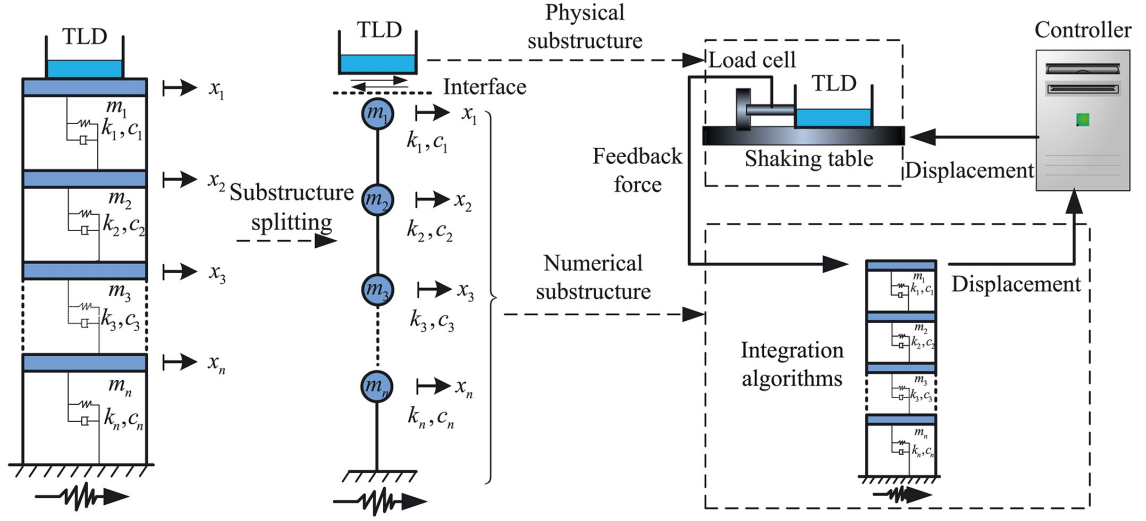


Figure 2.21: Example of Real Time Hybrid Testing process (Source: Wang et al. [202])

RTHT has a number of advantages, primarily relating to enabling the efficient modelling of large scale, complex structures without errors relating to model scaling. This form of testing is a practical and economical method to isolate and physically test the critical components of a system for which a numerical model is too complex or does not exist. For example, RTHT has been employed in a number of studies investigating the control of structures using TLDs [160, 181, 195, 198, 200, 202, 203]. As discussed, the behaviour of fluid within a TLD is complex and challenging to model without using a simplified equivalent numerical model [155]. Implementing RTHT allows for a numerical model of a structure to be used in conjunction with a physical model of a TLD. This negates the need for complex modelling or simplified representations of TLDs, capturing the actual behaviour of a full-scale TLD, without the need to create a scaled model of a structure.

Lee et al. [203] compared the response of a single storey structure equipped with a TLD from a RTHT with a traditional shake table test of a scaled model of the structure-TLD system. It was concluded that the RTHT provided a good estimate of the response and that a scaled model of the structure was not necessary. A further study was performed where a RTHT of a three storey structure (numerically modelled) with a TLD (physical model) was assessed to demonstrate that the dynamic response of the structure was reduced by the TLD.

Similarly, a study carried out by Wang et al. [202] also used shake table experiments of a two storey structure with TLD system to develop and validate a suitable procedure for RTHT. The validated methodology for RTHT was then used to parametrically assess the control of structures with different structural properties and number of storeys, altering only the numerical model. A parametric study like this would be time consuming, uneconomical and require a considerable amount of scaled model structures if it were reliant solely on shake table tests. The ability to simply change model parameters in a numerical model, whilst still having a physical model of the most complex aspect of the system, i.e. the force output of the TLD, demonstrates the potential of RTHT.

As RTHT testing procedures advanced, further studies have used this novel testing method to investigate the validity of equivalent models of TLDs [168, 198]. Roy et al. [160] used the RTHT facilities at Trinity College Dublin, Ireland to verify a nonlinear numerical model of a TLD based on shallow water wave theory and to investigate the application of TLDs for seismic induced vibration control. It was found that the numerical non-linear model of the TLD was in good agreement with the displacement and force responses of the TLD during the RTHT, validating the model. It was also concluded that the TLD was capable of reducing the seismic response of a structure, even under non-optimal conditions. Malekghasemi et al. [160] used results from RTHT of a TLD to investigate the suitability of three different simplified numerical models of TLDs and the finite volume method to develop a model of the liquid behaviour within the TLD, and hence the displacement response. The three numerical models assessed were those proposed by Sun et al. [204], Yu et al. [205] and Xin et al. [206]. Malekghasemi et al. [160] found that the finite volume method for modelling the TLD, when included in a FE model of the structure, provided the best match for the displacement response of the structure identified through RTHT. Of the three numerical models, the model proposed by Yu et al. [205] was closest to predicting the actual response of the structure-TLD system for a broad range of tuning ratios when compared to the RTHT results.

A series of work carried out by Zhang et al. [173, 200, 207] also using the RTHT facilities at Trinity College Dublin, Ireland, considered simplified models of TLDs and their appropriateness in controlling vibrations in large wind turbine towers. Zhang et al. [200] discusses using RTHT to assess the reduction in the vibration of wind turbine towers achieved using a full-scale TLD with and without

screens. A number of different wind speeds and tuning ratios were investigated, and in all scenarios the TLD was found to reduce the response. Altering the tuning ratio of the structure-TLD system was achieved by simply adjusting the level of water in the TLD, owing to the ease of tuning a TLD in comparison with other supplementary damping devices. Zhang [173] also used the RTHT testing facilities to investigate the response of sloped bottom TLDs and develop a finite difference method based numerical model. Various water levels, slope angles and the ratio of the horizontal projection length of the slope and the total length of the TLD were considered. It was found that the sloped bottom has a significant impact on the sloshing frequencies of shallow water TLDs but that this impact is reduced for deep water TLDs. The developed numerical model was concluded to be a useful tool for the future assessment of structure-TLD systems where slope bottomed TLDs are implemented. More recently, Zhang [207] also used the results from the RTHT to validate a reduced order model of the nonlinear fluid behaviour within a TLD under different excitation conditions. The use of the RTHT facilities at Trinity College Dublin, Ireland for testing of structure-TLD systems is well reported. The facilities enable the assessment of full-scale TLDs which has in turn permitted investigation of different simplified numerical models of TLDs, and the ability of TLDs to control vibrations induced from environmental loading such as wind and earthquakes as has been discussed.

Kwon and Kareem [195] used RTHT to investigate the ability of multiple stacked 'double-decker' TLDs to control the vibration response of a structure. A numerical model of a recently constructed reinforced concrete high-rise building was used during the RTHT of a single TLD. The response of the full-scale TLD tested was multiplied to account for the multiple TLDs in the supplementary damping system. This demonstrates the capability of RTHT further, showing that only one physical model of a complex element may be required to assess the response of a system containing multiples of these complex elements.

The use of RTHT to evaluate the dynamic response of TLDs and to validate simplified models has been well established. RTHT is a advantageous experimental technique for assessing the dynamic response of a structure-TLD system in an economic, cost-effective manner. As discussed in this Section, RTHT requires only one physical full-scale model of a TLD to parametrically assess the dynamic response of a structure equipped with a TLD, or multiple TLDs, under different loading

conditions. The experimental response feedback of the TLD can interact with a numerical model in a time-stepping procedure to obtain the dynamic response. The numerical model can be easily altered to assess different parameters and the TLD can be tuned by simply altering the depth of water in the tank. Therefore, it is clear that RTHT is a critical tool in validation of models of structure-TLD systems and in the parametric assessment of dynamic response control of structures.

2.9 Conclusions

Current published literature relating to topics considered by this thesis has been presented. Modular Construction, occupant comfort and habitability requirements, Operational Modal Analysis (OMA), mitigation of wind-induced vibrations, and Real Time Hybrid Testing (RTHT) have all been discussed extensively. It has been the aim of this chapter to provide suitable knowledge of the basic concepts and theory which is required to appreciate the motivation for this research and the understanding to comprehend topics discussed further in later chapters.

Chapter 3

Comparing Structural Modifications to Auxiliary damping for Mitigation of Wind-Induced Acceleration

3.1 Introduction

As discussed in Chapter 2, tall buildings experience wind induced motion which can result in serviceability and habitability issues associated with occupant discomfort. Modular construction continues to increase in height due to economic drivers, with building heights of over 160 m now realised [25]. However, as with other structural forms, habitability requirements associated with excessive acceleration response become the governing design criterion as building heights increase. Some Structural Engineers suspect that high-rise modular construction can be particularly susceptible to wind-induced accelerations due to the tall slender form of the RC core and the uncertain contribution of the modules to global lateral stiffness and damping. The purpose of this chapter is to investigate the scope of the susceptibility of high-rise modular buildings to vibrations in excess of habitability requirements and suitable mitigation measures. An idealised building model suitable for parametric analysis which employs standard design and analysis methods, by way of codes and standards, is used to investigate the wind-induced acceleration response. The purpose

of using the idealised building model is to relate the parametric study closely to engineering design practice.

Broadly speaking, for this form of construction, two approaches exist to mitigate excessive vibrations; (i) increasing core dimensions, or (ii) the addition of auxiliary damping. This chapter evaluates and compares the effectiveness of these two approaches in controlling wind-induced vibrations for a large number of archetype modular structures with a range of parameters typical to the case study structures described in Chapter 4. The work described in this chapter has previously been published in the journal *Structures* by Hickey et al. [20].

3.1.1 Idealised Structural Scheme

The main source of lateral load resistance in tall modular construction is typically a reinforced concrete core that is connected directly or indirectly to the modules [4, 5, 19, 28, 29, 53]. A typical modular scheme is illustrated in Figure 3.1, where modules are stacked around and connected to the core. There are many different schemes and materials used for the individual modules, typically incorporating some form of lateral bracing within module walls to satisfy transportation, erection and stability requirements. However, the contribution of the modules to global lateral stiffness also depends on intra- and inter-module connection stiffness which is challenging to estimate at a preliminary design stage [21, 40, 43]. As previously illustrated, there is no one connection type which is used predominantly within construction of modular buildings. This suggests that a connection type is yet to be developed which sufficiently supplies continuity to the structure and meets structural needs along with manufacturing and construction requirements [4, 52]. Furthermore, the potential increase in global lateral stiffness offered by greater in-situ continuity between individual modules and between the modules and the core is difficult to achieve within existing modular construction practice [21, 52]. There is insufficient conclusive research on optimal module connections and on modelling and estimating the behaviour and stiffness contribution of module connections [4, 21, 40, 43, 52]. Therefore in design, it is conservatively assumed that the entire lateral load resisting capacity of the structure is provided by the RC core; an assumption which is also adopted by this chapter.



Figure 3.1: Typical volumetric modular building with RC core (Source: Tide Construction [6])

Other properties that influence dynamic response can also be significantly different for modular construction, including the total building mass due to the combined core and modules, and the inherent damping of a hybrid structural form that employs both reinforced concrete and steel elements. As economic drivers and advancements in manufacturing and material technology lead to modular construction being employed in increasingly tall structures, the ability of the RC core to control the wind induced accelerations within acceptable limits is unproven [208], implying that it may be necessary to employ some form of acceleration control methods. Typical methods employed to control building accelerations include:

- Changing physical properties of the structure such as building shape, materials and dimensions to favourably alter aerodynamics. Varying the shape to improve aerodynamics is generally not possible in modular construction due to its cellular nature. However, it is often possible to adapt the lateral load resisting system, i.e. the RC core, to improve performance.
- Employing passive control systems such as the inclusion of viscoelastic materials, Tuned Mass Dampers (TMDs), Tuned Liquid Dampers (TLDs) or vari-

ations of TMDs and TLDs such as Tuned Mass Damper Inerter or a Tuned Liquid Column Damper, to name a few [118, 173]. Often where the decision is made to include an additional damping mechanism, a TLD is favoured due to its low maintenance, easy tuning and low cost compared to other forms of damper [173, 195, 209]. Another advantage of TLDs is their potential to be integrated into the water system of the building or to be used in fire resistance systems [15, 195].

- Employing active control systems incorporating a continuous monitoring strategy, a signal processing system and a damping mechanism such as a TMD which can be actively modified or induced to control accelerations based on the results of continuous monitoring.

This chapter examines two practical methods for controlling wind-induced accelerations in high-rise modular buildings: modifying the breadth and depth of the building's core, and installing a TLD. The goal of the work is to characterise the influence of key structural and control parameters on the acceleration response of modular high-rise buildings, providing novel insights into the behaviour of this form of construction when employed in high-rise buildings. Given the limited amount of data on existing structures of this kind, assessing the potential applicability of a modular scheme at a preliminary design stage can be challenging. In particular, outstanding questions exist about the maximum feasible height for single-core modular construction; at what height acceleration serviceability becomes a limiting factor in design and at what height interventions to control this become unfeasible or uneconomical. The aim, therefore, is to obtain new generic information that can be used early in the design of a wide range of modular towers; including what the maximum feasible building height may be and how structural properties can be altered to better control wind-induced motion. This will inform the preliminary design process and help mitigate the need to carry out detailed, building specific dynamic analyses at an early design stage.

In order to assess the practicality and efficiency of increasing core dimensions or adding a TLD, a sample set of 6,125 archetype buildings is generated. The archetypes cover a range of building heights, masses and dimensions representative of the feasible parametric range encountered in modular construction. A simple illustration of an archetype and its dimensions is shown in Figure 3.2. Peak acceleration response during a 1 year wind event is estimated for each archetype, and

in cases where this exceeds the ISO residential limit, the effectiveness of the two mitigation techniques is assessed and compared.

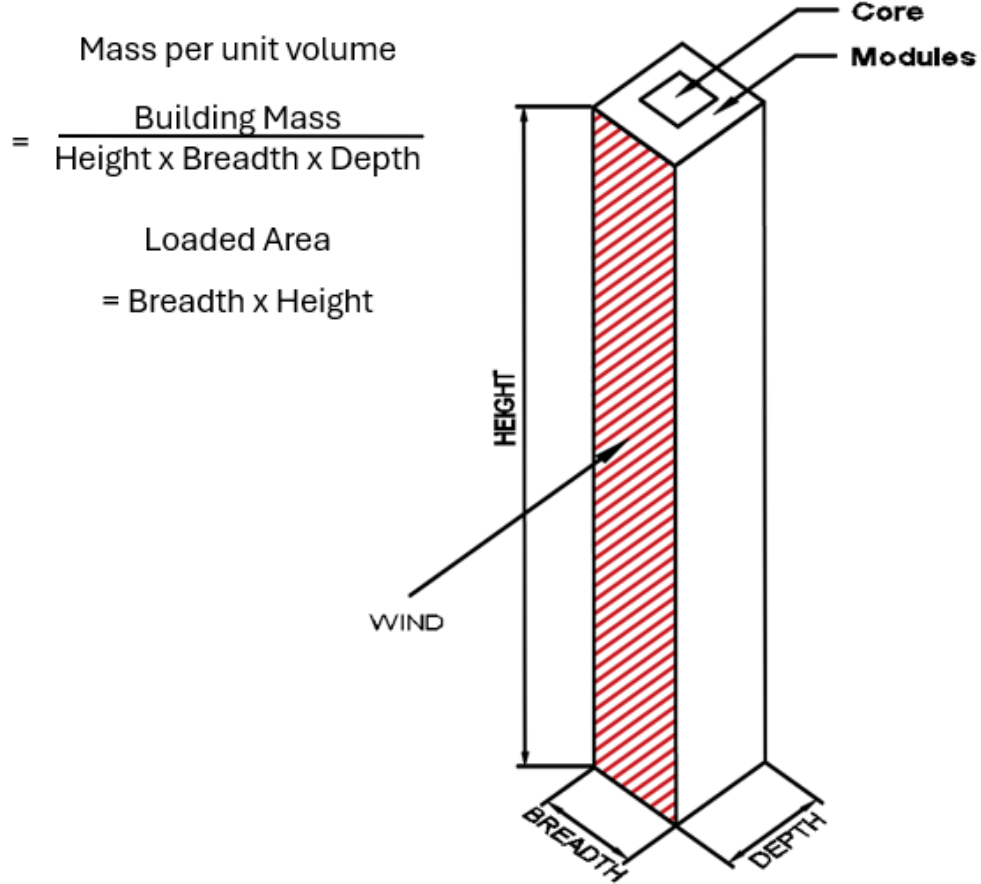


Figure 3.2: Simple representation of archetype building investigated

3.2 Generation of Structural Dataset

A sample set of 6,125 archetype buildings was generated in order to assess the effects of modifying building core dimensions and of adding auxiliary damping by means of a TLD. The sample buildings are all of single tower, single core form, generated for each possible combination of the parameters listed in Table 3.1. These parameters were chosen as they cover a range of values which are representative of a series of design possibilities for high-rise modular buildings.

Table 3.1: Archetype building properties investigated

Property	Symbol	Archetype Property Values
Building Height (m)	H	100, 110, 120, 130, 140, 150, 160
Mass Excluding Core (Mg/m^3)	M_{exC}	0.14 ,0.17, 0.20, 0.23, 0.26, 0.29, 0.32
Building Breadth (m)	B	20, 25, 30, 35, 40
Building Depth (m)	D	20, 25, 30, 35, 40
Core Wall Thickness (mm)	t_{core}	400, 450, 500, 550, 600

Total building mass is calculated separately for the core and surrounding structure. This approach allows the effects of changing the core dimensions on the building mass, and ultimately acceleration, to be fully assessed. The mass of the core, M_c , is calculated using the core dimensions and density, which is taken as $2,500 \text{ kg/m}^3$, a value typical of highly reinforced concrete. The mass of the surrounding structure, M_{exC} , is one of the parameters varied in the generation of the archetypes. Typical timber or steel framed modules have an approximate self weight, W_m , of $4 - 6 \text{ kN/m}^2$, with modules that include thicker concrete floor slabs having self weights up to 9 kN/m^2 [4, 29]. Therefore, acceleration response is investigated for archetype buildings with mass per unit volume of 0.14 Mg/m^3 to 0.32 Mg/m^3 , which corresponds to module self weights from 4 kN/m^2 to 9 kN/m^2 for a 3 m storey height, as in eq. 3.1:

$$W_m = M_{exC} \times g \times h \quad (3.1)$$

where g is the acceleration due to gravity taken as 9.81 m/s^2 and h is the storey height of the archetype.

The core breadth and depth, i.e. the outer core dimensions, labelled b_{core} and d_{core} respectively, are calculated as building height/13, but these are limited to a maximum of building breadth or building depth/3 in the relevant direction, meaning that:

$$d_{core} = \min \left(\frac{Height}{13}, \frac{Depth}{3} \right) \quad (3.2)$$

$$b_{core} = \min \left(\frac{Height}{13}, \frac{Breadth}{3} \right) \quad (3.3)$$

with these values being rounded to the nearest 250 mm.

The core wall thickness was limited to between 400 mm and 600 mm, which are typical values for buildings up to 200 m tall [210].

The fundamental natural frequency of each archetype was calculated using eq. 3.4, which is the standard expression for the natural frequency of a cantilever with a uniform mass per unit length.

$$f_s = \frac{3.5161}{2\pi} \times \sqrt{\frac{EI}{MH^3}} \quad (3.4)$$

where E is the Young's Modulus of the reinforced concrete in the core, I is the second moment of area of the core and $M = M_C + M_{exC}$ is the total mass of the archetype, considering both the core and surrounding modules. Therefore, the lateral stiffness of each archetype was assumed to come entirely from the building core with no contribution from the modules, which as discussed previously is a common design assumption. However, modules are considered in the calculation of the building mass, and consequently the natural frequency. The loaded area also takes into account the presence of modules. Therefore, following typical design assumptions and for this simplified study, modules are assumed to not contribute to lateral stiffness, however their presence does strongly influence wind load and structural response.

3.3 Calculation of Building Acceleration Response

3.3.1 Calculation Process

The peak acceleration response of each archetype building is estimated using the method proposed in Eurocode 1991-1-4 Annex C. The calculations are performed assuming a wind climate typical of the London region. The fundamental basic wind velocity, $v_{b,0}$, is 21.50 m/s , corresponding to the value specified by the UK National Annex to the Eurocode 1991-1-4 [73]. The surrounding terrain is assumed to be urban, or Class IV in Eurocode; the 10-minute mean wind velocity, $v_m(z)$, and the turbulence intensity, $I_v(z)$ are calculated using these assumptions.

Return period, which as mentioned is 1 year for the ISO limits, is considered in Eurocode 1991-1-4 through the c_{prob} factor, which is calculated using eq. 3.5:

$$c_{prob} = \left(\frac{1 - 0.2 \ln(-\ln(1 - p))}{1 - 0.2 \ln(-\ln(0.98))} \right)^{0.5} \quad (3.5)$$

where p is the annual probability of exceedance. This is often assumed to be the inverse of the return period, however this assumption is not valid for short return periods like 1 year. In this study a Poisson distribution is assumed [33] and annual probability of exceedance is calculated using eq. 3.6:

$$p = 1 - e^{-1/T} \quad (3.6)$$

For a return period, T , of 1 year this leads to a c_{prob} value of 0.749. Once the natural frequency and wind excitation have been established, the peak acceleration of each archetype structure can be calculated using the Eurocode 1991-1-4 Annex C approach. This is based on the spectral approach initially proposed by Davenport [211], with Steenbergen et al [212] providing a detailed derivation. In this approach the standard deviation of the along-wind acceleration, $\sigma_{\ddot{x}}$, is given by:

$$\sigma_{\ddot{x}} = c_f \times \rho \times I_v(z_s) \times V_m(z_s)^2 \times \frac{K_y \times K_z \times R}{\mu_{ref}} \quad (3.7)$$

3.3. Calculation of Building Acceleration Response

where c_f is the force coefficient (dependent on building shape), ρ is the density of air, $I_v(z_s)$ is the turbulence intensity at height z_s ($z_s = 0.6H$ for regular buildings), $V_m(z_s)$ is the mean wind speed at height z_s , K_y and K_z are non-dimensional coefficients representing aerodynamic admittance and are functions of mode shape and μ_{ref} is the mass per unit area. R is a factor which represents resonance between the building's natural frequency and wind gusts given by:

$$R^2 = \frac{\pi^2}{2\delta} \times S_L(z, f) \times K_s(f) \quad (3.8)$$

where $K_s(f)$ is a size reduction function accounting for variation in the wind load over the height of a tall structure, δ is the logarithmic decrement of damping and $S_L(z, f)$ is the value of the non-dimensional wind speed spectrum given by the equation:

$$S_L = \frac{6.8f_L(z, f)}{(1 + 10.2f_L(z, f))^{\frac{5}{3}}} \quad (3.9)$$

where f_L is a non-dimensional frequency defined in the code as a function of the local wind climate and building natural frequency.

In this study, the inherent damping of each archetype was assumed to be 1.27%. This figure was obtained from the value for the logarithmic decrement of damping, 0.08, recommended by EC 1991-1-4 Annex F for mixed concrete and steel composite structures. As previously mentioned, there is a large level of uncertainty surrounding the appropriateness of this damping value for use in the design of modular buildings. This is addressed further in Chapter 5.

The peak response is calculated from the standard deviation using a standard gust factor approach [211], in which the peak acceleration is given by:

$$\ddot{x}_{max} = k_p \times \sigma_{\ddot{x}} \quad (3.10)$$

where the gust factor, k_p is calculated as:

$$k_p = \sqrt{2 \times \ln(600 \times f_s)} + \frac{0.6}{\sqrt{2 \times \ln(600 \times f_s)}} \quad (3.11)$$

where f_s is the building natural frequency, calculated in equation 3.4

There are a number of simplifications in this approach, primarily that only along-wind acceleration response is considered. To account for this any archetypes deemed susceptible to vortex shedding, which can induce significant across-wind accelerations, were removed from the dataset. This was done by calculating the critical wind velocity for vortex shedding for each archetype using the method given in Eurocode 1991-1-4 Annex E and removing any case in which this was less than 1.25 times the one-year wind velocity, which is the criteria specified by the Eurocode for a structure to be deemed susceptible to vortex shedding. This resulted in the removal of 66 archetypes (or approximately 1% of those considered) from the data set.

3.3.2 Calculation Results

The method described above was employed to calculate the peak acceleration responses of 6,059 archetype buildings employing all combinations of the parameters listed in Table 3.1, excluding those where vortex shedding was identified as a potential issue. A number of trends emerge from the results of these calculations. An example of a 120 m archetype is used to demonstrate these trends as shown in Figures 3.3, 3.4 and 3.5. It is evident that most buildings 120 m in height experience peak accelerations in excess of 4 milli-g and so, by comparison with Figure 2.11, potentially exceed the ISO habitability limit for residential buildings. It may also be seen that the peak acceleration is generally reduced for archetypes with larger mass per unit volume. This is as expected; accelerations are known to reduce with building mass, to the extent that some forms of construction deliberately include additional mass to control wind induced accelerations [39].

In Figure 3.3 it can be seen that for low mass the range of accelerations is quite large, with maximum values of up to 15 milli-g, but as mass increases this range becomes smaller. It is generally the aim of the module fabricator to reduce the

module self weight as this reduces material costs and waste, and improves construction time, ease of transport and lifting. However, this can negatively impact the dynamic performance of modular high-rise buildings with respect to wind induced accelerations.

Peak acceleration may be expected to reduce with increased wall thickness due to greater flexural stiffness and mass, however, Figure 3.4 indicates that this effect may not be highly significant. Average peak accelerations are seen to decrease slightly as core wall thickness increases, but this trend is weak. Moreover, the range of peak accelerations observed is similar for each core wall thickness value examined. These results suggest that the increased mass and stiffness across the range of core wall thicknesses examined are not sufficient to substantially reduce peak accelerations.

The effects of building shape on the peak acceleration are also quite clear. Figure 3.5 shows the predicted accelerations for the 120 m archetypes; it is observed that as the breadth of the building (dimension perpendicular to the acting wind force) decreases, so too does peak acceleration. The reverse is true for the depth of the building (dimension parallel to the acting wind force), with the peak acceleration decreasing as this increases. This is due to the extra mass and greater core depth, and therefore stiffness, of a deeper building. The variation of peak acceleration with breadth is not as stark, as the increase in mass and subsequent acceleration reduction is offset by the increase in area loaded by the wind.

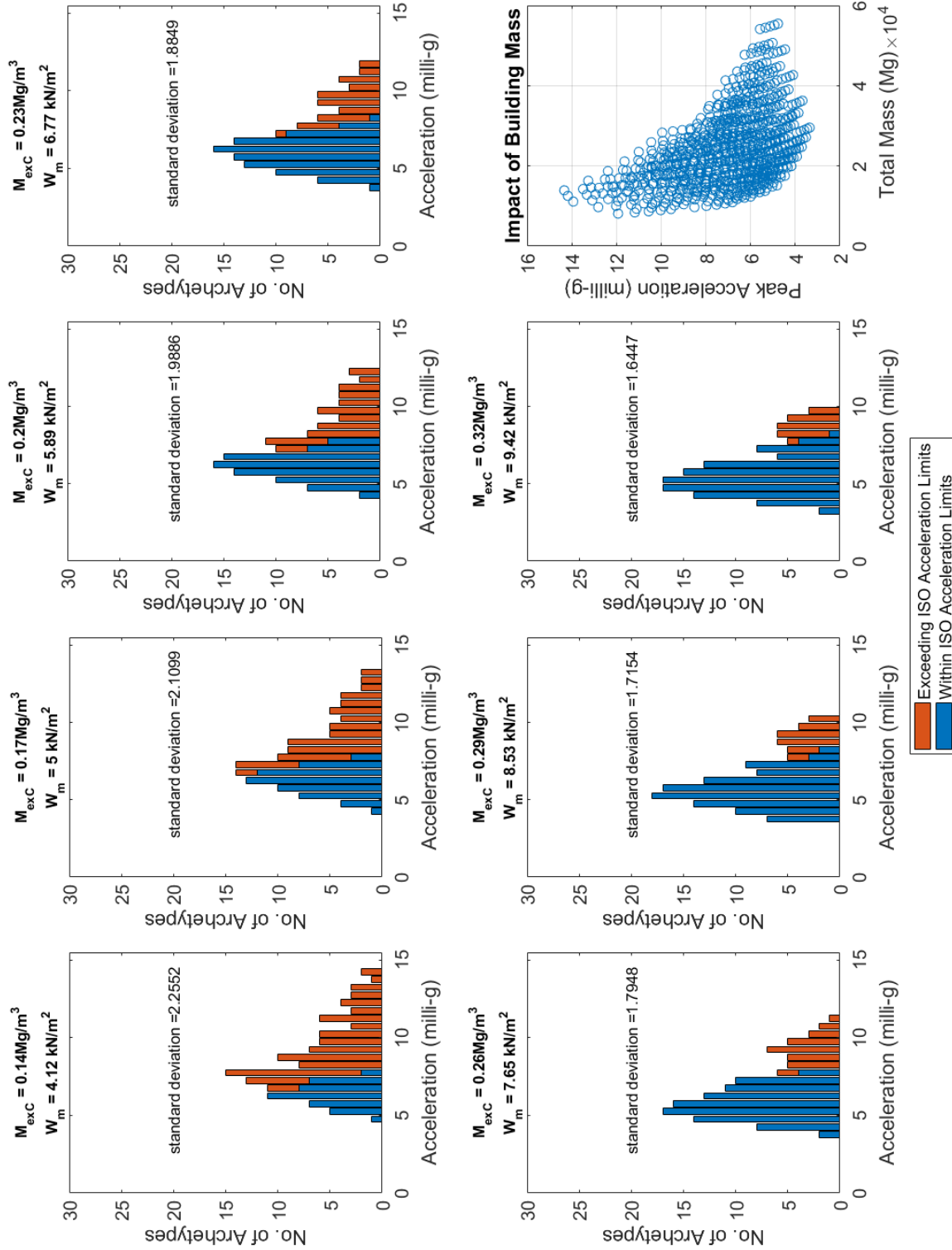


Figure 3.3: Relationship between peak acceleration and building mass, shown in both mass per unit volume (Mg/m^3) and module self weight (kN/m^2), for a 3 m storey height) for the 120 m archetypes

3.3. Calculation of Building Acceleration Response

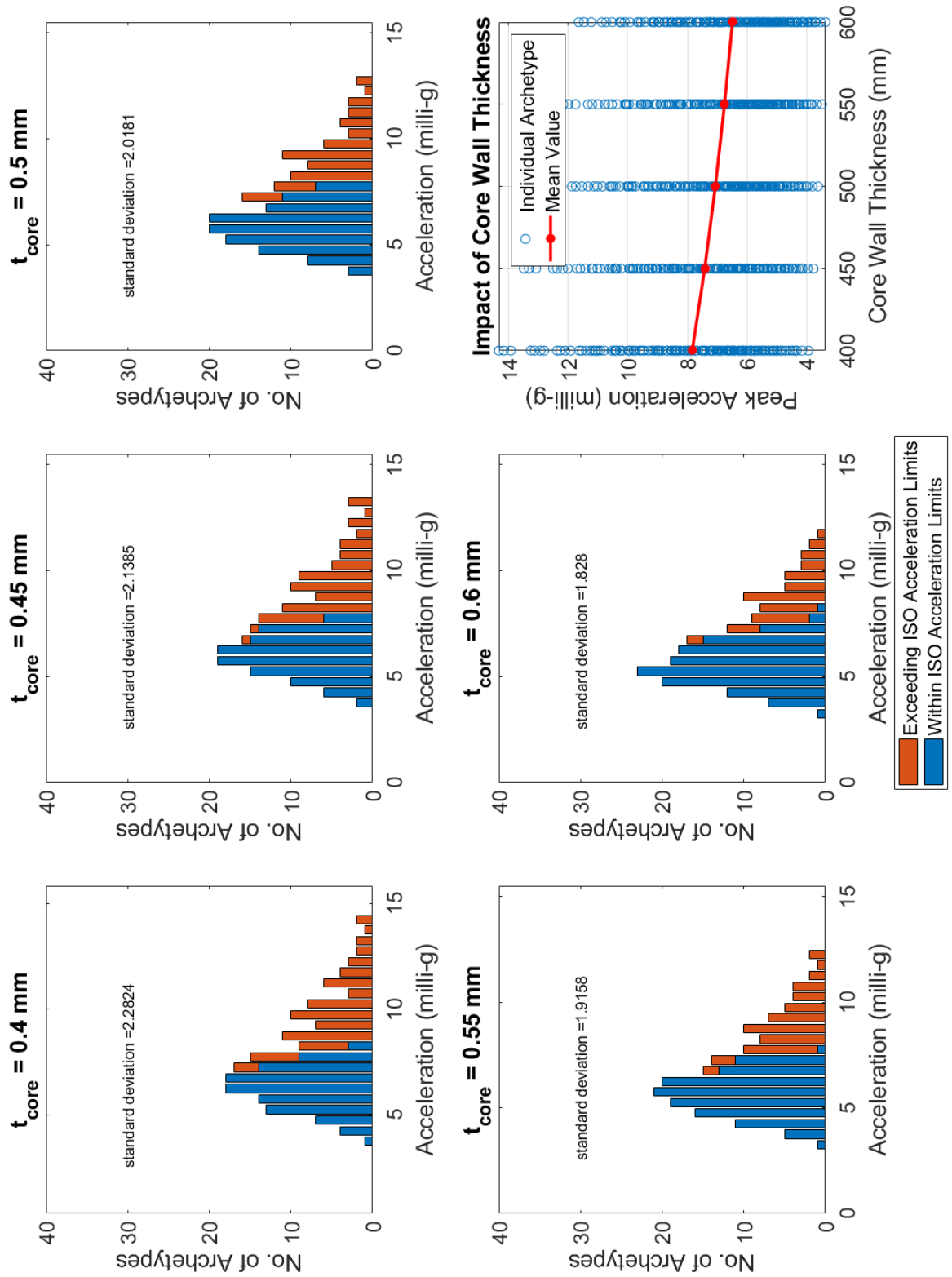


Figure 3.4: Relationship between peak acceleration and core wall thickness for 120 m Archetypes

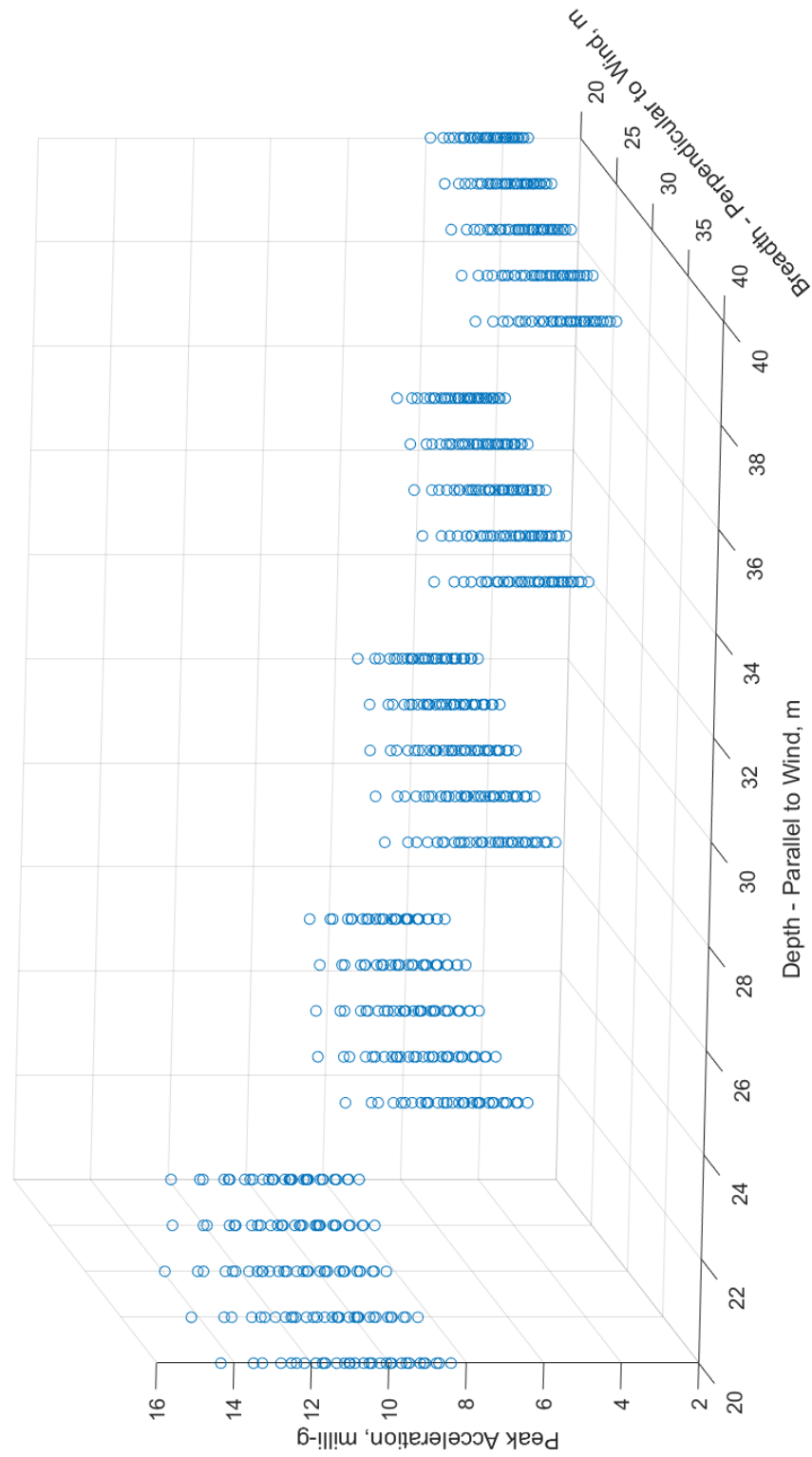


Figure 3.5: Impact of building plan dimensions on peak accelerations for 120 m archetypes

3.3.3 Required Acceleration Reduction

For each archetype, the difference between the predicted peak acceleration from eq. 3.10 and the frequency dependent ISO peak acceleration limit for a residential building was calculated. Of the 6,125 archetypes considered, 3,615 fall within ISO acceleration limits for residential buildings and require no additional measures to meet serviceability criteria. Modification of the remaining 2,510 archetypes was required to reduce the wind induced response and eliminate acceleration serviceability issues. Figure 3.6 shows the extent of the acceleration reduction required for each archetype.

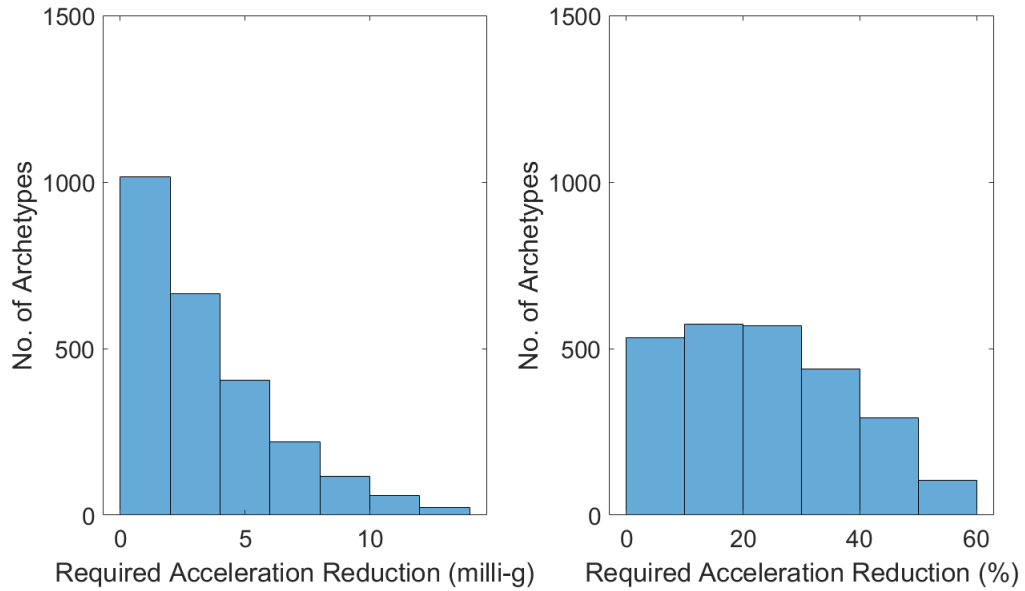


Figure 3.6: Required acceleration reduction for archetypes exceeding ISO acceleration limits

In most cases an acceleration reduction of less than 5 milli-g is required to satisfy ISO acceleration limits. However, there are some extreme cases in which archetypes require as much as a 15 milli-g reduction in accelerations. In percentage terms, for the majority of cases where the acceleration limit is exceeded, less than a 40% reduction in acceleration is required. However in a small number of scenarios, reductions of up to 60% are required. The extreme cases which require reductions in the order of 15 milli-g or 60% are often somewhat unrealistic archetypes where the combination of parameters from Table 3.1 leads to a structure unlikely to be constructed in reality; for example the archetype in the dataset with the highest acceleration value is a 160 m tall structure with a 400 mm thick, 6.75 m deep core.

If only cases with core slenderness less than 16 are considered, 1,033 archetypes out of 4,550 require remedial action, with this ratio reducing to 280 out of 2,625 for structures with core slenderness less than 13.

Figure 3.7 shows the percentage of archetypes requiring acceleration reduction interventions for the different module self weights considered. It can be seen that for lower module self weights, which correspond to the values typically seen in timber modules, the majority of accelerations exceed the ISO limit irrespective of building height. However, as module self weight is increased, the number of archetypes exceeding the limits decreases and building height begins to have a greater influence on whether or not the limit will be exceeded.

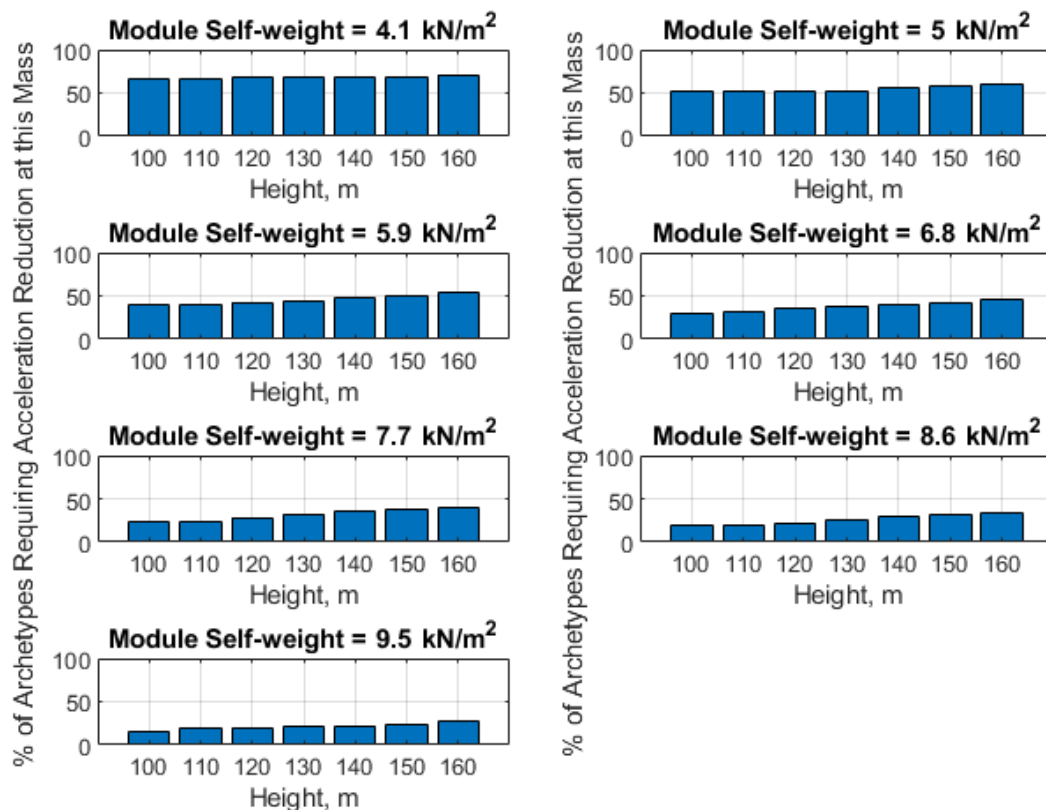


Figure 3.7: Number of archetypes requiring acceleration reduction by module self weight and building height

3.4 Modifications to Mitigate Excessive Accelerations

In the cases where the peak acceleration of an archetype exceeds the ISO limits, design modifications are required to meet serviceability criteria. As mentioned, in this study the modification methods considered to reduce accelerations are an increase in core dimensions and the installation of a TLD.

3.4.1 Required Increase in Core Dimensions

For each archetype, the increase in core dimensions, if any, required to reduce response accelerations sufficiently to satisfy the ISO residential limit was determined. It is important to note that the core wall outer dimensions, rather than core wall thickness, was altered to reduce accelerations. Core wall thickness is one of the parameters varied in the generation of the archetypes, but not adapted in the optimization, as Figure 3.4 showed that this is not an efficient response mitigation approach. Instead the breadth and depth of the core are increased. This concept is illustrated in Figure 3.8.

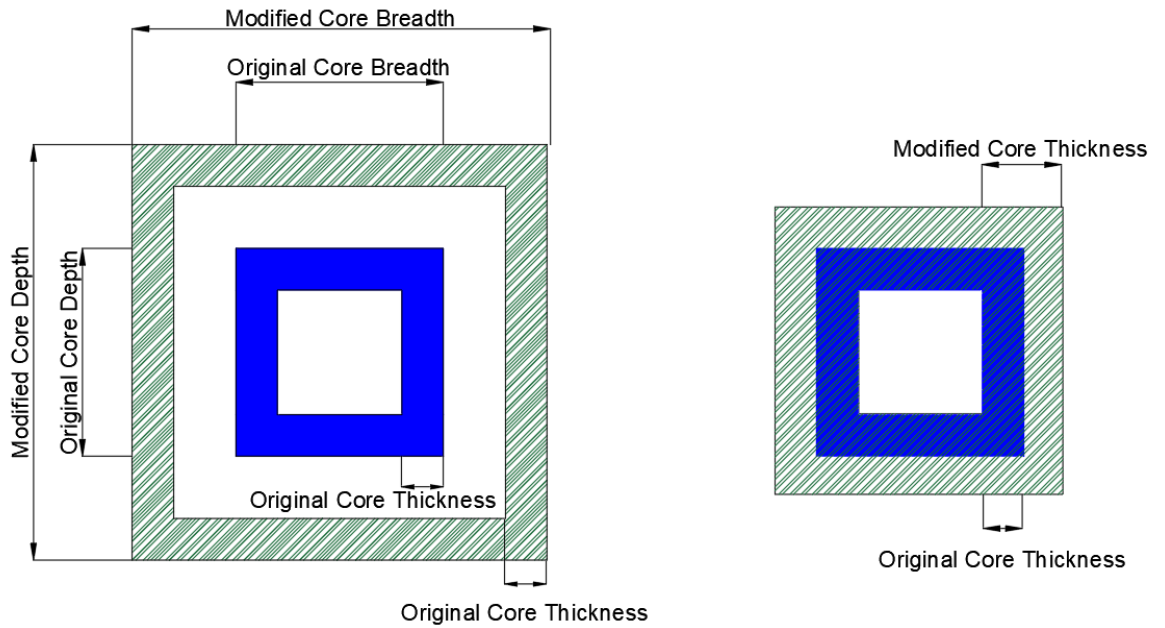


Figure 3.8: Illustration of increasing core outer dimensions (left) and increasing core thickness (right). The method on the left was employed in this study

An incremental approach was employed to find the increase in core size required to bring the peak acceleration within ISO limits. The outer and inner core dimensions were increased in 100 mm steps and the peak acceleration recalculated for the modified archetype using the methods described in Section 3.3.1. This process of core resizing and acceleration recalculation was repeated until the acceleration of the archetype with increased dimensions was found to be less than the ISO residential limits, as illustrated in Figure 3.9 for two archetypes.

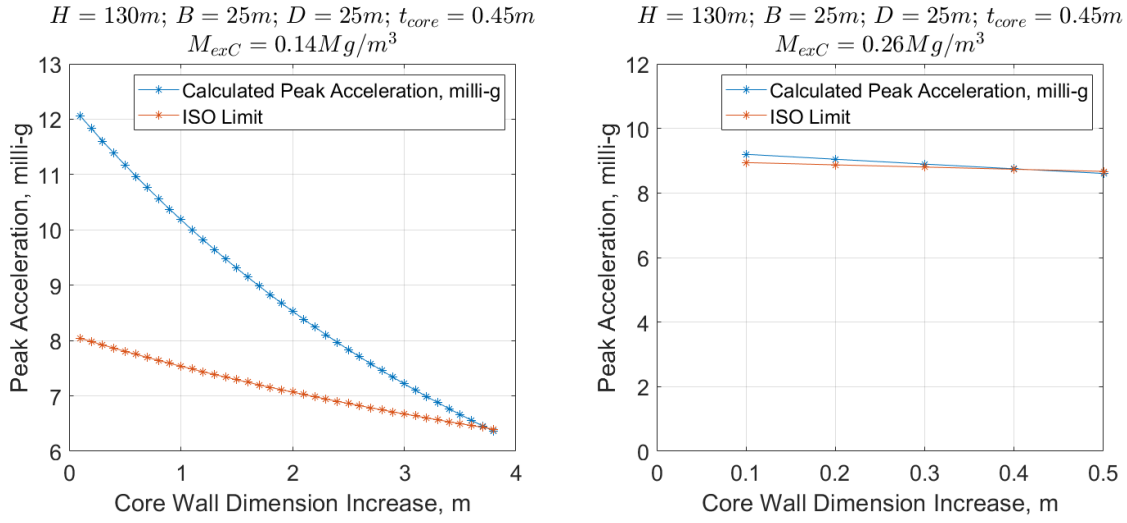


Figure 3.9: Incremental increase in core dimensions to reduce response acceleration below ISO limits for two example archetypes

The results of this process for all archetypes are shown in Figure 3.10 and 3.11. The rate of acceleration reduction with increased core outer dimensions appears relatively low. Therefore, quite large increases in core dimensions are often required to bring accelerations within the serviceability limits. In many cases these increases are in the order of meters; in some instances it is above 5 meters. Figure 3.11 shows that the majority of cases do not require increases in core dimensions as extreme as 5 meters even though these instances tend to dominate Figure 3.10 visually.

From examining Figure 3.10 it can be seen that an increase in core dimensions of 1000 mm leads to an average 10% decrease in peak accelerations. This relationship becomes clearer when only the archetypes which required less than a 2000 mm increase in core dimensions are considered as in Figure 3.12.

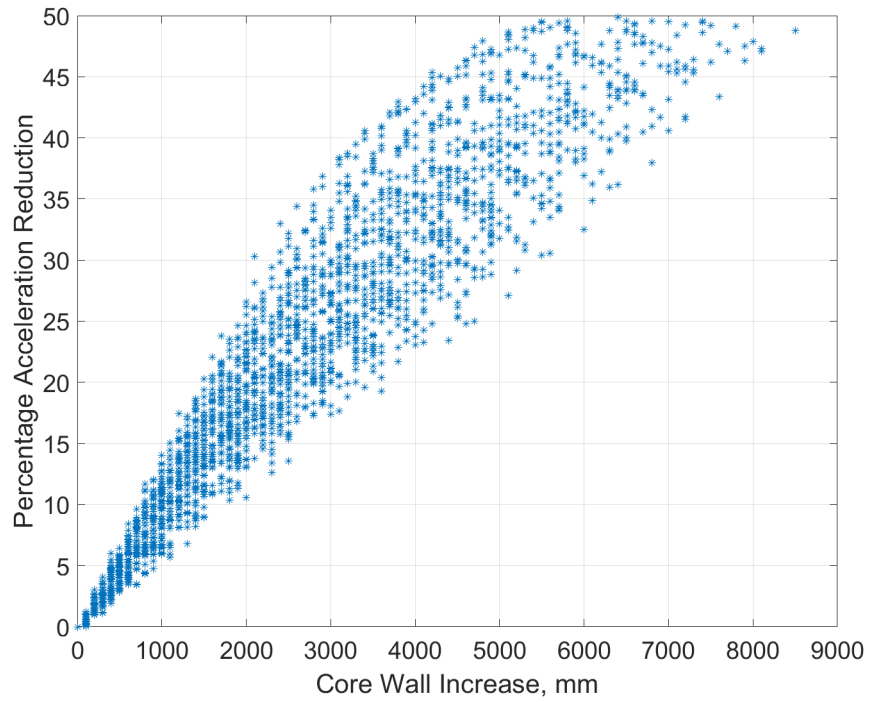


Figure 3.10: Required increase in core dimensions to achieve percentage reduction in acceleration response

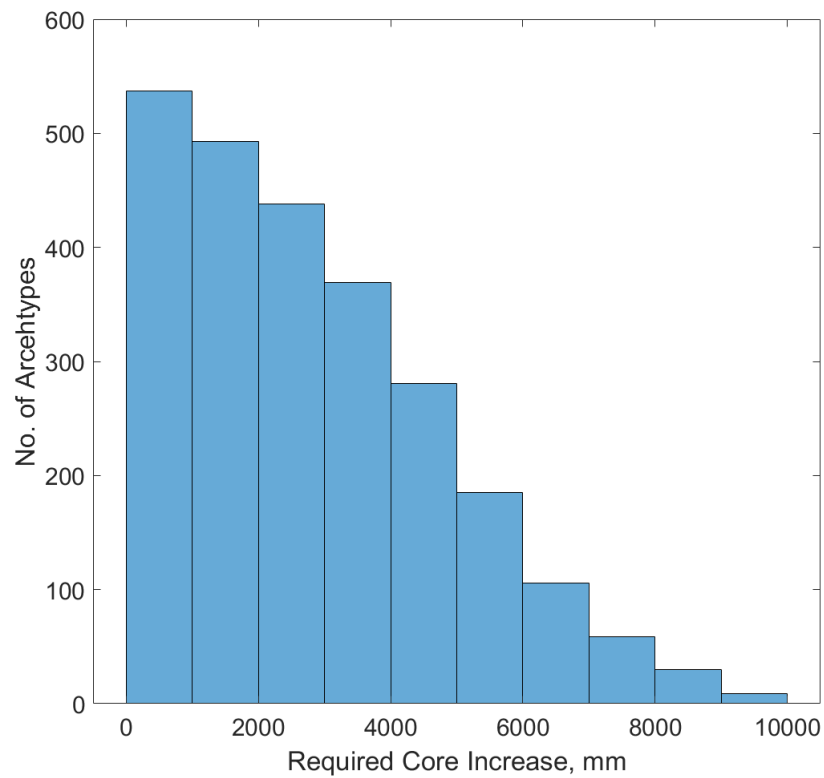


Figure 3.11: Extent of core increase required to satisfy acceleration limits

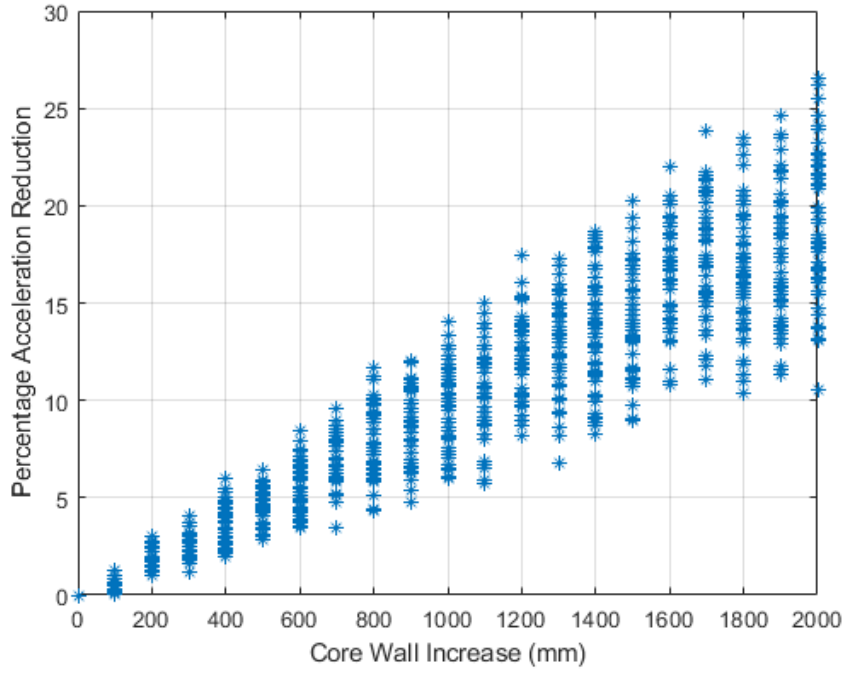


Figure 3.12: Percentage reduction provided by increasing core dimensions for cases where less than a 2000 mm increase is required

The feasibility of any increase in outer core dimensions is likely to be dictated by the financial loss associated with the loss of saleable floor area. Assuming the overall building footprint cannot be increased, increasing the core dimensions reduces the floor area outside the core. For tall modular construction, where typically the saleable area is limited to the area outside the core, even relatively minor increases in core dimensions can result in a substantial reduction in the value of a development. A square metre of residential property in London was valued at £8091 in 2019 [213]. Assuming only floor area outside the core can be sold, the reduction of income for the developer can be estimated as the product of the lost floor area per story, the number of storeys and the value of saleable floor area. For example, for a 100 m tall archetype with an 8 x 8 m core and 3 m storey height, a 100 mm increase in core outer dimensions translates to a loss in income of approximately £430,000 ($1.61 \text{ m}^2/\text{storey} \times 33 \text{ storeys} \times 8091 \text{ £/m}^2$), a 1000 mm increase equates to £4,500,000 and a 5000 mm increase leads to a loss of just under £30,000,000. Although these relations are approximate, it can be reasonably asserted that for tall buildings any increase in core dimensions greater than several hundred millimetres is likely to be prohibitively expensive, particularly as a response measure to address a serviceability limit state issue. Therefore, comparing the cost and the extent of the

required increases, it can be concluded that resizing core dimensions does not offer a feasible mitigation approach for the vast majority of potential modular buildings in which response acceleration is a design issue.

3.4.2 Required Additional Damping

A similar procedure was employed to calculate the additional damping required to bring the acceleration of each archetype below the ISO residential limit. As before, this was done in increments, by increasing the damping of the archetype in steps of 0.1%, and recalculating the peak acceleration as described in section 3.3.1 until it reduces below ISO limits.

Figure 3.13 shows the additional damping above the assumed inherent value of 1.27% required to bring accelerations below the ISO limits. Also shown is the analytical relationship between additional damping, ζ' , and percentage reduction in acceleration $\Delta\ddot{x}$, which can be derived from manipulation of eq. 3.7 and the standard expression for the relationship between logarithmic decrement and damping ratio to give:

$$\Delta\ddot{x} = 100 \left(1 - \left(\frac{1}{1 + \frac{2\zeta\zeta' + \zeta'^2}{\zeta^2 - \zeta^4 - \zeta^2\zeta'^2 + 2\zeta^3\zeta'^2}} \right) \right) \quad (3.12)$$

As mentioned previously, the majority of archetypes examined require a reduction in peak acceleration of 40% or less. Figure 3.13 shows that a 40% reduction can be achieved by providing additional damping of 2.4%.

In Figure 3.14, the additional damping and additional core depth required to satisfy the ISO limit are compared for each archetype. While the trend displayed is not linear, an approximate relationship which can be inferred from this data is that roughly a 2100 mm increase in core wall depth and breadth corresponds to a 1% increase in inherent damping.

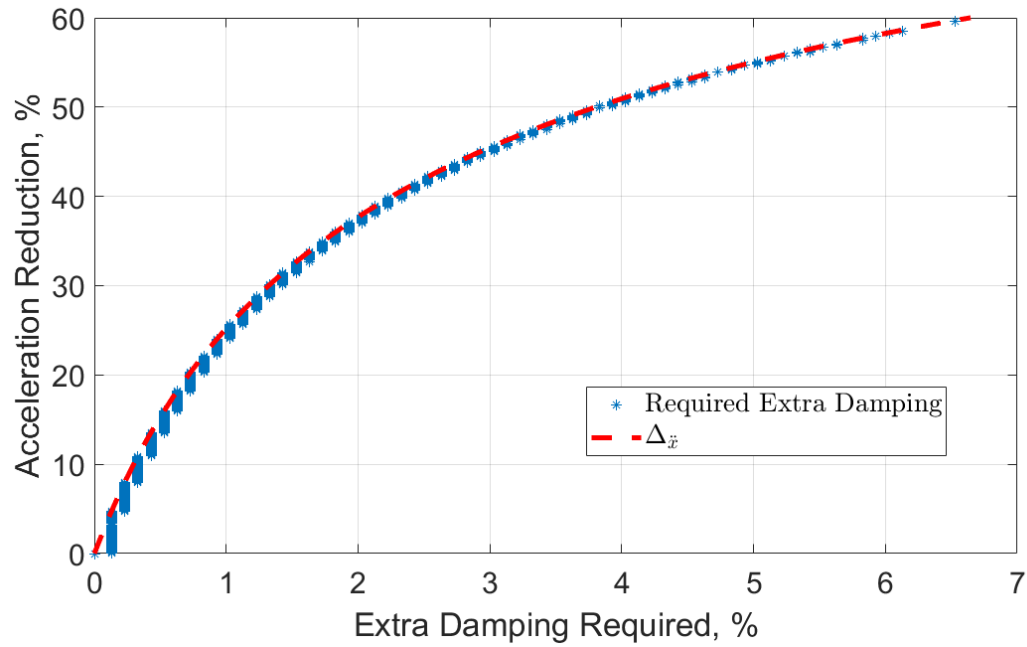


Figure 3.13: Percentage reduction in acceleration provided by increase in damping

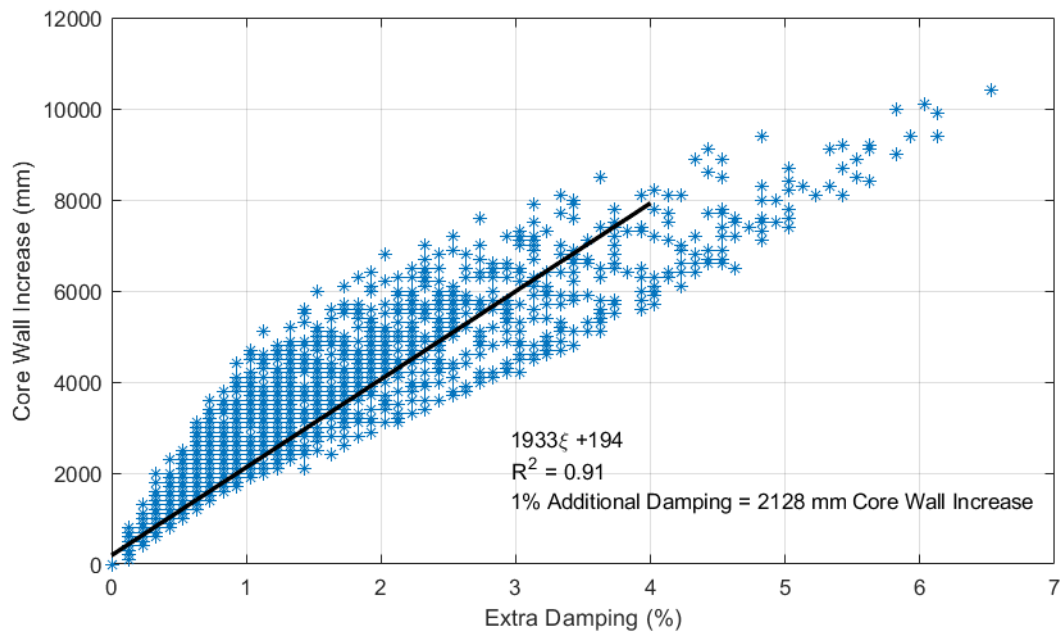


Figure 3.14: Comparison of required increase in core depth and required additional damping for each archetype

3.4.3 Additional Damping Provided by TLD

In order to assess whether the required additional damping can feasibly be provided, the maximum damping which can be provided by a TLD given the characteristics of each archetype was estimated using the method proposed by Tait [155] for the preliminary design of a TLD. The method proposed by Tait [155] models the TLD using equivalent properties of a TMD, which has a more straightforward design process.

In the assessment detailed below it is assumed that, as in Annex F of Eurocode 1991-1-4, the additional damping from a TLD can simply be added to the inherent structural damping to give an overall value for damping. A schematic of a typical TLD with dimensions labelled is shown in Figure 3.15, where h is the depth of water in the TLD, b_d is the breadth of the TLD and L_d is the length of the TLD.

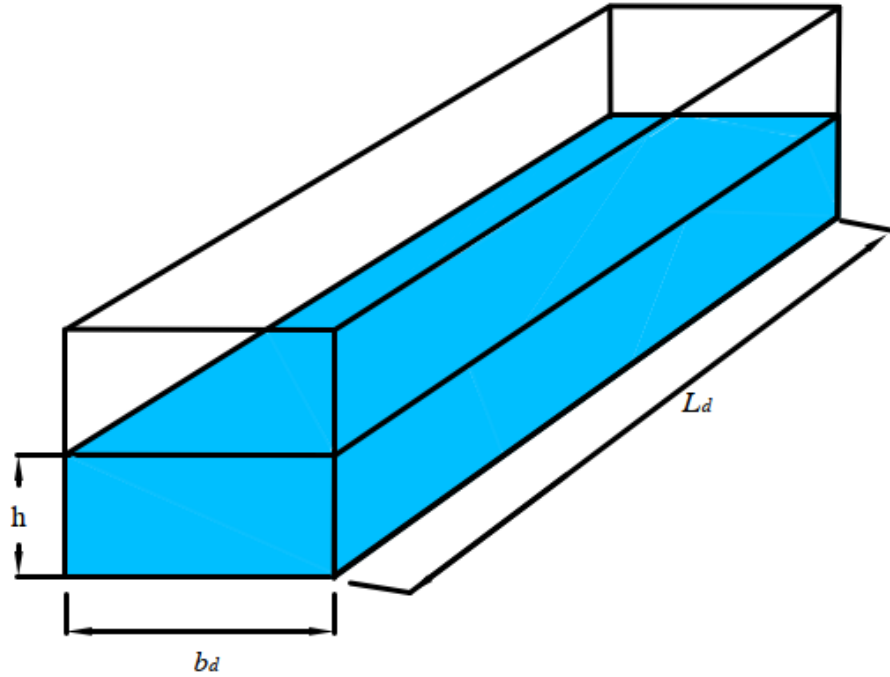


Figure 3.15: Schematic of TLD

As proposed in Tait [155], the method begins with the additional damping required, and a series of steps are followed to calculate the damper dimensions needed to provide this level of damping. The process is reversed in this study; damper plan dimensions, i.e. damper breadth, b_d , and depth, L_d (see Figure 3.15), are established based on the archetype properties and then the level of damping

provided by a damper with these dimensions is calculated. The steps undertaken to do this are as follows:

- Using the plan dimensions of the TLD and the natural frequency of the archetype, f_s , the height of water in the TLD, h , for which the sloshing frequency matches the building natural frequency is calculated as:

$$h = \frac{L_d}{\pi} \tanh^{-1} \left(\frac{4 \pi f_s^2 L_d}{g} \right) \quad (3.13)$$

- Once the height of water and the damper plan dimensions are established, the equivalent mass of water sloshing in the fundamental mode can be calculated as:

$$m_{eq} = \frac{8 \rho b_d L_d^2}{\pi^3} \tanh \left(\frac{\pi h}{L_d} \right) \quad (3.14)$$

- Using this value of m_{eq} , the mass ratio of the damper, μ , can then be calculated using eq. 3.15:

$$\mu = \frac{m_{eq}}{M_s} \quad (3.15)$$

where M_s is the first modal mass of the building, which in this study is estimated as the first modal mass of a cantilever with a uniformly distributed mass, plus the mass of the non-participating component of the liquid associated with the fundamental sloshing mode.

- Once μ is calculated, an estimate of the additional damping provided by a TLD can be made using eq. 3.16 [155, 214] :

$$\zeta_{eff} = \frac{1}{4} \sqrt{\frac{\mu + \mu^2}{1 + \frac{3\mu}{4}}} \quad (3.16)$$

In the first step of this process it is assumed that the TLD is tuned to the building natural frequency. However the optimal sloshing frequency of a TLD, f_d , is given by eq. 3.17.

$$f_d = f_s \frac{\sqrt{1 + \frac{\mu}{2}}}{1 + \mu} \quad (3.17)$$

Examination of eq. 3.17 shows that for the low values of mass ratio (less than 0.1) typically encountered in TLD design, f_d will be very close to f_s . Therefore, at least at the preliminary design stage, it is reasonable to assume that the TLD can be tuned to the natural frequency of the archetype. This assumption avoids the need for any iteration in the calculation process.

The process outlined above is initialised by establishing the damper dimensions. For this study, this is done by constraining the maximum plan dimensions of the TLD, L_d and b_d , to match the core dimensions. This means that the TLD can be positioned on top of the archetype core without extending over the surrounding modules. This constraint ensures that no additional vertical load is imposed on the modules. Given that tall modular construction is often executed using a crane supported on top of the core, the critical vertical design load is often the temporary case during construction. Therefore, post-construction when the crane is dismantled, there is often unutilised vertical load carrying capacity in the core that can be exploited to support the mass of a damper. Consequently, by limiting the dimensions to match the core, it can be assumed that the damper can be installed without requiring consideration of additional dead load imposed on the modules. The alternative approach of strengthening the modules so that they can support some of the damper is not considered here. Distribution of multiple small TLDs which suit the form of modular construction are investigated in Chapter 7.

Figure 3.16 shows the results of this calculation process for the 120 m and 150 m archetypes. It can be seen that for these archetypes, the estimated feasible additional damping ranges between approximately 0.5% and 3%. This value is a function of the mass ratio (eq. 3.15), which depends on the first modal mass of the building and the size of the damper that can be installed, which in this study is controlled by the core size. Therefore, the amount of damping that can be provided generally increases with core size, as a larger core leads to a larger damper and a greater mass ratio, but decreases with building mass, as a larger building mass increases the modal mass and therefore reduces the mass ratio. Figure 3.16 demonstrates this for the 120 m and 150 m archetypes.

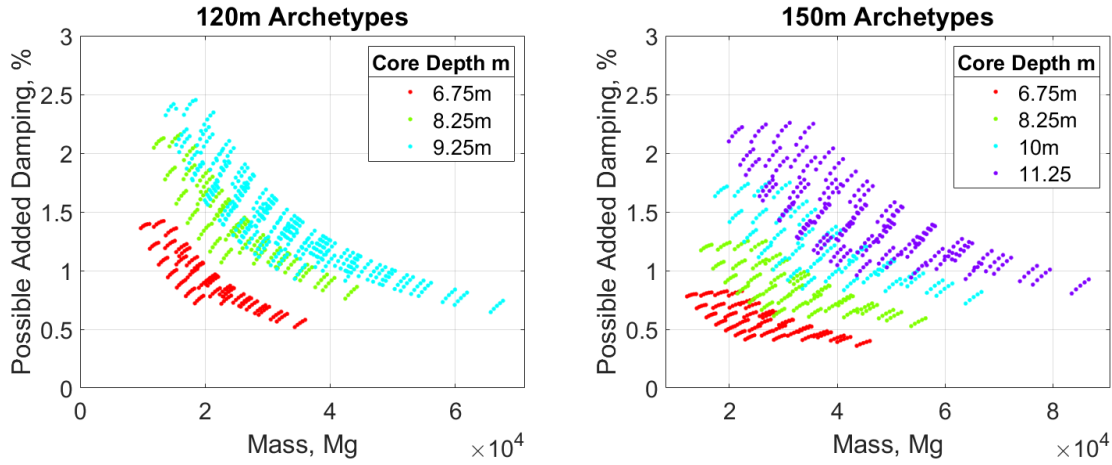


Figure 3.16: Variation in added damping with building core size and total mass for the 120 m and 150 m archetypes

In Figure 3.17, the maximum possible additional damping available for each archetype is shown in blue, while the damping required to bring the accelerations within ISO acceleration limits is shown in red. The Archetypes are numbered incrementally, therefore the possible additional damping for each archetype (blue) corresponds directly to the required additional damping (red) at the same location on the x-axis. In order for the TLD to provide sufficient auxiliary damping, the maximum damping available from the TLD must exceed the required damping. The charts show that whilst the possible additional damping exceeds the required additional damping for the majority of archetypes, as building height increases fewer archetypes can be made to satisfy the ISO serviceability limit by the addition of a TLD. This is because the required level of damping is generally greater for taller archetypes, as taller buildings tend to experience greater accelerations. However, the available additional damping tends to decrease with building height, as taller buildings have greater mass leading to TLDs with smaller mass ratios. Furthermore, in Figure 3.17 the mass of the building increases as the archetype number increases, i.e. the mass increases moving from left to right in each plot. Hence, it can also be seen that it is more difficult to control accelerations with a TLD in buildings with lighter modules.

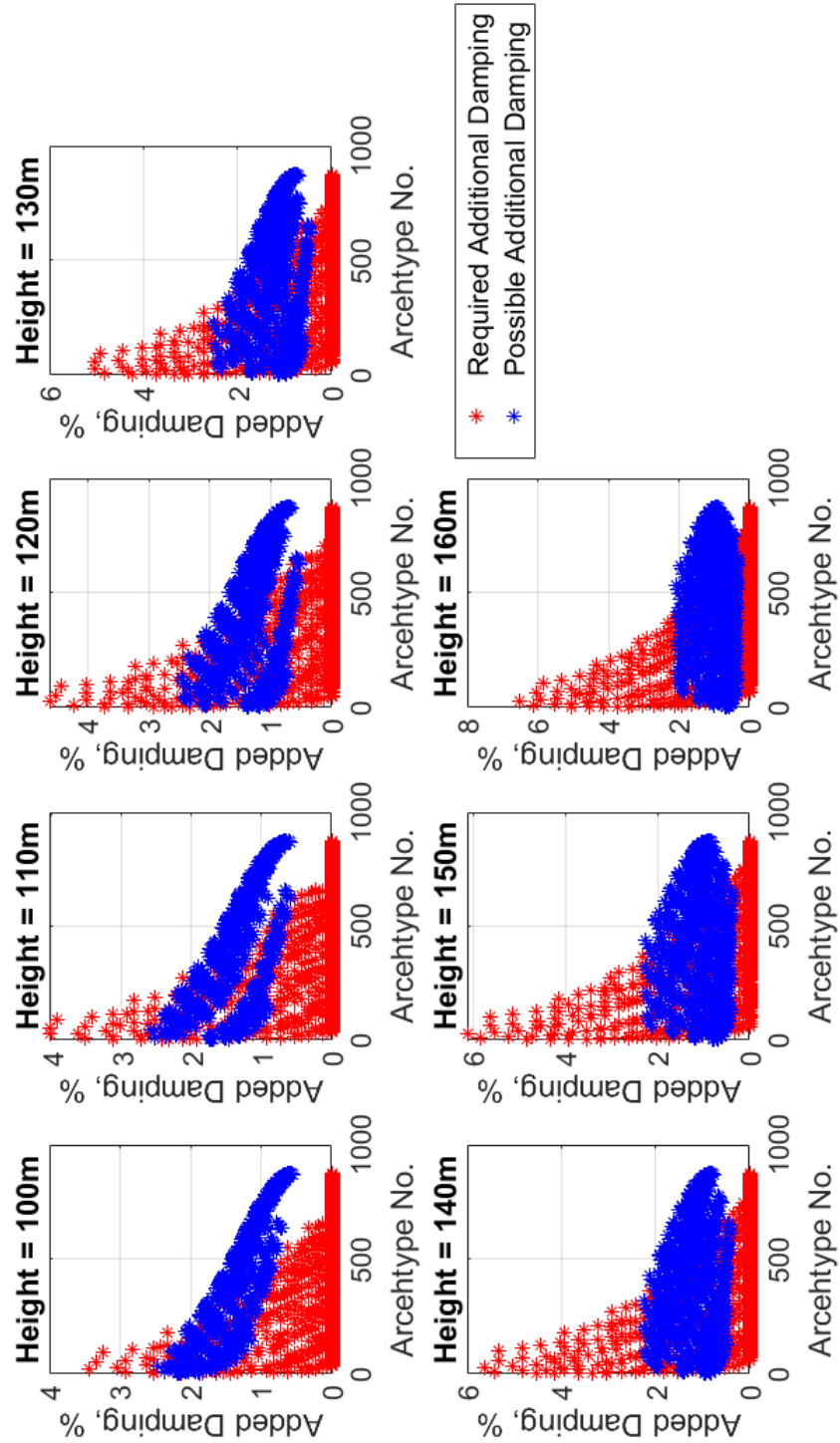


Figure 3.17: Comparison of required damping and predicted available damping from an optimal TLD for archetypes

3.5 Maximum Feasible Height of Modular Buildings

The methods presented above for calculating the response acceleration of a high-rise modular tower with a TLD tuned to the fundamental frequency of the building are used to evaluate the maximum feasible height that a modular building can attain without violating the serviceability limit state. It is shown that there is no unique value for the maximum height of a single core building which can be controlled by a TLD due to the dependence of acceleration response on both building mass and plan dimensions. However, it is possible to develop a series of curves which show the maximum feasible height as a function of building plan dimensions for a given mass per unit volume (excluding core) and core wall thickness.

For a series of archetypes of incrementally increasing height, with a specific module self weight and core wall thickness, the peak acceleration was predicted using the method described in Section 3.3.1. The additional damping required and the maximum damping a TLD can provide were calculated using methods described in Sections 3.4.2 and 3.4.3. It is then possible to determine the maximum feasible height as the greatest height at which both: (1) maximum damping provided by the TLD is sufficient to maintain accelerations within the ISO limits for a residential building and (2) vortex shedding is not deemed problematic. Performing this process for a number of different building dimensions enables a curve to be generated which shows the approximate maximum feasible height of a building with a particular module self weight (excluding core) and core wall thickness given its smallest dimension on plan.

Figure 3.18 shows examples of such curves for two combinations of module self weights and core wall thicknesses. In the calculation framework adopted here, the critical case that dictates maximum height is that with the larger plan dimension perpendicular to the wind and the smaller plan dimension parallel to the wind, as this orientation results in the greatest loaded area and least stiffness. Examining the curves in Figure 3.18 the most obvious trend is that the predicted maximum achievable height rises as the smaller plan dimension, i.e. building depth, is increased. For example in all four plots the estimated maximum height increases from about 100 m to 200 m as the building depth increases from 20 m to 30 m. This increase is attributable to the increased mass and stiffness of a deeper building.

The larger plan dimension, i.e. the breadth perpendicular to the wind, has much less influence on estimated maximum height. This is because the increase in loaded area associated with a wider building is offset by proportional increases in mass and stiffness. By comparing the four different combinations of building mass and core wall thickness in Figure 3.18 it can be seen that estimated maximum feasible height increases with mass, and to a lesser extent, core wall thickness. However, the influence of these parameters is minor compared to that of the smaller plan dimension. It is noted that the curves are not smooth and trends are not uniform with building depth. There are a number of reasons for this; firstly the ISO limit is a piecewise function meaning the required acceleration reduction is not smooth, and secondly the assumptions that govern core dimensions in this study result in abrupt changes in building frequency, and therefore response, with height.

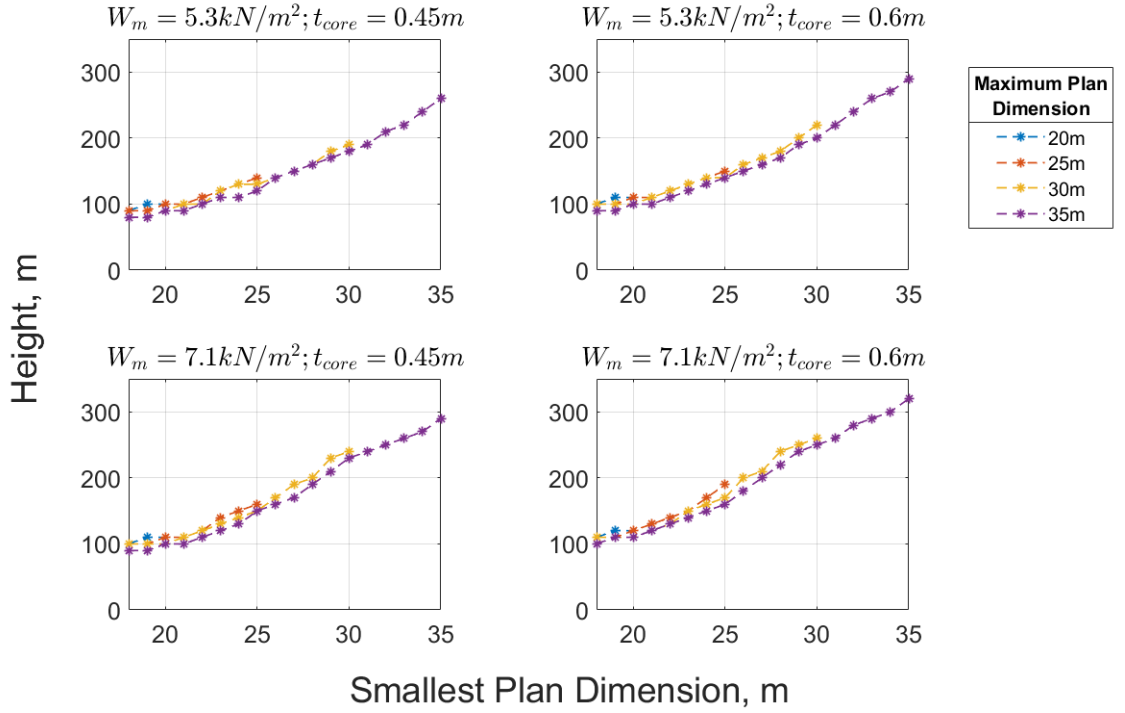


Figure 3.18: Approximated maximum feasible height of single core modular buildings with varying combinations of self weight equipped with a TLD (approximated for a 3 m storey height) and core wall thickness

The calculation process was also performed to determine the maximum feasible height of a modular building without a TLD. In this case, the tallest archetype for each combination of parameters which had an acceleration response within ISO limits is deemed the maximum feasible height. Figure 3.19 shows examples of the plots for the same two combinations of module self weights and core wall thicknesses

as in Figure 3.18. It is observed that the addition of a TLD increases the maximum feasible height of a single core modular building across each set of parameters. In particular, the possible height of modular buildings with lower module self weight is increased substantially with the increase less pronounced for structures with a larger self weight.

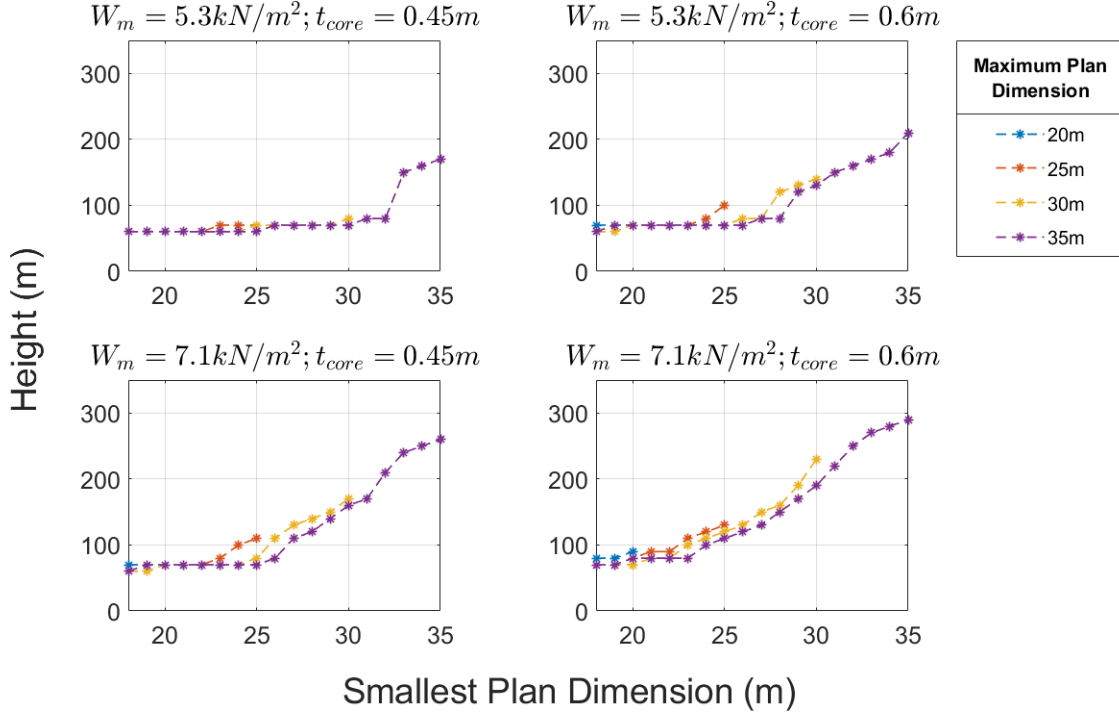


Figure 3.19: Approximated maximum feasible height of single core modular buildings with varying combinations of self weight with no supplementary damping mechanism (approximated for a 3 m storey height) and core wall thickness

Considering a single core modular building with a core wall thickness of 0.6 m, module self weight of 7.1 kN/m^2 and plan dimensions of 35 m as seen in the last plots of Figures 3.18 and 3.19, the maximum feasible height of the building without a TLD is 290 m whereas, with the addition of a TLD the maximum feasible height increased to 320 m. This is equivalent to an additional 10 stores of 3m storey heights, providing more than $12,000 \text{ m}^2$ of additional floor area. As previously discussed in Section 3.4.1, in 2019 property in London was valued at approximately $\text{£}8091 / \text{m}^2$ [213]. Therefore, in this example, an additional $\text{£}9.9$ million of value can be added to a single core modular building through addition of a TLD. However, this does not account for the cost of design and install of a TLD which is variable and not well reported. Nor does it account for the difference in property prices at higher storeys of a structure. Irregardless, it is clear that a TLD can aid in increasing the

height of a modular building, providing more available floor area for the same plan area.

3.6 Conclusions

The wind-induced acceleration responses of 6,125 archetype high-rise modular buildings have been evaluated and their dependency on the key parameters of core and module mass, core stiffness and inherent and added damping assessed. The effectiveness of two methods for controlling response accelerations have been compared: increasing the breadth and depth of the core and installing a TLD. Both methods were assessed for their ability to control peak accelerations of the archetypes within ISO 10137 acceleration limits for residential buildings.

Whilst both methods can achieve the desired acceleration reduction for the majority of the archetypes studied, the practicality of increasing core dimensions to control accelerations is limited. On average, to reduce accelerations by 10% the core breadth and depth must be increased by approximately 1000 mm; this is inefficient, uneconomical and in many cases not feasible. The additional damping that could be provided by a TLD was estimated for each archetype and in the majority of cases this additional damping is sufficient to bring accelerations within the ISO limits. However, providing the required damping becomes more challenging as building height and mass increase.

A method for predicting the maximum feasible height of a single core modular tower has also been presented. The method produces curves that, for any combination of building mass per unit volume and core wall thickness, show the approximate maximum feasible height of a single core tower which can be controlled by a TLD given the building's plan dimensions. These curves provide a useful tool for preliminary design of modular high-rise buildings, and demonstrate that wind-induced dynamic response issues should not prevent modular buildings achieving greater heights in the future.

Despite the helpful preliminary design tools presented by this chapter, significant uncertainty still surrounds the inherent properties of modular structures. This study relies on Eurocode 1991-1-4 [73] estimates of damping for composite concrete and steel structures; in reality the true inherent damping ratio of modular buildings

is unknown. Furthermore, the unknown stiffness contribution of the modules to the dynamic response has been neglected in this study as it also remains an area which has yet to receive significant research. To enable continued progress of modular high-rise structures it is necessary to further investigate a number of areas. These include:

- Identification of the inherent damping ratio of high-rise modular buildings through in-situ testing of full-scale structures. This is addressed in Chapter 5.
- The bending stiffness of a modular structure with an RC core must be studied further. Both the stiffness contribution of the modules and the RC core should be quantified. This is evaluated in Chapter 6.
- Further research into the ability of TLDs to control wind-induced accelerations of modular high-rise structures and the distribution of multiple smaller TLDs to allow for easier incorporation of a supplementary damping system in the design of a modular building is necessary. This is considered in Chapters 7 and 8.

Chapter 4

Case Study Structures

4.1 Introduction

The purpose of this research is to provide better understanding of the inherent properties and dynamic response of high-rise modular structures under ambient wind loads, and to evaluate suitable mitigation strategies. With the intention of strengthening this understanding, two of the world’s tallest volumetric modular buildings have been studied. These buildings serve as primary examples providing foundational data and insights that support the various concepts and methods discussed in this body of research. The two structures which have been studied are *Ten Degrees* and *College Road*, both located in Croydon, London, United Kingdom. This chapter aims to provide the reader with sufficient information about the structures studied to form a basis for the research discussed in subsequent chapters. Below, overviews of the examined structures are presented, including descriptions of the structural design, finite element models which were created and the results of wind tunnel tests conducted prior to construction.

4.2 Ten Degrees, Croydon

The first building which has been studied is *Ten Degrees* located in Croydon, London, United Kingdom. Upon completion this became the tallest volumetric modular building in the world [65] and won the Council of Tall Buildings and Urban Habitat’s 2022 Structural Engineering Award [215]. The full structure, location shown and

pictured in Figure 4.1, is a 44 storey, 135 m tall residential building. The structure consists of two slip-formed reinforced concrete (RC) cores, a transfer slab at level 4 and two adjoined towers of 44 (Tower A) and 37 (Tower B) storeys that consist of volumetric corner post modules stacked around and connected to the two RC cores. Due to the depth of the transfer slab, it was assumed to behave rigidly. The structure has a slenderness ratio (Height/Breadth) of 8.

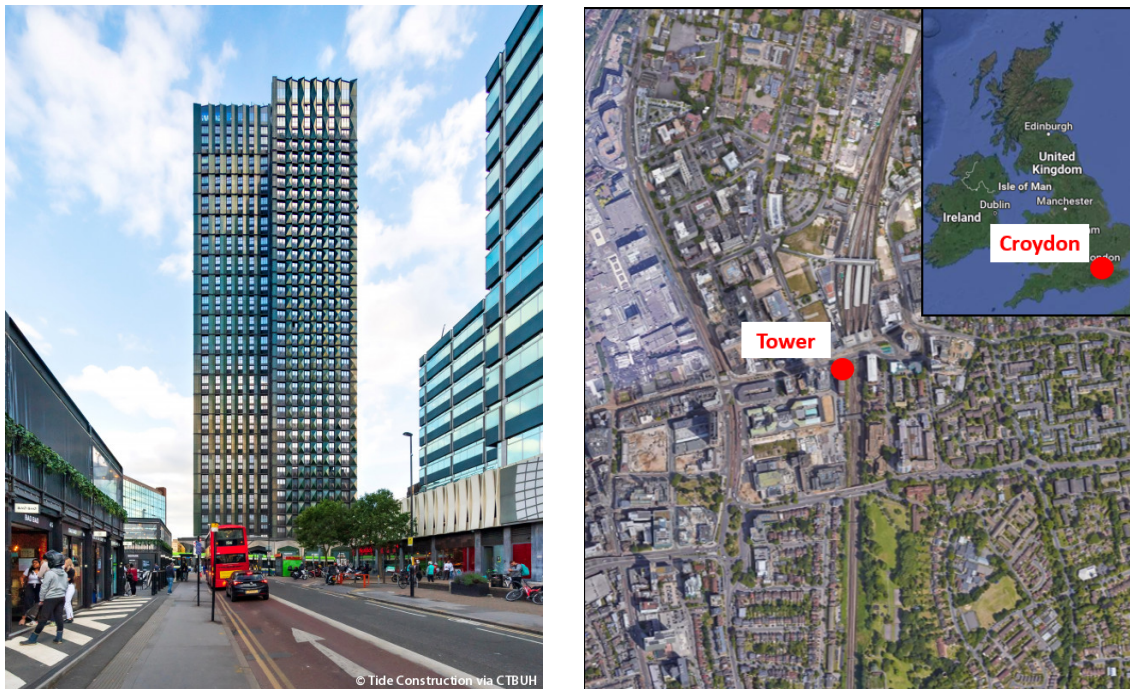


Figure 4.1: View of the *Ten Degrees* Tower and location

4.2.1 Structural Design

Ten Degrees was developed and constructed by *Tide Construction*. The civil and structural engineers responsible for Ultimate Limit State (ULS), Serviceability Limit State (SLS) and lateral stability were *Barrett Mahony Consulting Engineers*. *MJH Structural Engineers* were responsible for the structural design of the volumetric modules and *Vision Modular Systems* constructed and delivered the modules to site. A plan view of the building is provided in Figure 4.2.

The construction of this structure took a total of 26 months, with modular construction enabling a 42% quicker construction programme. More than 1,500 volumetric modules were installed with 1.5 storeys of modules installed on average

each week, equating to 21 homes a week. In total 546 A-rated low-energy homes are provided by this building. It is also claimed that by implementing modular construction technology, 24,229 tonnes of embodied carbon were saved on the project due to more efficient design, less waste and fewer vehicle movements [216]. A high-level visual of the construction programme is shown in Figure 4.3.

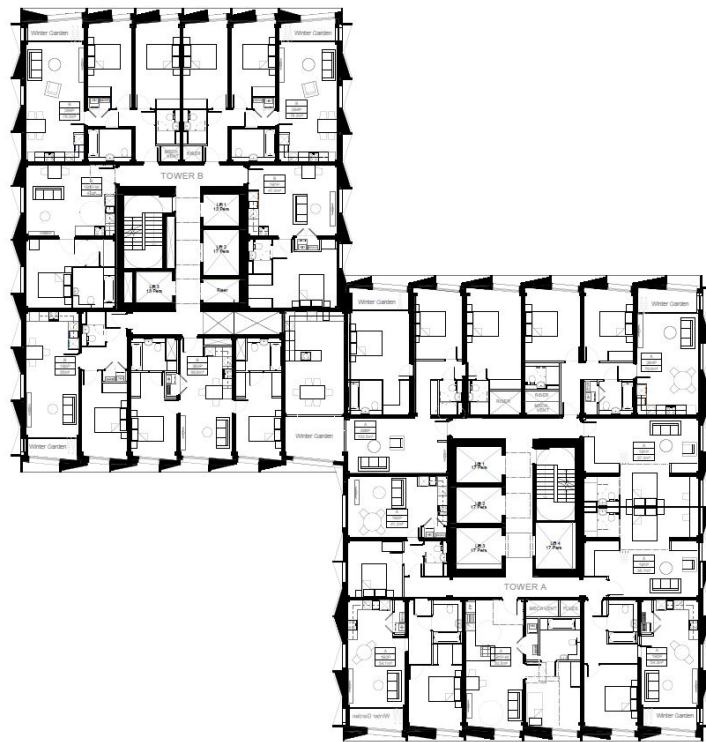


Figure 4.2: Plan view of *Ten Degrees*

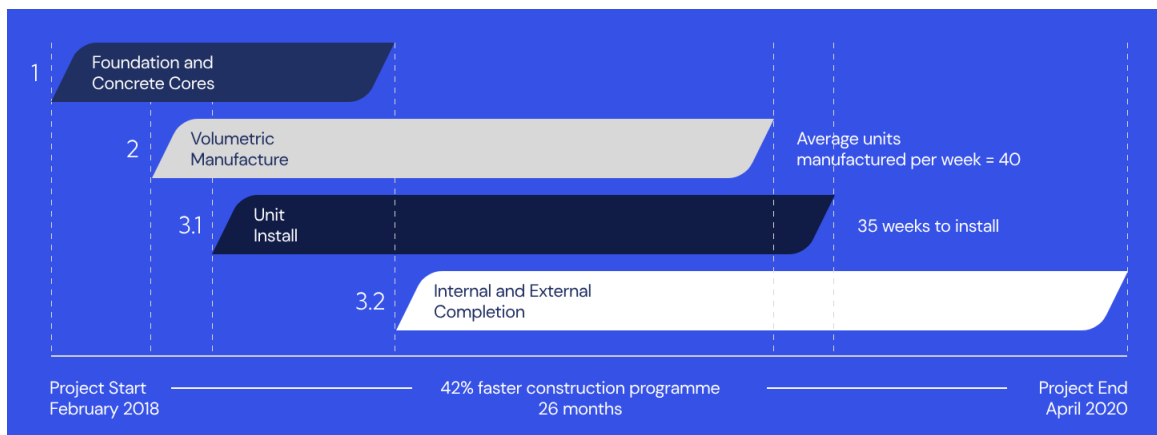


Figure 4.3: High-Level construction programme for *Ten Degrees, Croydon* (Source: Tide Construction [216])

The two reinforced concrete cores act as the main element for lateral load resistance, with the modules assumed to provide no additional stiffness to the structure. This was to ensure conservative calculations throughout the design process as the contribution of the modules to the lateral stability or stiffness of the finished structure remained unknown. The modules were designed to resist vertical loads and to transfer lateral loads through their floor slabs to the concrete cores. The concrete cores are approximately 8 x 8 m in plan and have walls which range from 300 mm to 450 mm thick. The landing slabs within the core are 300 mm deep. The RC cores were tied together during construction using 300x300x16 RHS members to alleviate inter-module connection forces in the temporary design state. As more storeys of modules were installed, the ties were removed and the connected storeys of modules began to act in their intended finished capacity as the ties between the two cores. Figure 4.4 shows the ties between the cores as modules were installed.



Figure 4.4: Ties between concrete cores during module installation (Source: Tide Construction [6])

The volumetric modules used in *Ten Degrees* are corner supported modules. The dimensions of the modules are typically 2.875 m tall, and are limited in length and width to approximately 6 m and 3.5 m respectively due to transportation and lifting constraints. They have a typical self weight of 7 kN/m^2 . Each module consists of a 150 mm thick reinforced concrete slab encased in steel perimeter channels, four square hollow sections acting as corner post columns, four square hollow sections as beams and ten bracing members, five on each of the longer faces of the module. Bracing members are included to ensure structural integrity of the module during transport and lifting, they are not designed to resist lateral loads. The roof of the module is a suspended plaster ceiling, supported by the square hollow section beams, which conceals services and inter-module connections. The member sizes within the modules are highly optimised and change from storey to storey to enhance efficiency and reduce costs. A representative module is shown in Figure 4.5 with the typical shape and size of each member indicated. The abbreviations seen are as follows: Square Hollow Hot Formed (SHHF), Square Hollow Cold Formed (SHCF), Rectangular Hollow Hot Formed (RHHF), and Parallel Flange Channel (PFC). Whilst this is a simple representation of the form of modules used in *Ten Degrees*, the modules throughout the structure are versatile. The length, breadth and shape of the modules varies across the structure, with member dimensions and the location of bracing members also changing. The variation of the modules across the structure adds a further layer of complexity to the analysis of the case study buildings.

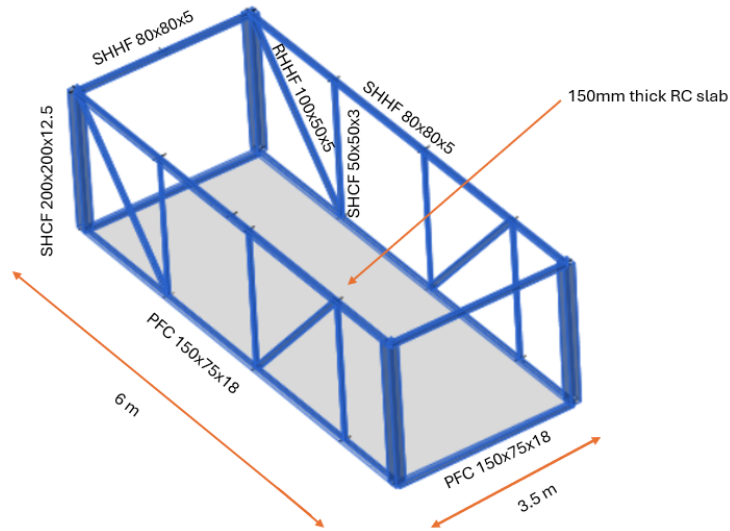


Figure 4.5: Typical volumetric corner post module installed in *Ten Degrees*

Ten Degrees employs a wide variety of intra- and inter-module connections as well as connections between the modules and the RC cores. The specifics of the connection types used in the case study structures is commercially sensitive information and full details cannot be provided. However, an overview of each connection type is presented to demonstrate the idealised behaviour of the connections.

The majority of the intra-module connections used are welded connections and are similar to those utilised in typical steel construction. In total, three types of inter-module connections were employed within the investigated building: connections between adjacent vertical members, connections between adjacent horizontal members and vertical ties. These connections use a combination of welded plates, brackets and dywidag bars. The connections between adjacent vertical and horizontal members consist of tie plates which are typically welded to both members at multiple locations along the member lengths to create rigid connections and diaphragm action between modules allowing for lateral load transfer. Ties between vertical columns are facilitated by dywidag bars. The two towers of the structure are joined together through the modules using these inter-module connections, providing continuity to the structure and creating diaphragm action across the floors of the multi-storey building. Hence, the stiffness of these connection should be expected to influence the overall dynamic response of the structure.

The presence of the reinforced concrete cores which act as the lateral stability element in this building results in another type of connection; connections between the modules and the RC core. These connections facilitate the transfer of lateral wind loads from the exterior modules to the interior cores. The connections between the RC core and modules used in *Ten Degrees* utilise a combination of embedded steel plates, brackets and welds. Steel plates embedded in the floor slab of the modules slot into brackets embedded in the concrete core. This allows for vertical movement between the modules and RC cores which may occur due to axial shortening whilst still providing a suitable connection for the transfer of lateral loads from the module slabs to the RC cores. The stiffness of this type of connection is significant in the dynamic response of the overall structure as it has impact on the stiffness contribution of the modules. The connections used in *Ten Degrees* and *College Road* have been designed by *MJH Structural Engineers* and are not investigated as part of this thesis. Further reading on the connections used can be found in work carried out by Heng et al. [45, 217].

4.2.2 Finite Element Models

During the design process a comprehensive Finite Element Model (FEM) of *Ten Degrees* was created using the software ETABs by *MJH Structural Engineers*. Within the model, the concrete cores are modelled using shell elements. Each individual member of the modules are modelled with the corresponding member sizes. The slabs of each module are modelled as individual diaphragms. Connections between the modules and between the modules and core are modelled as link elements with modified properties to reflect the stiffness of the more complex reality of the connections. The relevant dead and live loads are applied to each member and wind loads are applied as lateral line loads along the edge of the slabs of the modules at the outer edge of the building. This model was used to estimate the behaviour of the structure in ULS and SLS under the appropriate load combinations and in different stages of completion. The model is extensive and serves its purpose for assessing gravity loads and ULS design states. Figure 4.6 shows a 3D image of the Finite Element Model created by *MJH Structural Engineers*.

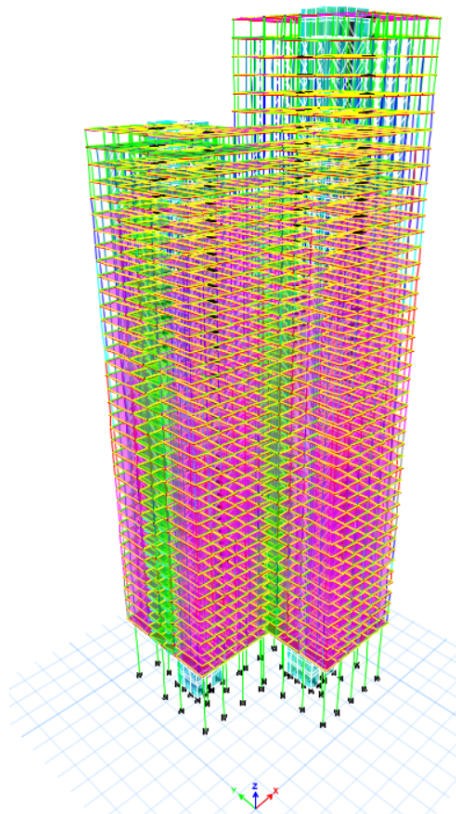


Figure 4.6: Image of the FE model created of *Ten Degrees* by *MJH Structural Engineers*

There are no specific software available for performing FE analysis of modular structures. A significant issue with using current modelling practices arises from the nature of volumetric modular construction. A typical modular building has a double member at every external beam and column, quadruple members in place of every internal beam and column and adjacent mirrored bracing members. This is due to each module being a separate structural system with a full set of supporting members of its own. The stacking and aligning of modules next to each other to complete the finished structure results in duplicates of every internal member.

There are a total of 19 beam, column and shell elements in the detailed model of one single module. As *Ten Degrees* contains over 1,500 modules, this results in the FE model having over 28,500 beam, column and shell elements for the modules alone. This value excludes the joints which are used to model each member within the module. In total, the FE model of the full structure contains approximately 260,000 elements; this value includes joints, elements used to model connections and the elements used to model the RC core. This is a large FE model which can be difficult to use; simply opening the model takes significant computational power. Light usage such as changing the view or selecting a member to identify its size results in large lags and latency issues. Full static analysis of the model on a PC with 32 GB of RAM and an 8 core Intel i9 processor takes more than 12 hours. It takes approximately three hours to run an eigensolution modal analysis of this model on the same PC.

This FE model of *Ten Degrees* was used to assess static behaviour and to identify the natural frequencies of the structure. Under static conditions, the structure was sufficient and the design requirements of the Eurocodes were met. However, through performing eigenanalysis of the FE model, low natural frequencies of the building were identified. These values are presented in Table 4.1.

Table 4.1: Natural Frequencies of *Ten Degrees* identified from FE model

Direction	East-West	North-South
	X-X	Y-Y
Natural Frequency	0.18 Hz	0.19 Hz

Due to the novelty of high-rise modular construction, the slenderness of the building and the calculated low natural frequencies, a wind tunnel study of the structure was performed.

4.2.3 Wind Tunnel Testing

A small-scale boundary layer wind tunnel study was performed on *Ten Degrees* during the design process to assess the wind effects relevant to the structural and serviceability design. The tests were conducted by BMT Fluid Mechanics Ltd (BMT) and detailed in a report issued to *Barrett Mahony Consulting Engineers* [218].

The boundary layer wind tunnel used to perform the tests had a working section of 4.8 m wide, 2.4 m high and 15 m long with a 4.4 m diameter multiple plate turntable and a remotely controlled 3-dimensional traversing system. A turbulent boundary layer, representative of the conditions at the site, was set up using an arrangement of roughness elements distributed over the floor of the wind tunnel and a 2-dimensional barrier with square posts or spires at the entrance to the test section.

A 1:300 scale model was used in the study. At this scale the model is sufficient to allow a good representation of the details that are likely to affect the local and overall wind flows at full scale. In addition, this scale enables a good simulation of the turbulence properties of the wind to be achieved. At the time when the study was conducted, the area immediately surrounding the proposed development principally comprised mid-rise developments typical of urban Greater London terrain. The terrain and buildings in the immediate vicinity were also scaled and included in the wind tunnel study. Figure 4.7 shows the wind tunnel setup with the scaled model of *Ten Degrees* shown in blue and the surrounding conditions in neutral colours.

The fluctuating wind pressures along the surface of the building were measured using pressure transducers a full range of wind directions in increments of *Ten Degrees*. The pressure transducers used were Sensor Techniques low range SLP004D differential transducers with a frequency response of 2000Hz. The transducers were grouped in blocks of eight, with a common reference pressure on each block, providing the means for in-situ calibration and checks of the calibration throughout the tests. During the measurements the reference pressure was connected to the static pressure in the wind tunnel test section.



Figure 4.7: Wind tunnel set up showing scaled model of *Ten Degrees* in blue and the surrounding conditions

The measured responses were employed in dynamic analyses using structural and dynamic properties provided by *Barrett Mahony Consulting Engineers* shown in Table 4.1. These analyses also investigated the acceleration response for natural frequencies $\pm 10\%$ of those estimated from the FE model to account for uncertainty in the estimation from the FE model and the eigenanalysis which was performed. A damping ratio of 2% was assumed by *Barrett Mahony Consulting Engineers*, in line with the Eurocode 1991-1-4:2004 [73] recommended value for concrete structures. The damping value for concrete was adopted due to the primary lateral stability elements in the structure being the two RC cores. Damping ratios of 2% $\pm 10\%$ (1.8% and 2.2%) were also tested as part of a sensitivity analysis.

The study defines design wind loads of the proposed development based on design wind speeds derived from the 50-year return period wind speed of 21.5m/s (referenced to a 10-minute averaging period at 10m height over open terrain) as stipulated in BS EN 1991-1-4:2005 and the related UK national annex (UK NA to BS EN 1991-1-4:2005) [73], both representing the current UK code of practice for the assessment of wind action on buildings. For serviceability design, namely wind-induced building accelerations, the building responses were assessed based on the 1- and 10-year return period design wind speeds in conjunction with the use of

the directionality information derived for extreme wind events in the region. The accelerations identified from the study are shown in Table 4.2 with values exceeding ISO 10137 [84] occupant comfort criteria shown in red.

Table 4.2: Peak acceleration response of *Ten Degrees* Tower A and Tower B for 1-year and 10-year return period wind speeds with values exceeding ISO 10137 [84] occupant comfort criteria highlighted in red

Storey	Damping	Acceleration Response for Natural Frequency -10% (milli-g)		Acceleration Response for Natural Frequency (milli-g)		Acceleration Response for Natural Frequency +10% (milli-g)	
		1-year	10-year	1-year	10-year	1-year	10-year
Tower A, Storey 44	1.8%	11.2	18.8	10.0	17.5	9.2	16.1
Tower A, Storey 44	2.0%	10.6	17.9	9.5	16.7	8.7	15.3
Tower A, Storey 44	2.2%	10.1	17.1	9.1	15.9	8.3	14.6
Tower B, Storey 37	1.8%	8.6	14.4	7.9	13.6	7.3	12.5
Tower B, Storey 37	2.0%	8.3	13.7	7.5	12.9	7.1	11.8
Tower B, Storey 37	2.2%	7.9	13.1	7.1	12.3	6.6	11.3

From Table 4.2 it can be concluded that for almost all assumed damping and natural frequency values, the peak accelerations identified from the wind tunnel study at the top storey of Tower A exceed ISO 10137 [84] criteria described in Section 2.5. Figure 4.8 provides a visual representation of this.

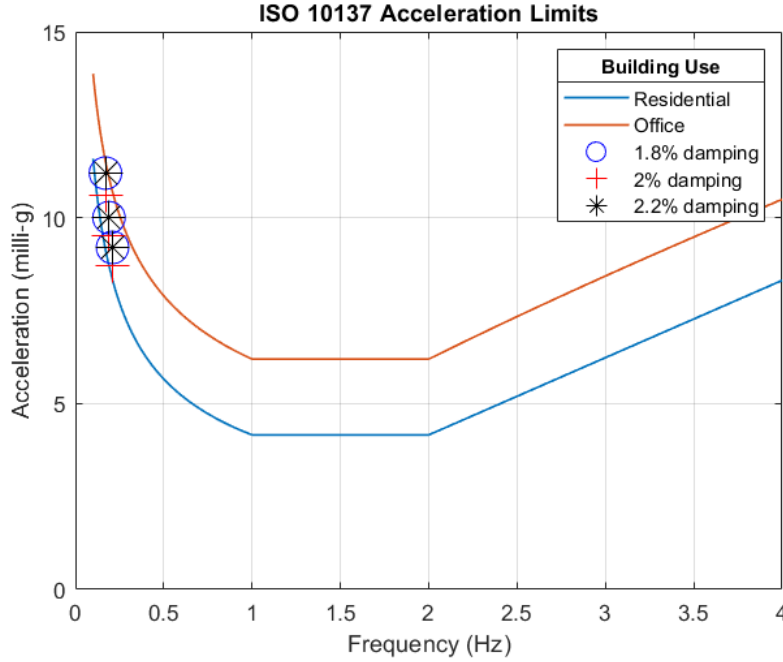


Figure 4.8: ISO 10137 acceleration limits for occupants comfort compared to top storey accelerations of *Ten Degrees* estimated through wind tunnel tests and dynamic analysis

As a result of the identified risk of not satisfying occupant comfort criteria and the novelty of modular high-rise construction, it was decided to perform full-scale in situ monitoring of the *Ten Degrees* structure. The aim of this testing was to assess the dynamic behaviour of the building and determine whether a supplemental damping device would be required to control the wind-induced vibrations of the structure. This testing is discussed further in Chapter 5.

4.3 College Road, Croydon

The second building which has been studied is *College Road* located beside *Ten Degrees* in Croydon, London, United Kingdom. *College Road* is a 163 m tall structure consisting of a 50-storey tower, Tower A, with 817 co-living homes and a 35-storey tower, Tower B, with 120 affordable homes. The structure consists of two slip-formed reinforced concrete cores, a transfer slab at Level 3 with traditional RC construction below and two adjoined towers of corner post volumetric modules stacked around the RC cores above. Upon completion in 2022 it was the tallest volumetric modular building in Europe [25], surpassing *Ten Degrees* by 28 m. In the East-West direction

the building has a slenderness ratio (Height/Breadth) of 3 and in the North-South direction the slenderness ratio is 8. Figure 4.9 shows *Ten Degrees* and *College Road* standing side by side.



Figure 4.9: *Ten Degrees* and *College Road*, Europe’s tallest volumetric modular buildings (Source: Tide Construction [25])

4.3.1 Structural Design

The design team for *College Road* was the same as the team for *Ten Degrees*. The building was developed and constructed by *Tide Construction*. *Barrett Mahony Consulting Engineers* were the civil and structural engineers responsible for Ultimate Limit State (ULS), Serviceability Limit State (SLS) and lateral stability and *MJH Structural Engineers* were responsible for the structural design of the volumetric modules. *Vision Modular Systems* performed the off-site construction and delivery of the modules. A plan of the building is provided in Figure 4.10. Construction of *College Road* took a total of 28 months, with time spent on traditional construction aspects accounting for 94 weeks (approx. 70%) and modular construction taking 40 weeks (approx. 30%) of the total construction programme. In total, 1,725 corner post volumetric modules were installed in 40 weeks. In comparison to *Ten Degrees*, *College Road* had a construction programme which was two months longer, however, over 200 more modules were installed than in *Ten Degrees*.

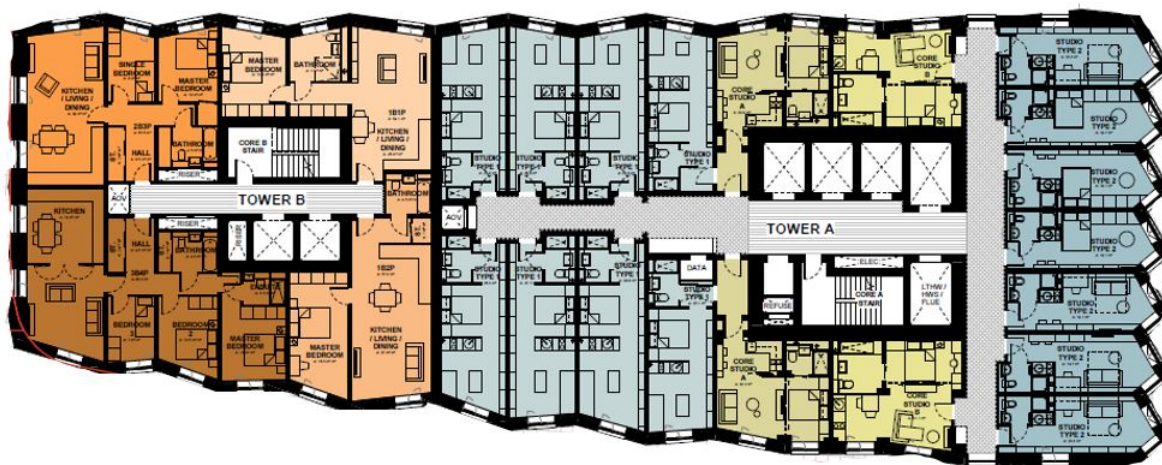


Figure 4.10: Plan view of *College Road*

The two reinforced concrete cores were treated as the primary lateral stability elements throughout the structural design process. This was due to the same uncertainty in the design of *Ten Degrees* surrounding the stiffness contribution of the modules to the completed building. As with *Ten Degrees*, the modules were designed for vertical loading and it is assumed that lateral loads are transferred through diaphragm action of the floor slabs back to the RC cores. The RC cores in *College Road* differ in size, with the core of the 35 storey Tower B measuring 8 x 8 m in plan and the core of the 50 storey Tower A measuring 10 x 10 m. The core walls in the smaller core are 450 mm thick, the same as the cores in *Ten Degrees*. The core walls in the Tower A are 600 mm thick. The landing slabs within the core are 1250 mm deep. Ties between the cores were again implemented during construction in a phased process to alleviate inter-module connection forces during temporary completion.

The module layouts and connection types used in *College Road* are largely the same as those used in *Ten Degrees* as shown in Figure 4.5. Modules are typically 2.875 m tall with lengths and widths which vary more than in *Ten Degrees*. Modules as long as 10 m and as wide as 4 m are used throughout the building. Despite the different dimensions to *Ten Degrees*, the self weight of the modules does not differ and is typically 7 kN/m^2 . The floor slabs within the modules were the same for the two structures, 150 mm thick RC slabs encased in steel perimeter channels. Four square hollow sections act as the corner posts and four square hollow sections act as beams. Five bracing members make up each of the longer faces of the modules, as shown in Figure 4.5, and are designed for transport and lifting purposes, not for

the permanent lateral stability of the structure. The roof of the modules consist of suspended plaster ceilings which conceal services and inter-module connections.

Connection systems used within *College Road* are the same as those used in *Ten Degrees*. Intra-module connections are constructed off-site in a factory setting with a high level of tolerance and use techniques seen in traditional construction such as welds and bolted connections. Three types of inter-module connections are used including connections between adjacent vertical members, connections between adjacent horizontal members and vertical ties, as discussed previously. Welded plates, brackets and dywidag bars are utilised in the various types of inter-module connection. Continuity and diaphragm action is created across the structure through the use of these connections. Connections between the modules and the RC cores allow for the transfer of lateral wind loading from the exterior modules to the interior RC cores. Embedded steel plates, brackets and welds are used in this form of connection. The intra- and inter-module connections as well as the connections between the modules and the RC cores were designed by *MJH Structural Engineers* and are commercially sensitive. They have been the focus of further research conducted by Heng et al. [45, 217].

4.3.2 Finite Element Models

College Road was modelled and analysed extensively using the FE software ETABs. The FE models were created by the design teams in *MJH Structural Engineers* and *Barrett Mahony Consulting Engineers*. The RC cores are modelled using shell elements. Each individual beam, column and bracing member is modelled with beam elements of the corresponding section size. Slabs of each module are modelled using shell elements and an individual diaphragm is assigned to each one. Link elements are modified with properties defined to accurately represent the stiffness of the different connection types. Dead and live loads are defined and applied to each member in correspondence with Eurocodes. Wind loads have been applied as lateral line loads along the edge of the slabs of the modules on the exterior of the building. Analysis of the structure was performed to assess the behaviour at ULS and SLS.

Similar to *Ten Degrees*, the FE model of *College Road* is considerably larger than models of similarly sized buildings employing traditional forms of construction.

This is due to the duplication of members throughout the structure as the modules are stacked. In total, there are more than 455,000 elements in the FE model of the full structure. The usability of a model of this size is greatly reduced, with static analysis taking more than 14 hours to run on a PC with 32 GB of RAM and an 8 core Intel i9 processor. The size of the FE models makes it prohibitively difficult to perform any parametric studies. Figure 4.11 shows the FE model of *College Road*.

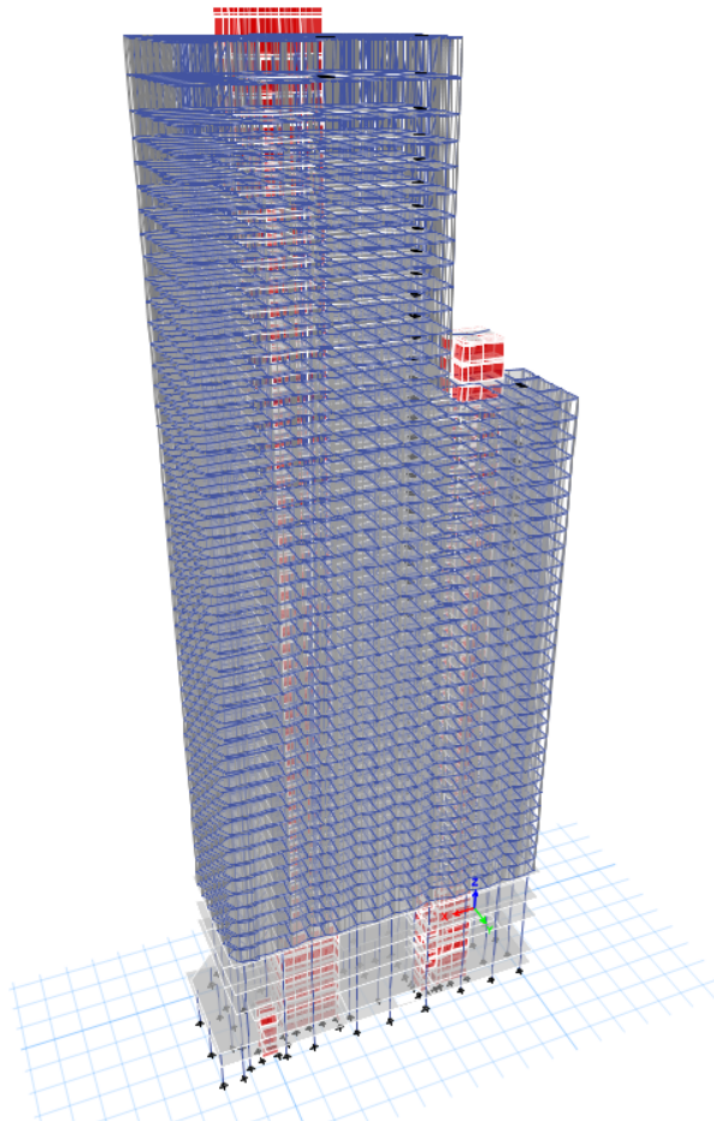


Figure 4.11: FE model of *College Road*

The FE model of *College Road* was used to assess the static behaviour and identify the natural frequencies of the structure. Design wind loads were applied to the structure, however only static ULS conditions were considered. Under static

conditions, the structure was found to satisfy all ULS criteria. The natural frequencies of the structure were found by performing eigenanalysis and are shown in Table 4.3. Similar to *Ten Degrees*, the low natural frequencies identified from the FE model led to a wind-tunnel study of *College Road* being performed.

Table 4.3: Natural Frequencies of *College Road* identified from FE model

Direction	East-West	North-South
	X-X	Y-Y
Natural Frequency	0.18 Hz	0.20 Hz

4.3.3 Wind Tunnel Testing

A boundary layer wind tunnel study was performed on *College Road* during the design stage to assess the wind induced response of the structure. The study was carried out by *RWDI Consulting Engineers* [219]. RWDI's boundary layer wind tunnel facility simulated the mean wind speed profile and turbulence of the natural wind approaching the modelled area by having a long working section with a roughened floor and specially designed turbulence generators, or spires, at the upwind end. Floor roughness and spires were selected to simulate 'Urban' terrain conditions.

A 1:300 scaled model of *College Road* and its surrounding terrain was built including scaled models of neighbouring *Ten Degrees* and other structures in the vicinity. Figure 4.12 shows the wind tunnel setup with *College Road* in red and *Ten Degrees* to the immediate left in grey.

A full range of wind directions were tested and the fluctuating wind pressures measured. Assessment of the acceleration response was performed using dynamic analysis with structural properties provided by *Barrett Mahony Consulting Engineers*. These included the natural frequencies from the FE model of *College Road* as shown in Table 4.3. The damping ratio adopted by *Barrett Mahony Consulting Engineers* was 1.2%. This is based on the value given by Eurocode 1991-1-4:2004 [73] for concrete and steel composite buildings. There was no sensitivity analysis performed, only the estimated natural frequencies and the assumed damping ratio were assessed.



Figure 4.12: Wind tunnel set up showing scaled model of *College Road* in red, *Ten Degrees* to the left in grey and other surrounding structures

A 50-year basic 10 minute wind speed of 21.5 m/s at 10m height in open terrain was used for the wind loading predictions. For serviceability design, i.e. assessing the wind-induced response, dynamic behaviour under a 1 year and 10 year return period were considered. The accelerations of the scale model were recorded at the top of the two towers, Tower A and Tower B. The peak accelerations calculated during the study are given in Table 4.4.

Table 4.4: Peak acceleration response of *College Road* Tower A and Tower B for 1-year and 10-year return period wind speeds

Storey	Damping	Peak Acceleration Response (milli-g)	
Return Period		1-year	10-year
Tower A, Storey 50	1.2%	6.4	11.7
Tower B, Storey 35	1.2%	4.4	7.9

The peak accelerations identified for the 1 year return period satisfy ISO 10137 [84] criteria for occupancy comfort of residential buildings. Figure 4.13 compares the calculated peak accelerations for Tower A and B considering the first two natural frequencies listed in Table 4.3 with ISO 10137 criteria for occupant comfort.

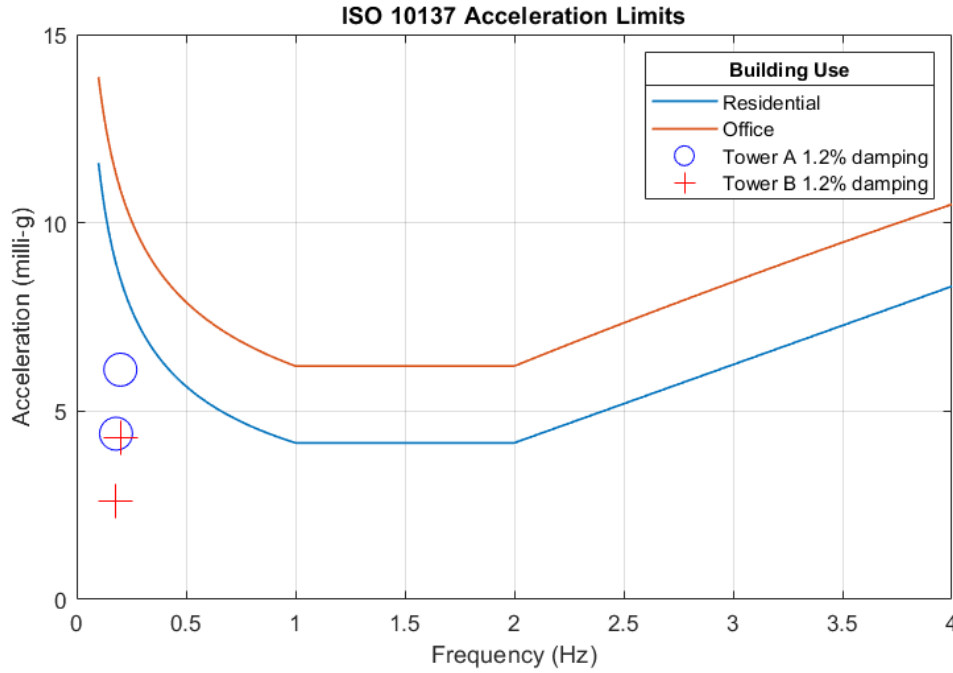


Figure 4.13: ISO 10137 acceleration limits for occupants comfort compared to top storey accelerations of *College Road* estimated through wind tunnel tests and dynamic analysis

Despite wind tunnel tests showing that the *College Road* building would satisfy occupant comfort criteria, given the novelty of modular high-rise construction, it was decided to perform further in situ monitoring of the structure. The aim of this being to gather further fundamental understanding of the inherent properties and dynamic response of this form of construction.

4.4 Conclusions

This chapter outlines the structural design, finite element models and wind tunnel studies that were performed on two high-rise modular buildings, *Ten Degrees* and *College Road*. These buildings will be the subject of much of the research outlined in this thesis, including in situ monitoring, modal identification, surrogate modelling and parametric studies of mitigation measures. From the information outlined in this chapter, it can be seen that there are still significant research gaps in volumetric modular construction. The lack of design codes and available research surrounding high-rise modular construction, made apparent from the above analysis of the design and wind tunnel studies conducted on *Ten Degrees* and *College Road*, have

informed the progression of the research contained in this thesis in order to achieve the overall objective; to provide improved insight into the inherent properties of volumetric modular buildings, their wind-induced dynamic response and suitable control methods. An outline is provided below of each knowledge gap which has been identified and how this has been addressed within the different chapters of this thesis.

1. Uncertainty Surrounding the Inherent Properties

Considering the natural frequency and damping ratio values supplied for the wind tunnel testing of the two case study structures, it is clear that uncertainty surrounds the inherent properties of volumetric modular buildings. This is particularly true of the damping ratio which can not be calculated in the same way as the natural frequency. As previously discussed, the inherent damping ratio of a structure significantly influences its dynamic response. It can be seen that values for concrete and values for steel and concrete composite structures outlined by Eurocode 1991-1-4:2004 [73] formed the best estimate for further analysis of the wind tunnel studies outlined in this chapter. In reality, the actual value is unknown. The level of uncertainty surrounding this value could completely undermine the results of the wind tunnel studies and result in modular high-rise structures being built which cause occupant discomfort. Therefore, the damping ratio of volumetric modular buildings is a topic which requires significant further research to enable this form of construction to continue to push to new heights. The uncertainty surrounding the damping ratio of modular buildings is addressed in Chapter 5 of this thesis where the results from in situ testing and Operational Modal Analysis (OMA) of the two case study structures is discussed. Estimates of the natural frequencies and damping ratios of *Ten Degrees* and *College Road* are presented, along with measures of the uncertainty of the estimates, hence improving the limited research available on the damping ratio of volumetric modular buildings.

2. Difficulty Modelling the Dynamic Response

The lack of modular specific FE or dynamic analysis software available is one apparent limitation to the progress of high-rise modular construction. Whilst using current FE techniques to model a modular building, in which every element is modelled individually, may be possible for smaller modular structures, modelling of each element in tall modular buildings with over 40 storeys becomes problem-

atic. The number of elements needed to be modelled becomes considerably large and requires significant computational power and run time to use and analyse the model. Whilst a global model of a modular structure with every element modelled individually is useful for checking capacities, stresses and the localised behaviour of members, there is a need to develop alternative models that represent the properties of a modular structure for the specific purpose of ensuring that habitability requirements are satisfied. This is important because the computational demands associated with the dynamic analysis of the response of buildings to wind loading can be much greater than those required for static analysis of gravity load response. On the other hand, the low amplitude vibrations associated with habitability requirements imply a small displacement, elastic response that can be well represented by modal properties, suggesting considerable scope for computational efficiency. In Chapter 6 of this thesis, a data-informed MDOF surrogate model is developed using the inherent properties of modular buildings identified. This MDOF surrogate model has been validated and shown to be capable of quickly predicting the dynamic response of a high-rise modular buildings.

3. Mitigation of Wind-Induced Vibrations

When concerns arose around the ability of *Ten Degrees* to meet habitability requirements, it became necessary to consider measures for mitigating the dynamic response of high-rise modular buildings. Traditional methods such as the provision of outriggers were quickly disregarded due to the form of modular construction. The two methods for reducing the dynamic response which were considered were stiffening of the RC cores and the addition of supplementary damping devices. Whilst both *Ten Degrees* and *College Road* were proven to conform with ISO 10137 occupant comfort criteria, the question remains: how can the wind-induced response of future high-rise modular buildings be effectively controlled? Chapters 3, 7 and 8 of this thesis address this question. Chapter 3 parametrically assessed the effects of increasing the dimensions of the concrete core on reducing the wind-induced peak acceleration response of modular buildings and found that supplementary damping, in the form of TLDs, is more effective at reducing excessive vibrations. Chapters 7 and 8 further consider the ability of TLDs to control the oscillations of a modular building and proposes a method of distributing TLDs vertically in order to incorporate supplementary damping in the structures more seamlessly, without occupying significant space at the top of a building. Hence, appropriate solutions to

the question of controlling the dynamic response are provided.

This chapter provides the necessary knowledge of the two high-rise modular buildings which are studied further in later chapters. A synopsis of their design and subsequent wind tunnel studies have been discussed. The knowledge gaps identified in this chapter highlight the motivation for much of the work which forms this thesis and how it has been shaped to achieve the primary objective.

Chapter 5

Operational Modal Analysis of Case Study Structures

5.1 Introduction

As an innovative form of construction, high-rise modular buildings have not been the subject of previous vibration monitoring investigations, implying an absence of essential information relating to their dynamic performance. Hence, it is crucial for the further development of modular buildings that the wind-induced acceleration response of this form of construction is better understood and characterised.

The damping ratio is a key indicator of an individual structure's susceptibility to dynamic loads, including the level of discomfort experienced by the occupants of a tall building subjected to wind loading. While computational models, laboratory studies and empirical data can provide estimates of structural damping, the most reliable way to evaluate true damping ratio values is through modal identification using data from field tests on full-scale finished structures. Furthermore, in situ testing can be used to identify the natural frequency of a structure. For modular buildings, this presents a unique opportunity to evaluate the stiffness of the structure and to quantify the unknown contribution of the modules to the overall bending stiffness. Therefore, in situ field measurements were recorded for two of the world's tallest modular buildings, *Ten Degrees* and *College Road*, under ambient wind loading in order to identify their inherent properties and provide a better understanding of the dynamic behaviour of modular high-rise buildings.

5.1.1 Factors which may Influence Damping Ratio Estimates

Field monitoring is the most accurate method for understanding the true behaviour of any structure. As previously discussed in Chapter 2, damping ratio values used in the design of buildings are mostly based on empirical values which were identified through full-scale tests of structures. However, variation was noted between the values recommended for steel and also for the values recommended for concrete. This is likely due to the uncontrolled nature of field testing. As the structures are tested in their natural environment, and not in a controlled laboratory setting, there are a number of factors which cannot be controlled leading to variation in the response of the structure, and therefore, in the results of the testing. These include the height and dimensions of the buildings, the loading of the structures, the specification of the sensors used, the locations of the sensors and the Operational Modal Analysis (OMA) methods used to calculate the damping ratio. Furthermore, it is important to note that in most cases the structure being monitored was not constructed for the purpose of testing; the structure was built primarily as a form of infrastructure with the full-scale testing being a product of striving to better understand its behaviour. Therefore, field testing is often governed by what is possible and not what is desired.

For the two high-rise modular structures studied within this chapter, there are a number of factors which may influence the estimated value of the damping ratio. These factors include:

1. **Stage of Construction:** Both *Ten Degrees* and *College Road* were under construction as monitoring took place and the response of the structures were recorded at various stages of completion.
2. **Different Structures:** Whilst the two structures use the same structural system, they are still two separate buildings of different heights and dimensions. This will result in variation of the identified inherent properties from the two structures.
3. **Direction of Response:** The response of the structures in their orthogonal directions, for example North-South and East-West, will differ. They are not symmetrical structures.

4. **Natural Frequency:** Relationships between the natural frequency and the damping ratio of a structure have been empirically reported [10, 11, 12]. Therefore, the natural frequency of the structures may influence the estimate of the damping ratio.
5. **RMS Acceleration:** The ambient loading conditions, namely the wind loading which excites the structures, will vary across measurements and cannot be controlled. This will result in varying RMS accelerations of the structures, which in turn may influence the damping ratio estimate from each measurement, as has been shown in previous studies [10, 11, 12].
6. **Sensors used for Monitoring:** The number and type of sensors and data loggers used to monitor the two buildings were different. Furthermore, the sensors used to monitor the two structures were placed in different locations in both buildings.
7. **OMA Techniques:** The OMA techniques which were used to analyse the dynamic response of the buildings and calculate the damping ratio may be inaccurate.

The design of the field testing and operational modal analysis of the response of the case study structures attempted to account for each of the factors above. Whilst the structures were studied at different stages of completion, multiple measurements of the response of the finished structures were also recorded, thus providing more than one estimate of the damping ratio of the completed buildings. Studying two structures using the same structural system enabled comparison of the results and for the influence of dimensions and response direction to be identified. High quality triaxial accelerometers were also used to ensure the response in both the North-South and East-West directions were measured and could be analysed. Through studying the structures at varying degrees of completion, the influence of natural frequency on the damping ratio could be studied. For *Ten Degrees*, wind speed measurements were captured to allow for insight into the ambient loading conditions exciting the structure during each measurement. The RMS acceleration of each response has also been calculated. The sensors used to record the response of the two structures differed. However, it was ensured that all equipment used was suitable and of high resolution. The positions of the sensors within the structures were located as close to the centre of the RC core as possible and at one external

position to measure the response at more than one location. However, the exact location and number of sensors used was limited due to the ongoing nature of the construction and the budget available for field testing. The use of multiple OMA methods which quantify uncertainty to process the response allows for validation of the estimates of the damping ratio. Therefore, whilst practical limitations did exist, the field testing procedure designed attempted to account for the different factors which could lead to variation in estimates of the damping ratio. Most significantly, the results from the full scale monitoring provided multiple estimates of the inherent properties which were used to obtain average values and estimate variations, from which important trends and relationships were observed.

5.1.2 Scope of Work

Within this chapter, details of the monitoring of the two case study modular structures, *Ten Degrees* and *College Road*, are presented. Further details of these structures can be found in Chapter 4. Four different OMA techniques were applied to the recorded acceleration responses from the structures in order to identify key inherent modal properties. These include the Bayesian Fast Fourier Transform (BFFT), Random Decrement Technique (RDT), Analytical Mode Decomposition Random Decrement Techniques (AMD-RDT) and a hybrid bootstrapped AMD-RDT which are discussed further in Section 5.2. Particular focus is given to identifying the natural frequency and damping ratios of each structure in the East-West and North-South directions.

The identified inherent properties of the case study buildings are presented. This includes the natural frequency of the structures at different stages of completion, which is further used in Chapter 6 to quantify the bending stiffness of modular buildings, and the damping ratio. As discussed, the damping ratio is the modal parameter which is hardest to predict accurately without full scale testing and is significantly influential in the dynamic response of a structure. By reporting the first measured damping ratios for a tall modular structure, this chapter makes an important contribution to knowledge about the vibration properties of an emerging form of construction. Reporting values of damping ratios obtained from live monitoring is a crucial advancement for modular construction and, ultimately, will allow for more informed designs in the future.

5.2 Operational Modal Analysis Methods

As discussed, estimating the damping ratio of a structure relies on empirical data from monitoring a full-scale building. Full scale monitoring of a structure can take the form of forced vibration experiments or ambient data collection. Forced vibration experiments involve applying a known input loading to the structure and recording the output. Often forced vibration experiments require an impractically large loading, cannot be performed while the structure is operational and are uneconomical. For this reason, ambient vibration tests have become increasingly popular in both structural health monitoring and modal identification technique development [71, 110]. The tests typically involve the placement of sensors, such as accelerometers, along the geometrical planes of the structure. A data acquisition system is employed to record and save the data from the sensors. The structure will naturally be excited by wind loading and other dynamic loads as it is operational during these field measurements. The loading the structure experiences is unquantified, however, it is assumed to be statistically random [110]. The acceleration response of the structure is recorded by the accelerometers and is considered ambient response data. This ambient data can then be processed using modal identification techniques to identify the key parameters of the structure such as natural frequencies, damping ratios and mode shapes.

Many different modal identification techniques are proposed in literature, some of which operate in the time domain, whilst others operate in the frequency domain. This study considers four modal identification methods: the Bayesian Fast Fourier Transform (BFFT) which operates in the frequency domain, the Random Decrement Technique (RDT) which operates in the time domain, a hybrid of the RDT which involves first decomposing the response signal into subsignals using Analytical Mode Decomposition (AMD), here on referred to as AMD-RDT, and a Bootstrapped version of the combined AMD-RDT method.

5.2.1 Bayesian Fast Fourier Transform

The first method employed to estimate the damping ratio of the structure is the Bayesian Fast Fourier Transform (BFFT). The BFFT is a method commonly used in modal identification to predict the modal properties of a structure from ambient data and has been investigated extensively [90, 108, 109, 110, 111, 220, 221, 222,

223]. Ambient data is frequently described as data in which the loading is unknown but is assumed to be statistically random and stationary [90, 110, 221]. The principle of the BFFT is that both the real and imaginary parts of frequency domain data, obtained from an FFT of the recorded time-domain acceleration response of a structure experiencing broad-band excitation will have a joint Gaussian distribution that can be described analytically by a set of modal parameters, θ . The modal parameters contained in θ are the natural frequency f , damping ratio ζ , mode shape Φ , entries of the force spectral density matrix $\{S_{ij}\}$ and the spectral density of the prediction error σ^2 , for any given mode. Frequency domain data is used to maximise the posterior probability density function (PDF) of the modal parameters and hence, find the most probable value (MPV) of each of the modal properties given the measured response.

The measured acceleration can be modelled as $\hat{\mathbf{x}}_j = \mathbf{x}_j(\theta) + \epsilon_j$ where $\mathbf{x}_j(\theta)$ is the acceleration response of the structural system calculated from fundamental structural dynamics theory given a set of modal parameters, θ , and ϵ_j is the measured error in the acceleration response; that is the difference between the actual and modelled response of the structural system.

The output of an FFT at a given frequency k is a complex number Z_k , with a real part F_k and imaginary part G_k . It has been shown that for long duration data the real and imaginary components of Z_k are jointly Gaussian with zero mean and a covariance matrix defined as:

$$\mathbf{C}_k = \frac{1}{2} \begin{bmatrix} \Phi(\text{Re}\mathbf{H}_k)\Phi^T & -\Phi i(\text{Im}\mathbf{H}_k)\Phi^T \\ \Phi(\text{Im}\mathbf{H}_k)\Phi^T & \Phi(\text{Re}\mathbf{H}_k)\Phi^T \end{bmatrix} + \frac{\sigma^2}{2} \mathbf{I}_{2n} \quad (5.1)$$

where $\Phi \in R^{n \times m}$ is the mode shape matrix of the measured DOFs; σ^2 is the constant spectral density level of the prediction error; $\mathbf{I}_{2n}$ denotes the $2n \times 2n$ identity matrix; $\mathbf{H}_k$ is the spectral density matrix of the model response and its (i, j) entry is given by eq. 5.2.

$$\mathbf{H}_k(i, j) = S_{ij} [(\beta_{ik}^2 - 1) + \mathbf{i}(2\zeta_i\beta_{ik})]^{-1} [(\beta_{jk}^2 - 1) + \mathbf{i}(2\zeta_j\beta_{jk})]^{-1} \quad (5.2)$$

where $\beta_{ik} = f^{(i)}/f_k =$ frequency ratio; $f^{(i)}$ and ζ_i are the natural frequency and damping ratio of the i th mode respectively and S_{ij} is the force cross spectral density between the i th and j th modal excitation. The modal identification is accounted for by the first term in eq. 5.1 and the prediction error is accounted for by the second term.

Using Bayes Theorem, the posterior PDF of θ given the FFT data is proportional to the likelihood function $p(\{\mathbf{Z}_k\}|\theta)$ under the assumption that the prior distribution is uniform, sometimes termed an uninformative prior [110, 111, 221]:

$$p(\theta|\{\mathbf{Z}_k\}) \propto p(\{\mathbf{Z}_k\}|\theta) \quad (5.3)$$

It can be shown that the distributions of $\mathbf{Z}_k$ at different frequencies are independent. Therefore, the distribution considering all frequencies in an FFT is given by the product of the distribution at each individual frequency. Thus, the posterior PDF, which is to be maximised, is proportional to the product of likelihood functions such that:

$$p(\theta|\{\mathbf{Z}\}) \propto p(\{\mathbf{Z}\}|\theta) = (2\pi)^{-(N_q-1)/2} \left[\prod_{k=2}^{N_q} \det \mathbf{C}_k(\theta) \right]^{-1/2} \times \exp \left[-(1/2) \sum_{k=2}^{N_q} \mathbf{Z}_k^T \mathbf{C}_k(\theta)^{-1} \mathbf{Z}_k \right] \quad (5.4)$$

The dependence of $\mathbf{C}_k$ on θ is highlighted in the equation, so it is possible to calculate, and therefore maximise, $p(\mathbf{Z}|\theta)$ for a given θ . While the posterior distribution $p(\theta|\mathbf{Z})$ is unknown, thanks to the proportionality defined in eq. 5.4 it can be maximized by maximizing the known $p(\mathbf{Z}|\theta)$ distribution. This process is made more efficient if eq. 5.4 is written in terms of the log-likelihood function $L(\theta)$:

$$p(\theta|\{\mathbf{Z}_k\}) \propto \exp[-L(\theta)] \quad (5.5)$$

with $L(\theta)$ defined as in eq. 5.6:

$$L(\theta) = (1/2) \sum_{k=2}^{N_q} [\ln \det \mathbf{C}_k(\theta) + \mathbf{Z}_k^T \mathbf{C}_k(\theta)^{-1} \mathbf{Z}_k] \quad (5.6)$$

It can be shown that the posterior PDF can be approximated by a Gaussian PDF for a sufficiently large data set [110]. To do this, the second order approximation of $L(\theta)$ is taken by letting $\hat{\theta}$ be the most probable value (MPV) that minimises L . By expanding as a second order Taylor series about $\hat{\theta}$ with the first-order term vanishing due to optimality of $\hat{\theta}$.

$$L(\theta) \approx L(\hat{\theta}) + \frac{1}{2}(\theta - \hat{\theta})^T H_L(\hat{\theta})(\theta - \hat{\theta}) \quad (5.7)$$

where the Hessian of L at the MPV is $H_L(\hat{\theta})$. The posterior PDF becomes a Gaussian PDF by substituting into eq. 5.5.

$$p(\theta | \{\mathbf{Z}_k\}) \propto \exp \left[- (1/2)(\theta - \hat{\theta})^T \hat{\mathbf{C}}^{-1}(\theta - \hat{\theta}) \right] \quad (5.8)$$

where $\hat{\mathbf{C}}$ is the posterior covariance matrix

$$\hat{\mathbf{C}} = H_L(\hat{\theta})^{-1} \quad (5.9)$$

The calculation of both the MPV and covariance matrix are required for the computation of the Gaussian PDF. Therefore, the goal of the BFFT method is to estimate these two variables so that $L(\theta)$ is minimised.

Computationally, this can be challenging, both in terms of time and achieving convergence. Au [110] proposed a more efficient method to perform the minimisation that avoids the need to calculate the determinant and inverse of $C_k(\theta)$, as required in eq. 5.6. It is shown that if it is assumed that the signal-to-noise ratio is high, an approximate, or ‘asymptotic approach can reduce the dimensionality of the minimisation problem and further improve computational speed and convergence. In this study damping is estimated using the asymptotic approach.

As ambient acceleration signals collected by monitoring campaigns are often considerably long to ensure significant enough data is available for processing, the data set can be broken down into non-concurrent shorter subsignals and processed separately. The estimated natural frequency and damping ratio from each subsignal can then be averaged to obtain the estimates from this method. The purpose of the signal decomposition is to improve efficiency in processing time and account for possible outliers resulting from numerical issues related to convergence of the minimisation problem.

5.2.2 Random Decrement Technique

The second approach employed to estimate the damping ratio of the structure is the Random Decrement Technique (RDT). This was first proposed for use in the aerospace industry by Cole [93] and has been applied extensively throughout the fields of Civil and Structural Engineering for damping estimation of structures [70, 71, 94, 106]. The RDT is based on the basic idea that white noise excitation of a single degree of freedom (SDOF) system will result in a response $x(t)$, which consists of the response due to initial displacement x_{x_0} , the response due to the initial velocity $x_{\dot{x}_0}$ and forced response due to random excitation x_F such that

$$x_{ni}(t) = x_{x_0} + x_{\dot{x}_0} + x_F \quad (5.10)$$

The RDT can be used to estimate the damping experienced by a linear structural system examining a ‘signature’ obtained from averaging segments of the response [70, 224]. These segments are sub-sections of the time history of the acceleration response which follow a threshold condition [13, 70, 93, 94], typically a triggering value, x_p , which satisfies both amplitude and slope criteria. By setting x_p at an appropriate level to intercept the time series of the data $x(t)$, multiple sub-signals $x_{ni}(t)$ can be obtained. Assuming the forced vibration and the initial velocity of the structure, are zero-mean stationary Gaussian variables, the expectation of the sub-excitation $E[x_F]$ and of the sub response $E[\dot{x}_0]$ are zero. Therefore, by averaging a large number of sub-signals with identical triggering conditions, the ensemble average of the initial velocity and forced vibration responses reduce to zero, leaving only the response due to the initial displacement:

$$E[x_{ni}(t)] = E[x_{x_0}] + E[x_{\dot{x}_0}] + E[x_F] = E[x_{x_0}] \quad (5.11)$$

This means that essentially the random component of the response is removed leaving a signal comprising only the free decay response of the structure. This concept is visualised in Figure 5.1.

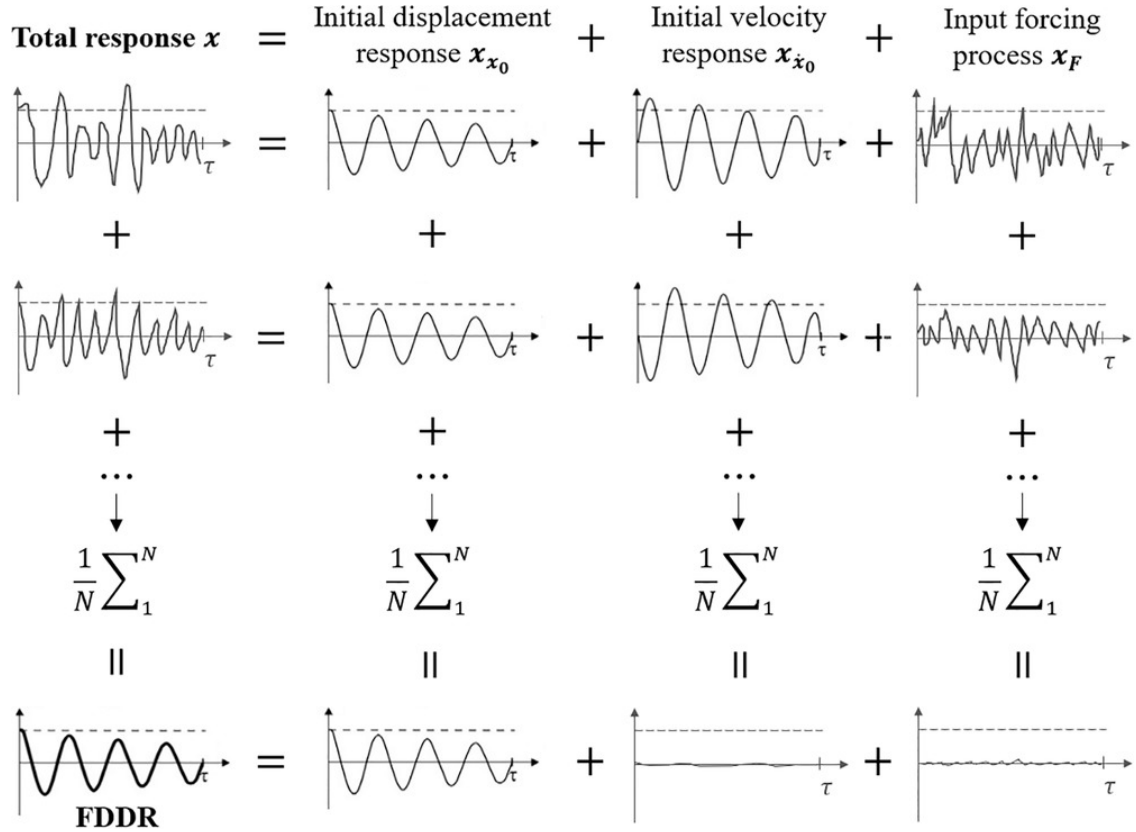


Figure 5.1: Conceptualisation of the Random Decrement Technique (Source: Zhou and Li [70])

Due to the limited duration of data measurement in practice, the expectation of the sub response is taken as the arithmetic mean across each sub-signal extracted. Therefore the random decrement signature, $\delta(t)$, is defined as:

$$\delta(t) = \frac{1}{N} \sum_{k=1}^N x_{ni}(t_k + \tau) \quad (5.12)$$

where N = number of subsamples and $\tau = t - t_i$.

Once the free vibrational response has been determined, the Hilbert transform is applied to the random decrement signature $\delta(t)$ to approximate the free decay response and determine the damping ratio ζ_i [94, 225]. For multi-degree of freedom systems, simply assuming a Single Degree of Freedom system and applying the RDT, termed the ‘Traditional RDT’ by Tamura et. al [226] can efficiently evaluate the natural frequency and damping ratio if the modes are well separated [226, 227].

5.2.3 Analytical Mode Decomposition

While the Traditional RDT can work for MDOF systems if the modes are well separated, if there are closely located predominant frequency components, a beating phenomenon is observed in the random decrement signature and the RDT cannot be used for evaluation of damping ratio [106, 225, 226]. To overcome this issue the RDT is commonly combined with a signal decomposition method. A conceptually straight-forward decomposition approach is to employ a band-pass filter, as in Zhou and Li [70] for example. However it is difficult to design a filter for a complete separation of the two frequency components as the spacing of the two frequencies becomes very small [99]. Empirical Mode Decomposition (EMD) is one method which has been shown to yield good results when combined with the RDT [71, 225]. However EMD has many shortfalls, the most problematic of which is that it does not accurately decompose signals with closely spaced frequencies or beating which are common in structural systems [97, 98, 99, 106]. Others have examined the use of Analytical Mode Decomposition (AMD) for signal decomposition and found it superior to EMD and to be highly effective for signals with highly coupled modes [97, 98, 99]. In this study, the AMD is combined with the RDT for modal identification from ambient data as presented by [97, 106]. The AMD decomposes a signal into two components, separated by a bisecting frequency ω . Each sub-signal can then be analysed using the RDT outlined in Section 5.2.2 to extract the free decay response of the structure and determine its damping ratio.

Let $\mathbf{x}(t)$ denote the measured acceleration data containing a number of frequency components $(\omega_1, \omega_2, \dots, \omega_n)$ where n is the number of sub-signals in which the data is to be decomposed. The subsignals, $\mathbf{x}_i(t) (i = 1, 2, \dots, n)$ have Fourier Spectra $\hat{\mathbf{X}}(\omega)$ which cover n mutually exclusive frequency ranges such that $(|\omega| < \omega_{b1}), (\omega_{b1} < |\omega| < \omega_{b2}), \dots, (\omega_{(bn-2)} < |\omega| < \omega_{(bn-1)})$ and $(\omega_{(bn-1)} < |\omega|)$.

$\omega_{bi} \in (\omega_i, \omega_{i+1}) (i = 1, 2, \dots, n - 1)$ are the bisecting frequencies. Therefore,

$$\mathbf{x}(t) = \sum_{i=1}^n \mathbf{x}_i(t) \quad (5.13)$$

Each of the modal responses has a narrow bandwidth in the frequency domain and can be determined by

$$\mathbf{x}_i(t) = \mathbf{s}_i(t) - \mathbf{s}_{i-1}(t), \dots, \mathbf{x}_n(t) = \mathbf{x}(t) - \mathbf{s}_{n-1}(t) \quad (5.14)$$

$$\mathbf{s}_i(t) = \sin(\omega_{bi}t)H[\mathbf{x}(t)\cos(\omega_{bi}t)] - \cos(\omega_{bi}t)H[\mathbf{x}(t)\sin(\omega_{bi}t)] \quad (5.15)$$

where $H[\cdot]$ represents the Hilbert Transform.

The AMD method provides an alternative to the band pass filtering technique which, as mentioned, performs poorly when modes become very close [99]. Other than possible end effects due to the finite length of a structural systems response, the AMD separates the frequency contents of a time series exactly. It has been shown that the selection of the bisecting frequencies does not influence the response decomposition [97, 99]. However, typically bisecting frequencies, ω_{bi} , are selected around the average of the two adjacent natural frequencies [97, 98, 99, 106].

After application of the AMD method to create sub-signals of the measured acceleration response, the RDT is applied to obtain the free vibrational response of the structural system and identify the damping ratio. In this study, damping is calculated using the RDT with and without AMD.

5.2.4 Bootstrapped AMD-RDT

Whilst use of the RDT is prominent in the literature on modal identification [70, 71, 224], previous applications of the method have not provided any measure of the uncertainty in its damping estimates. To address this deficiency, a novel hybrid method is developed which incorporates the statistical procedure known as Bootstrapping within the combined AMD-RDT method to obtain a statistical measure

of the error in the calculated damping ratio values, and hence, an understanding of the uncertainty of damping estimates obtained using the AMD-RDT. Bootstrapping is a statistical procedure involving the resampling of a single dataset to create a number of simulated datasets in which statistical parameters such as standard errors and confidence intervals can be calculated. This research is the first of the authors knowledge that considers the combined method of AMD and the RDT, known as the AMD-RDT, merged with bootstrapping to provide estimates of the damping ratio alongside its associated standard error.

Bootstrapping is a computationally expensive statistical procedure which enables descriptive features of a sample to be assessed. The principle of bootstrapping is to treat a sample as though it is the population and randomly sample from this to produce an empirical estimate of the statistic's sampling distribution [70, 100, 228]. The bootstrap procedure is applied in this methodology as first presented by Efron et al. [228] and involves the following steps:

1. A random independent sample, $X = (x_1, x_2, \dots, x_n)$ with a statistic of interest $\hat{\theta} = s(X)$ is drawn from an unknown identical distribution F.
2. The original data is sampled with replacement to create a bootstrap sample $X^* = (x_1^*, x_2^*, \dots, x_n^*)$ with a corresponding estimator $\hat{\theta}^* = s(X^*)$.
3. The bootstrap operation in step 2 is repeated B times to create a bootstrap ensemble containing B number of replicates $(\hat{\theta}_1^*, \hat{\theta}_2^*, \dots, \hat{\theta}_B^*)$.
4. The histogram of the bootstrap ensemble can be produced in order to identify the probability density function and hence calculate the bootstrap mean, $\bar{\theta}^*$, considered the optimal estimate by the bootstrap method using eq. 5.16, assuming a normal distribution. The standard deviation s_{θ}^* , which can be regarded as an estimate of the standard error of $\hat{\theta}$ can also be found using eq. 5.17 for a normal distribution. The coefficient of variation can be found using eq. 5.18.

$$\bar{\theta}^* = \mu(\hat{\theta}_b^*) = \frac{1}{B} \sum_{b=1}^B \hat{\theta}_b^* \quad (5.16)$$

$$\hat{s}_{\theta}^* = \sigma(\hat{\theta}_b^*) = \sqrt{\frac{1}{B-1} \sum_{b=1}^B (\hat{\theta}_b^* - \bar{\theta}^*)^2} \quad (5.17)$$

$$CV = \frac{\hat{s}_{\theta}^*}{\bar{\theta}^*} \quad (5.18)$$

where $\mu(\blacksquare)$ and $\sigma(\blacksquare)$ represent the operation of calculating the mean value and standard deviation respectively. Figure 5.2 illustrates the bootstrap procedure as applied to the AMD-RDT.

Relating the statistical method of bootstrapping to the context of the AMD-RDT as shown in Figure 5.2, the statistic of interest $\hat{\theta} = s(X)$ is the damping ratio, ζ . The sample population is obtained from segments of the acceleration response, after it has undergone analytical mode decomposition, which exceed the threshold value. As previously stated, the threshold value is set as the standard deviation of the acceleration response. The length of each segment is set as fifty times the period of the structure. The segments which form the sample can be overlapping. This is known as ‘level-crossing overlapping’ criterion and was found to be the criterion that yields the least error by Zhou et al. [70]. The sample population is then randomly sampled to create a ‘bootstrap sample’ containing ‘n’ number of segments. In this case ‘n’ was set as 1000. This process was repeated ‘B’ times, in this case 1000 times, to create a set of 1000 bootstrap samples.

The RDT is then applied to each bootstrap sample to obtain an estimate of the damping ratio, $E_{\zeta b}^*$, known as a bootstrap replicate. A histogram of the estimates from each bootstrap sample can then be plotted to confirm the distribution. Eq. 5.16 - 5.18 can be used to calculate the mean estimate of the damping ratio, $\bar{E}_{\zeta}^*$, the standard error of the estimate, $\hat{s}_{E_{\zeta}}^*$, and the coefficient of variation, CV, if the distribution is found to be normal. In eq. 5.16 - 5.18, $\bar{\theta}^*$ is the optimal or mean estimate from the bootstrap method, $\bar{E}_{\zeta}^*$, $\hat{\theta}_b^*$ is the bootstrap replicate for each sample, $E_{\zeta b}^*$, and $\hat{s}_{\theta}^*$ is the standard error of the estimate, $\hat{s}_{E_{\zeta}}^*$.

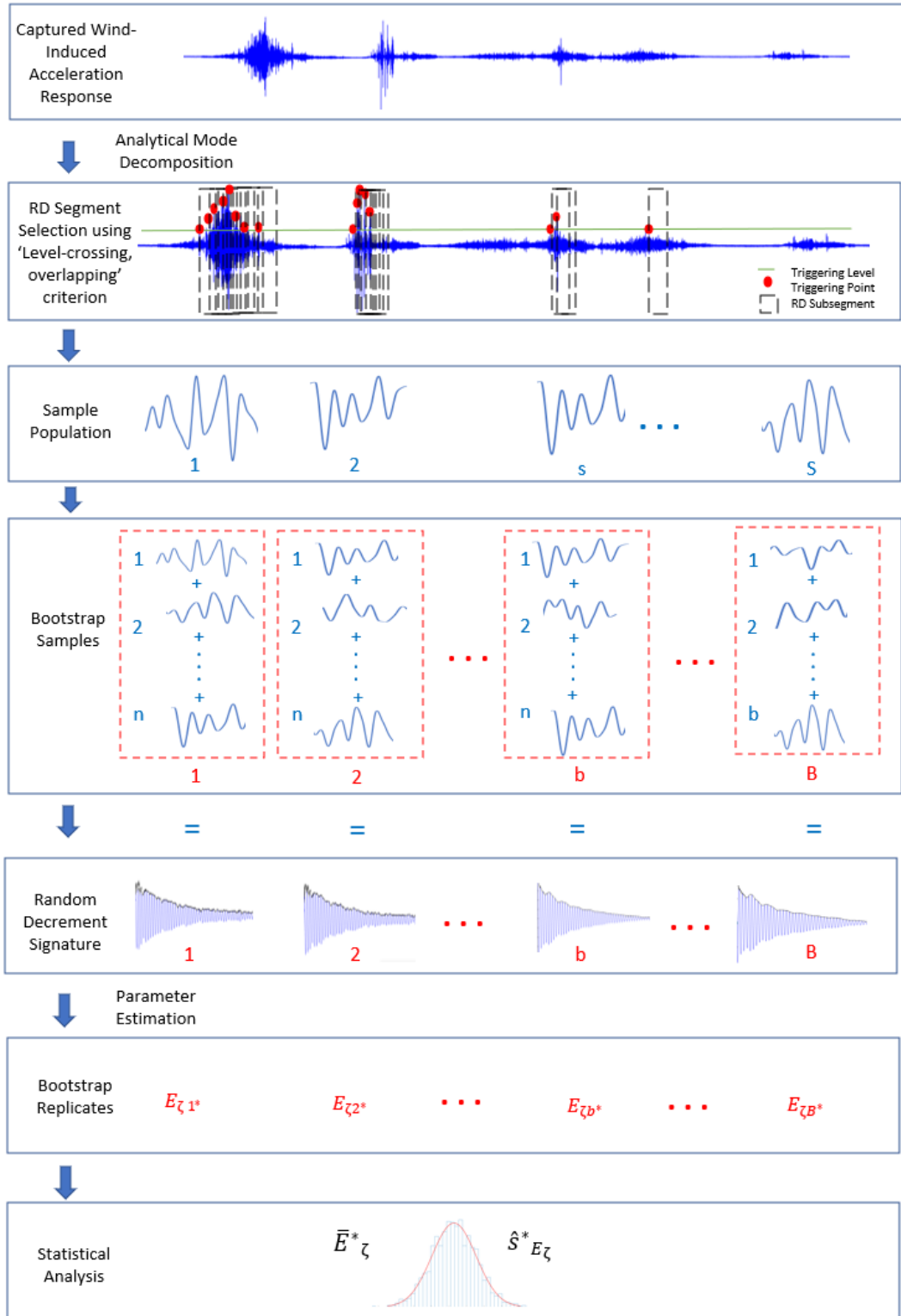


Figure 5.2: Illustration of the Bootstrap method as applied to the AMD-RDT method

5.3 Monitoring

The details of the monitoring which was completed as part of the field testing of *Ten Degrees* and *College Road* are provided below. This includes details of the equipment used, the locations in which sensors were located to record measurements and the stage at which construction was at when each measurement was taken.

5.3.1 Ten Degrees

The first case study structure to be monitored was *Ten Degrees*. Details of this structure are provided in Section 4.2. Two three-axis MEMS accelerometers, (*CX1 Structural Response Monitor*, resolution $0.00001g$, range $\pm 1.5g$, frequency range $DC - 200Hz$) were directly mounted approximately 1.84 m above the floor slab at level 43, i.e. just below roof level in the taller core, as pictured in Figure 5.3. The sampling rates for the accelerometers was 10 Hz. The accelerometers were installed off centre to capture lateral and torsional vibrations as shown in Figure 5.4. On the roof of the core, a weather station was installed to record 10-minute averaging wind speed and direction, maximum/minimum wind speed and direction within each 10-minute window, temperature, humidity, atmospheric pressure, rain and its duration. A 3G router was also installed to allow for remote access to all data. Data from the weather station was continuously monitored.



Figure 5.3: Picture of accelerometer and data logger used in full-scale monitoring

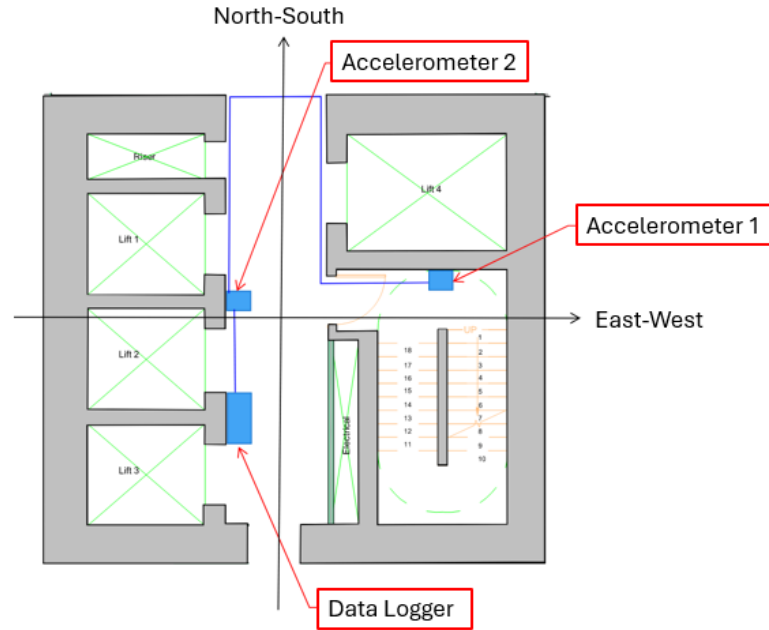


Figure 5.4: Location of accelerometers and data logger used in *Ten Degrees*

Construction of the cores to full height was completed in late 2018, prior to installation of any modules. As pictured in Figure 5.5, volumetric modules were stacked around two RC cores. Monitoring of the structure was undertaken over a two month period, beginning in late August 2019 and ending in late October 2019. Not all modules were installed when the monitoring campaign began. Therefore, initial measurements were taken on a partly completed structure.

In total, 10 sets of measurement data were recorded. A measurement set comprises of 12 hours of acceleration and wind speed data from before and after an instance in which the recorded acceleration exceeded a predetermined threshold value for at least 10 seconds. As mentioned, the module installation was not complete when monitoring commenced and continued over the period when measurements were taken, resulting in monitoring being performed on an evolving structure. In this study, the measurement data are presented in chronological order with the structure becoming progressively closer to completion as the sets of measurements were taken. The structure was fully complete by measurement 8 with all floors of modules installed. Table 5.1 shows the dates and number of storeys of modules installed for each measurement taken. Figure 5.6 presents a graphical representation of the storeys of modules installed when each set of measurement data was recorded.

Comparing Figures 5.5 and 5.6, it can be appreciated how initial measurements were performed on a structure approximately corresponding to the right hand picture in Figure 5.5. Furthermore, from Figure 5.6 it can be seen that there was a significant increase in modules installed between measurements 5 and 6.



Figure 5.5: The *Ten Degrees* building in Croydon, London, pictured at two points in time during module installation (Source: Tide Construction [216])

The evolution of the structure as measurements were taken results in damping ratios and natural frequencies that differ between measurements and reflect the change in the inherent properties of the structure. As modules are added to the upper levels of the structure, the mass of the structure increases. This is expected to affect the dynamic response significantly as the mass increase occurs at the top of the structure where it has most influence. The increase in the number of modules will also influence overall lateral stiffness. However, in contrast to the additional mass, the stiffness may not increase significantly due to the modules being added at the top of the structure, further from the point of most influence at the base. Therefore, a decrease in the natural frequency estimated by the modal analysis performed on each consecutive set of measurement data is expected, which affects response amplitude observed in each measurement, as presented later.

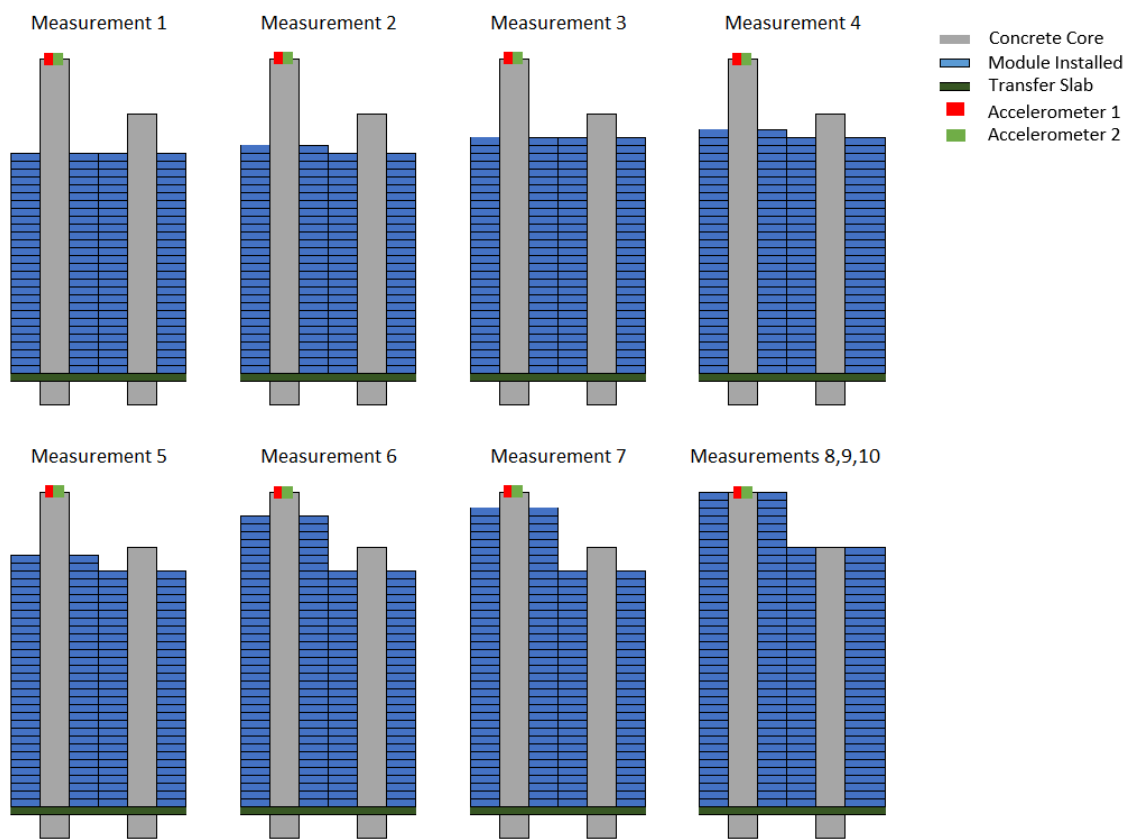


Figure 5.6: Graphical representation of the number of levels of modules installed at *Ten Degrees* for each set of measurement data

Table 5.1: Dates measurement data was recorded *Ten Degrees* and the number of floors of modules installed

Measurement	Date	Number of Storeys completed	
		Tower A	Tower B
1	31/08/2019	32	32
2	02/09/2019	33	32
3	06/09/2019	34	34
4	09/09/2019	35	34
5	11/09/2019	36	34
6	27/09/2019	41	34
7	29/09/2019	42	34
8	11/10/2019	44	37
9	13/10/2019	44	37
10	26/10/2019	44	37

5.3.2 College Road

College Road is the second of the two case study structures to be studied through field testing. More details of the structural form of *College Road* are provided in Section 4.3. Two three-axis MEMS accelerometers, (*LSM6DS3 Inertial Measurement Unit*, resolution 0.00001g, range $\pm 2g$, frequency range DC 800Hz) were used to measure the ambient acceleration response of *College Road*. The sampling rates for the accelerometers was 20 Hz. Like *Ten Degrees*, *College Road* was monitored both during the construction process as modules were being installed, and after all modules had been installed for five months respectively. During the initial five months as modules continued to be installed, one accelerometer, Accelerometer 1, was placed at the top storey of the core of Tower A and the other, Accelerometer 2, at the top storey of the core of Tower B. Both accelerometers were mounted as close to the centre of the core as possible, approximately 0.8 m above the respective core slab on a rigid, steel post. Upon completion of construction, Accelerometer 2 located at the top of Tower B was moved to the top of Tower A and mounted directly to a module slab at the outer edge of the building. The purpose of this was to allow scope for future work relating to the torsional modes. The locations

of Accelerometer 1 and 2 are shown in Figure 5.7 (left) for the initial five months of monitoring during module installation and in Figure 5.7 (right) for the final five months when all modules had been installed. Remote access to the recorded data was enabled by using off the shelf smartphones which are manufactured with high specification accelerometers. The phones were given a constant power supply and had 3G fixed wireless Internet access.

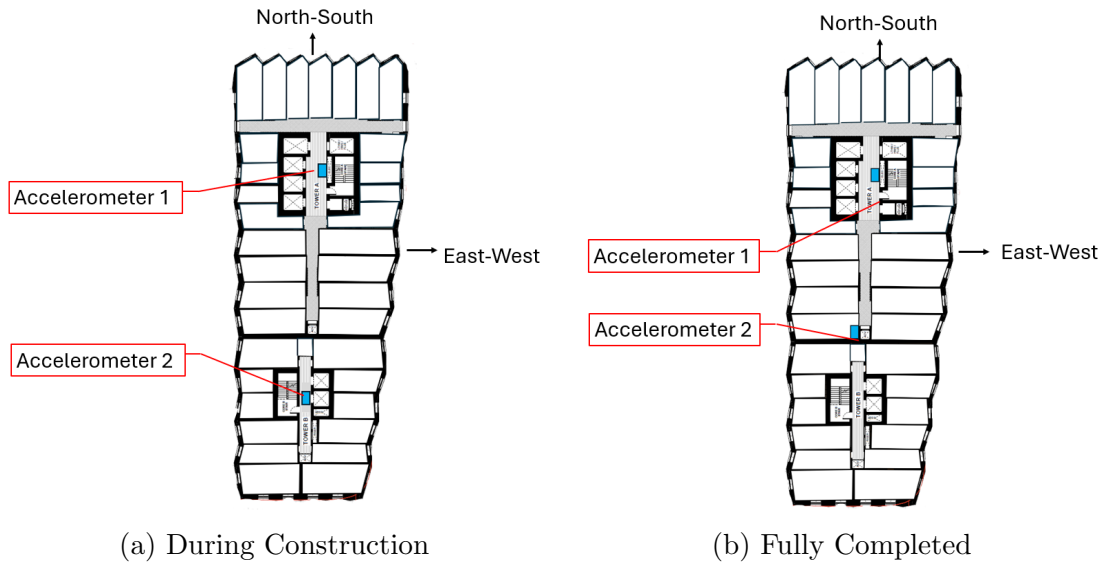


Figure 5.7: Location of accelerometers during initial 5 months (a) and final 5 months (b) of the monitoring campaign

The RC concrete cores of *College Road* were completed in late 2021. The installation of modules in Tower A commenced in November 2021 and in Tower B in May 2022. The initial five months of monitoring, during which modules continued to be installed, took place between December 2021 and April 2022. Throughout this phase of monitoring the first 20 storeys of modules were installed in Tower A and no modules were installed in Tower B. This resulted in Accelerometer 1 on Tower A capturing the initial stages of module installation, in contrast with the monitoring of *Ten Degrees* where monitoring captured the final storeys of modules being installed. During the entire initial monitoring of *College Road*, Tower B had no modules installed and the freestanding dynamic response of the RC core was captured by Accelerometer 2.

Installation of the modules was completed in October 2022. The second phase of monitoring began in December 2022 and finished in April 2023. During this phase construction was ongoing, with electrical and mechanical fit out and external

cladding being installed. However, excessive construction noise in the acceleration signals which were used for modal identification was avoided by using data recorded between the hours of 9pm and 5am when there was no construction work taking place.

In total, seven measurements of one hour length from each of the accelerometers were chosen from the initial set of monitoring during module install. Four measurements were chosen from the data recorded when the structure was fully complete. However, power was cut to the second accelerometer for the fourth measurement and so there is no data available for Measurement 11 from Accelerometer 2. Measurements were chosen by correlating large accelerations in the measurements with significant wind events noted in records from The Met Office for Heathrow Airport, London, United Kingdom [229]. Consideration was also given to choosing measurements where the RMS acceleration of the response was similar to the RMS accelerations during measurements of *Ten Degrees*. Table 5.2 shows the measurement number, date and the number of storeys of modules which were installed in the building. Figure 5.8 visually depicts the progress of construction at the time of each measurement.

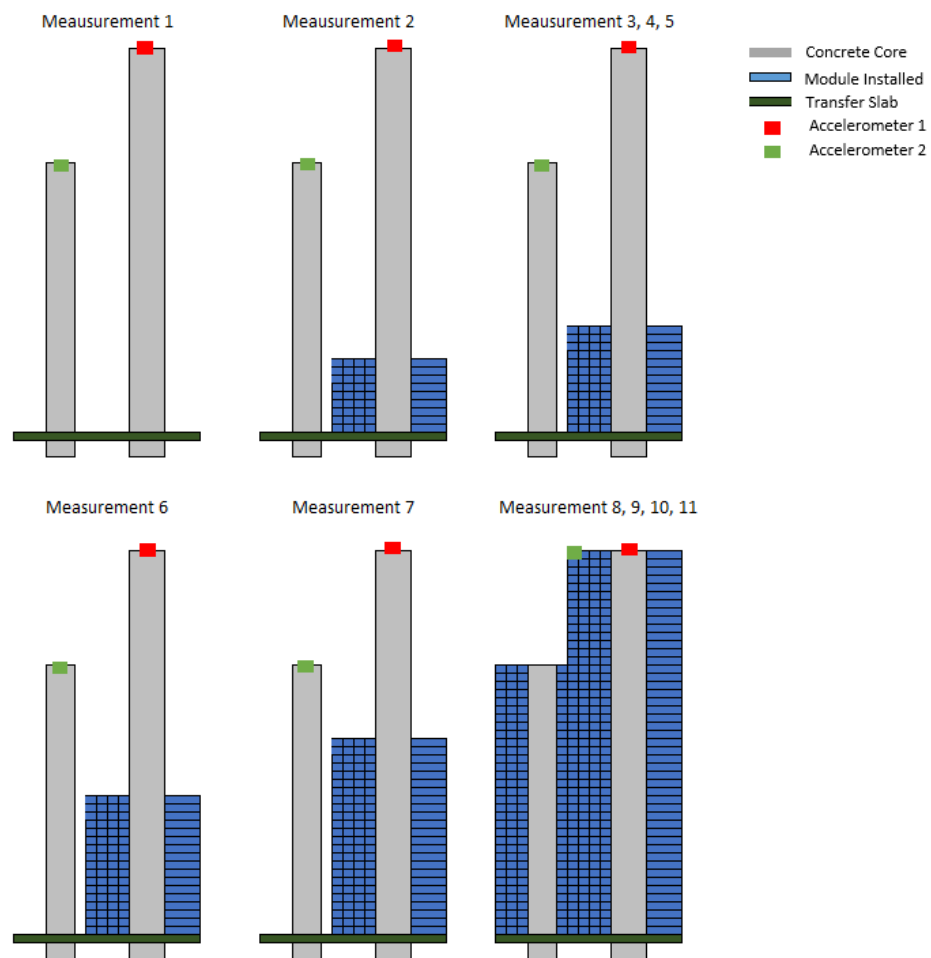


Figure 5.8: Graphical representation of the number of levels of modules installed at *College Road* for each set of measurement data

Table 5.2: Dates measurement data was recorded at *College Road* and the number of floors of modules installed

Measurement	Date	Number of Storeys completed	
		Tower A	Tower B
1	07/12/2021	0	0
2	30/01/2022	9	0
3	17/02/2022	13	0
4	18/02/2022	13	0
5	18/02/2022	13	0
6	19/03/2022	17	0
7	07/04/2022	24	0
8	13/01/2023	50	35
9	18/02/2023	50	35
10	13/03/2023	50	35
11	12/04/2023	50	35

5.4 Processing Measurements from In Situ Monitoring

5.4.1 Ten Degrees

Assessment of the ten wind and acceleration measurements recorded at *Ten Degrees* are provided below. The identified inherent modal properties of the structure are presented.

5.4.1.1 Measured Wind Loading

The ambient wind speeds were recorded during monitoring of *Ten Degrees*, this allowed for identification of significant wind loads and correlation between the observed wind load and measured acceleration response of the structure. As the data recorded is ambient and this is not a controlled test, the wind exciting the structure and therefore the accelerations experienced vary across measurements.

Figure 5.9 shows the average wind speed for each 10 minute period over the course of each of the 10 measurements. This allows the variation in average wind speed both within and between measurements to be examined. Average wind speeds were more significant for later measurements (after Measurement 6). This may be due to these measurements occurring later in September and October when storms and strong winds are typically more common. Another possible explanation for this is that more modules were installed in this structure for the later measurements, the structure was close to complete/completed and therefore there was more significant surface area for winds to be deflected upwards and towards the weather station. However, this is difficult to prove without completing CFD or wind tunnel tests for the structure at different stages of completion.

The Average and Maximum Wind Speed for each measurement are given in Table 5.3. Again, it can be seen that both average and max wind speeds have increased for Measurement 6 onward. For context, it is worth highlighting that the basic 10-minute average 50-year wind speed at 10 m is approximately $21.5m/s$ and the one-year 10-minute average wind speed at 10 m used to assess serviceability limit state performance is approximately $16.11 m/s$.

Table 5.3: Average and maximum wind speeds for each in situ measurement of response recorded at *Ten Degrees*

Measurement	Average Wind Speed (m/s)	Max Wind Speed (m/s)
1	7.00	15.93
2	6.66	14.04
3	8.23	16.83
4	6.28	13.78
5	8.99	17.50
6	11.65	23.92
7	12.35	20.20
8	13.40	23.23
9	10.99	20.47
10	12.22	25.54

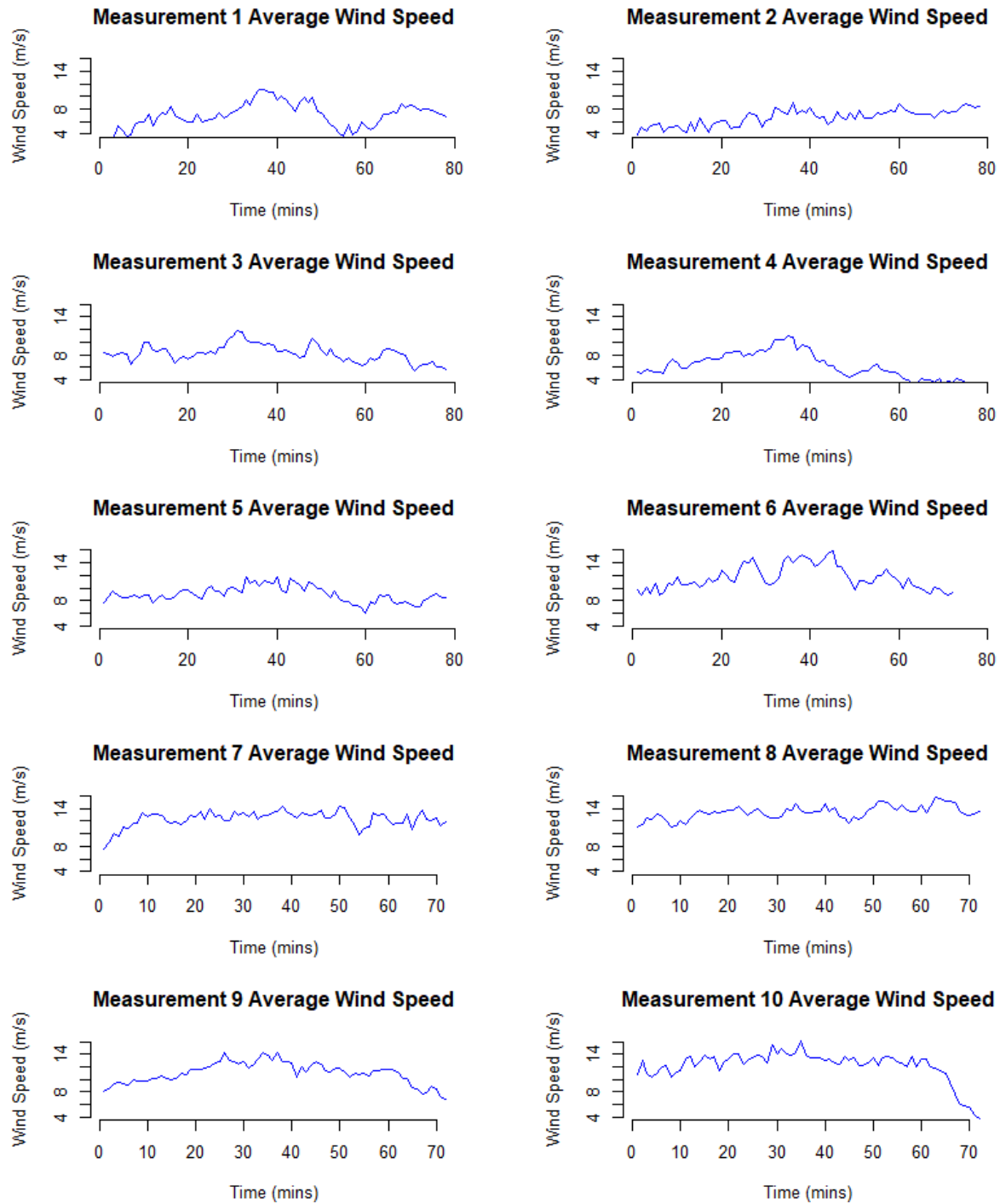


Figure 5.9: Average wind speed recorded at *Ten Degrees*

5.4.1.2 Acceleration Response

Due to the nature of the ambient in situ tests, the uncontrolled and varying wind speeds result in the acceleration response of the structure fluctuating and differing between measurements. Furthermore, as the testing was conducted whilst the structure was under construction, it is possible that the measured acceleration responses can include noise. Through qualitative assessment of the raw acceleration responses of the structure it is possible to note variation between measurements and to identify any significant noise in the acceleration signals recorded.

Figures 5.10 and 5.11 show the acceleration response in the X (roughly East West) in red, Y (roughly North-South) in blue and Z (roughly vertical) in green directions for both accelerometers overlaid by the average wind speeds. The z-direction accelerations have had acceleration due to gravity removed. For Measurement 1, Measurement 6 and Measurement 8, Accelerometer 1 shows time segments with significant accelerations in the z-direction. This can be identified as noise due to construction equipment being used within the structure. It is possible that the signals from these measurements will be significantly distorted and may not yield useful results during modal analysis. Measurements 7 - 10 show accelerations which appear to match the profile of the average wind speeds quite well. However, larger accelerations can be seen at less significant wind speeds for Measurements 1-6; this is likely due to the structure not yet being complete.

Tables 5.4 and 5.5 show the maximum and RMS accelerations for each accelerometer, in the two primary directions, EW being X and NS being Y, for each measurement. The large values seen from Accelerometer 1 for Measurement 8 can be attributed to construction noise as previously discussed.

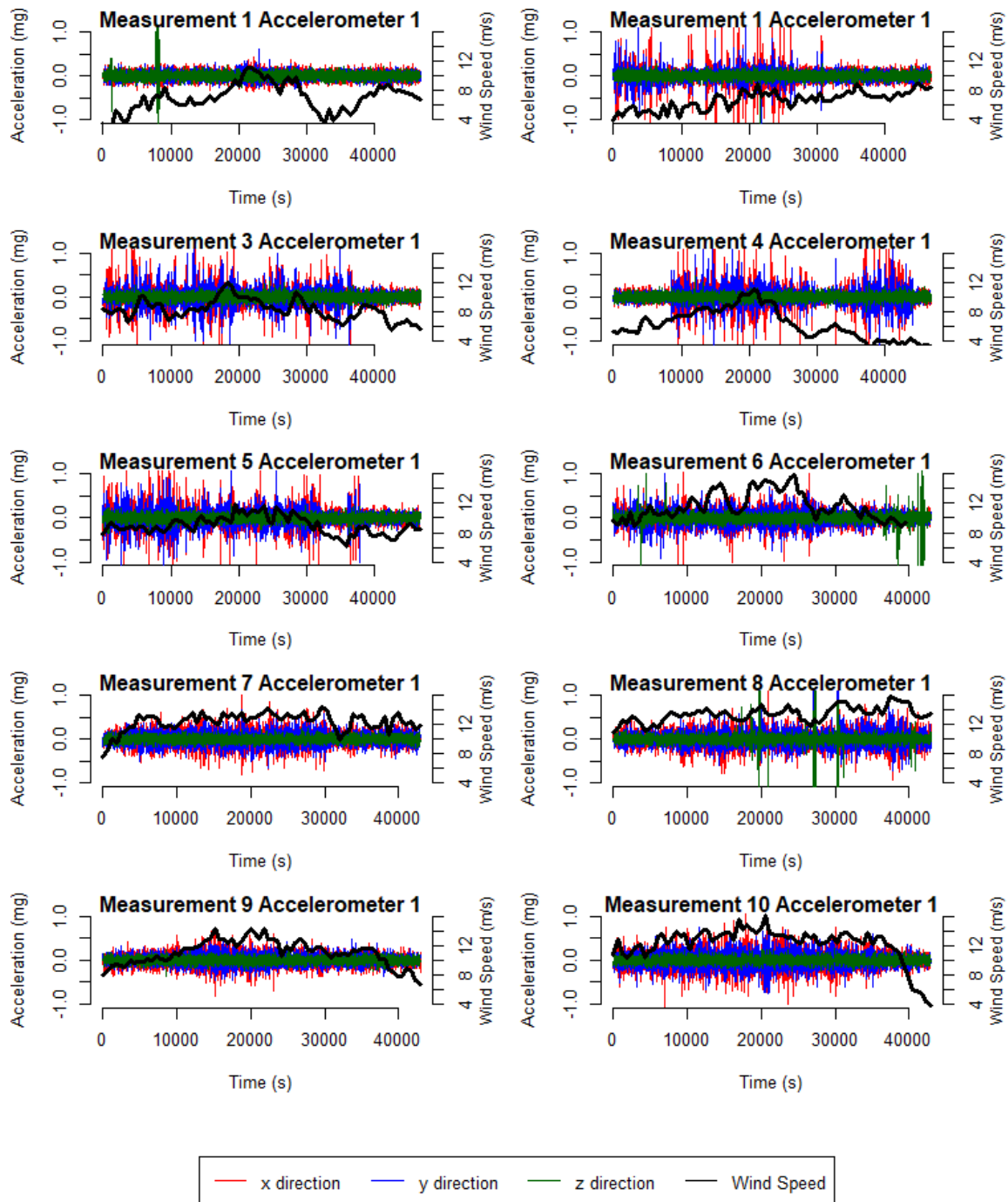


Figure 5.10: Acceleration and wind time histories for accelerometer 1 measured at *Ten Degrees*

5.4. Processing Measurements from In Situ Monitoring

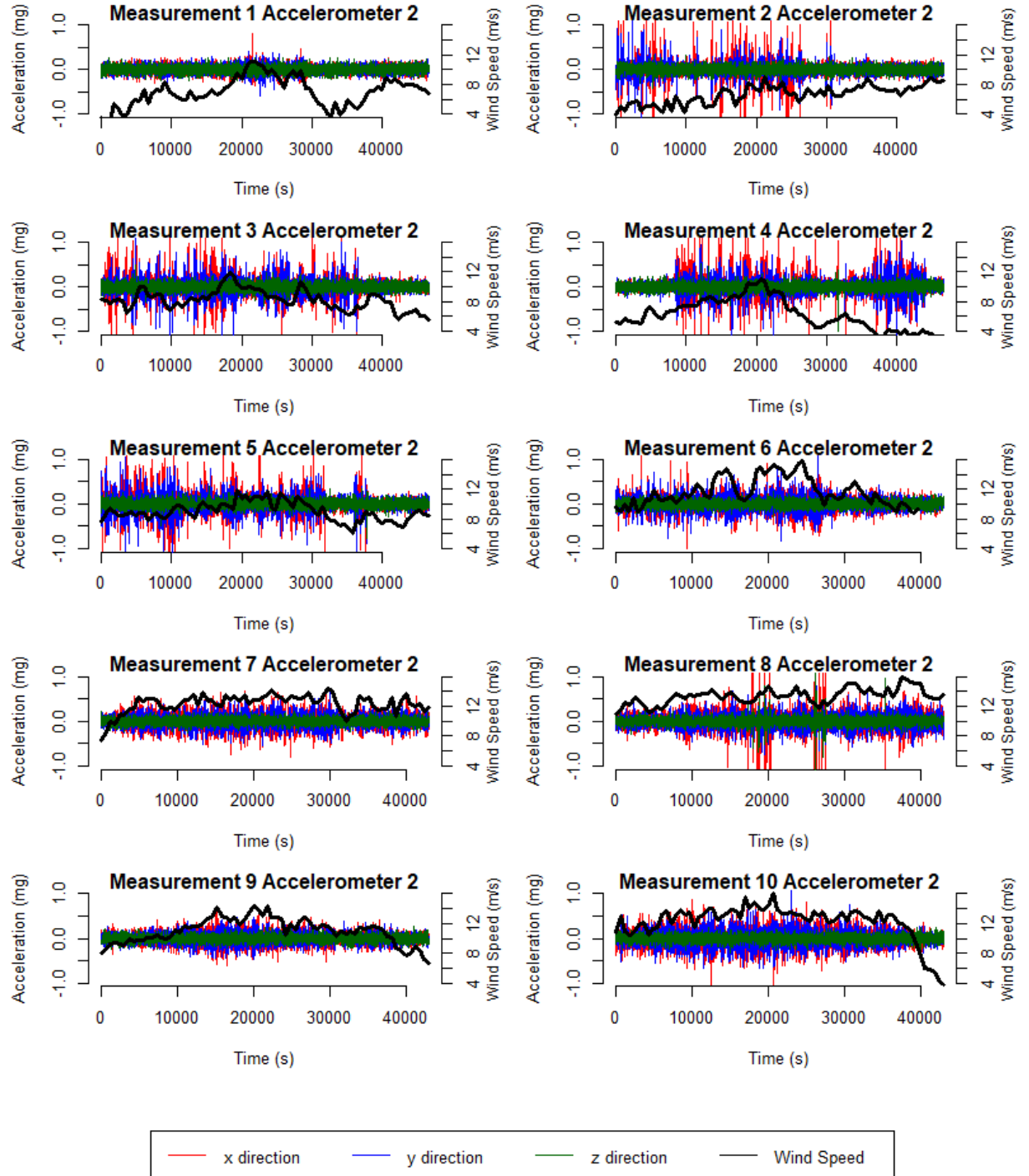


Figure 5.11: Acceleration and wind time histories for accelerometer 2 measured at *Ten Degrees*

Table 5.4: Maximum accelerations for each measurement taken at *Ten Degrees*

Measurement	Accelerometer 1		Accelerometer 2	
	Maximum	Maximum	Maximum	Maximum
	Acceleration EW (milli-g)	Acceleration NS (milli-g)	Acceleration EW (milli-g)	Acceleration NS (milli-g)
1	0.868	15.586	2.185	0.058
2	3.005	4.756	2.770	2.493
3	3.512	1.744	12.847	1.964
4	3.262	3.329	15.157	3.351
5	3.159	2.753	3.024	2.831
6	2.658	2.089	2.520	1.644
7	1.322	0.854	1.023	0.805
8	15.577	74.869	61.829	7.281
9	0.967	16.309	1.258	0.751
10	1.709	1.154	2.002	1.315

Table 5.5: RMS accelerations for each measurement taken at *Ten Degrees*

Measurement	Accelerometer 1		Accelerometer 2	
	RMS	RMS	RMS	RMS
	Acceleration EW (milli-g)	Acceleration NS (milli-g)	Acceleration EW (milli-g)	Acceleration NS (milli-g)
1	0.103	0.092	0.091	0.089
2	0.205	0.138	0.198	0.137
3	0.228	0.177	0.224	0.177
4	0.223	0.178	0.221	0.177
5	0.226	0.183	0.219	0.184
6	0.179	0.150	0.173	0.155
7	0.178	0.142	0.170	0.139
8	0.210	0.834	0.630	0.160
9	0.148	0.124	0.136	0.115
10	0.211	0.172	0.205	0.170

5.4.1.3 Results from OMA

Natural Frequency

Prior to application of each of the modal identification methods, Fast Fourier Transforms of each of the acceleration responses were taken in order to identify the natural frequency of the structure. Figure 5.12 shows the Fast Fourier Transform of the acceleration data from Accelerometer 1 and 2 for Measurement 10 in both x and y directions. Close inspection suggests two separate fundamental bending modes are found near 0.31 Hz and 0.36 Hz. The BFFT was applied first in order to find the first natural frequencies in both the X (EW) and Y (NS) directions. Table 5.6 shows the natural frequencies calculated from both accelerometers, in both directions, for each measurement. It can be concluded that the first natural frequency of the measurements taken from the completed structure (Measurements 8, 9 and 10) in the first mode in the EW direction is approximately 0.314 Hz and in the NS direction is approximately 0.370 Hz.

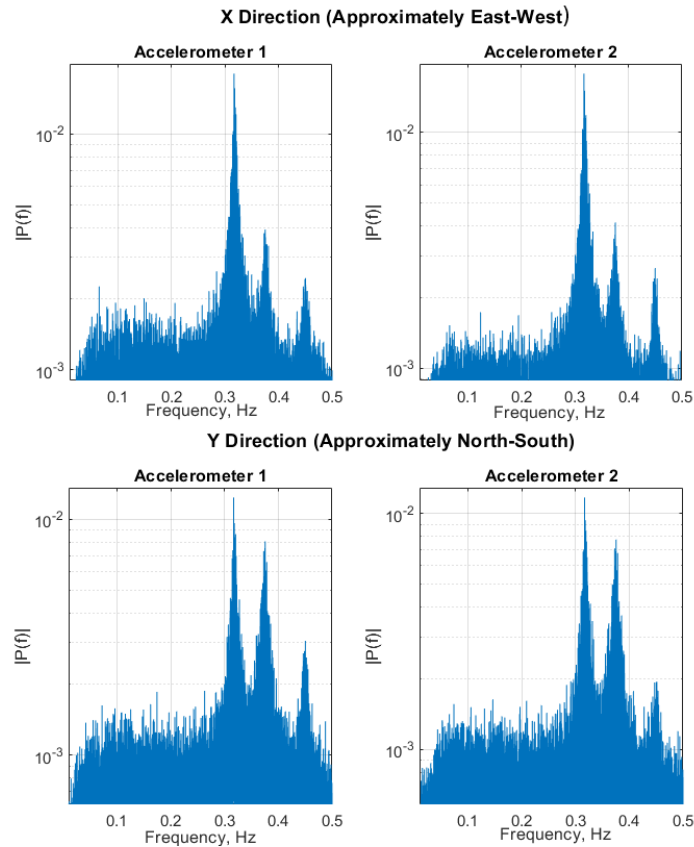


Figure 5.12: Fast Fourier Transform of Measurement 10 recorded at *Ten Degrees*

Measurement	Accelerometer 1		Accelerometer 2	
	Natural Frequency EW (Hz)	Natural Frequency NS (Hz)	Natural Frequency EW (Hz)	Natural Frequency NS (Hz)
1	0.370	0.417	0.370	0.417
2	0.368	0.415	0.368	0.415
3	0.360	0.410	0.360	0.410
4	0.359	0.407	0.359	0.407
5	0.350	0.406	0.351	0.401
6	0.321	0.375	0.321	0.375
7	0.323	0.377	0.323	0.377
8	0.316	0.368	0.315	0.371
9	0.317	0.374	0.317	0.374
10	0.308	0.365	0.307	0.365
Average (Measurements 8-10)	0.314	0.369	0.313	0.370

Table 5.6: Natural frequency calculated using the FFT of each measurement recorded at *Ten Degrees*

It can be seen that the values are in relatively close agreement between the two accelerometers and do not vary significantly across measurements. These values can also be compared to theoretical values calculated using eq. 3.4 in Section 3 of 0.35 Hz in the EW direction and 0.39 Hz in the NS direction, showing good agreement. The natural frequencies are slightly higher for earlier measurements, as is evident in Figure 5.13. This can be explained by the structure not yet being complete and therefore having a lower modal mass. For earlier measurements there are less modules and therefore, less mass and stiffness. As modules are added, the mass of the structure increases, as does the stiffness. However, since the natural frequency reduces as more modules are added, it can be concluded that the impact on natural frequency of any increase in the stiffness of the structure due to the additional modules is not as significant as the impact due to increased mass.

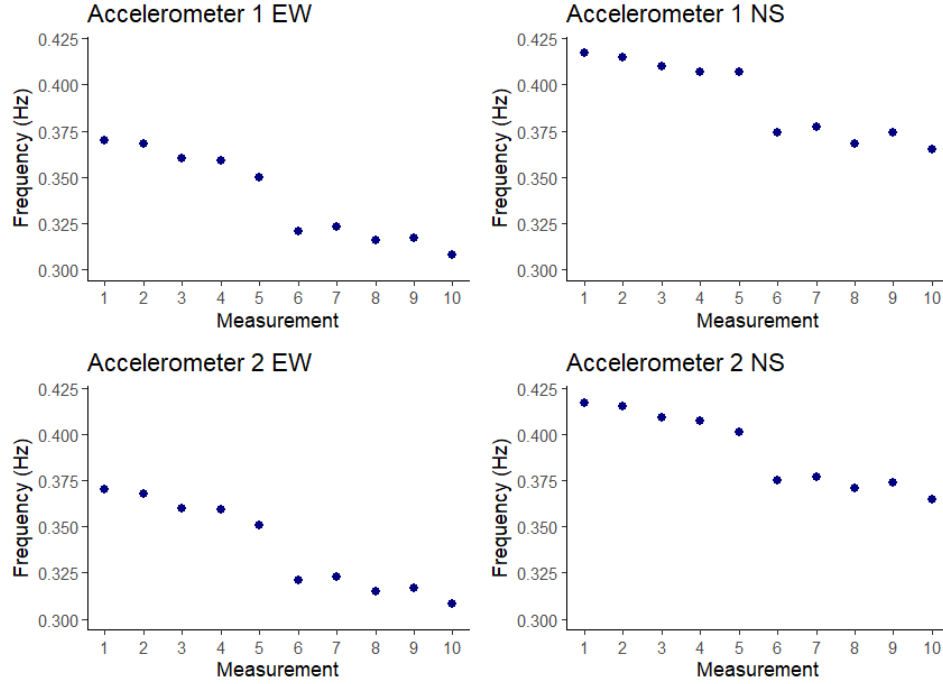


Figure 5.13: Natural frequencies for each measurement recorded at *Ten Degrees*

Damping Ratio

Each of the four modal identification techniques discussed in Section 5.2 were applied to the ambient data for each of the 10 measurements. Figure 5.14 shows the damping ratios calculated using the BFFT method for each of the 10 measurements. The damping ratios estimated by the BFFT method lie between 1 % and 2% and can be seen to be quite consistent across measurements.

Figure 5.15 shows the calculated damping ratio for the two described RDT methods for each of the 10 measurements. It can be seen that both methods are in close agreement for most sets of measurement data. However, the AMD-RDT method presents slightly lower estimates in most cases, particularly for measurements 4 and 8. The estimates from the AMD-RDT method are the same as those from the bootstrapped AMD-RDT method. The estimates from both the AMD-RDT and RDT decrease for later measurements, somewhat reflecting the decrease in natural frequency which is shown in Figure 5.13. Figure 5.16 shows the similar trend followed by the damping ratio estimated using the AMD-RDT method and the natural frequency.

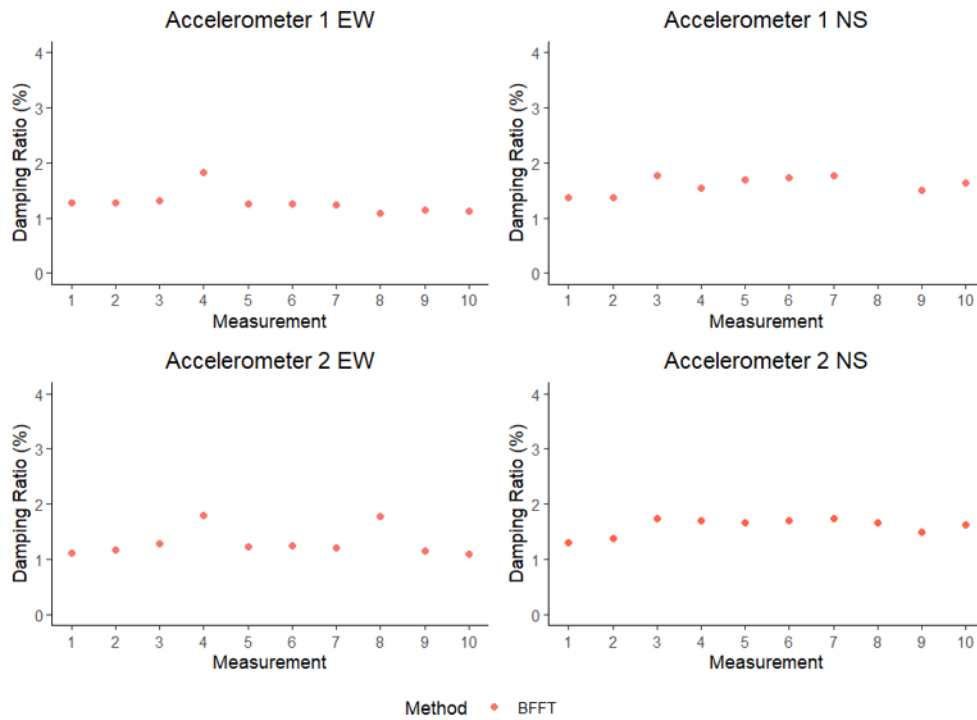


Figure 5.14: Estimated damping ratios of *Ten Degrees* calculated using the BFFT method

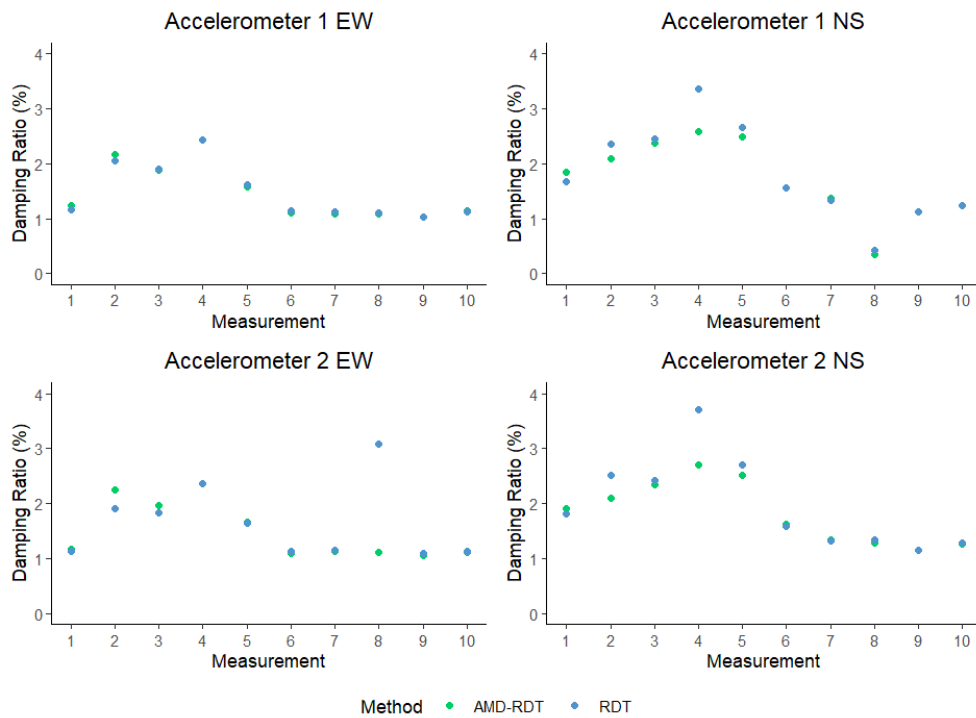


Figure 5.15: Estimated damping ratios of *Ten Degrees* calculated using the RDT and AMD-RDT methods

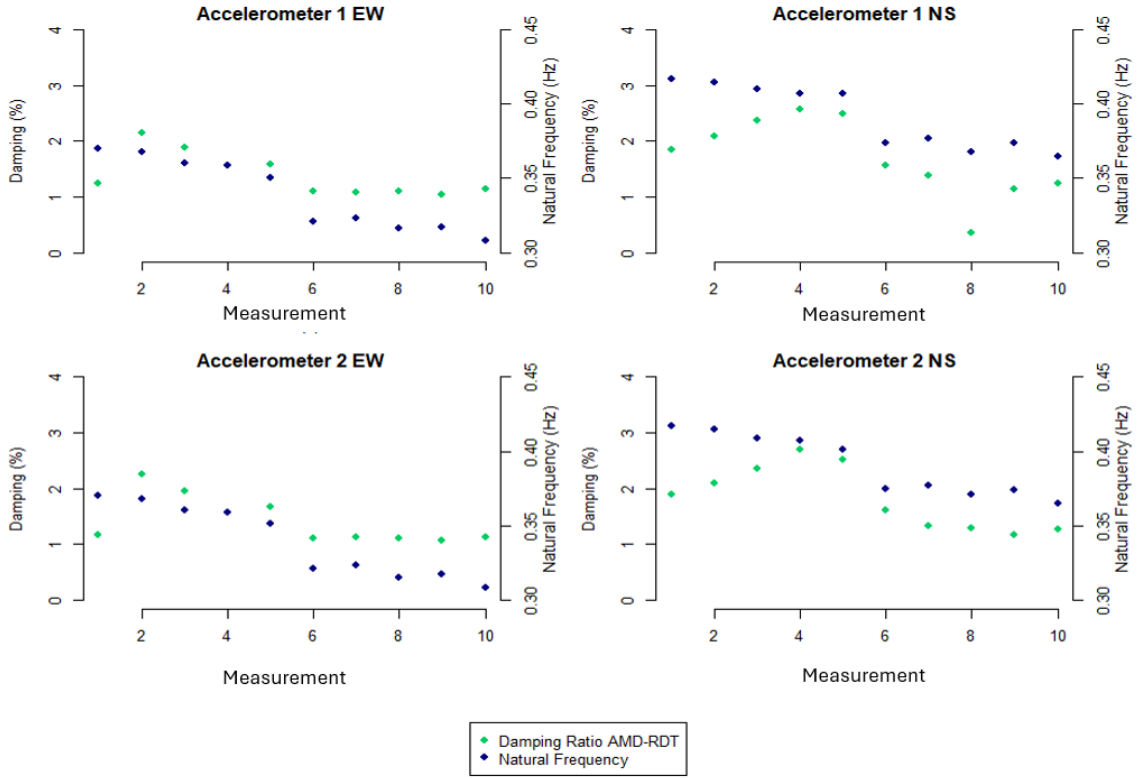


Figure 5.16: Decreasing trend identified in AMD-RDT estimate of damping and the natural frequency of *Ten Degrees*

Figure 5.17 compares the calculated damping ratio for the BFFT and AMD-RDT methods for each of the 10 measurements. The estimates are similar and indicate the damping ratio of the completed structure is close to one in the EW direction and slightly higher in the NS direction. However, there is variation in the estimates from each method. The AMD-RDT method shows lower estimates in the EW direction for measurements 6-10 but generally higher estimates for the earlier measurements in the NS direction. However, there does appear to be some convergence on values for the later events in both directions. In general, it can be observed that the AMD-RDT method provides more conservative estimates of the critical damping ratio of the completed structure.

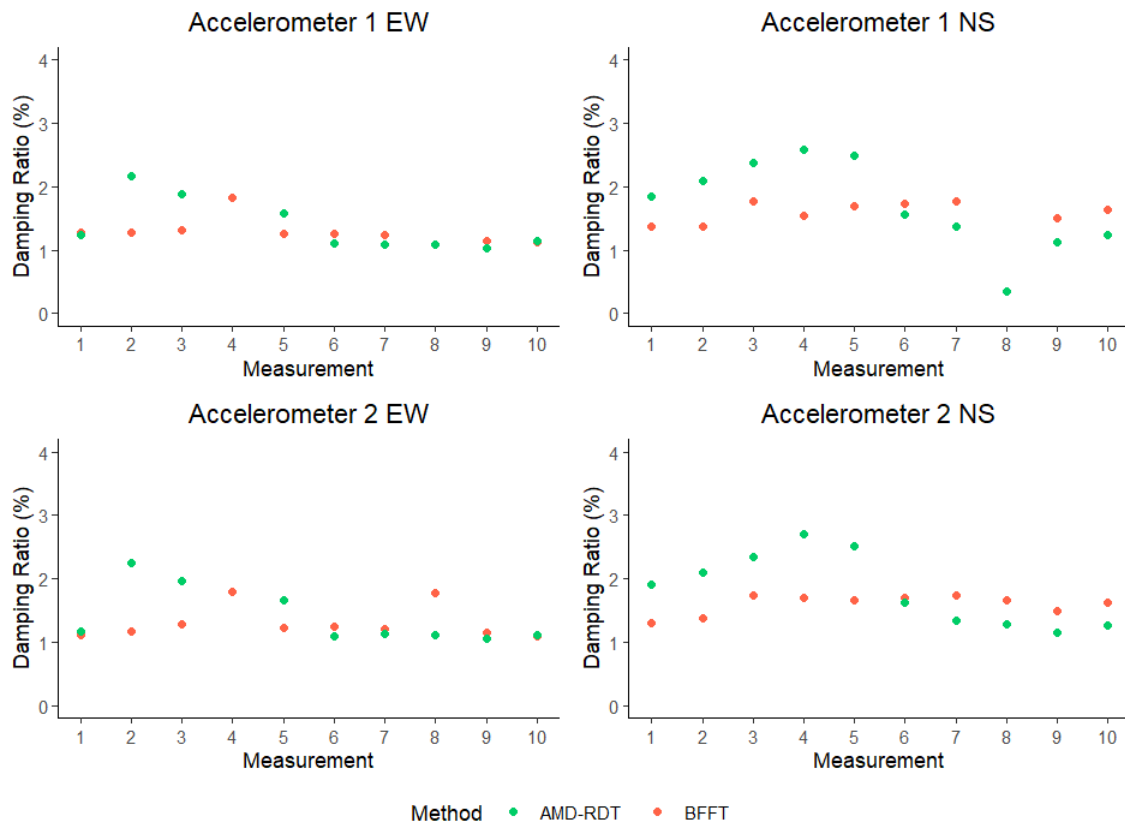


Figure 5.17: Comparison of calculated damping ratios from AMD RDT and BFFT methods for *Ten Degrees*

Table 5.7 presents the average damping ratios estimated using each method for measurements 8-10. It can be seen that the damping ratio values given by the BFFT, RDT and AMD RDT are close in value. Therefore, from these results it can be conservatively concluded that the true damping ratio of the measured data in the EW direction likely lies between 1 and 1.1% and in the NS direction between 1 and 1.3%. Furthermore, these values reflect the Eurocode 1991-1-4 recommended value for steel and concrete structures of 1.27%.

Table 5.7: Average identified Damping Ratio of *Ten Degrees* for Measurements 8-10

Method	Accelerometer 1		Accelerometer 2	
	EW	NS	EW	NS
BFFT	1.15 %	1.64 %	1.31 %	1.63 %
RDT	1.09 %	1.03 %	1.61 %	1.28 %
AMD RDT	1.09 %	1.02 %	1.10 %	1.26 %

The Coefficient of Variation (CV) of each of the damping ratios is estimated as part of the BFFT method. These values are presented in Table 5.8. There is a significant CV value for Measurement 8 in the NS direction, this is likely due to the noted construction noise in this acceleration response. All other estimates of the damping ratio have a CV of $< 9\%$, which is a low value and provides confidence in the precision of the damping ratio estimates.

Table 5.8: CV of BFFT estimates of the damping ratio of *Ten Degrees*

Measurement	Accelerometer 1		Accelerometer 2	
	EW (%)	NS (%)	EW (%)	NS (%)
1	5.4	6.36	5.06	6.09
2	5.43	6.37	4.99	6.37
3	5.54	6.62	5.46	8.53
4	7.91	2.51	7.78	8.33
5	6.08	8.33	5.88	7.94
6	2.08	8.64	5.24	8.47
7	5.22	8.89	5.02	8.72
8	5.17	88.95	7.98	8.43
9	5.52	7.38	5.52	7.21
10	5.38	6.35	5.30	8.44

The bootstrapped AMD-RDT method as described in Section 5.2.4 was used to obtain the CV of the damping ratio estimates from the AMD-RDT method. For appropriate use of the bootstrapped AMD-RDT method it is necessary to first confirm that the estimates from the bootstrap samples are normally distributed. Figure 5.18 shows the distributions of all bootstrap samples of the acceleration measurements from Accelerometer 1 in the East-West (x) direction. All bootstrap samples taken from all measurement data from both accelerometers in both directions are similarly normally distributed. Therefore, application of the bootstrapping method to obtain a measure of error in the damping estimates is suitable. The CV values for the bootstrapped AMD-RDT method are shown in Table 5.9. Again, a high CV is noted for Measurement 8 in the NS direction of Accelerometer 1.

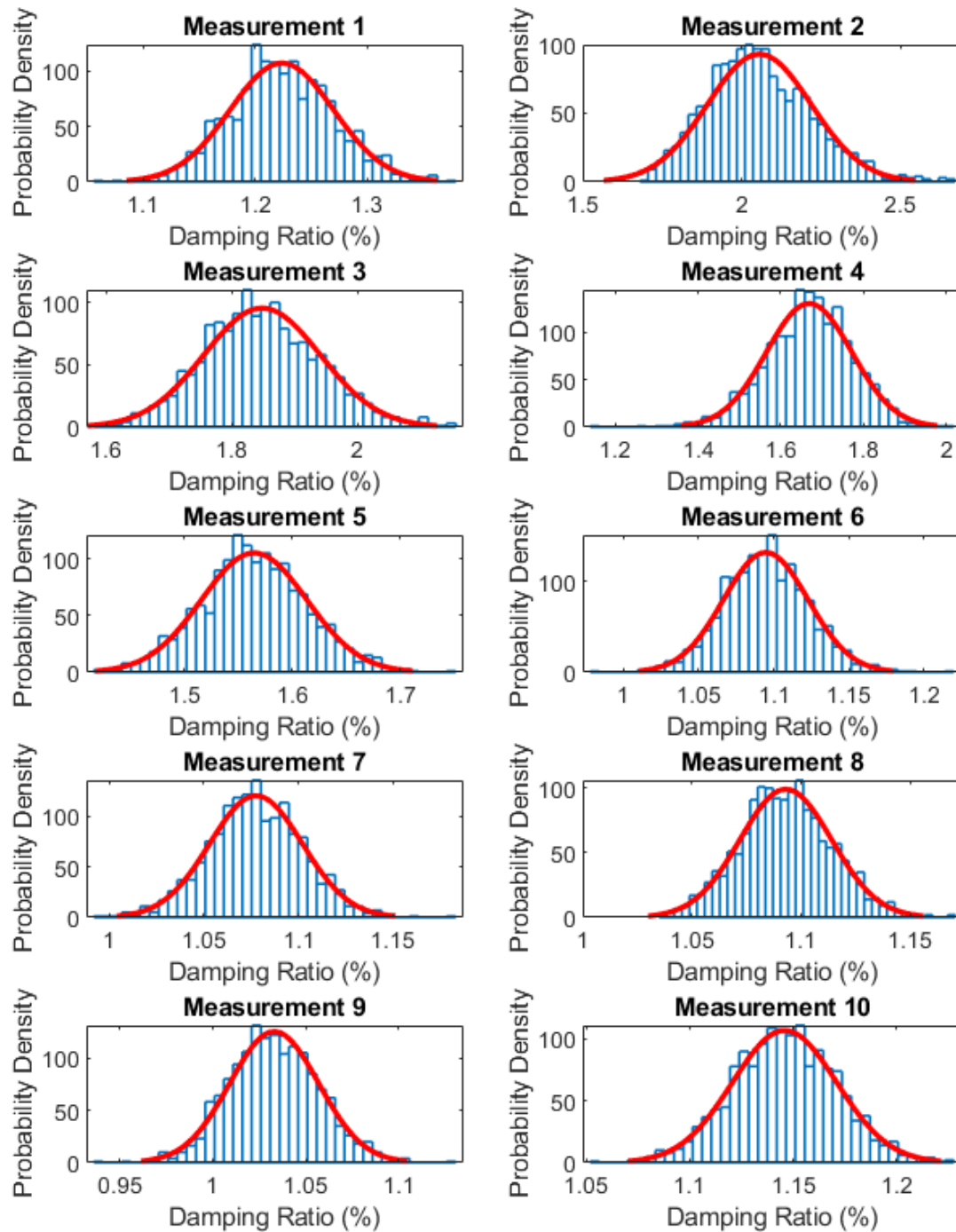


Figure 5.18: Probability distribution function of bootstrap samples of measurements from accelerometer 1 in East-West (x) direction from *Ten Degrees*

Table 5.9: CV of bootstrapped AMD-RDT estimates of the damping ratio of *Ten Degrees*

Measurement	Accelerometer 1		Accelerometer 2	
	EW (%)	NS (%)	EW (%)	NS (%)
1	3.82	4.83	3.55	5.11
2	2.282	4.28	1.94	4.65
3	5.042	3.41	5.04	3.28
4	6.18	3.98	5.01	3.78
5	3.13	2.51	3.3	2.36
6	2.59	2.93	2.44	2.98
7	2.27	2.76	2.26	2.95
8	1.92	24.04	2.44	2.46
9	2.31	2.79	2.37	2.74
10	2.17	2.71	2.27	2.60

Comparison of the results presented in Tables 5.8 and 5.9 shows that for the majority of measurements 1-10, the CV for the bootstrapped AMD-RDT is lower than the relative CV for the BFFT method. CV is a ratio of the standard deviation of an estimate to the mean and is a standardised measure of dispersion of a probability distribution. It is a useful tool for comparing the variation of a dataset based on one standard deviation. The average CV for the bootstrapped AMD-RDT is 3.23% while for the BFFT it is 6.46%. This suggests that variation in an estimate when using the AMD-RDT is approximately half that of the BFFT. Figure 5.19 compares the damping and CV values obtained with the two methods. There is some overlap between the ribbons of the CV of the BFFT and AMD-RDT estimates, however there are a number of measurements for which this is not the case. CV is based on one standard deviation, which empirically includes 68% of possible estimates. Therefore, not all estimates of the damping ratio lie within the ribbons and there is not complete overlap of results.

As before, the variation in the damping estimates across the measurements from the bootstrapped AMD-RDT method is greater than the variation in the estimates from the BFFT method. However, it is worth noting that each acceleration

measurement should be expected to give rise to a unique damping estimate. Each set of measurement accelerations was obtained under different ambient wind loading conditions and, in many cases, different stages of construction of the structure. The bootstrapped AMD-RDT results all display a consistent change in damping between measurements 5 and 6 during which time extra modules had been added at the top of the structure, changing its modal properties. Therefore, the higher level of variance between damping estimates from the bootstrapped AMD-RDT may be a further indicator of the capability of this method as it reflects its ability to capture changes in the damping ratio more acutely.

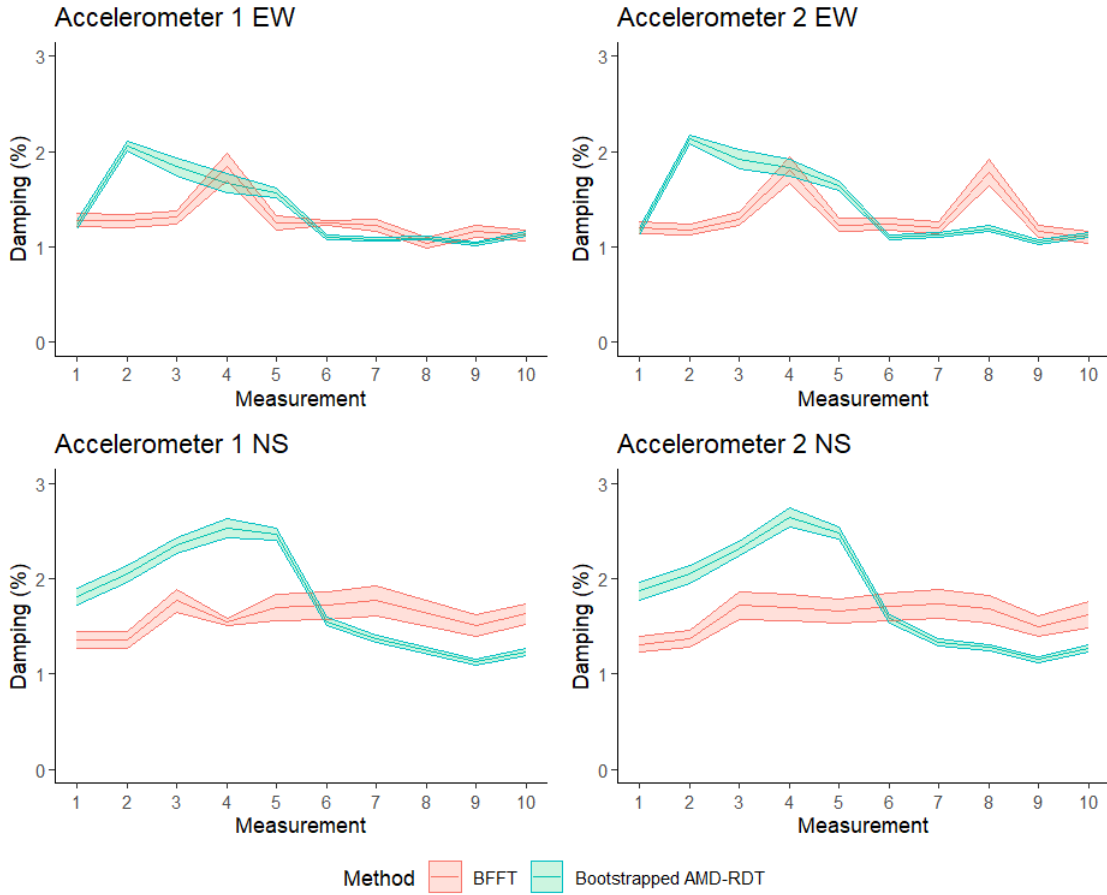


Figure 5.19: Comparison of damping estimates and their possible range using CV values from both the BFFT and bootstrapped AMD-RDT method for *Ten Degrees*

Figure 5.20 shows how the estimates vary with one another for each method. It can be seen that the highest damping levels occur at higher estimates of natural frequency suggesting a weak positive relationship between the two. This positive relationship between damping and natural frequency has been reported in literature

[12] also reflects higher RMS accelerations during early events. However, there is no relationship observed between natural frequency and damping ratio for estimates made using the BFFT method.

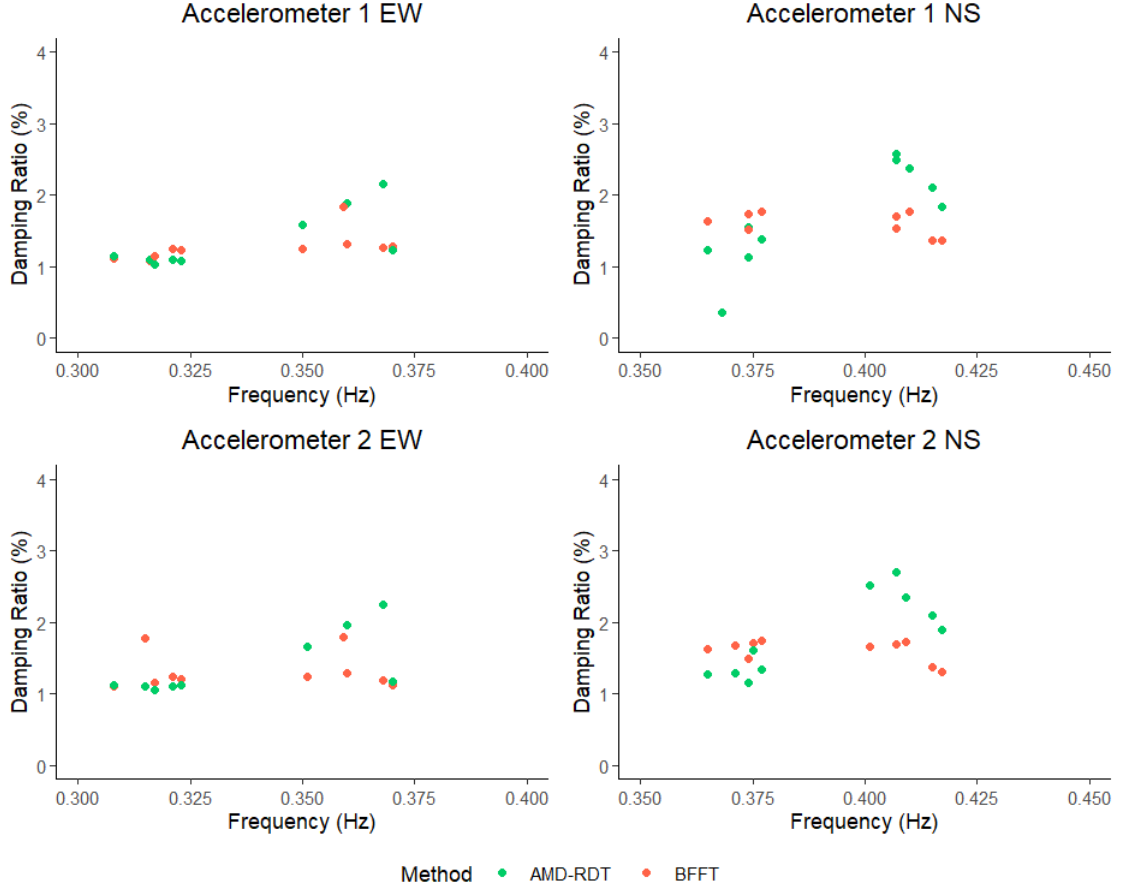


Figure 5.20: Estimated damping ratio of *Ten Degrees* vs estimated natural frequency

Figures 5.21 and 5.22 show the relationship between RMS acceleration and the natural frequencies and damping ratios estimated from the measurements. Literature examining empirical data from in situ testing has shown that natural frequency decreases and damping ratio increases with acceleration response in the low amplitude range due to higher inertial forces and energy dissipation [11, 12, 230]. However, from these figures there does not appear to be a strong trend linking natural frequency to acceleration. Given only ten measurements are available for analysis, more measurements of the response at a wider variation of acceleration responses could aid in confirming whether a relationship between the two exists.

There is a weak trend which appears to show damping ratio increasing as RMS acceleration increases. Generally speaking, higher damping ratios were ob-

served when RMS accelerations were higher. At lower RMS accelerations damping does not drop below 1% and at higher RMS accelerations the range of observed damping values increases with values above 2% determined in some events, depending on the processing method employed. Previous studies on other types of buildings have shown similar results; that damping tends to increase with acceleration response (e.g. [11, 230, 231]). Whilst this same relationship can be observed from the measurement data recorded during this study, it does not appear to be exceptionally strong. However, it is worth noting that the range of acceleration amplitudes observed in this monitoring campaign is not very large. The wind speeds reached when measurements were taken were low, far from inciting a structural response which would give rise to habitability issues. As the data recorded is ambient, there was no control over how significant the wind speed was and in reality the measured wind speed was far less than the 1 in 50 year wind speed in which the structure is designed for. A measurement of acceleration response from a period of high wind loading may offer more insight into the variation in acceleration with natural frequency or damping ratio.

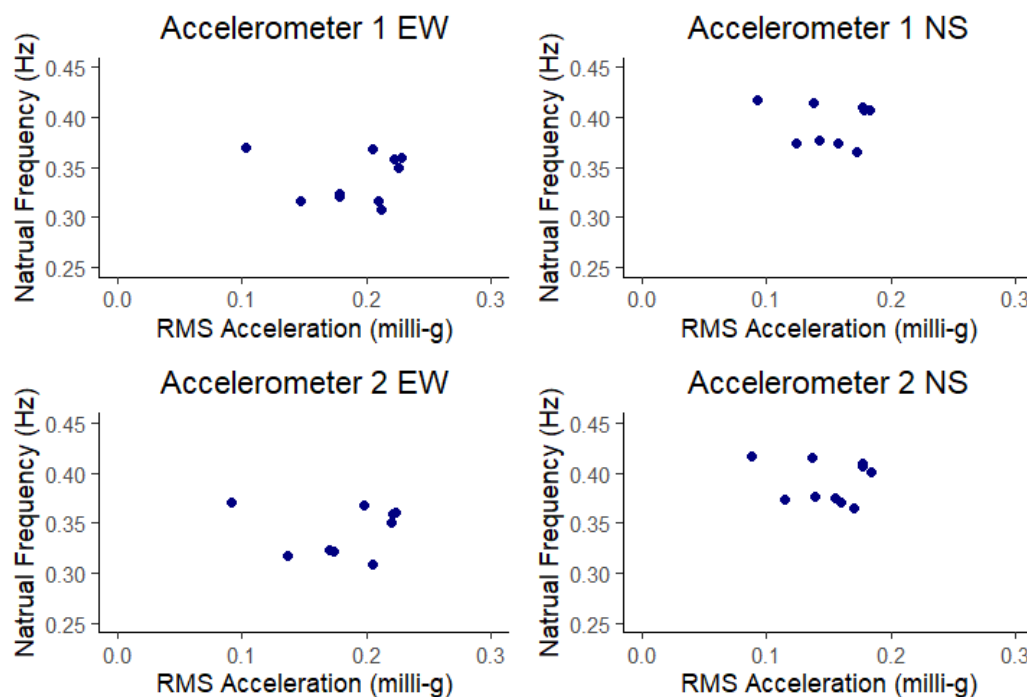


Figure 5.21: Estimated Natural Frequency of *Ten Degrees* vs RMS acceleration

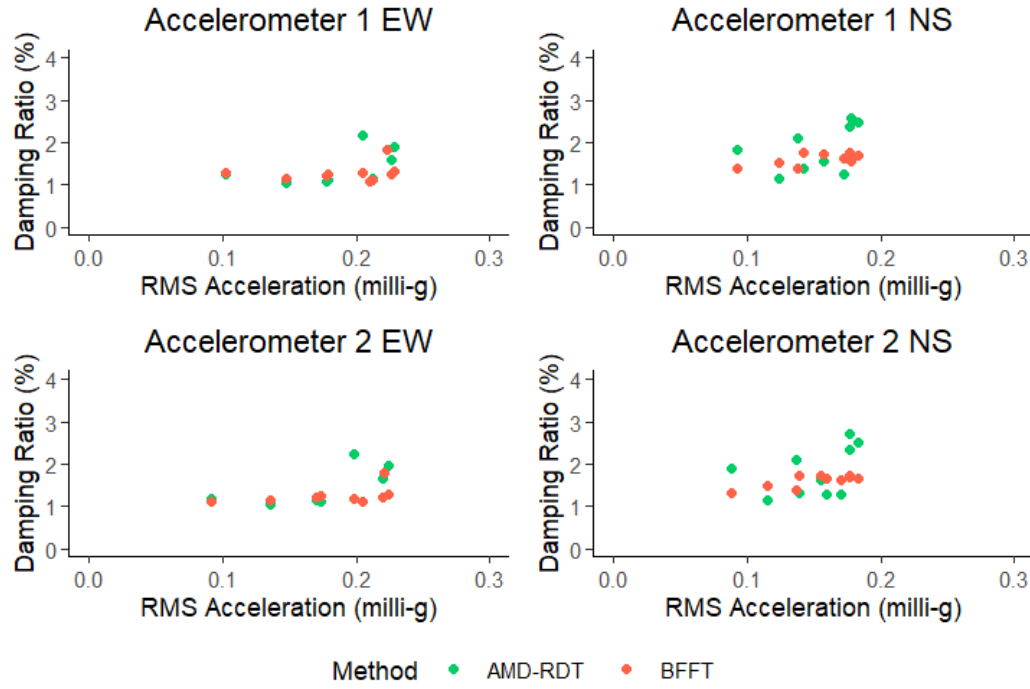


Figure 5.22: Estimated damping ratio of *Ten Degrees* vs RMS acceleration

5.4.1.4 Identified Inherent Properties

Through monitoring, recording of measurements and applying operational modal analysis techniques, the inherent properties of *Ten Degrees* in the first mode in both the X (East-West) and Y (North-South) directions were identified. These properties are summarised in Table 5.10.

Table 5.10: Identified inherent properties of *Ten Degrees*

Inherent Property	EW	NS
Natural Frequency (Hz)	0.314	0.370
BFFT Damping Ratio (%)	1.23 ± 0.07	1.64 ± 0.12
RDT Damping Ratio (%)	1.35	1.16
Bootstrapped AMD-RDT Damping Ratio (%)	1.10 ± 0.02	1.14 ± 0.03

5.4.2 College Road

Details and analysis of the acceleration measurements recorded at *College Road* are presented below. The acceleration responses are processed using the OMA methods outlined in Section 5.2. A comparison to the analysis of *Ten Degrees* is also provided.

5.4.2.1 Wind Loading

The acceleration responses captured during the in situ monitoring of *College Road* were observed under ambient loading conditions. However, there were no measurements taken of the ambient weather conditions. Therefore, the wind speed during the measured acceleration responses which are used in identifying inherent properties of the structure are unknown. However, records of the hourly wind speed are available from the Met Office for Heathrow Airport, London [229]. Whilst these wind speeds are not specific for the location of *College Road* in Croydon, London, United Kingdom, they do provide an indication of the extent of the wind load in the greater area at the time when measurements were recorded. The average hourly wind speed for each measurement as recorded by The Met Office is shown in Table 5.11. It is noted that only for Measurement 5, which was recorded during Storm Dudley which affected the United Kingdom in February 2022, did the wind speed recorded at Heathrow Airport exceed the 1 in 1 year 10 minute average design wind speed of 16.11 m/s for Serviceability Limit State design of *College Road*.

Table 5.11: Average Hourly wind speeds recorded at Heathrow Airport
for each in situ measurement of response recorded at *College Road* [229]

Measurement	Average Hourly Wind Speed (m/s)
1	4.72
2	6.66
3	10.83
4	9.72
5	18.05
6	3.61
7	10.27
8	10.27
9	8.33
10	8.88
11	6.11

5.4.2.2 Acceleration Response

The acceleration response of *College Road* was recorded under ambient loading conditions. As such, the uncontrolled and fluctuating wind conditions resulted in a varied acceleration response which was not uniform across measurements. Moreover, as with *Ten Degrees* the monitoring of *College Road* was undertaken whilst the structure was under construction. Five months of continuous acceleration response was captured whilst the structure was under construction, and a further five months was captured when the structure was fully complete. Special consideration was given to choose measurements which were not corrupted with noise from the construction work and so, measurements of the acceleration response which were chosen to perform OMA were shorter and predominately recorded between the hours of 9pm and 5am. In total, seven of the acceleration responses (Measurements 1-7) were recorded during construction and four acceleration responses (Measurement 8-11) were recorded when the structure was fully complete and all modules had been installed. Each measurement contains one hour of the acceleration response in the X direction (roughly East-West), Y direction (roughly North-South) and the Z direction (vertical). The acceleration responses of each measurement are shown in Figure 5.23 and 5.24 for Accelerometer 1 and Accelerometer 2. It is worth noting that the Z-direction accelerations are reasonably large, suggesting noisy signals despite the measurements being taken when no construction activity was ongoing.

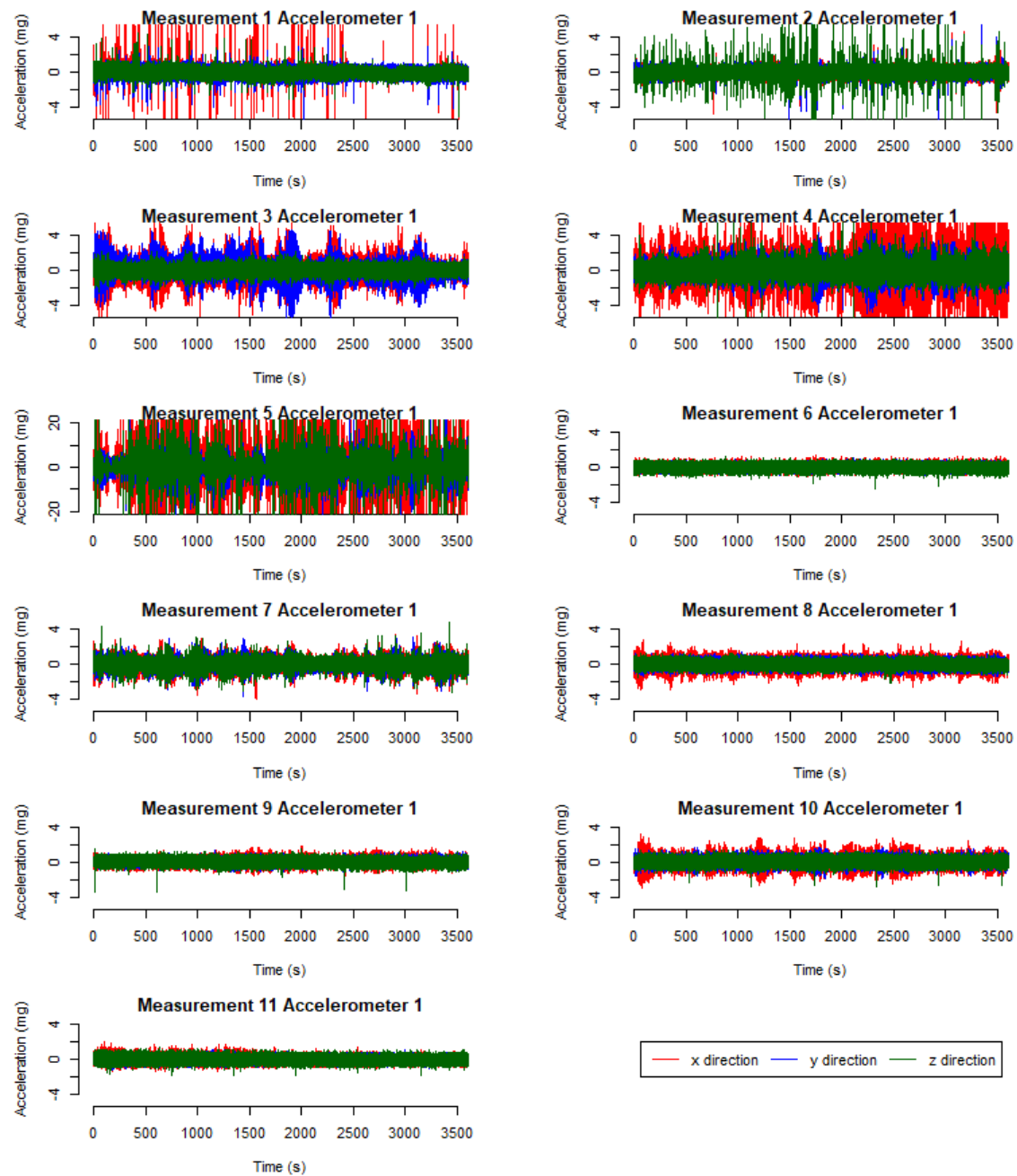


Figure 5.23: Acceleration time history for Accelerometer 1 measured from *College Road*

5.4. Processing Measurements from In Situ Monitoring

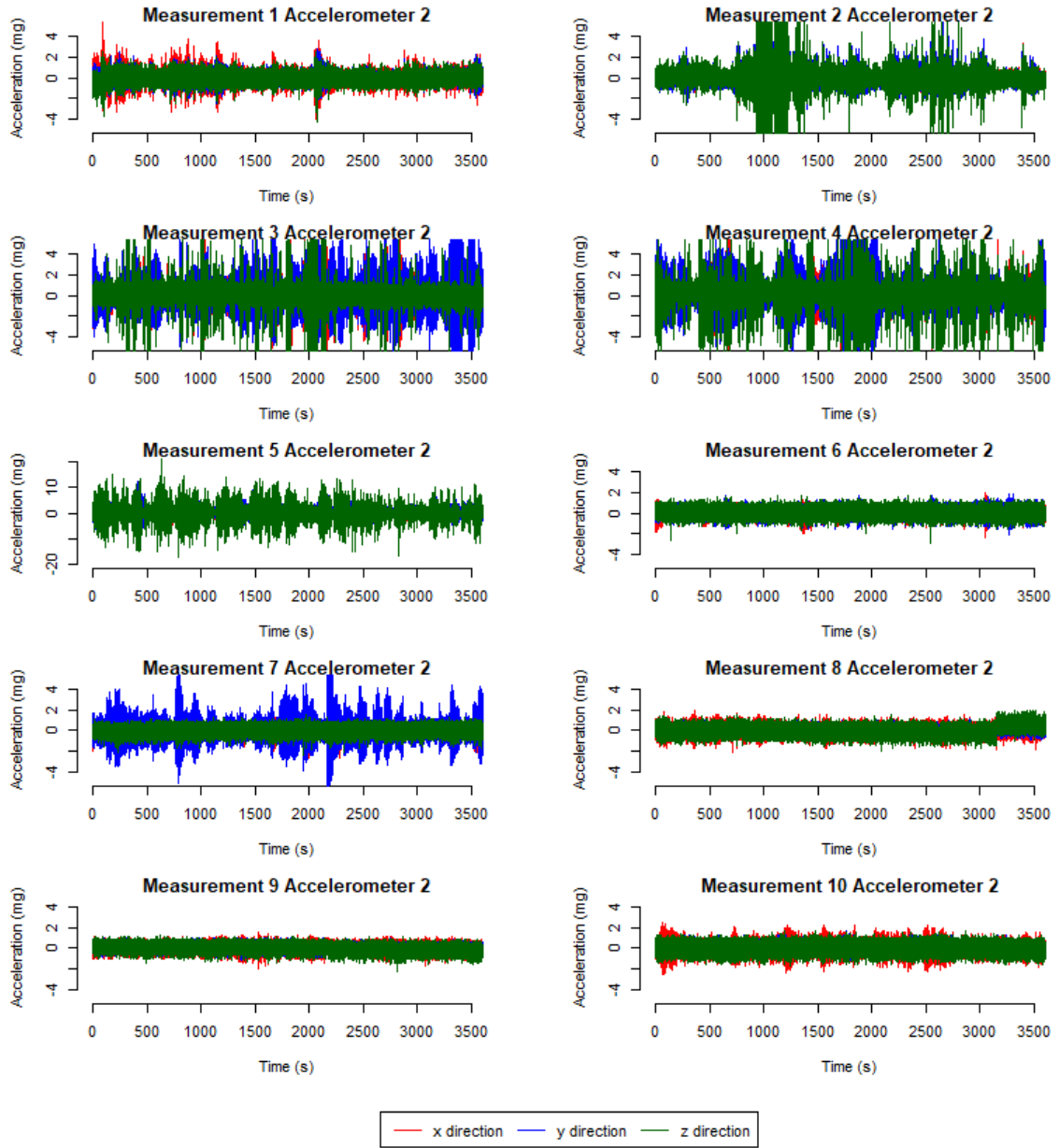


Figure 5.24: Acceleration time history for Accelerometer 2 measured from *College Road*

Tables 5.12 and 5.13 show the maximum and RMS accelerations for both accelerometers in the EW and NS directions for each measurement. Measurements 3, 4 and 5 were recorded during Storm Dudley which struck the United Kingdom from 14th to 19th February 2022. The large RMS and maximum accelerations observed in these measurements, particularly Measurement 5 reflect the increased ambient wind load which would have excited *College Road*. In fact, these are the only measurements where RMS accelerations exceeded 1 milli-g. These large accelerations are also prominent in Figures 5.23 and 5.24.

Table 5.12: Maximum accelerations for each measurement taken at *College Road*

Measurement	Accelerometer 1		Accelerometer 2	
	Maximum Acceleration EW (milli-g)	Maximum Acceleration NS (milli-g)	Maximum Acceleration EW (milli-g)	Maximum Acceleration NS (milli-g)
1	18.266	6.074	6.647	2.955
2	4.842	8.021	10.371	23.341
3	6.443	6.559	16.115	11.918
4	34.533	25.149	8.627	9.726
5	690.883	250.862	10.287	12.706
6	1.336	1.023	2.382	2.182
7	3.960	3.631	2.788	8.703
8	2.902	1.438	2.105	1.183
9	1.894	1.289	2.024	1.260
10	3.241	1.975	2.647	1.673
11	1.988	1.180	-	-

Table 5.13: RMS accelerations for each measurement taken at *College Road*

Measurement	Accelerometer 1		Accelerometer 2	
	RMS	RMS	RMS	RMS
	Acceleration EW (milli-g)	Acceleration NS (milli-g)	Acceleration EW (milli-g)	Acceleration NS (milli-g)
1	0.531	0.458	0.552	0.440
2	0.427	0.418	0.540	0.823
3	1.335	1.269	1.335	2.077
4	1.743	1.059	1.007	1.803
5	9.796	4.759	1.262	1.824
6	0.298	0.238	0.349	0.435
7	0.639	0.238	0.448	1.273
8	0.565	0.364	0.444	0.270
9	0.374	0.285	0.378	0.289
10	0.660	0.407	0.575	0.361
11	0.341	0.263	-	-

5.4.2.3 Results from OMA

Natural Frequency

Fast Fourier Transforms were taken of each of the measured ambient acceleration responses in order to identify the natural frequency of the structure in the X-direction (approximately East-West) and the Y-direction (approximately North-South). Figure 5.25 shows the Fast Fourier Transform which was taken of the acceleration responses from both accelerometers in both directions for Measurement 10. Whilst only the first mode is clearly identifiable in the acceleration response from the EW direction, both the EW and NS modes are identifiable in the NS acceleration responses.

Table 5.14 lists the natural frequencies calculated for each of the acceleration responses from both accelerometers for all measurements. It can be concluded that the finished structure has a natural frequency of 0.248 Hz in the first mode in the X (EW) direction and of 0.350 Hz in the first mode in the Y (NS) direction. The depth of the structure in the X direction is 21 m, less than half that of the Y direction which is 56 m. The greater flexibility of the structure in the X direction is apparent from the identified natural frequencies.

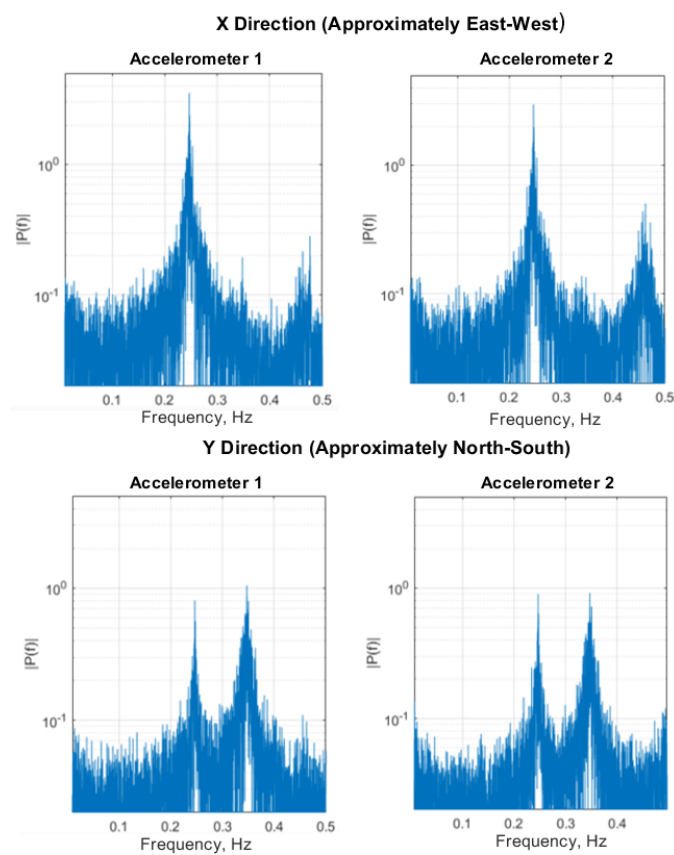


Figure 5.25: Fast Fourier Transform of Measurement 10 recorded at *College Road*

Table 5.14: Natural frequency calculated using the FFT of each measurement recorded at *College Road*

Measurement	Accelerometer 1		Accelerometer 2	
	Natural Frequency EW (Hz)	Natural Frequency NS (Hz)	Natural Frequency EW (Hz)	Natural Frequency NS (Hz)
1	0.275	0.318	0.481	0.511
2	0.288	0.332	0.480	0.509
3	0.292	0.337	0.475	0.506
4	0.230	0.344	0.477	0.505
5	0.278	0.352	0.468	0.500
6	0.312	0.354	0.475	0.504
7	0.325	0.379	0.469	0.507
8	0.250	0.350	0.250	0.350
9	0.250	0.353	0.250	0.353
10	0.246	0.347	0.246	0.347
11	0.247	0.347		
Average (Measurements 8-11)	0.248	0.349	0.248	0.350

It is worth noting that for Accelerometer 1, which was located at the centre of the core in Tower A, the estimated natural frequency increases by almost 20% from measurements 1-7 as the first 24 storeys of modules were added to the structure. The increase in the natural frequency indicates a significant increase in the stiffness of the structure as the storeys of modules are added. This trend is not seen in Accelerometer 2, which was located in the centre of the core in Tower B for the first 7 measurements where there were no modules installed. Therefore, it is clear that the modules do contribute to the bending stiffness of the completed structure. In Chapter 6, the evolving natural frequencies from *Ten Degrees* and *College Road* are used alongside a stepped beam numerical model to quantify the bending stiffness contribution of the modules.

The natural frequency of the finished structure (Measurements 8-11) is lower than that of the partially completed structure with 24 storeys of modules. Accelerometer 2 was moved to the edge of the modules in Tower A for measurements 8-10. The estimated natural frequency for the freestanding Core A (Accelerometer 1, Measurement 1) and Core B (Accelerometer 2, Measurements 1-7) is higher than

for the completed structure in both the X and Y directions. As was seen with *Ten Degrees*, the impact on the natural frequency of the added mass of the modules is greater than the impact of the additional stiffness they provide for modules added on the upper floors. Considering the changing natural frequencies as modules were added to both *Ten Degrees* and *College Road*, it is appropriate to conclude that the modules added towards the bottom of the tower contribute significantly to the bending stiffness of the structure and hence increase the natural frequency. However, as more storeys of modules are added, the stiffness contribution of the modules is surpassed by the effect of the added mass of the modules towards the top of the structure, reducing the natural frequency. Figure 5.26 shows the identified natural frequency of *College Road* for each of the recorded acceleration responses. The plots for Accelerometer 1 provide a visual representation of the increase in natural frequency as the first 24 storeys of modules are added prior to the drop in the value for the completed structure. Considering Accelerometer 2, the natural frequency is seen to remain near constant for the first 7 measurements as there were no modules added to the freestanding core during this time period.

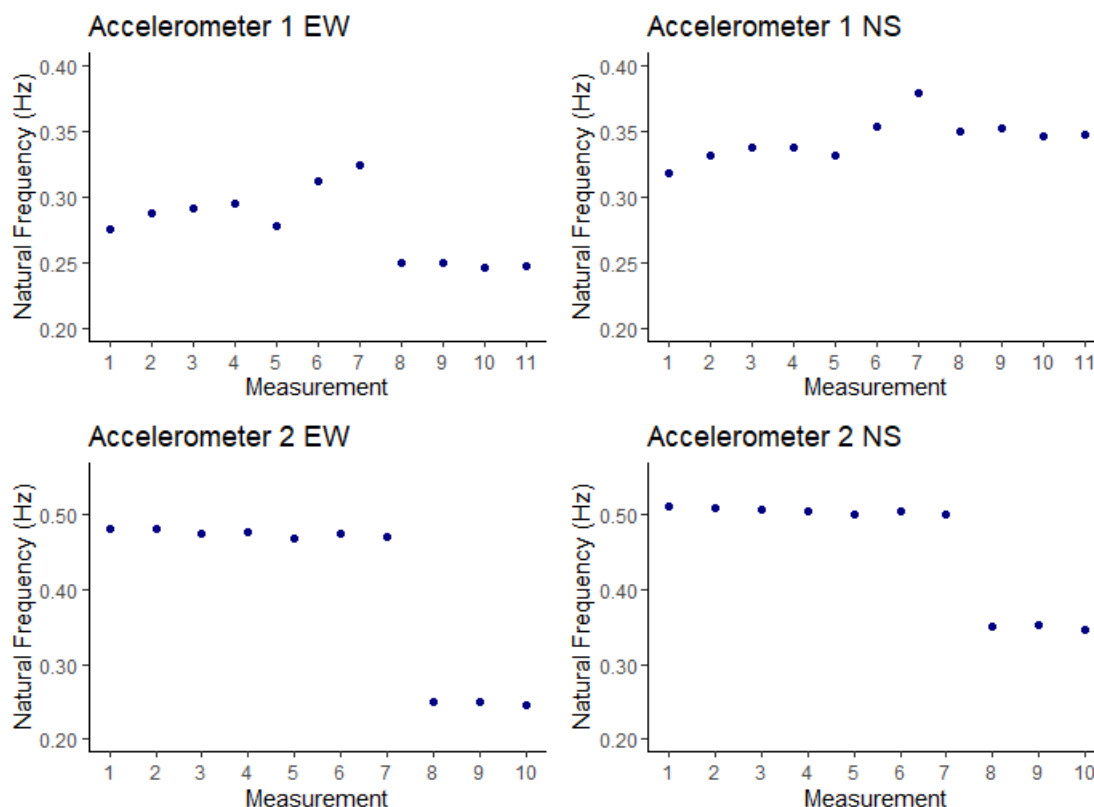


Figure 5.26: Natural frequencies for each measurement recorded at *College Road*

Damping Ratio

Each of the four modal identification techniques, presented in Section 5.2, were also applied to the acceleration response measurements recorded at *College Road* in order to estimate the inherent damping ratio of the volumetric modular structure. The first method applied was the BFFT, Figure 5.27 shows the estimated damping ratios for each measurement using this method.

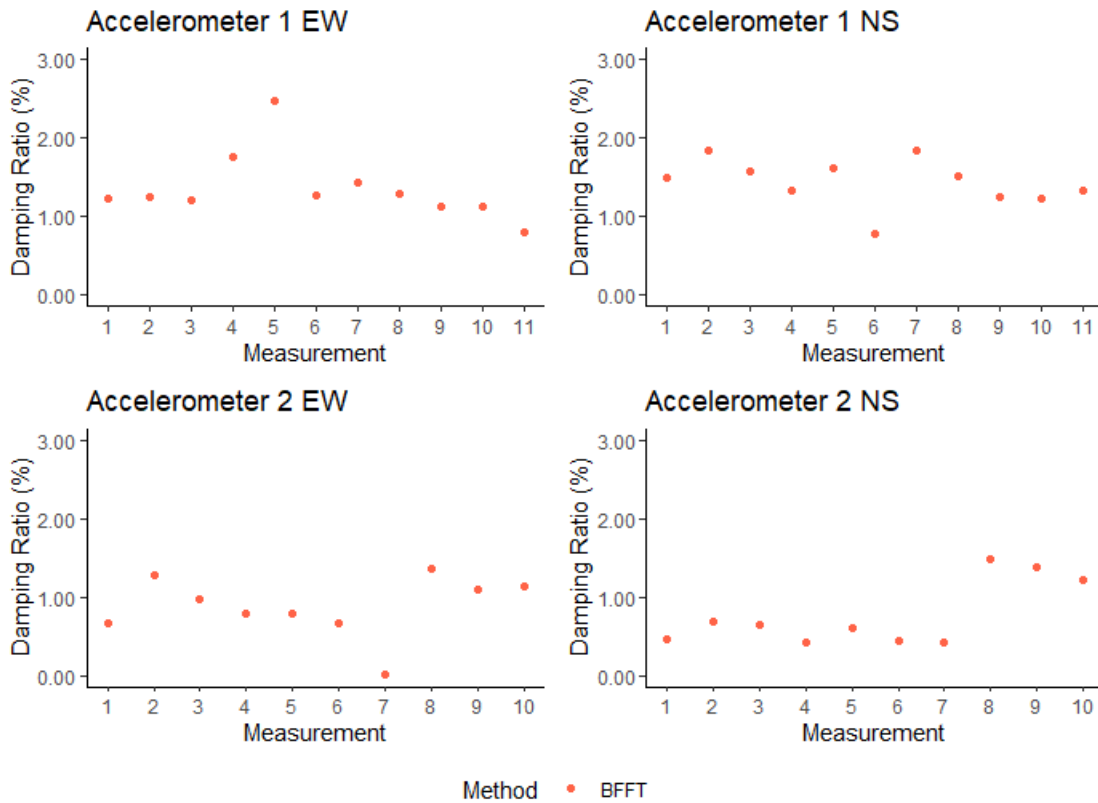


Figure 5.27: Estimated damping ratios of *College Road* calculated using the BFFT method

It can be observed that the estimate of the damping ratio from Accelerometer 2 is lower than Accelerometer 1 for the majority of estimates made from measurements 1-7 when Accelerometer 2 was installed in the RC core of Tower B with no modules installed. This may be due to the fact that Accelerometer 1 was installed in the core of Tower A which had thicker concrete walls (600 mm vs 450 mm). For the initial 7 measurements, Tower A had increasing storeys of modules installed which may have also contributed to the damping. However, if only Measurement 1 is considered, where both Tower A and Tower B had no modules installed, the damping estimates from Accelerometer 1 in Tower A in both EW

and NS directions are larger than from Accelerometer 2 in Tower B. Therefore it is likely the thicker walls of the RC core are a larger contributor to the damping ratio than the modules. The estimates from both accelerometers for Measurements 8-11 which were recorded on the finished structure are consistent, with estimates from Accelerometer 2 relocated at the edge of a module in Tower A being slightly higher than the equivalent measurements recorded with Accelerometer 1.

Figure 5.28 shows the damping ratio estimates for each measurement calculated using the RDT and AMD-RDT methods. The damping values estimated by the bootstrapped AMD-RDT are the same as for the AMD-RDT. Good agreement between the two methods can be seen from Accelerometer 2, with estimates from the two methods showing slightly more variation in measurements taken from Accelerometer 1. This may be due to the level of noise in the recorded measurements, with the AMD-RDT better equipped to deal with noisier signals. However, the RDT still reflects the AMD-RDT well for notably larger or smaller values such as for Measurement 5 in the EW direction from Accelerometer 1 and Measurement 6 in the NS direction from Accelerometer 2. As with estimates from the BFFT, Accelerometer 1 appears to provide larger estimates than Accelerometer 2 for a number of measurements, reinforcing the observation that the larger core walls of Tower A provide more damping. Overall, the estimates of damping ratio from the AMD-RDT and RDT methods are noticeably lower than those from the BFFT method. Figure 5.29 compares the estimates of damping ratio provided by the BFFT and AMD-RDT methods.

Observing Figure 5.29, the lower estimates of the damping ratio using the AMD-RDT method are apparent. However, both methods do seem to reflect a similar pattern of increases and decreases between measurements. The difference between the estimates of the two methods reflects the fact that the BFFT is a frequency domain method whereas the AMD-RDT is a time domain method. Therefore, the methods are based on different underlying theories and assumptions. In general, time-domain methods are more sensitive and capable of dealing with more complex systems such as those with closely spaced modes [86]. However, time-domain methods can be more sensitive to noise within the measured response. Conversely, frequency domain methods can be more sensitive to the resolution of the response but commonly require less computational power and smaller measurement lengths [87]. Thus, it is good practice to apply both a time-domain and frequency do-

5.4. Processing Measurements from In Situ Monitoring

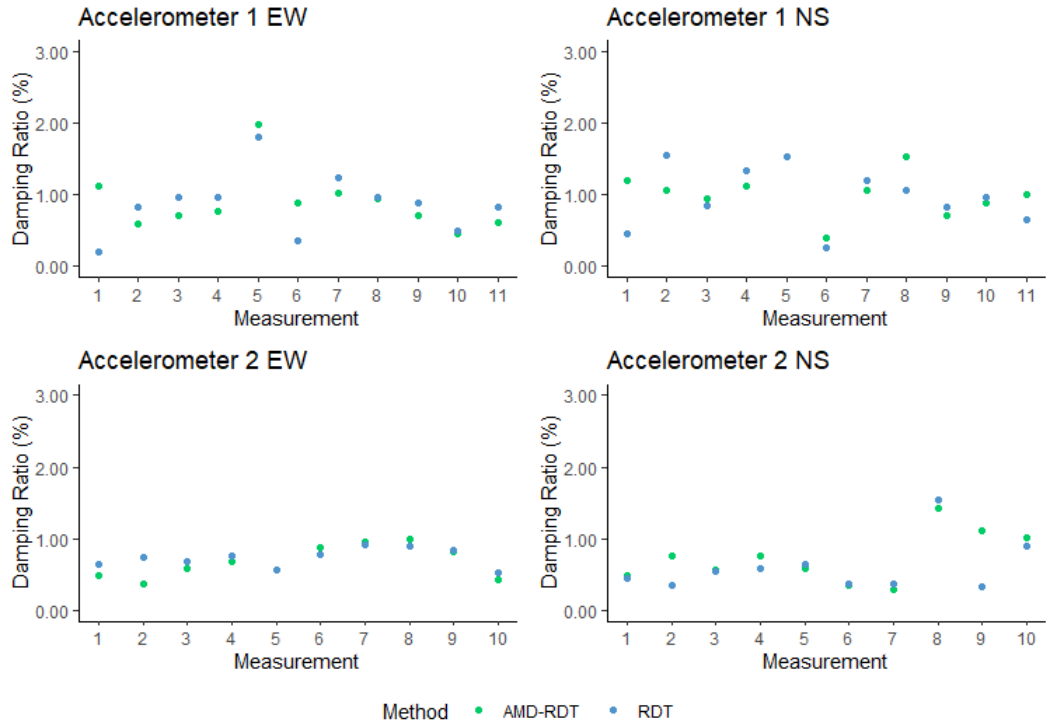


Figure 5.28: Estimated damping ratios of *College Road* calculated using the RDT and AMD-RDT methods

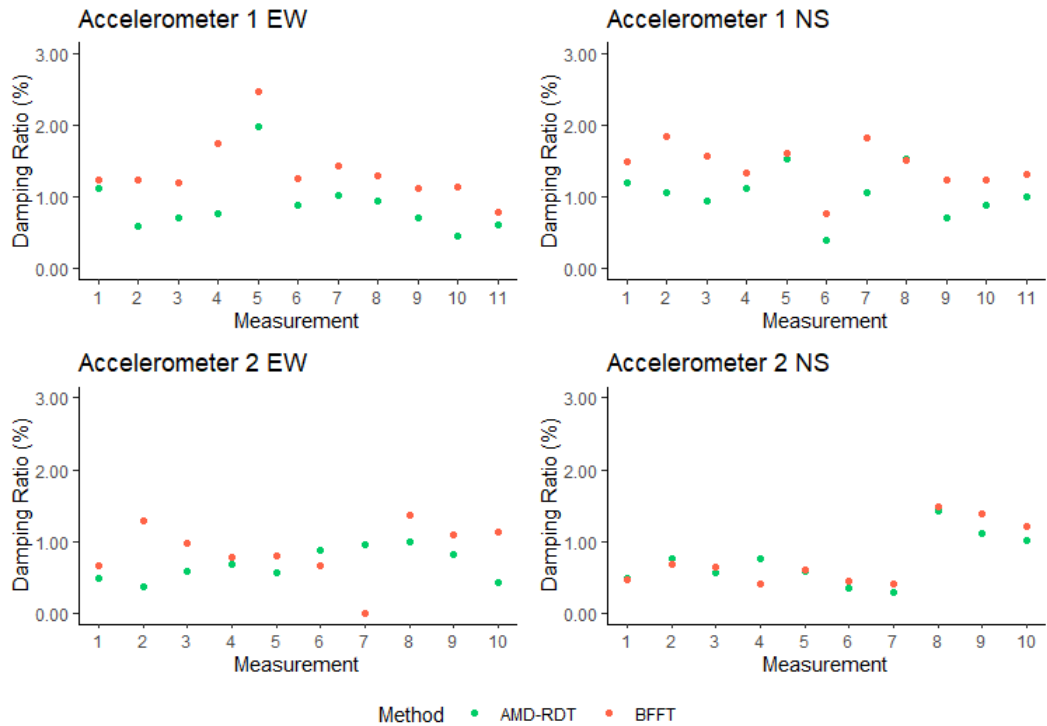


Figure 5.29: Comparison of calculated damping ratios from AMD RDT and BFFT methods for *College Road*

main method when performing OMA in order to compare estimates [86, 87, 88]. Whilst both OMA methods applied in this study differ slightly, they are generally in good agreement and produce estimates within the range of accuracy which can be expected from OMA.

Table 5.15 shows the average estimated damping ratio of the finished structure from each method. The damping ratio of the finished structure as estimated using the BFFT is consistently higher than that from the RDT and AMD-RDT methods. All methods show higher estimates of damping in the NS direction than in the EW direction. This reflects the higher observed natural frequency in the NS direction. It is also worth noting that whilst the BFFT method provides estimates of the damping ratio similar to those observed for the *Ten Degrees* and which are close to the Eurocode 1991-1-4 recommended value of 1.27% for mixed steel and concrete structures. However, the estimates from the RDT and AMD-RDT are lower, particularly in the EW direction.

Table 5.15: Average Identified damping ratio of *College Road* for measurements 8-11

Method	Accelerometer 1		Accelerometer 2	
	EW	NS	EW	NS
BFFT	1.09 %	1.33 %	1.20 %	1.36 %
RDT	0.79 %	0.88 %	0.76 %	0.92 %
AMD-RDT	0.68 %	1.03 %	0.75 %	1.19 %

The CV of the damping estimates was calculated using the BFFT and bootstrapped AMD-RDT methods to help provide error margins on the estimates. The CV for each estimate calculated using the BFFT method is provided in Table 5.16. The bootstrap samples taken from applying the AMD-RDT method were normally distributed. Table 5.17 shows the CV of each estimate from the AMD-RDT method calculated using the bootstrapped AMD-RDT.

The average CV for the BFFT method was 17.14 % and for the bootstrapped AMD-RDT was 5.71%. Comparing the average CV reported in Tables 5.16 and 5.17 to those obtained on estimates of *Ten Degrees* (6.46% for BFFT and 3.23% for bootstrapped AMD-RDT), it can be seen that the CV of the damping estimates for *College Road* are considerably larger. This is likely due to the noisier acceleration measurements which were recorded. The bootstrapped AMD-RDT exhibits smaller

Table 5.16: CV of BFFT estimates of the damping ratio of *College Road*

Measurement	Accelerometer 1		Accelerometer 2	
	EW (%)	NS (%)	EW (%)	NS (%)
1	20.36	24.49	10.62	8.66
2	3.18	23.843	19.46	12.14
3	17.92	22.16	14.90	11.62
4	24.76	11.21	12.14	7.94
5	32.24	22.84	11.87	10.46
6	18.40	12.18	11.15	8.32
7	20.40	22.34	18.72	8.02
8	19.62	27.86	20.56	24.62
9	17.78	20.36	17.37	23.76
10	17.79	20.06	17.79	19.723
11	13.98	21.60		

Table 5.17: CV of bootstrapped AMD-RDT estimates of the damping ratio of *College Road*

Measurement	Accelerometer 1		Accelerometer 2	
	EW (%)	NS (%)	EW (%)	NS (%)
1	4.92	2.87	3.81	5.04
2	5.49	5.49	9.11	5.02
3	5.67	5.50	7.63	5.55
4	4.88	9.19	5.96	5.29
5	5.61	6.65	4.94	4.25
6	4.11	3.54	3.73	5.43
7	4.99	8.00	5.84	5.21
8	8.22	3.45	9.36	5.17
9	5.26	5.06	3.84	4.70
10	10.50	5.44	9.82	6.25
11	4.14	4.61		

margins of error in estimates of damping ratio than the BFFT, despite the damping estimates being notably lower. This suggests the AMD-RDT is more precise than the BFFT but it is difficult to say which method is more accurate. The tighter margin of error from the bootstrapped AMD-RDT is visualised in Figure 5.30.

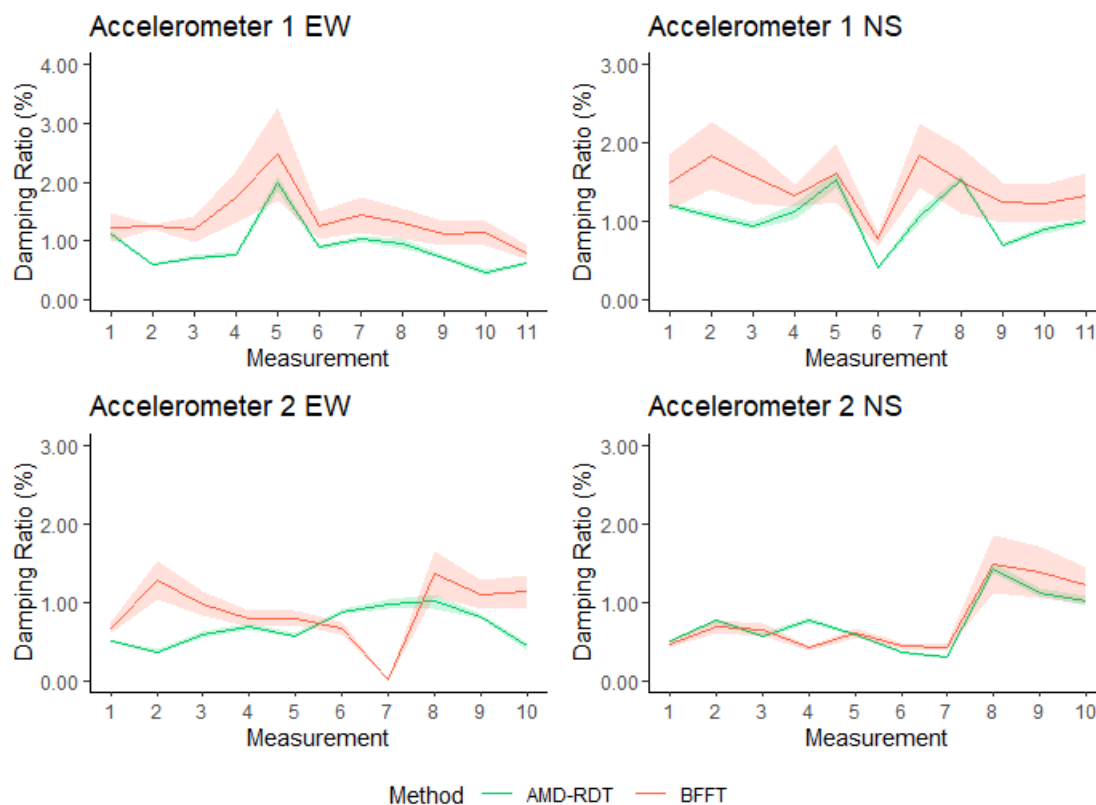


Figure 5.30: Comparison of damping estimates and their possible range using CV values from both the BFFT and bootstrapped AMD-RDT method for *College Road*

Figure 5.31 shows the relationship between estimates of the damping ratio and the natural frequency of *College Road*. Accelerometer 1 in the EW direction shows a weak positive relationship, however, this is not seen for the NS direction or in estimates made using measurements from Accelerometer 2. The plots for Accelerometer 2 show two clusters of results, the clusters are distinct due to the accelerometer recording results in Tower B initially, before being relocated to Tower A to measure the completed structure.

As was emphasised with *Ten Degrees* it is essential to note that each measurement of the acceleration response of a structure recorded under ambient loading conditions will result in a unique damping ratio estimate. Furthermore, the changing structure as more modules are added will also give rise to damping ratio esti-

mates which differ across measurements. Both the BFFT and AMD-RDT appear to sufficiently capture these differences.

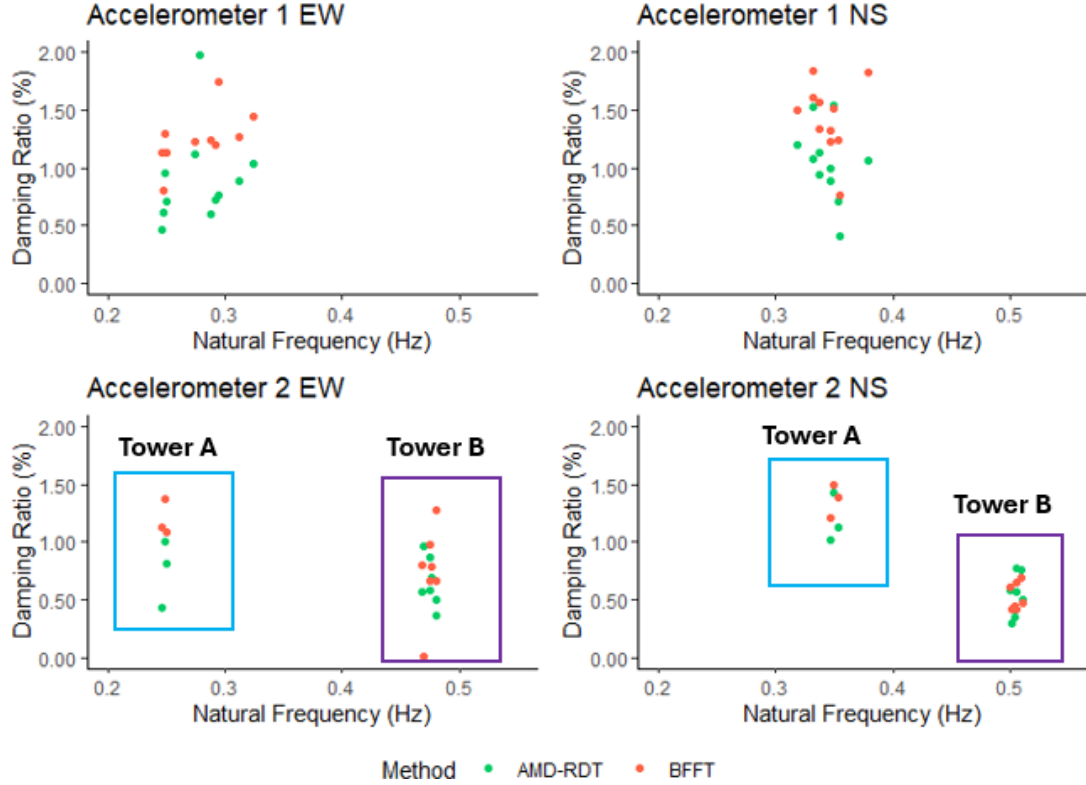


Figure 5.31: Estimated damping ratio of *College Road* vs estimated natural frequency

It is important to identify if the extent of the varying loading conditions, and hence, varied acceleration responses of the structure between measurements has a relationship with the observed natural frequency and estimated damping ratio of *College Road*. Figure 5.32 shows the relationship between the natural frequency of the structure and the RMS acceleration response. Accelerometer 2 again shows two clusters of results; one for when the accelerometer was in Tower B measuring the response of the freestanding RC core and another from when the accelerometer was relocated to the edge of Tower A to record the response of the completed building. Similar to *Ten Degrees* there appears to be no dependence of natural frequency on the recorded RMS acceleration response of the structure.

Considering the relationship between the estimated damping ratio and RMS acceleration response from measurements recorded at *College Road* as shown in Figure 5.33, there appears to be only a weak positive trend. For both directions of Accelerometer 1 and Accelerometer 2 in the EW direction, in particular below an

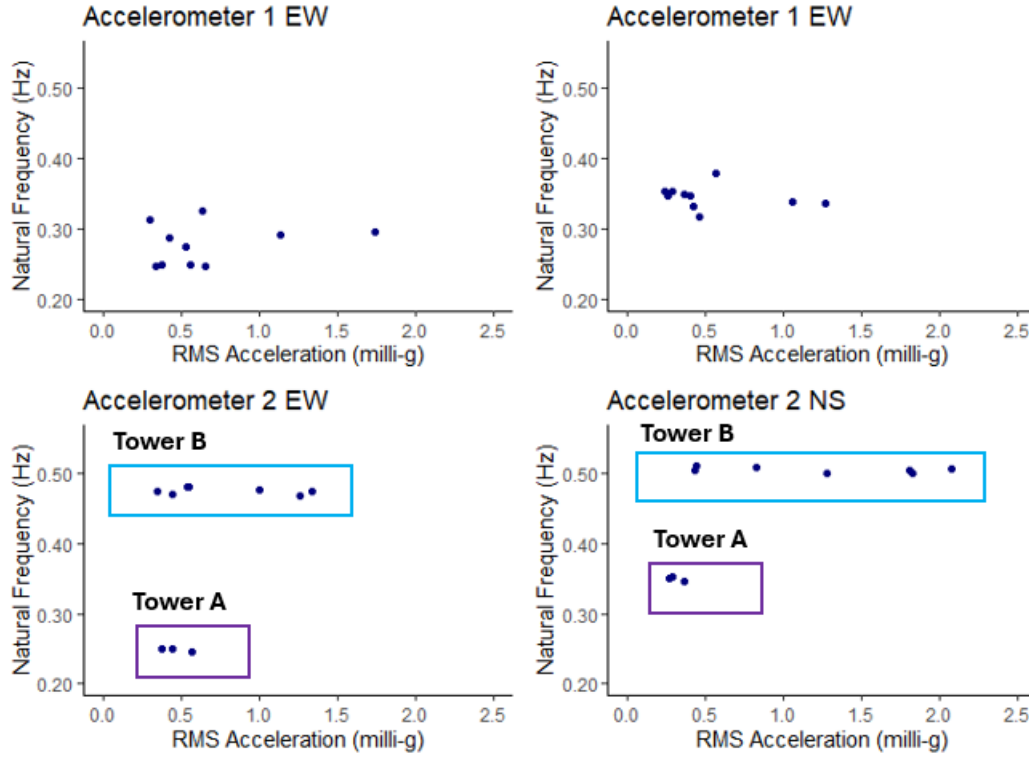


Figure 5.32: Estimated Natural Frequency of *College Road* vs RMS acceleration

RMS of 1 milli-g, damping ratio appears to increase with increasing RMS acceleration. This reflects the relationship observed for measurements from *Ten Degrees* where it is weakly observed that damping ratio estimates appear to increase as RMS acceleration increases. However, the plots for Accelerometer 2 include results for the measurements recorded from the freestanding RC core of Tower B and the completed structure when the accelerometer was relocated to Tower A. Therefore, it is difficult to draw conclusions using results from Accelerometer 2.

The RMS accelerations recorded from *Ten Degrees* were considerably lower than those recorded at *College Road*. The highest recorded RMS acceleration from non-corrupt responses at *Ten degrees* is 0.228 milli-g. For comparison the average RMS acceleration recorded at *College Road* is approximately 1 milli-g. However, it is worth noting that *Ten Degrees* was considerably closer to completion with more storeys of modules installed during the monitoring than *College Road*. Only in Measurement 5 are RMS accelerations observed at *College Road* which exceed ISO 10137 [84] guidelines for occupancy comfort. During this measurement, 13 storeys of modules were installed in Tower A, which is less than one third of the number of storeys in the completed structure. The average wind-speeds recorded at *Ten*

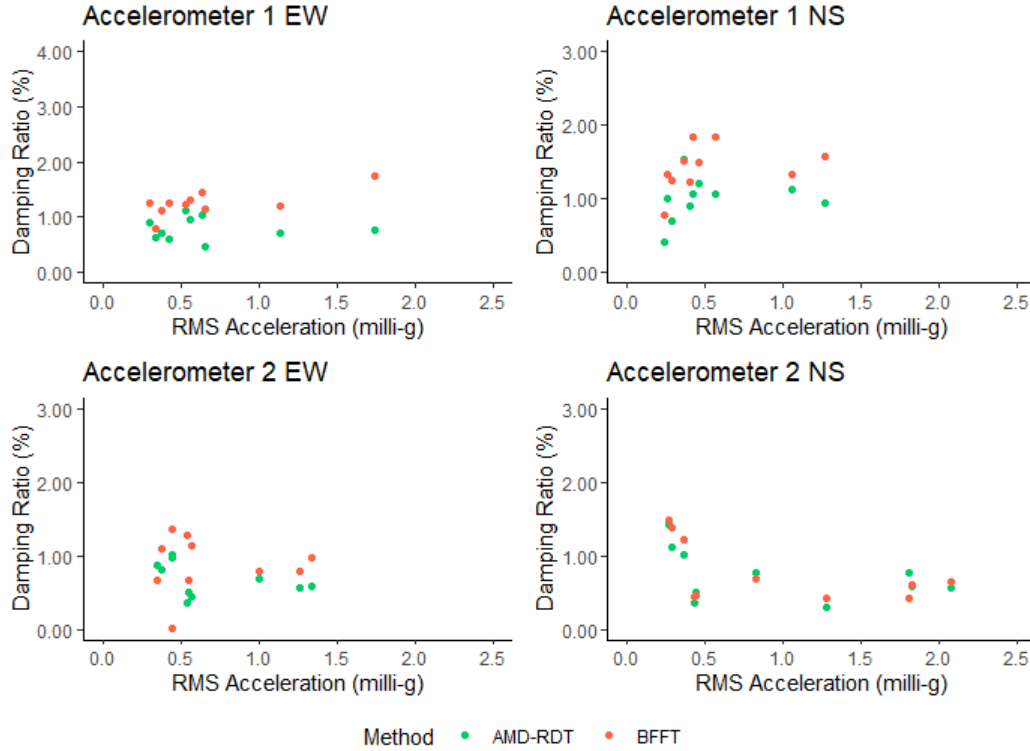


Figure 5.33: Estimated damping ratio of *College Road* vs RMS acceleration

Degrees were significantly lower than the 1 in 50 year design wind speed of 21.50 m/s for the structure. The wind speeds were not recorded at *College Road*, however, the recorded average wind-speeds at Heathrow Airport during the same time period as Measurement 5 reported by The Met Office [229] exceeded the equivalent 1 in 1 year 10 minute average design wind speed of 16.11 m/s. Without measurements of wind speed at the location of *College Road*, it is difficult to know whether the accelerations experienced were as a result of wind loading similar to the design wind speeds of the structure. More observations recorded during measured high wind speeds may result in a more significant trend between RMS acceleration and the inherent properties of the structure.

5.4.2.4 Identified Inherent Properties

A summary of the identified inherent properties of *College Road* in the first mode in both the X (East-West) and Y (North-South) directions is provided in 5.18. These values were estimated from large-scale monitoring, processing of recorded measurements and application of operational modal analysis methods.

Table 5.18: Identified inherent properties of *College Road*

Inherent Property	EW	NS
Natural Frequency (Hz)	0.248	0.350
BFFT Damping Ratio (%)	1.15 ± 0.21	1.35 ± 0.30
RDT Damping Ratio (%)	0.78	0.90
Bootstrapped AMD-RDT Damping Ratio (%)	0.72 ± 0.05	1.11 ± 0.05

5.4.3 Damping Ratio of Modular buildings

There are a number of points worth making about the estimated damping ratios for both *Ten Degrees* and *College Road*. Firstly, no two estimates of damping ratio for either structure were the same. This is because there is no one true damping ratio value of a structure. It is not an exact quantity and it varies as a result of a number of parameters such as the direction and force of loading, the building height and the natural frequency of the structure [11, 12, 230]. Some of these relationships have been identified as part of this research. The variation across estimates of the damping ratio can be related back to the factors expected to affect damping ratio estimates outlined in Section 5.1.1.

1. **Stage of Construction:** A weak relationship between the number of storeys of modules installed and damping ratio was observed for *Ten Degrees* using the AMD-RDT method. However this relationship was not observed for *College Road*. The natural frequency was seen to change as more storeys of modules were installed, it is possible that the weak relationship noted between storeys installed and damping ratio is a result of the relationship observed between natural frequency and damping ratio.
2. **Different Structures:** The damping ratio estimated of the completed structure of both *Ten Degrees* and *College Road* were found to be quite similar. Particularly considering the bootstrapped AMD-RDT method where the damping ratio was estimated to be close to 1.10% for natural frequencies of similar values. A more significant relationship is noted between natural frequency and damping ratio than between the damping ratio estimates from the same structure.

3. **Direction of Response:** The direction in which the response was measured on the structure, i.e. EW vs NS, was seen to influence the estimates of damping ratio. Particularly for *College Road* which is less of a symmetrical structure, the estimates of damping ratio were higher in the NS direction where the structure has a larger dimension. This trend is likely a result of the relationship between natural frequency and damping ratio which has been identified.
4. **Natural Frequency:** A relationship was identified between damping ratio and natural frequency. Higher natural frequencies resulted in higher damping ratios. This is noted across both of the structures studied and each of the OMA methods applied.
5. **RMS Acceleration:** A weak positive relationship was identified between the RMS accelerations of the response and the estimated damping ratios of both buildings. However, the recorded RMS accelerations of both of the completed structures were low and further testing of fully constructed high-rise modular buildings under more significant wind loads would provide further insight into this trend.
6. **Sensors used for Monitoring:** The sensors used to record the response of *College Road* resulted in noisier acceleration response measurements. This is reflected in the CV of the damping ratio estimates for both OMA methods. However, the uncertainty in the estimates is still relatively small, particularly for the AMD-RDT method.
7. **OMA Techniques:** Estimates of the damping ratio calculated using the different OMA methods largely agreed for measurements from *Ten Degrees*. However, more significant difference is seen in the estimates for *College Road* with the BFFT method estimating higher damping ratios. The difference between the estimates from the two methods for *College Road* is likely a result of the basis of the two methods; BFFT is a frequency domain method whereas the AMD-RDT is a time domain method. Time domain methods typically perform better for more complex systems with closely spaced modes whereas frequency domain methods can be more suitable where noisier or shorter responses are measured. Therefore, choosing both a time domain and frequency domain method has allowed for comparison and verification of damping ratio estimates.

It is interesting to note that the value of damping ratio obtained from the majority of measurements taken of the fully constructed *Ten Degrees* and *College Road* buildings are very similar to the value of 1.27% recommended in Eurocode 1991-1-4 for mixed steel and concrete structures. However, using the AMD-RDT method, the damping ratio of *College Road* in the EW direction is estimated to be quite low at 0.72%. The estimated damping ratios of both structures mostly lie within the range of values recommended by ASCE 7-10 of 1% for steel and 1.5% for concrete structures, and within the range of 1% - 1.2% recommended by ISO 4354:2009 in some cases. This contradicts the hypothesis that damping ratios may be higher for modular structures due to the greater potential for frictional energy dissipation in joints. At least for the levels of excitation examined during the described insitu monitoring, there appears to be no evidence of damping being higher than in other forms of construction. While damping values are highly specific to individual structures, the results in Tables 5.10 and 5.18 appear to suggest that it is reasonable to apply the values proposed by design codes for mixed buildings to modular structures. However, results from further monitoring of tall modular structures, particularly under higher levels of excitation, are needed to fully validate this conclusion. With the limited existing research in mind, it is conservative to recommend a value of 1% of critical damping be assumed by designers of mixed steel and concrete volumetric modular buildings with RC cores.

Additionally, it is also relevant to note that the estimated damping ratio of *Ten Degrees*, predominately for the AMD-RDT method, appears to decrease over the course of the monitoring campaign. In other words, as extra modules, and therefore extra inter-module movement joints, were added, the damping ratio decreased. Again, this appears to contradict the hypothesis that the additional joints in modular construction may provide additional opportunities for frictional energy dissipation. It is possible that the higher RMS accelerations for some of the earlier events compared to the lower values observed for later measurements recorded at both structures, could be a key factor in the lower damping ratios estimated for the later events. However, only weak relationships were noted between the RMS accelerations and the damping ratios for both *Ten Degrees* and *College Road* and further testing would be required to add more evidence to this claim.

Furthermore, Only two accelerometers were used to monitor both *Ten Degrees* and *College Road*, and one anemometer was used to measure wind speed

at a low sampling rate at *Ten Degrees*. This was due to budget constraints and limitations with installation of monitoring equipment as both structures were still under construction for a significant portion of the monitoring. The restricted monitoring equipment used during this study is a limitation and whilst the values and relationships identified provide helpful insights into the behaviour of modular structures, more extensive monitoring would be beneficial to the progress of modular construction. Ideally, a number of accelerometers distributed over the height of the structures and anemometers with a higher sampling frequency would provide more understanding of the behaviour of modular buildings and is an area for further research.

5.5 Conclusions

This chapter has provided critical insight into the dynamic behaviour of high-rise modular buildings and their inherent properties. Through measurement of ambient acceleration responses, application of existing and improved OMA techniques and analysis, the natural frequency and damping ratio of high-rise volumetric modular buildings have been identified in literature for the first time.

Within this chapter, details of the monitoring of the two case study structures, *Ten Degrees* and *College Road* are provided. Both involved placement of two triaxial accelerometers to capture the ambient acceleration response of the buildings under wind loading. For *Ten Degrees* the ten minute average wind speeds were also recorded. Both structures were still under construction throughout the measurement of the acceleration response. Accelerations at *Ten Degrees* were measured whilst the structure had 32 storeys of modules installed, until it was fully complete with 44 storeys of modules installed in Tower A. *College Road* was monitored from when there were no storeys of modules installed until there were 24 storeys of modules installed in Tower A and again when the structure was complete with 50 storeys of modules installed in Tower A and 35 storeys of modules installed in Tower B.

In total, ten measurements of the ambient acceleration response from *Ten Degrees* were processed using OMA techniques, the final three measurements were captured on the finished structure. Eleven ambient acceleration measurements from *College Road* were processed using OMA techniques, four of which were recorded upon completion of the structure. The OMA methods used to identify the inherent

damping ratio of the structures were the BFFT, RDT, AMD-RDT and a novel bootstrapped AMD-RDT which has enhanced capability for identifying closely spaced modes and provides a quantification of the error in the calculated damping ratio estimate. The traditional RDT and AMD-RDT methods do not provide errors of their estimates and this has been a limitation noted in literature [70, 100].

The identified natural frequencies and damping ratios of the case study structures are summarised in Table 5.19. It can be concluded that the estimated damping ratios of the two modular structures are close to the code recommended value of 1.27% for mixed steel and concrete structures (Eurocode 1991-1-4). This suggests that modular buildings do not exhibit higher damping ratios than more traditional steel and concrete structures. Furthermore, in some cases the estimated critical damping ratio is less than code recommended values and it is advisable to conservatively assume a value of closer to 1.0% in design of modular structures employing similar core-based structural schemes

Table 5.19: Identified inherent properties of both *Ten Degrees* and *College Road*

Inherent Property	<i>Ten Degrees</i>		<i>College Road</i>	
	EW	NS	EW	NS
Natural Frequency (Hz)	0.314	0.370	0.248	0.350
BFFT Damping Ratio (%)	1.23 ± 0.07	1.64 ± 0.12	1.15 ± 0.21	1.35 ± 0.30
RDT Damping Ratio (%)	1.35	1.16	0.78	0.90
Bootstrapped AMD-RDT Damping Ratio (%)	1.10 ± 0.02	1.14 ± 0.03	0.72 ± 0.05	1.11 ± 0.05

In total damping ratios were estimated from a total of 82 different wind-induced acceleration responses of high-rise modular buildings. The responses are a result of measurements recorded from both structures, in both directions, on two separate accelerometers. From these estimates, there are some weak trends which can be identified. The damping ratio appears to increase as RMS accelerations increase, although this is a weak trend it remains noteworthy. The RMS acceler-

ations recorded during each of the measurements were quite low, indicating that the estimates of damping ratio are conservative. It is possible that at higher RMS accelerations, the damping ratio may be larger. This would prove to be beneficial for taller modular buildings which will likely have lower natural frequencies and hence larger RMS acceleration responses. However, for *Ten Degrees*, a weak positive trend was noted between natural frequency and damping ratio; suggesting that taller modular structures with lower natural frequencies will exhibit a lower damping ratio. This could cancel out any trend between RMS acceleration and damping ratio. Interestingly, the damping ratios of the two structures do not differ significantly. In fact the damping ratio of both *Ten Degrees* and *College Road* are larger in the same direction in which the natural frequency is larger, NS in both cases. It appears there is a stronger relationship between natural frequency and damping ratio than there is between estimates of damping ratio from the same structure. The trends noted provide interesting topics for further research. In particular, monitoring of more high-rise modular buildings, specifically under more significant wind loads, may help further analyse trends between damping ratio, RMS accelerations and natural frequency. The damping ratios identified in this Chapter have been used in Chapter 6 to create a data-informed MDOF surrogate model for calculating the dynamic response of high-rise modular buildings.

Through capturing the response of the two structures during the construction process as storeys of modules were added, deepened understanding of the contribution of the modules to the bending stiffness of the structure and the overall dynamic response of the completed building has been provided. It was observed that as modules were added to higher storeys in *Ten Degrees* the natural frequency of the structure reduced. However, as modules were added to the lower storeys of *College Road*, the natural frequency initially increased prior to decreasing for the finished structure. This suggests that the contribution of the modules to global lateral stiffness makes a significant contribution to the dynamic performance of modular buildings, significantly at lower storeys. However, the mass contribution of modules at the top of the structure has a larger impact on the natural frequency. This observation of an evolving natural frequency as a modular structure progresses towards completion provides motivation for subsequent work on quantifying the stiffness contribution of modules to the global bending stiffness of a modular building completed in Chapter 6.

Whilst further studies of high-rise modular buildings are required to provide a full database of empirical measurements, the reported values provide an initial baseline. The inherent properties identified in this chapter pave the way for more informed design and assessment of SLS design of high-rise modular buildings.

Chapter 6

Stiffness Identification and Data-Informed Surrogate Models

6.1 Introduction

As the construction of modular buildings to new heights continues, issues with applying traditional modelling techniques to these structures are becoming apparent. It has been found that current modelling techniques do not cater for the variety of elements and connections seen in modular buildings and they do not capture the dynamic behaviour of modular buildings sufficiently [17, 232]. There is a significant lack of standards for modular buildings; both in the case of structural design and in applications of BIM [33, 233]. Structural design of modular buildings is currently completed using traditional, non-specific design codes with lateral stability elements such as reinforced concrete cores and modules designed independently rather than as a cohesive structure. Additionally, there are no standards for the modelling of modular buildings resulting in traditional BIM techniques being applied [4]. Using practices which were not created for this type of construction leads to inefficiency in design and modelling processes, impeding the progression of this form of construction [4].

Furthermore, the stiffness contribution of modules to the overall lateral stability of a modular structure is unknown [4, 17, 21, 69, 234, 235]. At present, tall modular structures are designed considering only the lateral stability of the RC concrete core and ignoring any stiffness that the modules may add. Limited studies

on the stiffness contribution of the modules have been performed and no method for calculating this stiffness has been developed. In reality, the modules must add some stiffness and it is imperative to the future design of modular structures, and assessment of habitability requirements, that this value is identified.

This chapter aims to identify the stiffness contribution of the modules in the two case study structures and to use these values to create a data-informed surrogate model suitable for the assessment of the dynamic response of modular buildings. Attempts at quantifying the stiffness value of the modules include comparison of natural frequency results from the extensive finite element models described in Chapter 4 with results from the in situ acceleration monitoring schemes described in Chapter 5. A stepped beam mathematically equivalent mechanical model which considers the natural frequency of the case study structures at their varying degrees of completion has been developed. This model, alongside the results from the in situ monitoring of the case study structures has been employed to efficiently identify the stiffness contribution of the modules in the case study structures, hence, providing the first reporting and analysis of the stiffness of a full-scale volumetric modular building.

Capitalising on the results obtained thus far from Finite Element (FE) models, in situ monitoring, and equivalent models, a data-informed multi degree of freedom surrogate model specifically for modular buildings is developed. The surrogate Multi Degree of Freedom (MDOF) model has been developed using masses identified from FE models, stiffness identified from the equivalent Stepped Beam Model and damping ratios identified from in situ monitoring. This MDOF model is one of the first surrogate models developed specifically for modular buildings, enabling efficient prediction of the acceleration response of high-rise modular buildings and hence, the ability to assess compliance with habitability requirements.

6.2 Finite Element Models of Modular Buildings

It has been widely reported that traditional methods for modelling modular buildings using FE software are largely unsuitable [4, 5, 21, 32, 33, 66]. There have been challenges identified both in creating full-scale FE models of modular buildings and in creating simplified models, particularly of the bespoke connections typical in modular buildings [50, 51, 53]. The full-scale models are too large and cumbersome

[5, 33, 36] and the simplified models lack sufficient experimental evidence to be validated [17, 18, 236].

6.2.1 Finite Element Models Reported in Literature

Some FE models which have been described in literature have been discussed in Chapter 2. These include:

- Research completed by Chua et al. [17] which compared the use of translational spring models and simplified joint models to capture the transfer of lateral forces within an FE model of a 40 storey modular structure. It was found that the translational spring model was more suitable.
- Work by Lacey et al. [18] which compared theoretical models of inter-module connections with limited available experimental results and found that the current simplified models do not accurately capture the shear stiffness, tensile stiffness or moment rotation of the actual connections.
- A study performed by Shan et al. [236] which considered a piecewise FE model where connections were modelled separately in ABAQUS software and the resultant forces represented as springs in a 3D model of the full structure created in ETABs. This work was not validated with experimental results.

As seen above and found throughout existing literature [21, 42, 54, 67, 237], the most common method for modelling modular buildings involves FE models with simplified representation of the more complex inter-module connections. This is largely due to how substantial FE models become when each connection is modelled within the model of the full structure. The simplified connections are often represented by one dimensional elements with stiffness properties set from results of more detailed FE models or experimental tests. Each of the above studies concluded that more experimental studies of modular buildings were required to accurately create simplified models of modular buildings. Many studies have considered modelling the dynamic response of modular buildings to environmental conditions [21, 36, 50, 54, 67, 42, 237], however, there is a noted lack of studies which validate multi-storey models with experimental or in situ testing.

Peng et al. [67] created a detailed FE model of two stacked modules in ABAQUS software in order to assess the lateral stability of an ‘elementary unit’ of a modular building. Each member was modelled individually, as were the bespoke inter-module connections between the stacked modules. Lateral loads were applied in the x and y directions in order to identify the elastic stiffness and load-resisting capacity of the structure. Two different profile steel sections were compared as the module columns. It was found that for square hollow sections, similar to those used in the two case-study structures investigated by this research, the elastic stiffness was approximately 1.8 kN/mm and the load resisting capacity was approximately 125 kN. The results of the detailed analysis were used to create a simplified FE model in ABAQUS in which connections were modelled as ties, the composite floor system was modelled as a slab using shell elements and the properties of the sections are modified to reflect the stress profiles of the more detailed model. The simplified model is then used to evaluate wind loads on a 12 storey modular building in Sydney. Results of the wind-induced response of the model are compared to Australia’s design code for wind AS 1170.2 and found to satisfy ultimate limit state requirements but fail to adhere to serviceability limits. The simplified model was not validated using any experimental data. The authors noted a need for improved research into the dynamic response of modular buildings and more experimental data to allow for validation of models.

Similarly Munmulla et al. [42] investigated the wind-induced response of modular buildings. A FE model of a 25 storey case study structure was created in SAP-2000, a software in which ETABs is built from. Instead of relying on properties from detailed FE models of connections as was seen in work by Chua et al. [17], Shan et al. [236] and Peng et al. [67], Munmulla et al. [42] uses rigid connections between modules and verifies these results using the stiffness of connections derived from limited previously reported experimental studies of connections by Chen et al. [51]. It was identified that addition of concrete cores had a larger impact on the stiffness of the structure and satisfaction of the serviceability limit state set out by AS 1170.2 than the inter-module connections. Both Chen et al. [51] and Munmulla et al. [42] identified that the available experimental data for verification of models of connections is limited.

Ye et al. [50] also used limited experimental data to verify simplified models of modular buildings. This work performed a review of lateral structural stability in

multi-storey modular buildings by considering four different stability systems: (1) module connections, (2) module structure, (3) layout pattern and (4) supplemental stability elements. Ye et al. [50] considered existing literature and created a rating system for the different stability mechanisms which allows for evaluation and comparison between the different options. Simplified two dimensional numerical models were created in OpenSees and pushover analysis was performed to investigate the influence of each lateral stability system. Experimental data is used from cyclic tests of the lateral stability of single modules carried out by Annan et al. [234] and Hong et al. [238] in a laboratory. Annan et al. reported the elastic stiffness of a single module as 14.3 kN/mm, whereas Hong reported the elastic stiffness of a single module as 0.98 kN/mm. The difference in these values is likely due to the different structural form of the modules tested with Annan et al. using braced modules and Hong et al. considering modules with double skin steel panels. Both Annan et al. [234] and Hong et al. [238] created two dimensional models in OpenSees which were validated using results from experimental tests on one single module. These models were then scaled and used by Ye et al. [50] to perform push-over analysis of multi-storey modular buildings using the different stability systems. It was found that RC cores had the largest influence on lateral stability, noting that the diaphragm stiffness and bracing in the modules can also affect the stiffness of a modular building. However, this two dimensional simplified model of a multi-storey structure has not been validated with in situ tests.

Further studies which have attempted to experimentally validate models of modular buildings include work by Alembagheri et al. [239] and Bi et al. [69]. Alembagheri et al. [239] tested a single full-scale volumetric module equipped with accelerometers and a data acquisition system in order to study the different dynamic characteristics of modules with different infill wall systems. The module was tested on a shaking table under ambient loading conditions for a number of different infill wall conditions including a stud wall, and different proportions of the module wall infilled with gypsum plasterboard. The module with a stud wall infill had a natural frequency of 6.39 Hz and a damping ratio of 4.53% in the first mode. With a fully infilled wall, the natural frequency of the module was 19.14 Hz and the damping ratio was 2.08%. It was concluded that light-weight steel framing alone had insignificant effect on the inherent dynamic properties of the volumetric module. Furthering this work, Rashidi et al. [240] tested additional infill wall conditions of the full-scale module and used the experimental results to validate a micro-level FE

model of a module in ABAQUS software and to calibrate a macro-level strut model in which the infilled wall is replaced with a single strut. The micro-level model was found to have less than a 3% deviation in prediction of the natural frequency of the module with no infill and up to 6% deviation with the infilled module. Whilst this study does show good agreements between the FE model and actual experimental results, it is noted that it only considers one single module and further research into the dynamic behaviour and modelling of full-scale modular buildings still requires further investigation.

The work described in Bi et al. [69] is, to the best of the author's knowledge, the only full-scale in situ testing of a modular building which has been reported on in literature. In this study, field measurements of Gangnan Road Public Rental Housing Community in Zhenjiang City, Jiangsu Province were recorded. The structure consists of 18 storeys with a total of 324 steel volumetric modules stacked around two reinforced concrete cores. The structure was monitored under ambient conditions using an anemometer and two accelerometers. Measurements were recorded over a 14 hour period in January 2018 and the maximum 10 minute average wind-speed was 10.10 m/s. The natural frequency of the structure in the x and y direction in the first mode were identified as 1.1475 Hz and 0.9956 Hz respectively by taking the power spectral density of acceleration response. The damping ratios of the structure were not reported. A FE model of the structure was built using ABAQUS software and verified using the natural frequencies identified from the in situ testing. The natural frequencies predicted using the FE model were found to be within 5% of the values identified from the ambient measurements. The FE model was deemed to be a fair representation of the actual dynamic behaviour of the modular building. The research reported on by Bi et al. [69] provides an initial insight into the accuracy of FE models in capturing the dynamic response of modular buildings. However, this study is limited to one 18 storey structure and there is significant scope for further research of this nature.

It is important to note that within the studies discussed above, any FE models which have been verified using experimental results are not substantial in size. As structures become increasingly larger, the practicality of modelling each element becomes questionable and there is little evidence to suggest that accuracy in the FE model predictions of dynamic behaviour is improved by more detailed models. Therefore, there is a significant demand for simplified models of high-rise

modular buildings which accurately capture the dynamic response of this novel form of construction. These simplified models will aid in preliminary design of these novel structures in order to ensure adherence to habitability requirements.

6.2.2 Finite Element Models of Case Study Structures

As discussed in Chapter 4, the FE models of the two case study structures which have been studied as part of this research were created by the Structural Engineers in FE software ETABs during their respective design process. Within these models, the RC cores and each individual member of each module are modelled using the appropriate material properties and member sizes. The inter-module connections and the connections between the modules and the RC cores are modelled using steel plates with standard moment connections between the adjacent members. Two primary difficulties become notable when using these models:

1. Difficulties relating to the size, usability and computational expense of the model
2. Discrepancy relating to the accuracy of this model in predicting the dynamic response of the structures, particularly the natural frequency.

It is well reported that models of modular buildings are significantly larger than for more traditional forms of construction [5, 32, 36, 54, 64, 67]. This remains true despite using simplified models of connections and is primarily a result of the duplication of members that occurs as modules are stacked. The size of the FE models for the 44 and 50 storey *Ten Degrees* and *College Road* are exceptionally large with 260,000 and 455,000 elements respectively. This results in FE models which are difficult to work with, limiting the ability to perform parametric analysis and hence, hindering assessment of the effects of member size, connection types and core dimensions on the dynamic response of the structures. Furthermore, these models are computationally expensive to solve for both static and dynamic analysis.

On the second difficulty with these FE models, namely their accuracy in predicting the dynamic response of the structures, it can be seen that the natural frequencies estimated from the FE models do not agree with the measured natural frequencies observed from using OMA techniques on recorded field measurements as

described in Chapter 5. A comparison of the natural frequencies estimated from the FE models and the identified natural frequencies from in situ testing are provided in Tables 6.1 and 6.2. It can be seen that the difference between the estimates is significant, with the FE models providing a substantial underestimate of the inherent dynamic property.

Table 6.1: Natural Frequencies of *Ten Degrees* identified from FE model compared with natural frequencies identified from field measurements

Method	East-West X-X	North-South Y-Y
Natural Frequency (FE model)	0.18 Hz	0.19 Hz
Natural Frequency (Field measurements)	0.314 Hz	0.370 Hz
Difference	43%	49%

Table 6.2: Natural Frequencies of *College Road* identified from FE model compared with natural frequencies identified from field measurements

Method	East-West X-X	North-South Y-Y
Natural Frequency (FE model)	0.18 Hz	0.20 Hz
Natural Frequency (Field measurements)	0.248 Hz	0.350 Hz
Difference	27%	43%

The natural frequency of a structure is dependent on its mass and stiffness as shown in eq. 6.1, where ω is the natural frequency of a structure in rad/sec, k is a structures bending stiffness in N/m and m is a structures mass in kg. The mass of a structure is often accurately calculated using an FE model as the mass of each component is easily quantified and the total mass of a structure is the sum of its parts. The stiffness of a structure is more difficult to calculate using FE models and is dependent on the global interactions of the stiffness of each member. Therefore, it is more likely that inaccuracy in the natural frequency predicted by the FE models of *Ten Degrees* and *College Road* is a result of the modelled stiffness of the global structure. Given that the natural frequencies are underestimated by the FE model when compared to the values obtained through in situ testing, it is

valid to infer that the FE models of the modular structures underestimate their global stiffness. Design and modelling of concrete is a well established process with appropriate codes and guidelines such as Eurocode 2 [241]. It is possible that the FE model underestimated the stiffness of the RC core, however given the extent of research and application of concrete as a construction material, this is unlikely. Therefore, it is reasonable to conclude that the modelling of the modules is the primary contributor to the underestimated natural frequencies.

$$\omega = \sqrt{\frac{k}{m}} \quad (6.1)$$

The stiffness contribution of the modules to the overall lateral resistance of the structure is a well established uncertainty in the design of modular buildings [4, 5, 17, 52]. This uncertainty was addressed during the design process of *Ten Degrees* and *College Road* by conservatively designing the concrete cores to be adequately stiff enough to resist all lateral loading and designing the modules to resist vertical loads and transfer all lateral loading to the RC cores through sufficiently stiff inter-module connections. Considering the underestimation of natural frequencies by the FE model, the contribution to lateral stiffness of the modules is likely to be more significant than the FE model captures.

In the case of the two volumetric high-rise modular case study structures, it can be concluded that the FE models used were difficult to work with and did not accurately estimate the natural frequencies of the structures. Whilst the FE model is suitable for assessing performance of modular structures under gravity load and ULS conditions, it does not appear to be accurate in predicting lateral dynamic response for assessing the structure in SLS and hence determining compliance with habitability requirements. Furthermore, as modular structures continue to increase in height, the size, usability and computational expense will continue to grow and become even more unmanageable. Therefore, there is an evident need for a simplified and more accurate surrogate model for assessing the dynamic performance of modular buildings.

6.2.3 Macro Module Finite Element Model

Given the nature of modular buildings as repeatable units stacked around a lateral stability system, there is considerable capacity for efficient modelling techniques to be employed where a single element can represent one module. One such technique which was investigated was to create a macro ‘module’ element in the FE software ETABs which reflects the inherent properties of a single module. This macro module element could then replace the multiple elements used to model each module in the global FE model, hence reducing the model time, complexity and the computational expense of performing eigenvalue analysis in order to identify the dynamic properties of the structure.

To create a macro module element, an FE model of a typical module with all elements modelled was first created. The FE software ETABs was used. The size of the steel members in each module varies across the structure, with members at upper storeys being smaller due to decreasing vertical loading. Typical member sizes were chosen to create a module which is representative of the ‘average’ module used in both *George Street* and *College Road*. In the FE model of the single ‘average’ module, steel members were modelled using frame elements of the relevant dimensions and the RC slab of the module was designed as a 150 mm thick slab element. The size of the members of the average module are given in Chapter 4 in Figure 4.5 and the material properties used are given in Table 6.3.

Table 6.3: Material properties used in FE model of single module

Material	Property	Value
Concrete	Strength	C35/45
	Weight per unit Volume	25 kN/m^3
	Modulus of Elasticity	$30,250 \text{ MPa}$
	Poisson’s Ratio	0.2
Steel	Grade	S355
	Weight per unit Volume	76.8195 kN/m^3
	Modulus of Elasticity	$205,000 \text{ MPa}$
	Poisson’s Ratio	0.3

The size, mass, stiffness and natural frequency of the ‘average’ module were obtained from the FE model and used to develop and validate a suitable macro module element. The mass of the module was extracted by considering the base reactions of the FE model under dead loading. This mass is representative of the

mass of the module including the self-weight of non-structural elements as each module arrives to site fully fitted out. The stiffness of the module was found by applying a 100 kN force to the top of the FE model and recording the deflection. The stiffness could then be calculated using eq. 6.2.

$$k = \frac{F}{\delta} \quad (6.2)$$

where k is the stiffness in N/m, F is the applied lateral force, in this case 100 kN and δ is the lateral deflection in mm.

A modal analysis was run using ETABs in order to obtain the natural frequency of the individual module. The stiffness, mass and natural circular frequency of the module were verified using eq. 6.3.

$$k = \omega_n^2 m \quad (6.3)$$

where ω_n is the natural circular frequency in rad/sec, k is the stiffness of the module in N/m and m is the total mass of the module in kg. Once the stiffness was found, the Young's Modulus, E , of the macro module was found using eq. 6.4.

$$E = \frac{k L}{I} \quad (6.4)$$

where E is the Modulus of Elasticity in Pa, L is the length of the module in m and I is the Second Moment of Area of the module in m^4 is calculated using eq. 6.5.

$$I = \frac{b d^3}{12} \quad (6.5)$$

where b is the total breadth of the module in m and d is the total height of the module in m.

The properties in the x-direction, i.e. along the length of the module, derived from the detailed model of the individual module and adopted for use in the macro module element, are shown in Table 6.4.

Table 6.4: Properties of single module in the x-direction

Property	Value
Length, L	8.887 m
Breadth, b	3.472 m
Height, h	2.875 m
Second Moment of Area, I	6.876 m^4
Mass, m	16.927 kg
Stiffness, k	191,990,266 N/m
Natural Circular Frequency, ω_n	106.50 rad/sec
Natural Frequency, f	16.95 Hz
Modulus of Elasticity, E	248 MPa

A material property, named ‘module’, was created in ETABs using the mass and modulus of elasticity of the single module as listed in Table 6.4. The macro module element was modelled as a singular beam element in ETABs with a solid rectangular section of the same dimensions of a single module using the new ‘module’ material property. Figure 6.1 shows the macro module element and the x and y directions referred to in Tables 6.4 and 6.5. Eigen-analysis was performed in ETABs in order to identify the natural frequency of the macro module element. This value was compared to the natural frequency of the full module for validation. A comparison of the natural frequencies of both the full model and macro model of the module are shown in Table 6.5.

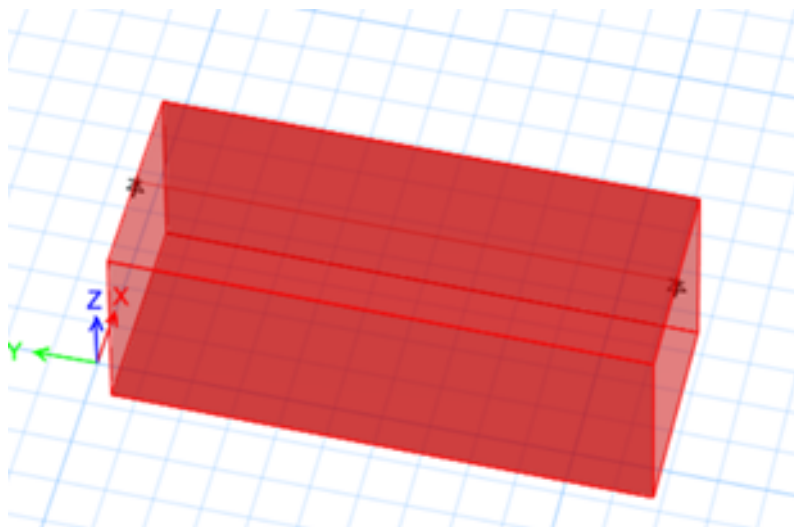


Figure 6.1: Macro module element

Table 6.5: Natural frequencies of the full model and macro model of a single module

Model	Natural Frequency (Hz)	Natural Frequency (Hz)
	X-X Direction	Y-Y Direction
Full model of single module	16.95	12.34
Macro model of single module	16.99	12.36
Difference	0.24%	0.16%

It is clear that the natural frequencies obtained from both models are close in value. Although the properties of the macro module element were set to reflect the full module model, there is still a slight error in the natural frequency of the macro element. This is likely due to the lumped mass of the macro module element and negligible rounding of values. The primary advantage of the macro module element is that it is one single element which accurately captures the inherent dynamic properties of a single module without the need to model each of the 19 members which constitute each module. Hence, the complexity of a full model of a modular structure is greatly reduced.

The macro module element was used to replace each module in the full FE model of *Ten Degrees* in ETABs. The RC cores and connections between the modules and core and the inter-module connections remained the same and were modelled with steel plates. An eigen-analysis of the full structure with macro module elements was performed. The natural frequencies calculated by this analysis in ETABs are reported in Table 6.6.

By substituting the macro module element into the FE model of *Ten Degrees*, the model reduces from 260,000 elements to 60,000 elements when the lesser number of steel members, slabs, intra-module connections and joints are considered. Hence, the usability, ability to parametrically assess the dynamic behaviour, and the computational expense of the model is greatly improved. Performing eigen-analysis on the FE model with the macro module elements on a PC with 32 GB of RAM and an 8 core Intel i9 processor takes less than 7 minutes. This is significantly less than the almost three hours required to run eigen-analysis of the full FE model with each member of the modules modelled.

Table 6.6: Natural frequencies of the full model and macro model of a single module

Method	Natural Frequency (Hz)	Natural Frequency (Hz)
	X-X Direction	Y-Y Direction
	East-West	North-South
Full FE model	0.18	0.19
FE model with Macro module	0.18	0.18
Field Measurements	0.314	0.370

It can be seen that the FE model including the macro module element calculates natural frequencies similar to those estimated using the full FE model. This is not unexpected given the macro-module element was designed to replicate the dynamic properties of the full FE model of a single module. Whilst substituting a macro module element allows for a model which can be studied and solved more efficiently, it does not solve the inaccuracy in using FE models to predict the dynamic performance of high-rise modular buildings. The inaccuracy in modelling the dynamic response of the modular structures must lie elsewhere; for example in the global interactions between the RC cores and the modules. A possible way to capture these interactions and improve accuracy in predicting the dynamic behaviour of modular buildings is to quantify the inherent dynamic properties, such as bending stiffness, natural frequency and damping ratios, through analysing results from in situ testing of this novel form of construction.

6.3 Stepped Beam Model

An alternative numerical model was developed to quantify the stiffness contribution of the modules to the overall dynamic behaviour of the studied modular structures. This model is less computationally expensive than the finite element models and is one which suits the nature of modular construction better, more accurately representing the stiffness contribution of modules to the finished structure. The model uses the results from the in situ testing of *Ten Degrees* and *College Road* and capitalises on the monitoring of the structures through varying stages of construction.

The numerical model is based on the free vibration analysis of a stepped beam using the Adomian Decomposition Method as proposed by Mao and Pietzrko [242]. This Stepped Beam Model lends itself well to the in situ acceleration measurements obtained at different levels of completion. The model is verified using the natural frequencies estimated from the in situ acceleration measurements of *Ten Degrees* and *College Road*. It is then used to replicate the in situ structures at the varying levels of completion and compare the modelled natural frequencies with the values identified through field measurements.

6.3.1 Adomian Decomposition Method

The Adomian Decomposition Method (ADM) is a useful numerical method which can be employed in modelling real world physical problems to obtain analytical and numerical solutions of differential equations. The method is applied to solve ordinary and partial nonlinear differential equations using Adomian polynomials to converge on a solution of the nonlinear part of the equation without having to linearise the system. The method was first proposed by George Adomian from the 1970's - 1990's [243] and has been further developed and employed extensively to solve a number of complex physical problems [243, 244, 245, 246, 247].

The ADM as proposed by Adomian [244] is described as follows, first considering the general equation:

$$Fu(t) = g(t) \tag{6.6}$$

where F is a general nonlinear differential operator with both linear and nonlinear terms and $u(t)$ is the displacement response. The linear terms can be decomposed into $L + R$ where L can be inverted and R is the remainder of the linear operator. L is taken as the highest order derivative to avoid difficult integrations resulting when complicated Green's functions are involved. Therefore, eq. 6.6 can be rewritten as in eq. 6.7.

$$Lu + Ru + Nu = g \tag{6.7}$$

where Nu represents the nonlinear terms. Isolating to solve for Lu :

$$Lu = g - Ru - Nu \quad (6.8)$$

where L is invertible, the equivalent expression in eq. 6.9 is obtained.

$$L^{-1}Lu = L^{-1}g + L^{-1}Ru - L^{-1}Nu \quad (6.9)$$

If an initial value problem is considered, then the integral operator L^{-1} can be taken as the definite integral from 0 to t . If L is a second order operator, then L^{-1} is a two-fold integration operator and $L^{-1}Lu = u - u(0) - tu'(0)$. Indefinite integrals can be used for boundary problems with the constraints given from the conditions. Through solving eq. 6.9 for u , eq. 6.10 becomes:

$$u = A + Bt + L^{-1}g - L^{-1}Ru - L^{-1}Nu \quad (6.10)$$

As Nu is nonlinear, it can be equated to the sum from 0 to infinity of A_n as shown in eq. 6.11, where A_n are the Adomian polynomials outlined in eqs. 6.16 to 6.20.

$$Nu = \sum_{n=0}^{\infty} A_n \quad (6.11)$$

u can also be decomposed into its sum as in eq. 6.12, where $u_0 = A + Bt + L^{-1}g$:

$$\sum_{n=0}^{\infty} u_n = u_0 - L^{-1}R \sum_{n=0}^{\infty} u_n - L^{-1} \sum_{n=0}^{\infty} A_n \quad (6.12)$$

which can be written as:

$$u_1 = -L^{-1}Ru_0 - L^{-1}A_0 \quad (6.13)$$

$$u_2 = -L^{-1}Ru_1 - L^{-1}A_1 \quad (6.14)$$

$$\vdots$$

$$u_{n+1} = -L^{-1}Ru_n - L^{-1}A_n \quad (6.15)$$

The Adomian polynomials are calculated such that A_0 depends only on u_0 and A_1 depends only on u_0 and u_1 and so forth. Each u_n is solvable and the sum of A_n for Nu as shown in eq. 6.11 is a Taylor Series for $f(u_0)$.

Therefore, the Adomian polynomials can be written as:

$$A_0 = f(u_0) \quad (6.16)$$

$$A_1 = u_1(d/du_0)f(u_0) \quad (6.17)$$

$$A_2 = u_2(d/du_0)f(u_0) + (u_1^2/2!)(d^2/du_0^2)f(u_0) \quad (6.18)$$

$$A_3 = u_3(d/du_0)f(u_0) + u_1u_2(d^2/du_0^2)f(u_0) + (u_1^3/3!)(d^3/du_0^3)f(u_0) \quad (6.19)$$

$$\vdots$$

and can be summarised as:

$$A_n = \frac{1}{n!} \frac{d^n}{d\lambda^n} \left[N \sum_{i=0}^n \lambda^i u_i \right]_{\lambda=0}, \quad n \geq 1 \quad (6.20)$$

where λ is a grouping parameter.

The key advantage of ADM is its broad applicability. It can be directly applied to a range of differential equations regardless of their linearity, homogeneity or coefficients. This capability is the primary reason it was applied by Mao and Pietzrko [242] in the stepped beam methodology outlined below.

6.3.2 Application of the Stepped Beam to Case Study Structures

The stepped beam methodology proposed by Mao and Pietzrko [242] is adapted to study the stiffness contribution of the case study structures. This is achieved by considering the inherent natural frequencies identified from in situ monitoring of the structures in their partial completion state. The ‘step’ in the beam is considered as the step between the storeys in which modules are installed and the free-standing core above this level. The problem is set up by considering the ratio of mass per unit length (m) and stiffness (EI) of the sections of the building above and below the module installation level as shown in Figure 6.2 such that:

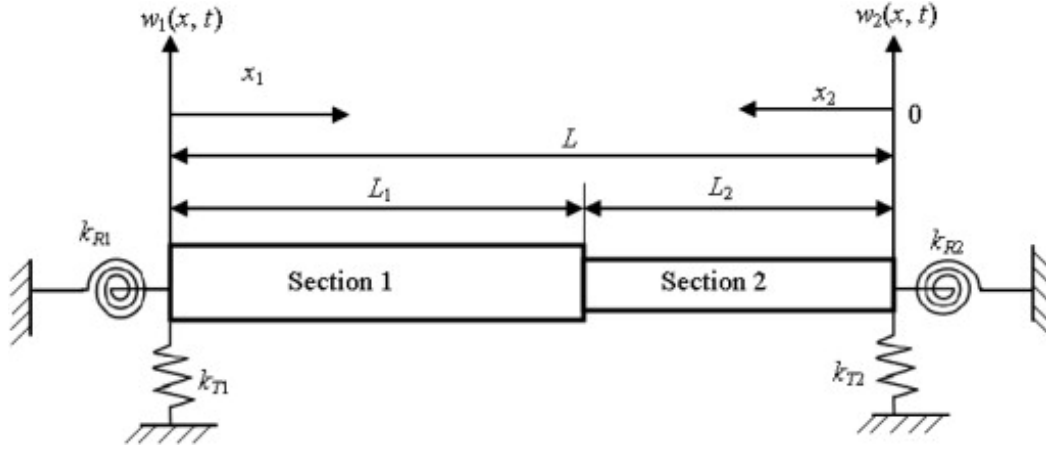


Figure 6.2: Coordinate system for the Euler Bernoulli stepped beam elastically restrained at both ends

$$m_{cm} = \beta m_c \quad (6.21)$$

$$E_{cm} I_{cm} = \alpha E_c I_c \quad (6.22)$$

where E is the Young’s modulus of the section, I is the second moment of area of the section and m is the mass per unit length of the section. The subscripts c and cm refer to the core and core-plus-modules (i.e. completed structure) respectively.

The ratio of the product of the mass and the stiffness of the two sections, i.e. the ratio of α and β , is also considered:

$$\mu = \frac{m_c}{m_{cm}} \frac{E_{cm} I_{cm}}{E_c I_c} = \frac{\alpha}{\beta} \quad (6.23)$$

The free vibration of an Euler-Bernoulli beam consisting of two uniform sections and elastically restrained at both ends as shown in Figure 6.2 is considered. The stepped beam consists of two sections each with their own system of reference, x_1 and x_2 . The partial differential equation which describes the free vibration of each section is shown in eq. 6.24 and 6.25.

$$\frac{\partial^4 w_1(x_1, t)}{\partial x_1^4} + \frac{\beta}{\alpha} \frac{m_c}{E_c I_c} \frac{\partial^2 w_1(x_1, t)}{\partial t^2} = 0, \quad x_1 \in [0, L_1] \quad (6.24)$$

$$\frac{\partial^4 w_2(x_2, t)}{\partial x_2^4} + \frac{m_c}{E_c I_c} \frac{\partial^2 w_2(x_2, t)}{\partial t^2} = 0, \quad x_2 \in [L_2, H] \quad (6.25)$$

where w_1 and w_2 are the displacements of the respective sections of the stepped beam, and t is time.

Following the method outlined by Mao and Pietrzko [242], by taking a modal analysis approach, $w_1(x_1, t)$ and $w_2(x_2, t)$, are separated into space and time components as shown in eq. 6.26 and 6.27:

$$w_1(x_1, t) = \Phi_1(x_1) e^{i\omega t} \quad (6.26)$$

$$w_2(x_2, t) = \Phi_2(x_2) e^{i\omega t} \quad (6.27)$$

where $\Phi_1(x_1)$, $\Phi_2(x_2)$ are the mode shape of each section, ω is the natural frequency of the relevant section and i denotes the imaginary unit such that $i = \sqrt{-1}$.

Eq. 6.26 and 6.27 can be substituted into eqs. 6.24 and 6.25. Separating for space and time, eqs. 6.28 and 6.29 are obtained.

$$\frac{d^4\Phi_1(x_1)}{dx_1^4} - \frac{m_{cm}\omega^2}{E_{cm}I_{cm}} \Phi_1(x_1) = 0 \quad (6.28)$$

$$\frac{d^4\Phi_2(x_2)}{dx_2^4} - \frac{m_c\omega^2}{E_cI_c} \Phi_2(x_2) = 0 \quad (6.29)$$

Rewriting eqs. 6.28 and 6.29 in dimensionless form to obtain eqs. 6.30 and 6.31:

$$\frac{d^4\Phi_1(X_1)}{dX_1^4} - \Omega_1^4 \Phi_1(X_1) = 0 \quad (6.30)$$

$$\frac{d^4\Phi_2(X_2)}{dX_2^4} - \Omega_2^4 \Phi_2(X_2) = 0 \quad (6.31)$$

where

$$X_1 = \frac{x_1}{L}, \quad X_2 = \frac{x_2}{L} \quad (6.32)$$

The dimensionless natural frequencies, Ω_1 and Ω_2 , of the stepped beam are defined in eqs. 6.33 and 6.34.

$$\Omega_1^4 = \frac{m_{cm} \omega^2 L^4}{E_{cm}I_{cm}} \quad (6.33)$$

$$\Omega_2^4 = \frac{m_c \omega^2 L^4}{E_cI_c} \quad (6.34)$$

where m, E, I are the mass, Young's Modulus and second moment of area of the respective sections denoted by subscripts cm for the modules and core and c for the core as previously. The natural frequency of the structure is ω and L is the combined length of both sections.

Applying Adomian Decomposition principles, $\Phi_1(X_1)$ and $\Phi_2(X_2)$ can be expressed as an infinite series as in eqs. 6.35 and 6.36.

$$\Phi_1(X_1) = \sum_{m=0}^{\infty} \Phi_1^{[m]}(X_1) \quad (6.35)$$

$$\Phi_2(X_2) = \sum_{m=0}^{\infty} \Phi_2^{[m]}(X_2) \quad (6.36)$$

where $\Phi_1^{[m]}(X_1)$ and $\Phi_2^{[m]}(X_2)$ can be solved recurrently.

According to Mao and Pietzrko [242], to solve eqs. 6.30 and 6.31, the linear operator $G = \frac{d^4}{dX^4}$ is imposed, with the inverse operator of G being a 4-fold integral:

$$G^{-1} = \iiint\int (...)dXdXdXdX \quad (6.37)$$

and

$$G^{-1}G(\Phi_1(X_1)) = \Phi_1(X_1) - \Phi_1(0) - \frac{d\Phi_1(0)}{dX_1}X_1 - \frac{d^2\Phi_1(0)}{dX_1^2}\frac{X_1^2}{2} - \frac{d^3\Phi_1(0)}{dX_1^3}\frac{X_1^3}{6} \quad (6.38)$$

$$G^{-1}G(\Phi_2(X_2)) = \Phi_2(X_2) - \Phi_2(0) - \frac{d\Phi_2(0)}{dX_2}X_2 - \frac{d^2\Phi_2(0)}{dX_2^2}\frac{X_2^2}{2} - \frac{d^3\Phi_2(0)}{dX_2^3}\frac{X_2^3}{6} \quad (6.39)$$

G^{-1} can then be taken of both sides of eqs. 6.35 and 6.36 to obtain eqs. 6.40 and 6.41.

$$G^{-1}G(\Phi_1(X_1)) = \Omega_1^4 G^{-1}(\Phi_1(X_1)) = \Omega_1^4 G^{-1}\left(\sum_{m=0}^{\infty} \Phi_1^{[m]}(X_1)\right) \quad (6.40)$$

$$G^{-1}G(\Phi_2(X_2)) = \Omega_2^4 G^{-1}(\Phi_2(X_2)) = \Omega_2^4 G^{-1}\left(\sum_{m=0}^{\infty} \Phi_2^{[m]}(X_2)\right) \quad (6.41)$$

Combining eqs. 6.38 and 6.39 with 6.40 and 6.41 respectively, eqs. 6.42 and 6.43 are obtained.

$$\Phi_1(X_1) = \Phi_1(0) + \frac{d\Phi_1(0)}{dX_1}X_1 + \frac{d^2\Phi_1(0)}{dX_1^2}\frac{X_1^2}{2} + \frac{d^3\Phi_1(0)}{dX_1^3}\frac{X_1^3}{6} + \Omega_1^4 G^{-1}\left(\sum_{m=0}^{\infty} \Phi_1^{[m]}(X_1)\right) \quad (6.42)$$

$$\Phi_2(X_2) = \Phi_2(0) + \frac{d\Phi_2(0)}{dX_2}X_2 + \frac{d^2\Phi_2(0)}{dX_2^2}\frac{X_2^2}{2} + \frac{d^3\Phi_2(0)}{dX_2^3}\frac{X_2^3}{6} + \Omega_2^4 G^{-1}\left(\sum_{m=0}^{\infty} \Phi_2^{[m]}(X_2)\right) \quad (6.43)$$

Mao and Pietrzko [242] then uses a recurrence relationship to find the approximate solution:

$$\Phi_1^{[0]}(X_1) = \Phi_1(0) + \frac{d\Phi_1(0)}{dX_1}X_1 + \frac{d^2\Phi_1(0)}{dX_1^2}\frac{X_1^2}{2} + \frac{d^3\Phi_1(0)}{dX_1^3}\frac{X_1^3}{6} \quad (6.44)$$

$$\Phi_1^{[m]}(X_1) = \Omega_1^4 G^{-1}(\Phi_1^{[m-1]}(X_1)), \quad m \geq 1 \quad (6.45)$$

$$\Phi_2^{[0]}(X_2) = \Phi_2(0) + \frac{d\Phi_2(0)}{dX_2}X_2 + \frac{d^2\Phi_2(0)}{dX_2^2}\frac{X_2^2}{2} + \frac{d^3\Phi_2(0)}{dX_2^3}\frac{X_2^3}{6} \quad (6.46)$$

$$\Phi_2^{[m]}(X_2) = \Omega_2^4 G^{-1}(\Phi_2^{[m-1]}(X_2)), \quad m \geq 1 \quad (6.47)$$

which can be approximated by a truncated series such that:

$$\Phi_1(X_1) = \lim_{M \rightarrow \infty} \Phi_1^{[M]}(X_1) = \sum_{m=0}^{M-1} \Phi_1^{[m]}(X_1) = \sum_{s=0}^3 \frac{d^s \Phi_1(0)}{dX^s} \sum_{m=0}^{M-1} \left[\Omega_1^4 \frac{X_1^{4m+n}}{(4m+n)!} \right] \quad (6.48)$$

$$\Phi_2(X_2) = \lim_{M \rightarrow \infty} \Phi_2^{[M]}(X_2) = \sum_{m=0}^{M-1} \Phi_2^{[m]}(X_2) = \sum_{s=0}^3 \frac{d^s \Phi_2(0)}{dX^s} \sum_{m=0}^{M-1} \left[\Omega_2^4 \frac{X_2^{4m+n}}{(4m+n)!} \right] \quad (6.49)$$

where the number of the series summation limit M is determined by convergence requirements and in this study is set to 10; no significant change in estimates were obtained when this value was increased.

The boundary conditions at the end of the stepped beam can be written as:

$$E_{cm} I_{cm} \frac{d^2 \Phi_1(x_1)}{dx_1^2} - k_{R,1} \frac{d\Phi_1(x_1)}{dx_1} = 0, \quad E_{cm} I_{cm} \frac{d^3 \Phi_1(x_1)}{dx_1^3} - k_{T,1} \Phi_1(x_1) = 0 \quad (6.50)$$

$$E_c I_c \frac{d^2 \Phi_2(x_2)}{dx_2^2} - k_{R,2} \frac{d\Phi_2(x_2)}{dx_2} = 0, \quad E_c I_c \frac{d^3 \Phi_2(x_2)}{dx_2^3} - k_{T,2} \Phi_2(x_2) = 0 \quad (6.51)$$

where $k_{R,1}$, $k_{R,2}$ are the stiffness of the rotational spring and $k_{T,1}$, $k_{T,2}$ are the stiffness of the translational spring at $x_1 = 0$ and $x_2 = L$ respectively.

In dimensionless form, eqs. 6.50 and 6.51 become:

$$\frac{d^2 \Phi_1(0)}{dX_1^2} - K_{R,1} \frac{d\Phi_1(0)}{dX_1} = 0, \quad \frac{d^3 \Phi_1(0)}{dX_1^3} - K_{T,1} \Phi_1(0) = 0 \quad (6.52)$$

$$\frac{d^2 \Phi_2(0)}{dX_2^2} - K_{R,2} \frac{d\Phi_2(0)}{dX_2} = 0, \quad \frac{d^3 \Phi_2(0)}{dX_2^3} - K_{T,2} \Phi_2(X_2) = 0 \quad (6.53)$$

where

$$K_{R,1} = \frac{k_{R,1} L}{E_{cm} I_{cm}}, \quad K_{T,1} = \frac{k_{T,1} L^3}{E_{cm} I_{cm}} \quad (6.54)$$

$$K_{R,2} = \frac{k_{R,2} L}{E_c I_c}, \quad K_{T,2} = \frac{k_{T,2} L^3}{E_c I_c} \quad (6.55)$$

By substituting eqs. 6.52 and 6.53 into eqs. 6.48 and 6.49 Mao and Pietzrko [242] obtains:

$$\begin{aligned} \Phi_1(X_1) = \Phi_1(0) & \left\{ \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{X_1^{4m}}{(4m)!} \right] - K_{T,1} \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{X_1^{4m+3}}{(4m+3)!} \right] \right\} + \\ \frac{d\Phi_1(0)}{dX_1} & \left\{ \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{X_1^{4m+1}}{(4m+1)!} \right] - K_{T,1} \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{X_1^{4m+2}}{(4m+2)!} \right] \right\} \end{aligned} \quad (6.56)$$

$$\begin{aligned} \Phi_2(X_2) = \Phi_2(0) & \left\{ \sum_{m=0}^{M-1} \left[\Omega_2^{4m} \frac{X_2^{4m}}{(4m)!} \right] - K_{T,2} \sum_{m=0}^{M-1} \left[\Omega_2^{4m} \frac{X_2^{4m+3}}{(4m+3)!} \right] \right\} + \\ \frac{d\Phi_2(0)}{dX_2} & \left\{ \sum_{m=0}^{M-1} \left[\Omega_2^{4m} \frac{X_2^{4m+1}}{(4m+1)!} \right] - K_{T,2} \sum_{m=0}^{M-1} \left[\Omega_2^{4m} \frac{X_2^{4m+2}}{(4m+2)!} \right] \right\} \end{aligned} \quad (6.57)$$

The stress concentration at the step location of the two sections of the beam is neglected and the continuity conditions in dimensionless form become:

$$\Phi_1(R_1) = \Phi_2(R_2), \quad \frac{d\Phi_1(R_1)}{dX_1} = -\frac{d\Phi_2(R_2)}{dX_2} \quad (6.58)$$

$$\frac{d^2\Phi_1(R_1)}{dX_1^2} = \frac{E_c I_c}{E_{cm} I_{cm}} \frac{d^2\Phi_2(R_2)}{dX_2^2}, \quad \frac{d^3\Phi_1(R_1)}{dX_1^3} = -\frac{E_c I_c}{E_{cm} I_{cm}} \frac{d^3\Phi_2(R_2)}{dX_2^3} \quad (6.59)$$

where $R_1 = \frac{L_1}{L}$ and $R_2 = \frac{L_2}{L}$ such that $R_1 + R_2 = 1$

Substituting eqs. 6.56 and 6.57 into eqs. 6.58 and 6.59 and writing in matrix formation:

$$\begin{bmatrix} D_1(R_1) & D_2(R_1) & -D_3(R_2) & -D_4(R_2) \\ \frac{dD_1(R_1)}{dX_1} & \frac{dD_2(R_1)}{dX_1} & \frac{dD_3(R_2)}{dX_2} & \frac{dD_4(R_2)}{dX_2} \\ \frac{d^2D_1(R_1)}{dX_1^2} & \frac{d^2D_2(R_1)}{dX_1^2} & -\frac{1}{\alpha} \frac{d^2D_3(R_2)}{dX_2^2} & -\frac{1}{\alpha} \frac{d^2D_4(R_2)}{dX_2^2} \\ \frac{d^3D_1(R_1)}{dX_1^3} & \frac{d^3D_2(R_1)}{dX_1^3} & \frac{1}{\alpha} \frac{d^3D_3(R_2)}{dX_2^3} & \frac{1}{\alpha} \frac{d^3D_4(R_2)}{dX_2^3} \end{bmatrix} \begin{bmatrix} \Phi_1(0) \\ \frac{d\Phi_1(0)}{dX_1} \\ \Phi_2(0) \\ \frac{d\Phi_2(0)}{dX_2} \end{bmatrix} = 0 \quad (6.60)$$

where:

$$D_1(R_1) = \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m}}{(4m)!} \right] - K_{T,1} \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m+3}}{(4m+3)!} \right], \quad (6.61)$$

$$D_2(R_1) = \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m+1}}{(4m+1)!} \right] + K_{R,1} \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m+2}}{(4m+2)!} \right], \quad (6.62)$$

$$D_3(R_2) = \sum_{m=0}^{M-1} \left[(\mu\Omega_1^4)^m \frac{R_2^{4m}}{(4m)!} \right] - K_{T,2} \sum_{m=0}^{M-1} \left[(\mu\Omega_1^4)^m \frac{R_2^{4m+3}}{(4m+3)!} \right], \quad (6.63)$$

$$D_4(R_2) = \sum_{m=0}^{M-1} \left[(\mu\Omega_1^4)^m \frac{R_2^{4m+1}}{(4m+1)!} \right] + K_{R,2} \sum_{m=0}^{M-1} \left[(\mu\Omega_1^4)^m \frac{R_2^{4m+2}}{(4m+2)!} \right], \quad (6.64)$$

Therefore, the dimensionless natural frequencies can be solved using eq. 6.65:

$$\det \begin{bmatrix} D_1(R_1) & D_2(R_1) & -D_3(R_2) & -D_4(R_2) \\ \frac{dD_1(R_1)}{dX_1} & \frac{dD_2(R_1)}{dX_1} & \frac{dD_3(R_2)}{dX_2} & \frac{dD_4(R_2)}{dX_2} \\ \frac{d^2D_1(R_1)}{dX_1^2} & \frac{d^2D_2(R_1)}{dX_1^2} & -\frac{1}{\alpha} \frac{d^2D_3(R_2)}{dX_2^2} & -\frac{1}{\alpha} \frac{d^2D_4(R_2)}{dX_2^2} \\ \frac{d^3D_1(R_1)}{dX_1^3} & \frac{d^3D_2(R_1)}{dX_1^3} & \frac{1}{\alpha} \frac{d^3D_3(R_2)}{dX_2^3} & \frac{1}{\alpha} \frac{d^3D_4(R_2)}{dX_2^3} \end{bmatrix} = 0 \quad (6.65)$$

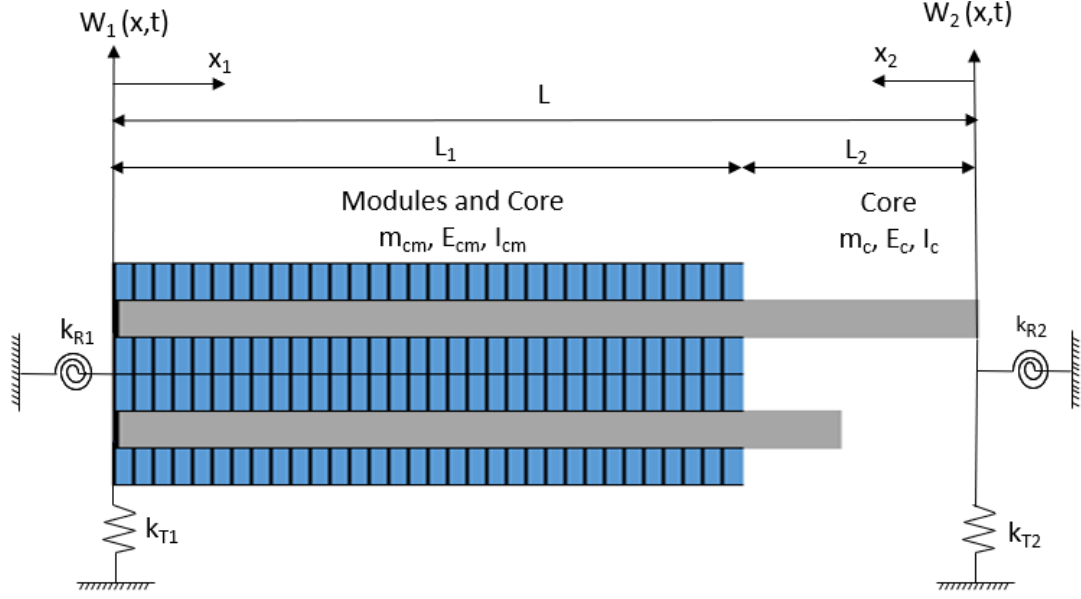


Figure 6.3: Partially completed modular structure represented as a stepped beam

For the purpose of this research, the modular structure is considered a cantilever stepped beam as shown in Figure 6.3, with fixed-free boundary conditions such that $k_{R,1}$ and $k_{T,1}$ are sufficiently large (1×10^9) and $k_{R,2}$ and $k_{T,2}$ are 0. Therefore:

$$K_{R,1} = 1 \times 10^9 \frac{L}{E_{cm} I_{cm}}, \quad K_{T,1} = 1 \times 10^9 \frac{L^3}{E_{cm} I_{cm}} \quad (6.66)$$

$$K_{R,2} = K_{T,2} = 0 \quad (6.67)$$

Thus, eqs. 6.61 to 6.64 can be written as:

$$D_1(R_1) = \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m}}{(4m)!} \right] - 1 \times 10^9 \frac{L^3}{E_{cm} I_{cm}} \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m+3}}{(4m+3)!} \right], \quad (6.68)$$

$$D_2(R_1) = \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m+1}}{(4m+1)!} \right] + 1 \times 10^9 \frac{L^3}{E_{cm} I_{cm}} \sum_{m=0}^{M-1} \left[\Omega_1^{4m} \frac{R_1^{4m+2}}{(4m+2)!} \right], \quad (6.69)$$

$$D_3(R_2) = \sum_{m=0}^{M-1} \left[(\mu\Omega_1^4)^m \frac{R_2^{4m}}{(4m)!} \right] \quad (6.70)$$

$$D_4(R_2) = \sum_{m=0}^{M-1} \left[(\mu\Omega_1^4)^m \frac{R_2^{4m+1}}{(4m+1)!} \right] \quad (6.71)$$

The Stepped Beam numerical model is applied to both of the case study structures, *Ten Degrees* and *College Road*, described in Chapter 4 to identify the stiffness contributions of the modules to the global stiffness of these novel high-rise modular structures. The stepped beam method as presented by Mao and Pietzrko [242] and described above is set up to calculate the natural frequency of the stepped beam. However, in this study, the natural frequency of the structure in the partially completed ‘stepped’ state and the completed state is known and it is the stiffness properties, $E_{cm}I_{cm}$, E_cI_c and the ratio of the two, that are the unknown parameters of interest. Therefore, one set of the stepped beam equations with the identified natural frequencies is generated for the partially complete structure and another set for the completed structure. These two sets of equations can then be solved simultaneously to identify the stiffness parameters of interest. The results are presented and discussed in Section 6.3.3.

6.3.3 Identified Bending Stiffness Contribution of Modules

6.3.3.1 George Street

For *Ten Degrees*, the completed modular structure above transfer level has $L = L1 = 126.5$ m and measured natural frequencies of 0.316 Hz in the x-direction and 0.371 Hz in the y-direction. The mass per unit length, m , of the structure in its completed state, was calculated as 263,738 kg/m using the FE model of *Ten Degrees*. These properties are used to establish the stepped beam equations for $E_{cm}I_{cm}$.

Another set of stepped beam equations was then established using the partially completed structure with 32 stories of modules installed. The properties used for this set of equations were $L = 126.5$ m, with $L1 = 92$ m and $L2 = 34.50$ m. The combined mass per unit length of the cores was calculated as 106,162 kg/m from the FE model. The mass per unit length of the section with modules and cores was

set as 263,738 kg/m as above. The corresponding natural frequencies identified for the structure with 32 storeys installed were 0.371 Hz in the x-direction and 0.417 Hz in the y-direction. These properties were then used to obtain a set of stepped beam equations for the partially completed structure with two sections of differing stiffness, $E_{cm}I_{cm}$ and E_cI_c .

The two sets of stepped beam equations were then solved simultaneously for both x and y directions to identify values for $E_{cm}I_{cm}$ and E_cI_c . The identified bending stiffness values for *Ten Degrees* are shown in Table 6.9.

Table 6.7: Identified mass per unit length found using the FE models *Ten Degrees*

m_{cm}	m_c	β
263738 m/kg	10612 m/kg	2.48

Table 6.8: Identified natural frequencies found using OMA of in situ acceleration of *Ten Degrees*

Storeys Installed	Natural Frequency X-X East-West	Natural Frequency Y-Y North-South
32	0.370 Hz	0.417 Hz
44	0.316 Hz	0.371 Hz

Table 6.9: Identified bending stiffness values of *Ten Degrees* found using the stepped beam numerical model and natural frequencies from modal identification

Direction	$E_{cm}I_{cm}$	E_cI_c	α
X-X			
East-West	$1.99 \times 10^{13} \text{ Nm}^2$	$7.98 \times 10^{12} \text{ Nm}^2$	2.5
Y-Y			
North-South	$3.80 \times 10^{13} \text{ Nm}^2$	$1.29 \times 10^{13} \text{ Nm}^2$	2.9

where $\beta = \frac{m_{cm}}{m_c}$ and $\alpha = \frac{E_{cm}I_{cm}}{E_cI_c}$ as in eqs. 6.21 and 6.22.

It can be seen from Table 6.9 that the section of the modular building with modules installed around the core is approximately 3 times as stiff as the freestanding cores. This suggests that the modules do add significant bending stiffness to a modular high-rise building and the assumption that the cores are the only source of lateral resistance is untrue.

6.3.3.2 College Road

The completed modular structure of *College Road* has 50 storeys with $L = L1 = 136.25$ m above the transfer slab and corresponding natural frequencies of 0.248 Hz in the x-direction and 0.349 Hz in the y-direction. The mass per unit length, m , of the structure in its completed state, was calculated as 240,695 kg/m using the FE model of *College Road*. These properties are used to establish the stepped beam equations for $E_{cm}I_{cm}$.

A second set of stepped beam equations were then established using the partially completed structure with 24 storeys of modules installed. The properties used for this set of equations were $L = 136.25$ m, with $L1 = 69$ m and $L2 = 67.25$ m. The combined mass per unit length of the cores was calculated as 119,097 kg/m from the FE model. The mass per unit length of the section with modules installed and cores was set as 240,695 kg/m. The corresponding natural frequencies identified for the structure with 24 storeys installed were 0.325 Hz in the x-direction and 0.379 Hz in the y-direction. These properties were then used to obtain a set of stepped beam equations for the partially completed structure with two sections of differing bending stiffness, $E_{cm}I_{cm}$ and E_cI_c .

As with *Ten Degrees*, the two sets of stepped beam equations were then solved simultaneously for both x and y directions to identify values for $E_{cm}I_{cm}$ and E_cI_c . The identified bending stiffness values for *Ten Degrees* are shown in Table 6.12.

Table 6.10: Identified mass per unit length found using the FE models *College Road*

m_{cm}	m_c	β
240,695 kg/m	119,097 kg/m	2.02

Table 6.11: Identified natural frequencies found using OMA of in situ acceleration of *College Road*

Storeys Installed	Natural Frequency X-X East-West	Natural Frequency Y-Y North-South
24	0.325 Hz	0.379 Hz
50	0.248 Hz	0.349 Hz

Table 6.12: Identified bending stiffness values of *College Road* found using the stepped beam numerical model and natural frequencies from modal identification

Direction	$E_{cm}I_{cm}$	E_cI_c	α
X-X			
East-West	$2.72 \times 10^{13} \text{ Nm}^2$	$1.69 \times 10^{13} \text{ Nm}^2$	1.6
Y-Y			
North-South	$1.05 \times 10^{14} \text{ Nm}^2$	$2.98 \times 10^{13} \text{ Nm}^2$	3.5

Similar to *Ten Degrees*, the section of *College Road* with modules installed is 3.5 times as stiff as the cores alone in the Y-Y direction. However, in the X-X direction the structure is only 1.5 times as stiff. This is expected as the identified natural frequency in the Y-Y direction is greater than the X-X direction. The higher bending stiffness in the Y-Y direction is likely due to the shape of the structure, with the width of the completed structure in the X-X direction being approximately 21 m long and the Y-Y direction being approximately 56 m long. For *Ten Degrees*, α was also seen to be greater in the Y-Y direction; the completed *Ten Degrees* is 44 m wide in the Y-Y direction compared to 41 m in the X-X direction. Validation of the identified bending stiffness values is presented in Appendix A.

6.4 Data-Informed Multi-Degree of Freedom Surrogate Model

As previously explored in Section 6.2, Finite Element models of modular structures are difficult to use, computationally expensive, proved to be inaccurate in the instance of the case study structures, and are ultimately unsuitable for performing dynamic analysis of modular buildings. Given the continued progression of modular construction to exceedingly new heights, accurate prediction of the natural frequency, dynamic response and controlled vibrations will require a more suitable model, focused on the reported inherent properties of modular buildings.

The primary focus of this thesis has been to identify inherent properties of modular buildings and suitable control methods. Through in situ testing and equivalent modelling, key inherent properties of modular buildings have been identified. Namely the damping ratio of this form of construction and the stiffness contribution

of modules to the overall dynamic response of a modular building. These properties are used to build a data-informed Multi Degree of Freedom (MDOF) surrogate model which better replicates the dynamic response of a modular building and allows for easier parametric analysis and investigation of suitable control methods.

The fundamental equation of motion in which forces at each of N Degree of Freedom (DOF) are equated as shown in eq. 6.72 forms the basis of the MDOF model.

$$[M]\{\ddot{x}\} + [K]\{\dot{x}\} + [C]\{x\} = F(t) \quad (6.72)$$

where $[M]$, $[K]$ and $[C]$ are $[N \times N]$ matrices of the relevant mass, stiffness and damping coefficients respectively at each DOF, $\{\ddot{x}\}$, $\{\dot{x}\}$ and $\{x\}$ are $[N \times t]$ vectors of the acceleration, velocity and displacements experienced at each DOF and $F(t)$ is an $[N \times t]$ vector of forces at each DOF, where t is the length of time in which a force is applied multiplied by the number of timesteps.

Each storey within the building represented by this model will have 3 DOF's. That is one vertical, one translational and one rotational about the top of the storey. Therefore, for i storeys, N will be equal to $3 \times i$ DOF's.

The mass matrix M is made up of the mass per storey entered into each DOF along the diagonal. Values for the mass per storey of the modular building are calculated from the finite element models and represent the mass of the combined core and modules. If the structure is partially complete, the mass values in the relevant DOF's will include the mass per storey of the core only. The mass values were obtained from the FE model of *Ten Degrees* and are presented in Table 6.13.

Table 6.13: Data-informed mass values

Mass of Core only, M_c	Mass of Modules and Core, M_{mc}
106162 kg/m	263738 kg/m

The stiffness matrix $[K]$ is made up of the combined local stiffness matrices for each storey. The arrangement of the local stiffness matrix is that of a fixed-fixed element and is shown in eq. 6.73. The bending stiffness, EI , is assigned based on whether the storey is fully complete or not. If the building being analysed is in a

state of partial completion, storeys fully completed will use the EI value for the modules and core combined, if the storey is yet to be completed, the EI value for the core alone is used. The EI values used are those which are reported in Section 6.3.3, identified using the numerical Stepped Beam Model.

$$K_{local} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & \frac{-EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{-12EI}{L^3} & \frac{-6EI}{L^2} \\ 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} & 0 & \frac{6EI}{L^2} & \frac{2EI}{L} \\ \frac{-EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & \frac{-12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{-6EI}{L^2} & \frac{2EI}{L} & 0 & \frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \quad (6.73)$$

The local stiffness matrices are then converted to global coordinates using the transfer matrix shown in eq. 6.74 and eq. 6.75. The storey stiffness matrix, K_{storey} is then entered into the global Stiffness matrix $[K]$ at the relevant DOF.

$$T = \begin{bmatrix} \cos\left(\frac{\pi}{2}\right) & \sin\left(\frac{\pi}{2}\right) & 0 & 0 & 0 \\ -\sin\left(\frac{\pi}{2}\right) & \cos\left(\frac{\pi}{2}\right) & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos\left(\frac{\pi}{2}\right) & \sin\left(\frac{\pi}{2}\right) & 0 \\ 0 & 0 & 0 & -\sin\left(\frac{\pi}{2}\right) & \cos\left(\frac{\pi}{2}\right) & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (6.74)$$

$$K_{storey} = T^T \cdot K_{local} \cdot T \quad (6.75)$$

The damping coefficient matrix, $[C]$, uses damping ratio and natural frequency values obtained from in situ monitoring outlined in Chapter 5 to obtain the Rayleigh Damping coefficients, α and β shown in eq. 6.76. The variable α is the mass related damping coefficient and β is the stiffness related damping coefficient. The value of α and β are found using eq. 6.77.

$$C = \alpha M + \beta K \quad (6.76)$$

$$\begin{bmatrix} \alpha \\ \beta \end{bmatrix} = \frac{1}{2} \begin{bmatrix} \frac{1}{\omega_1} & \omega_1 \\ \frac{1}{\omega_2} & \omega_2 \end{bmatrix} \times \begin{bmatrix} \xi_1 \\ \xi_2 \end{bmatrix}^{-1} \quad (6.77)$$

Therefore, it is possible to consider a wide range of damping ratios, including those identified from the full scale testing of *Ten Degrees* and *College Road* reported in Chapter 5.

The force matrix $F(t)$ can be assembled to allow for a wide range of forces to be applied to the structure. This can include impact loads at isolated storeys or time histories of forces including time histories of wind loads at each storey of the structure.

The natural frequencies and mode shapes of the structure modelled using the data-informed surrogate MDOF model are found by calculating the eigenvalues and eigenvectors such that:

$$[K]\Phi = \omega^2[M]\Phi \quad (6.78)$$

To calculate the dynamic response of the modular building and obtain the acceleration $\{\ddot{x}\}$, velocity $\{\dot{x}\}$ and displacement response $\{x\}$ of the structure modelled, the Newmark Beta method is used.

The model takes in a number of parameters to allow for analysis of a wide range of modular buildings and performance of parametric studies. These parameters include:

1. Building Height
2. Building Width
3. Building Length
4. Total Number of Storeys
5. Number of Storeys installed
6. Basic Wind Speed/Forces to be applied
7. Response direction

The adaptability of this data-informed surrogate model allows a range of modular buildings to be studied, enabling parametric assessment of their dynamic response. Moreover, the data-informed nature of this model, implementing identified inherent properties of the case study modular buildings, creates a surrogate model which is more fit for purpose for dynamic response assessment than traditional FE software which are designed for more traditional forms of modular buildings. This enhances the design process for modular high-rise buildings, improving understanding of their dynamic performance, and ultimately, ensuring adherence to habitability requirements.

6.4.1 Calibration of MDOF Surrogate Model

In order to calibrate the data-informed MDOF surrogate model, *Ten Degrees* and *College Road* are modelled and their natural frequencies estimated. Tables 6.14 and 6.16 show the parameters used in the MDOF surrogate model and the natural frequencies predicted using this data-informed model.

Table 6.14: Parameters used to predict the natural frequency of *Ten Degrees* using data-informed MDOF surrogate model

Parameter	Value
Building Height	126.5 m
Building Width	41m
Building Depth	44m
$E_{cm}I_{cm}$ (X-X)	$1.99 \times 10^{13} \text{ Nm}^2$
E_cI_c (X-X)	$7.98 \times 10^{12} \text{ Nm}^2$
$E_{cm}I_{cm}$ (Y-Y)	$3.80 \times 10^{13} \text{ Nm}^2$
E_cI_c (Y-Y)	$1.29 \times 10^{13} \text{ Nm}^2$
Total Number of Storeys	44 storeys
Number of Storeys Installed	44 storeys
Natural Frequency (X-X)	0.297 Hz
Natural Frequency (Y-Y)	0.410 Hz

Table 6.15: Parameters used to predict the natural frequency of *College Road* using data-informed MDOF surrogate model

Parameter	Value
Building Height	136.25 m
Building Width	21m
Building Depth	56m
$E_{cm}I_{cm}$ (X-X)	2.72×10^{13} Nm ²
E_cI_c (X-X)	1.69×10^{13} Nm ²
$E_{cm}I_{cm}$ (Y-Y)	1.05×10^{14} Nm ²
E_cI_c (Y-Y)	2.98×10^{13} Nm ²
Total Number of Storeys	50 storeys
Number of Storeys Installed	50 storeys
Natural Frequency (X-X)	0.248 Hz
Natural Frequency (Y-Y)	0.349 Hz

Table 6.16: Parameters used to predict the natural frequency of *College Road* using data-informed MDOF surrogate model

It is apparent that the data-informed MDOF surrogate model provides accurate estimates of the natural frequency of a modular building when compared to the actual natural frequencies of *Ten Degree* and *College Road* identified through applying OMA methods to in situ measurements of accelerations of the two buildings. These values are compared to those from the FE models of the two structures in Tables 6.17 and 6.18.

The natural frequencies predicted using the MDOF surrogate model are much closer to the actual values of the two case study structures than those predicted using FE models of the structure. This is likely a result of the data-informed model being developed purposefully for modular buildings, with the inherent properties of modular buildings, such as the mass, stiffness and damping ratios identified through in situ tests. Furthermore, the MDOF surrogate model is adaptable, easy to use, takes seconds to run and is not computationally expensive. Therefore, the data-informed MDOF surrogate model is an improved method for assessing the dynamic response of high-rise modular buildings.

Table 6.17: Natural Frequencies of *Ten Degrees* estimated using different models and in situ testing

Method	Natural Frequency X-X Direction (Hz)	Natural Frequency Y-Y Direction (Hz)
FE Model	0.18	0.19
Field Measurements	0.314	0.370
Data-Informed MDOF Surrogate Model	0.297	0.410

Table 6.18: Natural Frequencies of *College Road* estimated using different models and in situ testing

Method	Natural Frequency X-X Direction (Hz)	Natural Frequency Y-Y Direction (Hz)
FE Model	0.18	0.18
Field Measurements	0.248	0.349
Data-Informed MDOF Surrogate Model	0.272	0.376

6.5 Conclusions

This chapter considered the variety of ways in which modular buildings can be modelled, with a particular focus on modelling their dynamic response. Finite Element models of modular buildings reported in literature were discussed, alongside their limitations. Results from dynamic analysis of the two modular high-rise case study structures were reviewed and compared to the natural frequencies of the modular buildings identified through in situ testing in Chapter 5. The FE models of the case study structures were found to be incredibly cumbersome to work with and to significantly underestimate the actual natural frequencies of the structures. The need for a more suitable dynamic model of modular buildings which accurately captured their dynamic properties and was less computationally expensive drove the motivation behind the development of surrogate models.

One such model was an alternative macro-module element, developed to

capture the behaviour of an individual module with one single element. It was found that an element could be developed to reflect the bending stiffness and hence, natural frequency of an FE model of a module. Substituting the macro module elements for each module in the full-scale FE model of *Ten Degrees*, the model size, usability and computational expense was drastically reduced. However, the natural frequencies remained grossly underestimated. This is likely because the macro module element was designed to reflect the bending stiffness from the FE model of a module as opposed to the actual stiffness of a modular building.

Therefore, a stepped beam numerical model as proposed by Mao and Pietrzko [242] was employed alongside the identified natural frequencies of the case study modular buildings in their different states of completion in order to quantify the bending stiffness contribution of the modules to the global stiffness of the structures. It was found that the RC cores with modules stacked around them were up to 3 times stiffer than the freestanding RC cores. This discredits the overly conservative assumption that RC cores are the only form of lateral load resistance in modular buildings.

A data-informed MDOF surrogate model was created using the identified inherent properties of the case study structures. These properties include the mass, bending stiffness and damping ratios of the structures. This surrogate model is adaptable, easy to use, less computationally expensive and upon calibration, was found to be accurate in capturing the dynamic characteristics of the case study structures. The MDOF surrogate model enables quicker, easier and more accurate assessment of the dynamic response of modular buildings than traditional FE models, enhancing the design process, supporting parametric analysis and facilitating assessment of conformity to habitability requirements.

This chapter has made significant contribution to research on modelling high-rise modular buildings. To the best of the author's knowledge, this is the first time the bending stiffness of a full-scale modular building has been assessed and quantified using field measurements. Furthermore, through creation of a data-informed MDOF surrogate model, this work has created an opportunity to assess suitable control strategies for modular structures as they continue to reach new heights.

Chapter 7

Assessment, Validation and Testing of Suitable Control Methods

7.1 Introduction

In the previous chapter, a data-informed MDOF surrogate model which utilises inherent properties of modular structures identified through in situ testing and numerical modelling was presented. This model incorporates mass, stiffness and damping ratio values typical of *Ten Degrees* and *College Road* to estimate the dynamic response of high-rise modular buildings. During the design process of *Ten Degrees*, uncertainty in the dynamic behaviour of the structure due to the novelty of this form of construction and the lack of empirical in situ measurements, led to concerns that the wind-induced acceleration response of the structure would not satisfy habitability requirements. Furthermore, the stiffness contribution of the modules was unknown and assumed to be negligible, resulting in the FE models of *Ten Degrees* greatly underestimating the natural frequency of the structure. This led the designers to consider suitable methods for controlling potentially excessive vibrations. Ultimately, through in situ testing, the real dynamic behaviour of *Ten Degrees* was captured and the designers decided that no additional control methods were necessary. However, as modular structures continue to increase in height and for the future progress of this novel form of construction, it is imperative to assess

suitable methods to control the wind-induced vibration response.

As was explored in Section 3 including a Tuned Liquid Damper (TLD) in a modular building proved to be a more efficient and economical method for controlling the wind-induced vibration response of archetype structures than increasing the size of the core walls. The auxiliary damping provided by a TLD was shown to be an effective method of supplementing the damping of the archetype structures which did not meet habitability requirements. Therefore, the inclusion of TLDs in high-rise modular buildings is further investigated in this chapter.

Equipped with the presented data-informed MDOF surrogate model, this chapter presents an assessment of the dynamic response of modular buildings of varying height, and the ability to reduce this response using different configurations of TLDs. In particular the concept of Multiple Vertically Distributed Tuned Liquid Dampers (MVDTLs) is explored. The repeatable nature of the individual units in modular buildings is well suited to distributing the TLDs throughout the structure, including vertically over a number of storeys, rather than occupying significant space at the top storey of the structure as is conventionally done.

Within this chapter, a general description of TLDs and the equations which are used to model their behaviour is provided. The equivalent Tuned Mass Damper (TMD) model of a TLD, as proposed by Tait [155] is discussed in detail and the incorporation of this model in the MDOF surrogate model using methods described by Moon [183] and Elias et al. [196] are presented. This model was validated through laboratory tests on a prototype full-scale TLD carried out in the Structural Engineering Laboratory at Trinity College Dublin. Details of the laboratory testing and validation of the model are provided. The validated MDOF surrogate model with equivalent TMDs was then used to perform a parametric analysis of a modular building with a number of MVDTL configurations to assesses the suitability of MVDTLs for reducing the wind-induced response of modular buildings. Finally, fragility curves were developed which highlight the important role supplementary damping devices, in this case MVDTLs, can play in improving the reliability of modular high-rise buildings conforming to habitability requirements. Furthermore, the fragility curves can offer design guidance to Structural Engineers designing modular buildings.

7.2 Modelling of a Tuned Liquid Damper

Tuned Liquid Dampers are an effective, reliable and easy to implement supplementary passive control method often added to a structure to control wind-induced vibrations [166]. TLDs consist of a tank partially filled with a liquid, often water, which will slosh when a structure begins to move. The sloshing of the liquid results in energy dissipation which increases the damping of the structure in a similar way to a Tuned Mass Damper (TMD) [195]. TLDs can come in many shapes, including rectangular, square or circular, and dimensions are chosen to attain a specified mass ratio, natural frequency and level of supplemental damping. In some TLDs, flow damping devices such as screens or baffles are included in the tank to add damping and alter the natural frequency of the sloshing motion. For the purpose of simplicity within this study, rectangular TLDs with no flow damping devices are considered. TLDs have a number of advantages over TMDs, including ease of installation and maintenance, easier tuning and being more cost-effective [7, 16, 157, 198]. For this reason, the ability of TLDs to control wind-induced vibrations in high-rise modular structures is investigated as part of this research.

7.2.1 Equations Governing Behaviour of a TLD

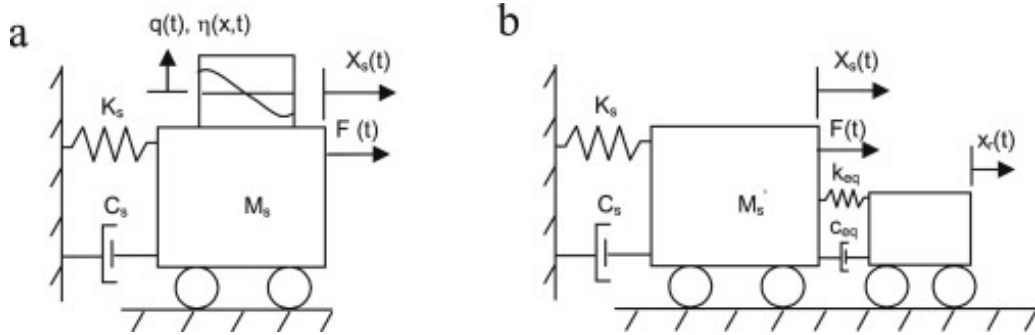


Figure 7.1: (a) Structure-TLD model and (b) linearised structure TLD model (Source: Tait [155])

Figure 7.1 shows (a) a typical structure-TLD system and (b) a linearised equivalent mechanical representation for a single degree of freedom system. In these models the mass, stiffness and damping of the structure are represented by M_s , K_s and C_s respectively. The external force on the structure is $F(t)$ and the response of the structure is $X_s(t)$. The equivalent TLD properties are m_{eq} , k_{eq} , c_{eq} and $x_r(t)$.

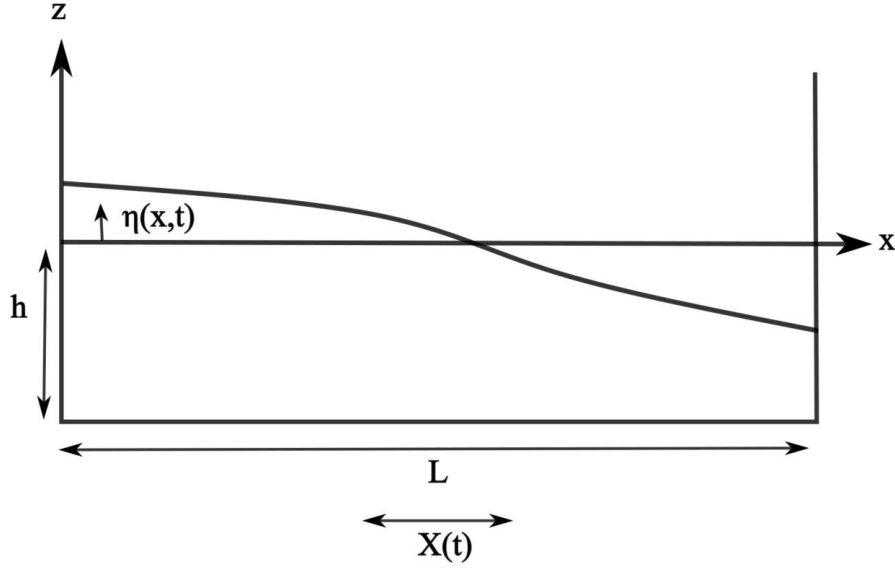


Figure 7.2: Liquid sloshing in a TLD

Considering the mechanical model of a rectangular TLD as shown in more detail in Figure 7.2, the liquid sloshing motion of a TLD can be formulated using potential flow theory as described by Sun and Fujino [204] and Tait [155]. From Figure 7.2, L is the length of the TLD, h is the quiescent fluid depth and b is the width of the tank. The fluid response amplitude is denoted by $\eta(x, t)$ and is assumed to be small compared to fluid depth. Inviscid, incompressible, irrotational flow and negligible surface tension are assumed. Based on shallow water wave theory, a gradient of the velocity potential, $\Phi(x, z, t)$ is used to represent the velocity of a liquid particle relative to the tank. Therefore, the kinematic continuity of incompressible flow becomes:

$$\frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial z^2} = 0 \quad (7.1)$$

with kinematic boundary conditions:

$$u(x, z, t) \big|_{x=0, x=L} = \frac{\partial \Phi}{\partial x} \big|_{x=0, x=L} = 0 \quad (7.2)$$

$$w(x, z, t) \big|_{z=-h} = \frac{\partial \Phi}{\partial z} \big|_{z=-h} = 0 \quad (7.3)$$

and the linearised boundary condition at the free surface, η is:

$$\frac{\partial \Phi}{\partial z} \Big|_{z=0} = \frac{\partial \eta}{\partial t} \quad (7.4)$$

Therefore, velocity potential can be rewritten to satisfy these boundary conditions as:

$$\Phi(x, z, t) = \sum_{n=1}^{\infty} \dot{q}_n(t) \cos\left(\frac{n\pi x}{L}\right) \frac{\cosh\left[\frac{n\pi(z+h)}{L}\right]}{\left(\frac{n\pi}{L}\right) \sinh\left(\frac{n\pi h}{L}\right)} \quad (7.5)$$

and the free surface can be taken as

$$\eta(x, t) = \sum_{n=1}^{\infty} q_n(t) \cos\left(\frac{n\pi x}{L}\right) \quad (7.6)$$

The kinematic energy of the liquid sloshing can therefore be completely described by the set of $q_n(t)$ for $n = 1, 2, 3, \dots$. Gravitational potential can be used to express the energy of the system of standing waves such that

$$V = \frac{1}{2} \rho b g \int_0^L \eta^2(x, t) dx \quad (7.7)$$

with kinematic potential

$$T = \frac{1}{2} \rho b \int_{-h}^0 \int_0^L \left[\left(\dot{X} + \frac{\partial \Phi}{\partial x} \right)^2 + \left(\frac{\partial \Phi}{\partial z} \right)^2 \right] dx dz \quad (7.8)$$

where ρ is the density of the liquid, b is the tank width, g is the acceleration due to gravity and $\dot{X}$ is the horizontal velocity of the tank.

These equations can be reformulated into the corresponding Lagrange equations to have the form

$$\frac{d}{dt} \left(\frac{\partial T}{\partial \dot{q}_n} \right) - \frac{\partial T}{\partial q_n} + \frac{\partial V}{\partial q_n} = Q_n, \quad n = 1, 2, 3, \dots \quad (7.9)$$

where Q_n are the non-conservative forces. The equation of motion can then be expressed as

$$m_n^* \ddot{q}_n(t) + c_n^* \dot{q}_n(t) + m_n^* \omega_n^2 q_n(t) = \gamma_n^* \ddot{X}(t), \quad n = 1, 2, 3, \dots \quad (7.10)$$

where m_n^* is the generalised mass such that

$$m_n^* = \frac{1}{2} \frac{\rho b L^2}{n \pi \tanh\left(\frac{n \pi h}{L}\right)} \quad (7.11)$$

ω_n^2 is the natural frequency of the n^{th} sloshing mode such that

$$\omega_n^2 = \frac{n \pi g}{L} \tanh\left(\frac{n \pi h}{L}\right) \quad (7.12)$$

and γ_n^* is the excitation factor, expressed as

$$\gamma_n^* = \rho b L^2 \frac{(1 - \cos(n \pi))}{(n \pi)^2} \quad (7.13)$$

Upon solution of $q_n(t)$ using eq. 7.10, eqs. 7.5 and 7.6 can be solved for the velocity potential, ϕ and the free surface η . Additionally, the fluid pressure $p(x, z, t)$ can be computed from the linearised Bernoulli equation using

$$p(x, z, t) = -\rho g z - \rho \frac{\partial \Phi}{\partial t} - \rho \left(x - \frac{L}{2} \right) \ddot{X}(t) \quad (7.14)$$

Through integrating the pressure force acting on the tank walls, the horizontal component of total sloshing force, F_s can be obtained

$$F_s(t) = b \int_{-h}^0 ([p]_{x=L} - [p]_{x=0}) dz \quad (7.15)$$

and substituting eqs. 7.5 and 7.14 into 7.15,

$$F_s(t) = \sum_{n=1}^{\infty} \gamma_n \ddot{q}_n(t) - m_0 \ddot{X}(t) \quad (7.16)$$

where m_0 is the total mass of the liquid. Of these equations, eqs. 7.10 and 7.16 are the most important for the analysis of the structure-TLD interaction. However, as it is assumed that the liquid in the tank behaves as an ideal fluid, the damping term is not included in eq. 7.10. In reality, sloshing energy is dissipated at the interface of the tank and liquid due to the effect of viscous shear. Therefore, the damping ratio, ζ_n of the n^{th} sloshing mode of a TLD can be calculated as

$$\zeta_n = \frac{1 + h/b}{2h} \sqrt{\frac{2\nu}{\omega_n}} \quad (7.17)$$

where ν is the kinematic viscosity of the liquid.

Considering the nonlinear motion of sloshing liquid in TLDs, the design of a TLD using the above equations can be challenging [184, 186]. For this reason, more straightforward models of TLDs which relate the behaviour of a TLD to an equivalent TMD have been developed [155, 161, 184, 186, 248, 249]. These equivalent TMD models allow for efficient design and understanding of the behaviour of TLDs [155, 184]. In particular, the equivalent TMD model as proposed by Tait [155] has been validated and used extensively in preliminary design of TLDs [156, 174, 182, 207, 249, 250]. Therefore, the equivalent TMD model is used within this research to parametrically assesses the performance of MVD TLDs in controlling the wind-induced response of tall modular buildings.

7.2.2 Equivalent Tuned Mass Damper Model

As discussed, modelling of a TLD as an equivalent TMD has been proposed by Tait [155] and demonstrated to be in good agreement with experimental testing of TLDs [182, 207, 249, 250]. Using an equivalent model for a TLD allows for TMD design theory to be applied to TLDs, aiding in preliminary design of structure-TLD systems [173].

The equivalent TMD model proposed by Tait [155] uses equivalent natural frequency, ω_{eq} , mass, m_{eq} , stiffness, k_{eq} and damping ratio ζ_{eq} values to model the behaviour of the TLD. These values are calculated based on the tank length, L , width, b , liquid height, h , liquid density, ρ and the kinematic viscosity of the fluid, ν , as shown in equations 7.18 - 7.22.

$$\omega_{eq} = \frac{\pi g}{L} \tanh\left(\frac{\pi h}{L}\right) \quad (7.18)$$

$$m_{eq} = \frac{8\rho b L^2}{\pi^3} \tanh\left(\frac{\pi h}{L}\right) \quad (7.19)$$

$$k_{eq} = \frac{8\rho b L g}{\pi^2} \tanh^2\left(\frac{\pi h}{L}\right) \quad (7.20)$$

$$\zeta_{eq} = \frac{1}{2h} \sqrt{\frac{\nu}{2\omega_n}} \left(1 + \frac{2h}{b} + SC\right) \quad (7.21)$$

$$c_{eq} = \zeta_{eq} 2m_{eq}\omega_{eq} \quad (7.22)$$

where SC is a surface contamination factor which is taken as 1 [155].

To maximise the performance of the TLD, the mass ratio, μ , of the TLD equivalent mass, m_{eq} to the mass of the structure M_s is defined as in eq. 7.23:

$$\mu = \frac{m_{eq}}{M_{structure} + \rho b h L - m_{eq}} \quad (7.23)$$

The value of the mass ratio μ typically lies between 0.5% and 5% [161, 171, 186], with a larger value providing more supplemental damping. The relationship between damping ratio, ζ and the mass ratio μ is shown in eq. 7.24.

$$\zeta_{eff} = \frac{1}{4} \sqrt{\frac{\mu + \mu^2}{1 + \frac{3\mu}{4}}} \quad (7.24)$$

The size of the TLD can be chosen in order to maximise μ or to achieve the desired level of supplemental damping. However, in some cases, such as with modular structures, the size of the TLD may be limited due to size constraints and in this case multiple TLDs may be used.

It is important that the TLDs have a similar natural frequency to the structure. This is so that the TLDs are activated when the structure begins to respond dynamically to environmental loading, and so that the motion of the liquid is tuned to that of the structure. The tuning ratio, Ω , is the ratio of the natural frequency of the TLD, ω_{eq} to the natural frequency of the structure, ω_s . To optimise the performance of a TLD, this value is chosen as close to 1 as possible [9, 155, 197].

$$\Omega = \frac{\omega_{eq}}{\omega_s} \quad (7.25)$$

The equivalent values, mass ratio and tuning ratio outlined above can be used in the method described in the following section, alongside the data-informed MDOF surrogate model described in Section 6.4 to assess the dynamic response of modular buildings equipped with MVDTLs.

7.3 Representing MVDTLs in the Data-Informed MDOF Surrogate Model

Rather than occupying a substantial amount of space at the top storey of a modular structure, it is possible to install multiple small TLDs distributed throughout modules and over multiple storeys of the structure, also known as Multiple Vertically Distributed Tuned Liquid Dampers (MVDTLs). Figure 7.3 shows how a number of damping devices can be distributed vertically over a structure. This

concept distributes the space requirements of supplementary damping devices and could be easily included in the off-site manufacturing of volumetric modules. It is possible that there could be a single module stacked with multiple TLDs on each of the upper storeys of a modular building or it could be the case that every module in these storeys includes a small TLD. MVDTLDS capitalise on the flexibility and repeatability of modular construction, offering a solution to wind-induced vibration control which complements the nature of this novel form of construction.

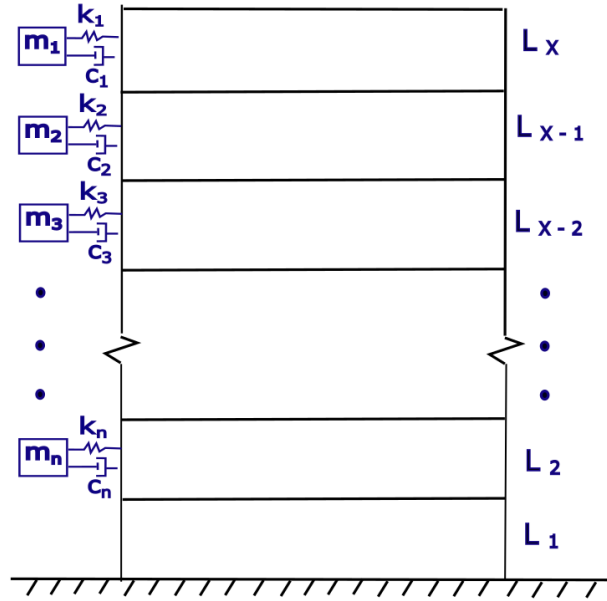


Figure 7.3: Multiple damping devices distributed vertically over the height of a structure

To incorporate MVDTLDS in the data-informed MDOF surrogate model, the equivalent TMD model can be employed. The equivalent TMD properties of the TLDs can be used to represent the TLDs at the relevant DOFs within the MDOF surrogate model. The MDOF surrogate model has been developed to accommodate any number of TLDs added to any storey over the height of the structure. This allows for parametric assessment of MVDTLDS and their ability to control wind induced vibrations. The methodology proposed by Elias et al. [197] and Moon [183] for incorporating vertically distributed TMDs has been used to incorporate the MVDTLDS in the data-informed MDOF surrogate model.

For an i storey structure there are $N = i \times 3$ DOFs, as described in Section 6.4, where each storey is represented by 3 DOFs, one in the axial, one in the translational and one in the rotational directions. For each of i storeys, there can be any number

7.3. Representing MVDTLs in the Data-Informed MDOF Surrogate Model

of TLDs installed. The number of storeys in which TLDs are installed is represented by j and this relates to n DOFs such that $n = j \times 3$ storeys, accounting for the 3 DOFs used to represent each storey. The mass, stiffness and damping matrices of the data-informed MDOF surrogate model with MVDTLs can be represented as shown in eqn's. 7.26 - 7.28.

$$[M] = \begin{bmatrix} M_1 & 0 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ 0 & M_2 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ 0 & 0 & M_3 & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & \dots & M_{N-1} & 0 & 0 & 0 & \dots & 0 \\ 0 & 0 & 0 & \dots & 0 & M_N & 0 & 0 & \dots & 0 \\ 0 & 0 & 0 & \dots & 0 & 0 & m_1 & 0 & \dots & 0 \\ 0 & 0 & 0 & \dots & 0 & 0 & 0 & m_2 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & m_n \end{bmatrix} \quad (7.26)$$

$$[K] = \begin{bmatrix} K_1 + K_2 + k_n & -K_2 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & -k_n \\ -K_2 & K_2 + K_3 + k_{n-1} & -K_3 & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ 0 & -K_3 & K_3 + K_4 + k_{n-2} & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & \dots & K_{N-1} + K_N + k_2 & -K_N & 0 & -k_2 & \dots & 0 \\ 0 & 0 & 0 & \dots & -K_N & K_N + k_1 & -k_1 & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \dots & \vdots & -k_1 & k_1 & 0 & \dots & 0 \\ 0 & 0 & 0 & \dots & -k_2 & 0 & 0 & k_2 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ -k_n & 0 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & k_n \end{bmatrix} \quad (7.27)$$

$$[C] = \begin{bmatrix} C_1 + C_2 + c_n & -C_2 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & -c_n \\ -C_2 & C_2 + C_3 + c_{n-1} & -C_3 & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ 0 & -C_3 & C_3 + C_4 + c_{n-2} & \dots & 0 & 0 & 0 & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & \dots & C_{N-1} + C_N + c_2 & -C_N & 0 & -c_2 & \dots & 0 \\ 0 & 0 & 0 & \dots & -C_N & C_N + c_1 & -c_1 & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \dots & \vdots & -c_1 & c_1 & 0 & \dots & 0 \\ 0 & 0 & 0 & \dots & -c_2 & 0 & 0 & c_2 & \dots & 0 \\ \vdots & \vdots & \vdots & \ddots & \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ -c_n & 0 & 0 & \dots & 0 & 0 & 0 & 0 & \dots & c_n \end{bmatrix} \quad (7.28)$$

where M_N, K_N and C_N represent the number of DOFs of the structure and m_n, k_n and c_n represent the number of DOFs the TLDs are installed over.

It is worth noting that as three DOFs are considered for each storey as previously mentioned, $M_1 = M_2 = M_3$ each represent the mass of the first storey. Similarly, K_1 is the stiffness in the vertical DOF of the first storey, K_2 is the stiffness of the first storey in the translational DOF and K_3 is the stiffness of the DOF representing rotation at the top of the first storey. The same DOFs also apply for C_1, C_2 and C_3 . As the TLDs are only investigated for control in one DOF at a time, only the DOFs relating to the horizontal displacement of storeys with TLDs will have non-zero values for c_n and k_n . The mass values for each corresponding storey with a TLD will be equal at each DOF, i.e. $m_1 = m_2 = m_3$. This is to avoid a determinant of zero within the mass matrix.

The mass, stiffness and damping matrices for the combined structure-TLD system can be expressed as a combination of the matrices for the structure provided in Section 6.4 and the equivalent TMD model in eqs. 7.26, 7.27 and 7.28. The combined matrices, along with the corresponding displacement, velocity and acceleration vectors are shown in eqns. 7.29, 7.30 and 7.31.

$$[M]\{\ddot{x}\} = \begin{bmatrix} [M_N]_{N \times N} & [0]_{N \times n} \\ [0]_{n \times N} & [0]_{n \times n} \end{bmatrix} \begin{Bmatrix} \{\ddot{X}_N\}_{N \times t} \\ \{0\}_{n \times t} \end{Bmatrix} + \begin{bmatrix} [0]_{N \times N} & [0]_{N \times n} \\ [0]_{n \times N} & [m_n]_{n \times n} \end{bmatrix} \begin{Bmatrix} \{0\}_{N \times t} \\ \{\ddot{x}_n\}_{n \times t} \end{Bmatrix} \quad (7.29)$$

$$[K]\{x\} = \begin{bmatrix} [K_N]_{N \times N} & [0]_{N \times n} \\ [0]_{n \times N} & [0]_{n \times n} \end{bmatrix} \begin{Bmatrix} \{X_N\}_{N \times t} \\ \{0\}_{n \times t} \end{Bmatrix} + \begin{bmatrix} [k_n]_{N \times N} & -[k_n]_{N \times n} \\ -[k_n]_{n \times N} & [k_n]_{n \times n} \end{bmatrix} \begin{Bmatrix} \{X_N\}_{N \times t} \\ \{x_n\}_{n \times t} \end{Bmatrix} \quad (7.30)$$

$$[C]\{\dot{x}\} = \begin{bmatrix} [C_N]_{N \times N} & [0]_{N \times n} \\ [0]_{n \times N} & [0]_{n \times n} \end{bmatrix} \begin{Bmatrix} \{\dot{X}_N\}_{N \times t} \\ \{0\}_{n \times t} \end{Bmatrix} + \begin{bmatrix} [c_n]_{N \times N} & -[c_n]_{N \times n} \\ -[c_n]_{n \times N} & [c_n]_{n \times n} \end{bmatrix} \begin{Bmatrix} \{\dot{X}_N\}_{N \times t} \\ \{\dot{x}_n\}_{n \times t} \end{Bmatrix} \quad (7.31)$$

The Force for the combined structure-TLD model then becomes:

$$[F] = \begin{bmatrix} [F(t)]_{N \times t} \\ [0]_{n \times 1} \end{bmatrix} \quad (7.32)$$

where t is the number of time steps.

The formulation of these matrices can be adapted to incorporate any number of TLDs which can be vertically or horizontally distributed across the structure, or a combination of both. In the case of horizontal distribution the m_{eq} , k_{eq} and c_{eq} of each damper is summed and lumped at the relevant DOF of the storey in the $[M]$, $[K]$ and $[C]$ matrices. For vertical distribution, the m_{eq} , k_{eq} and c_{eq} of each damper is applied directly at the relevant DOF of that storey in the $[M]$, $[K]$ and $[C]$ matrices.

The formulated matrices can be subbed into eq. 6.72 and solved using the Newmark Beta method as described in Section 6.4 to obtain a time series of displacements, velocities and accelerations.

The adaptability of this model allows for a wide range of parametric analyses to be performed to assess different vertical distributions of TLDs and their effectiveness in controlling wind-induced vibrations.

7.4 Validating the equivalent TMD Model of a TLD

Equivalent TMD models have been used widely to represent TLDs and perform preliminary analysis of the control of the dynamic response of structures [155, 156, 161, 186, 207]. Where experimental work has been carried out to validate the equivalent models, good comparison between the model and experimental tests has been shown [155, 186, 207]. To evaluate the specific methods used in this parametric study, experimental validation of the theoretical TLD, represented using the equivalent TMD model proposed by Tait [155], and incorporated in the data-informed MDOF surrogate model was undertaken. The validation studies examined a set of archetype structures subjected to harmonic and wind loading to compare the theoretical and experimental response of a prototype TLD. This was to assure confidence in the dynamic model including a theoretical TLD prior to performing parametric studies.

7.4.1 Laboratory Setup

The structural engineering laboratory facilities in Trinity College Dublin were used to perform the experimental testing for validation of the equivalent TMD model of a TLD. This was achieved by using the MTS real-time hybrid test system. The experimental hardware consists of a Series 111 MTS Accumulator and a 150 kN capacity high speed linear hydraulic actuator with a 250 mm (± 125 mm) stroke. The computer hardware comprises a Simulation Host PC, a Real-time Target PC, a Test PC and a two channel MTS 493 Real-time Controller. The Structural Test System (STS) software provides the user interface for PID (Proportional Integral Derivative) control and calibration of the actuator. More details on Real Time Hybrid Testing (RTHT) are presented in Chapter 8, however, for validation purposes, the full-scale experimental TLD is decoupled from the numerical model of the modular building and only the force response of the experimental TLD to a time series of displacement commands is measured using this system.



Figure 7.4: Full-scale TLD tested in the MTS real-time hybrid test system at Trinity College Dublin, Ireland

Figure 7.4 shows the laboratory setup, including the full-scale experimental TLD which was used for validation. The setup consists of the data-acquisition system, the hybrid testing frame and a MTS 493 hydraulic actuator with a load cell and internal LVDT. The actuator is connected to the left wall of the full-scale TLD and the large internal reaction frame. The TLD is suspended using 4 steel suspension cables attached to the top of the reaction frame to eliminate resistance whilst the TLD is moving. The TLD is a rectangular tank which is 1.93 m in length, L , 0.59 m in width, b , and 1.2 m tall. The depth of water, h of the tank is altered to match the natural frequency of each archetype modular building listed in Table 7.1. The dimensions of the TLD are not scaled and would be suitable for use in the full-scale archetype modular buildings investigated.

To perform the tests on the TLD, a time series of displacements is passed to the MTS controller via the Test PC using the STS software. The MTS controller then controls the displacement of the actuator, which, in turn applies the displace-

ments to the TLD. This simulates the displacements the TLD would experience if it were installed in the top storey of the archetype structure. The force and displacement feedback from the TLD can be monitored and recorded using the STS software. The test system has previously been used to carry out testing of TLDs and validation of equivalent models by Zhang et al. [173, 200, 207].

7.4.2 Archetype Structures

A series of theoretical archetype structures were used for validation of the equivalent TMD model. The archetype structures investigated are shown in Table 7.1. The TLD and experimental setup described in the previous section were used to perform the validation. For each of the archetypes, the depth of the water, h , is chosen using eqs. 7.18 and 7.25 to achieve a tuning ratio as close to, without exceeding, 1. The archetypes vary only in the number of storeys and hence, structure height, which leads to each structure having a different natural frequency. A typical storey height of 2.85 m for modular buildings is adopted for each structure.

Table 7.1: Archetype structures used for validation

Number of Storeys	Structure Height	Natural Frequency of the Structure, ω_s	Natural Frequency of the TLD, ω_{eq}	Liquid Depth in TLD, h	Tuning Ratio, Ω
	(m)	(Hz)	(Hz)	(m)	
40	115.00	0.4874	0.4765	0.39	0.9777
44	126.50	0.4113	0.4088	0.27	0.9940
46	132.25	0.3710	0.3649	0.21	0.9834
50	143.75	0.3203	0.3112	0.15	0.9715
55	158.13	0.2650	0.2555	0.10	0.9639
60	172.50	0.2212	0.2142	0.07	0.9650

7.4.3 Validation using Sinusoidal Loading

Initial validation of the equivalent TMD model was performed by applying a sine wave with a similar frequency to the translational DOF of the top storey of each

archetype structure using the MDOF surrogate model. There were no TLDs added to this model. The model was then solved to obtain time histories of the dynamic response. The resulting displacements at the top storey in the translational DOF were then applied to the full scale TLD in the laboratory which was tuned to the respective natural frequency. The force and displacement responses of the TLD were recorded. Figure 7.5 shows the process followed.

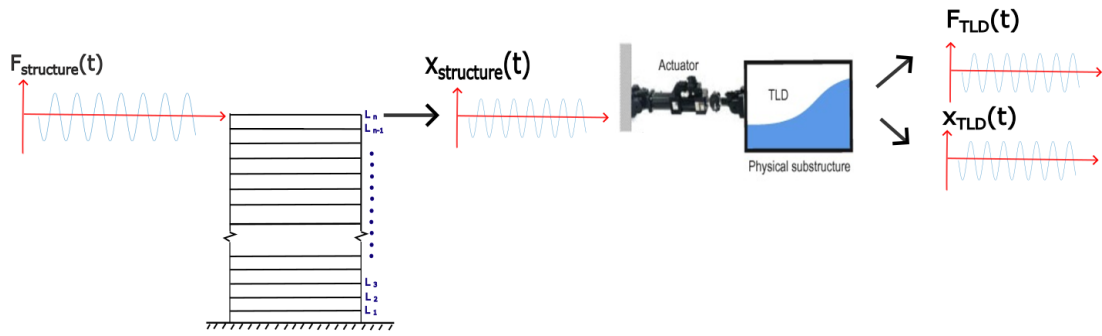


Figure 7.5: Process followed to validate MDOF surrogate model containing an equivalent TMD model of a TLD

The same sinusoidal force was applied to the MDOF surrogate model containing a singular tuned TLD at the top storey for each archetype structure. This was also solved to obtain the resulting theoretical displacement response of the TLD. A Fourier Transform was applied to the displacement response of the TLD to obtain its theoretical natural frequency. Similarly, a Fourier Transform was also applied to the experimental displacement feedback for the TLD obtained through the laboratory tests described above.

The natural frequency of the theoretical TLD was found to be quite similar to the natural frequency of the full-scale experimental TLD tested in the laboratory for each archetype. A comparison of the theoretical and experimental natural frequencies is given in Table 7.2 and visually shown in Figure 7.6.

From Figure 7.6 it is apparent that the natural frequency of the theoretical TLD, calculated using the equivalent TLD model, is in excellent agreement with the natural frequency of the in situ experimental TLD tested under cyclic loading. An almost exact 1-to-1 mapping exists between the two. Therefore, the equivalent TMD model appears to accurately capture the dynamic characteristics of a TLD.

Table 7.2: Theoretical and experimental TLD natural frequencies

Archetype Structure	Archetype Structure	Theoretical TLD	Experimental TLD
Storeys	Natural Frequency (Hz)	Natural Frequency (Hz)	Natural Frequency (Hz)
40	0.4874	0.4498	0.4515
44	0.4113	0.4106	0.4089
46	0.3710	0.3713	0.3698
50	0.3203	0.2930	0.2987
55	0.2650	0.2499	0.2489
60	0.2212	0.2000	0.1991

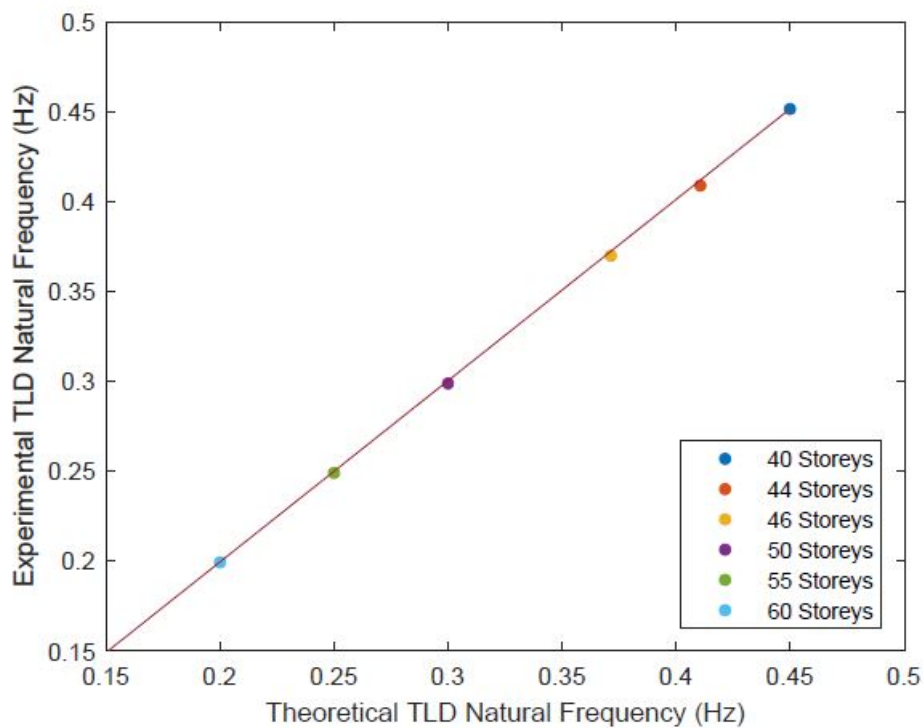


Figure 7.6: Comparison of the natural frequency of the equivalent TMD model and in situ TLD

The theoretical force response of the theoretical TLD can be calculated by considering the stiffness and damping properties of the TLD and the computed response as in eq. 7.33.

$$\begin{bmatrix} k_{eq} & -k_{eq} \\ -k_{eq} & k_{eq} \end{bmatrix} \begin{Bmatrix} x_{TLD} \\ x_{structure} \end{Bmatrix} + \begin{bmatrix} c_{eq} & -c_{eq} \\ -c_{eq} & c_{eq} \end{bmatrix} \begin{Bmatrix} \dot{x}_{TLD} \\ \dot{x}_{structure} \end{Bmatrix} = \begin{bmatrix} -F(t)_{TLD} \\ F(t)_{TLD} \end{bmatrix} \quad (7.33)$$

where x_{TLD} and $\dot{x}_{TLD}$ are the displacement and velocity responses of the translational DOF of the theoretical TLD in the MDOF surrogate model and $x_{structure}$ and $\dot{x}_{structure}$ are the displacement and velocity responses of the translational DOF at the top storey of the archetype structure with a TLD installed.

The RMS theoretical and experimental force response of the TLD are compared in Table 7.3. Considering the difference in the RMS response, it can be seen that the theoretical response is lower than the actual response of the TLD in all cases except for the 46 storey archetype.

Table 7.3: RMS force response of theoretical and experimental TLD

Archetype Structure (Storeys)	Theoretical RMS Force Response (N)	Experimental RMS Force Response (N)	% Difference
40	77.2285	207.2583	62.7381
44	126.1019	132.3596	11.3020
46	145.9956	98.2729	-37.1246
50	45.8019	73.7066	40.8652
55	18.3187	54.64928	66.4795
60	7.9320	55.6410	85.7443

Figure 7.7 shows plots of the force response for both the theoretical and experimental TLD. The difference in the magnitude of the theoretical and experimental force response can be appreciated. Beating of the response is apparent for the 40 and 44 storey archetypes. The TLDs for these archetypes have the largest mass due to the larger liquid depths needed to tune the TLD to the higher natural frequencies of these archetypes. The beating seen in the force response may be a

result of the increased mass and volume of liquid activated by the applied loading. In TLDs with larger liquid depths, standing sloshing waves occur which can reduce energy dissipation and damper effectiveness.

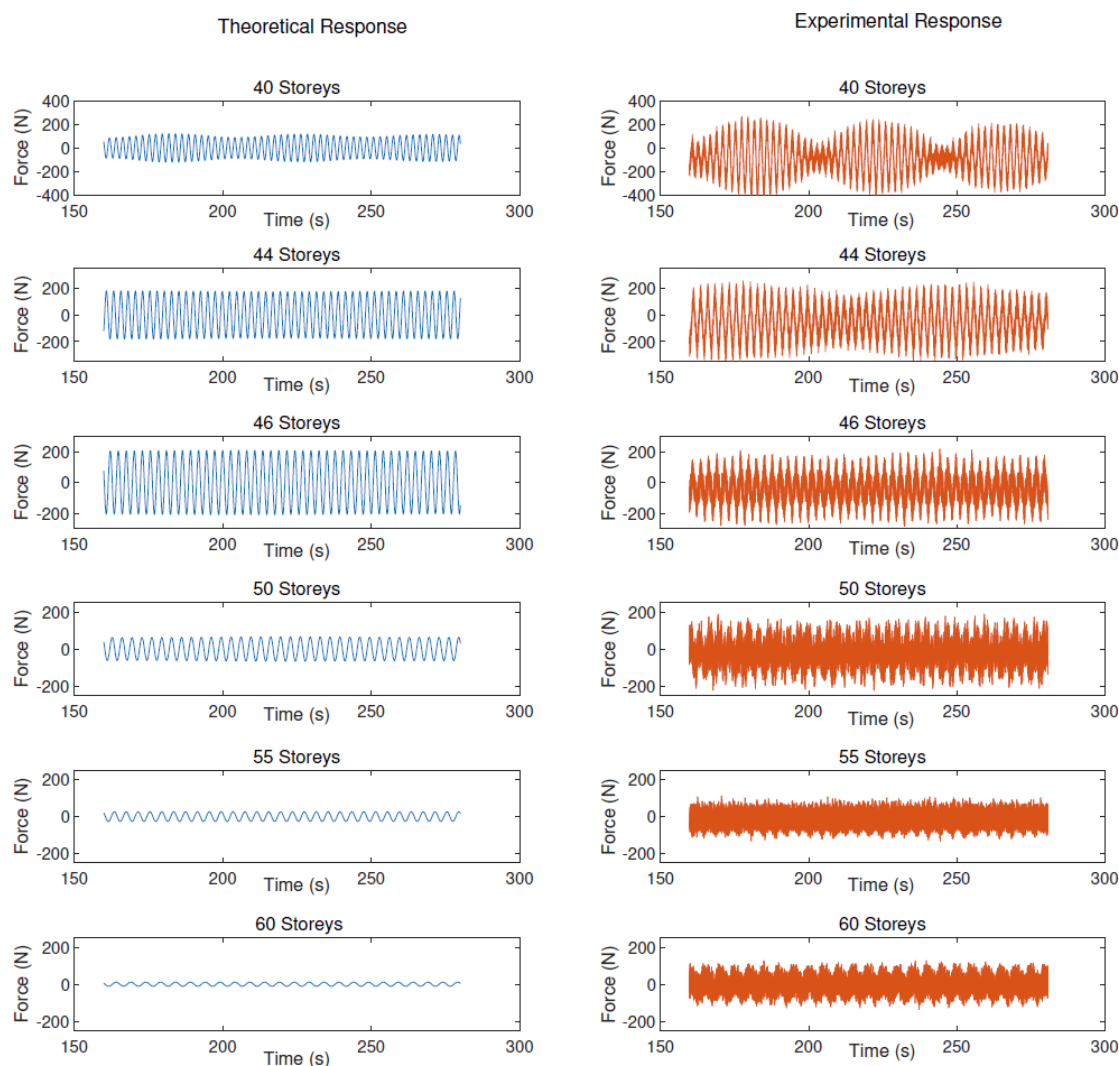


Figure 7.7: Theoretical vs Experimental TLD Force Response for sinusoidal loading

The comparison in Figure 7.7 and the RMS values in Table 7.3 suggest that the theoretical equivalent TMD model of the TLD provides an underestimate of the force response of a TLD. However, the equivalent TMD does provide an accurate model for tuning the TLD as the theoretical and experimental natural frequencies of the TLD are in good agreement as presented in Figure 7.6 and Table 7.2.

7.4.4 Validation for Wind-Induced Response

Further validation of the equivalent TMD model of a TLD presented by Tait [155] was performed by considering wind-induced forces and the response of the theoretical and experimental TLDs. White noise wind load time histories were calculated for 4 different wind speeds corresponding to a 1 in 1 year, 1 in 5 year, 1 in 10 year and 1 in 50 year wind events using Eurocode 1991-1-4 [73] and factors relevant to the site in London, United Kingdom of the two case study modular buildings. For simplicity, and as the TLD can only be tested in one direction, only along-wind forces are considered.

As described in Section 3.3.1, the fundamental basic wind speed, $v_{b,0}$, specified by the UK National Annex to Eurocode 1991-1-4 [73] is 21.50 m/s for a 50 year return period. Using eq. 3.5, the scaling factor, c_{prob} , can be calculated for a 5 year and 10 year return period and the wind speeds for each return period can be calculated by multiplying c_{prob} by $v_{b,0}$. However, as was discussed in Section 3.3.1, eq. 3.5 is not valid for a 1 year return period. Instead, a Poisson Distribution can be used to approximate the value of c_{prob} and hence, calculate the basic wind speed for a 1 year return period. The return periods, c_{prob} factors for each return period and basic wind speeds, $v_{b,0}$, used for validation are shown in Table 7.4.

Table 7.4: Wind speeds applied to archetype structures

Return Period	c_{prob}	Basic Wind Speed, $v_{b,0}$ (m/s)
1 year	0.7493	16.11
5 year	0.8614	18.52
10 year	0.9056	19.47
50 year	1	21.50

Each of the wind speeds in Table 7.4 were used to generate time histories of wind forces at each DOF of each of the archetype buildings using the methods outlined in Eurocode 1991-1-4 [73]. The surrounding terrain is assumed to be urban, or Class IV in Eurocode, similar to that of the location of the case study structures. The forces were calculated using a 0.015625 s time step, corresponding to a sampling frequency of 64 Hz. First, the basic wind speed, v_b , is calculated as in eq. 7.34.

$$v_b = c_{dir} \times c_{season} \times v_{b,0} \quad (7.34)$$

where c_{dir} and c_{season} are directional and seasonality coefficients, typically taken as 1. The terrain factor k_r is then calculated as:

$$k_r = 0.19 \times \left(\frac{z_0}{z_{0,II}} \right)^{0.07} \quad (7.35)$$

where z_0 is the roughness length which depends on the terrain and for ‘Urban’ terrain is 1, and $z_{0,II}$ is taken as 0.05. k_r is used in calculating the roughness factor $c_r(z)$ such that:

$$c_r(z) = k_r \times \ln \left(\frac{z}{z_0} \right) \quad (7.36)$$

where z is the height above ground at which the wind force is calculated. This value changes for each storey of the archetype buildings. The wind velocity at each storey, $v_m(z)$ is calculated using eq. 7.37.

$$v_m(z) = c_r(z) \times c_o(z) \times v_b \quad (7.37)$$

where $c_o(z)$ is the orography factor taken as 1.

Next, the wind turbulence, $I_v(z)$ is calculated:

$$I_v(z) = \frac{\sigma_v}{v_m} = \frac{k_I}{c_o(z) \times \ln \frac{z}{z_0}} \quad (7.38)$$

where σ_v is the standard deviation of the turbulence and k_I is the turbulence factor, typically taken as 1. The turbulent length scale, $L_t(z)$, is calculated as:

$$L_T(z) = 300 \left(\frac{z}{200} \right)^{0.67+0.05 \ln(z_0)} \quad (7.39)$$

$L_T(z)$ is used in calculating the non-dimensional frequency spectrum, f_L such that:

$$f_L(z, n) = \frac{n L_T(z)}{v_m(z)} \quad (7.40)$$

where n is the frequency. The power spectral density function, $S_L(z, n)$ is then calculated as:

$$S_L(z, n) = \frac{6.8 f_L(z, n)}{(1 + 10.2 f_L(z, n))^{5/3}} \quad (7.41)$$

This wind spectrum can then be used to calculate the wind speed and corresponding wind force at each DOF of the structure. This process was carried out for each of the four basic wind speeds in Table 7.4 and the wind forces applied to the MDOF surrogate model of each archetype building. Each wind force which has been applied to the MDOF surrogate model of the archetypes is shown in Figure 7.8.

The displacement, velocity and acceleration responses of the MDOF surrogate model of each archetype building under wind loading with the four different return periods were solved using the Newmark Beta method. The displacement response at the top storey in the translational DOF was then applied to the experimental TLD using the process described in 7.4.3 and the laboratory set up described in Section 7.4.1. In the case of the 60 storey archetype under a 1 in 50 year wind load, the displacements at the top storey were greater than 250 mm and hence exceeded the maximum actuator stroke. Therefore, this scenario was not tested and does not form part of the dataset.

For validation purposes, a single equivalent TMD was added to the top storey of the MDOF surrogate model of each archetype. Each wind load was applied as the forcing function for the archetype and the dynamic response of the theoretical TLD was calculated. Table 7.5 and Figure 7.9 show the natural frequencies obtained by performing a Fourier transform of the theoretical and experimental displacement response of the TLD for each archetype and wind speed.

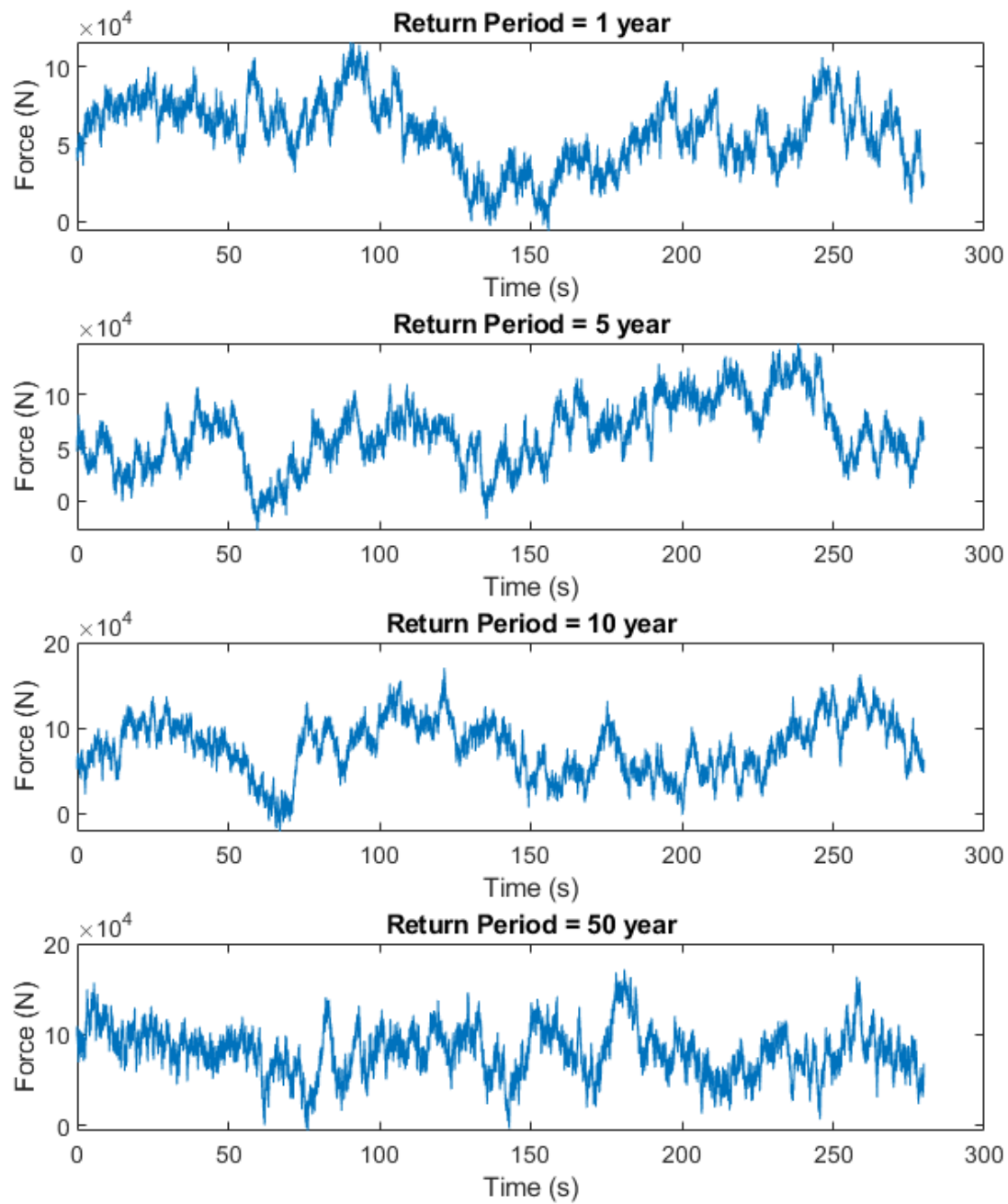


Figure 7.8: Wind forces applied to each archetype building

7.4. Validating the equivalent TMD Model of a TLD

Table 7.5: Natural frequencies of the theoretical and experimental TLD

Archetype (Storeys)	Return Period (years)	Natural Frequency of Theoretical TLD (Hz)	Natural Frequency of Experimental TLD (Hz)
40	1	0.4748	0.4871
	5	0.4748	0.4871
	10	0.4713	0.4871
	50	0.4748	0.4871
44	1	0.4213	0.4124
	5	0.4177	0.4089
	10	0.4177	0.4076
	50	0.4213	0.4053
46	1	0.3713	0.3698
	5	0.3642	0.3698
	10	0.3641	0.3767
	50	0.3677	0.3698
50	1	0.3070	0.3200
	5	0.3142	0.3164
	10	0.3106	0.3200
	50	0.3106	0.3164
55	1	0.2642	0.2361
	5	0.2535	0.2667
	10	0.2335	0.2595
	50	0.2499	0.2702
60	1	0.2142	0.2204
	5	0.2142	0.2204
	10	0.2214	0.2204

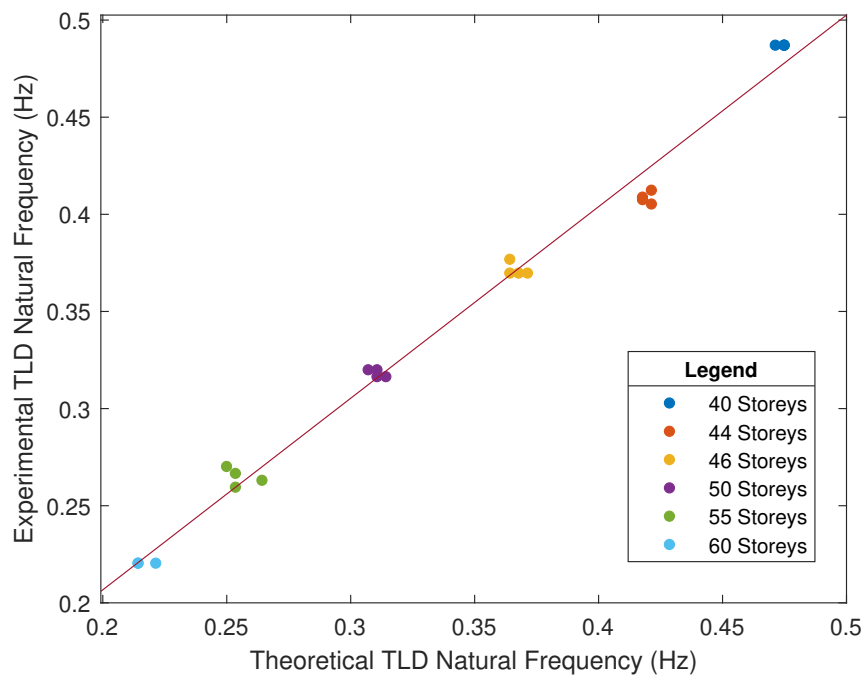


Figure 7.9: Comparison of the natural frequency of the equivalent TMD model and in situ TLD under wind loading

The natural frequencies of the theoretical TLD and experimental TLD are in good agreement, further validating that the equivalent TMD model of a TLD proposed by Tait [155] is a sensible representation of a TLD for the preliminary stages of design.

Using eq. 7.33 the theoretical force response of the equivalent TMD model was calculated for each archetype and wind speed. Observing Figures 7.10 - 7.15 a number of key identifying features can be seen in both the calculated theoretical force response to the observed experimental force response. The overall form of the force response appears similar across both the theoretical and experimental responses for all cases with the exception of the 44 storey structure. Furthermore, the difference in the magnitude of the force responses of the different structures are equally captured by the theoretical and experimental responses. Therefore, these results suggest the theoretical equivalent TMD model is a suitable preliminary modelling method for assessing the force output of a TLD.

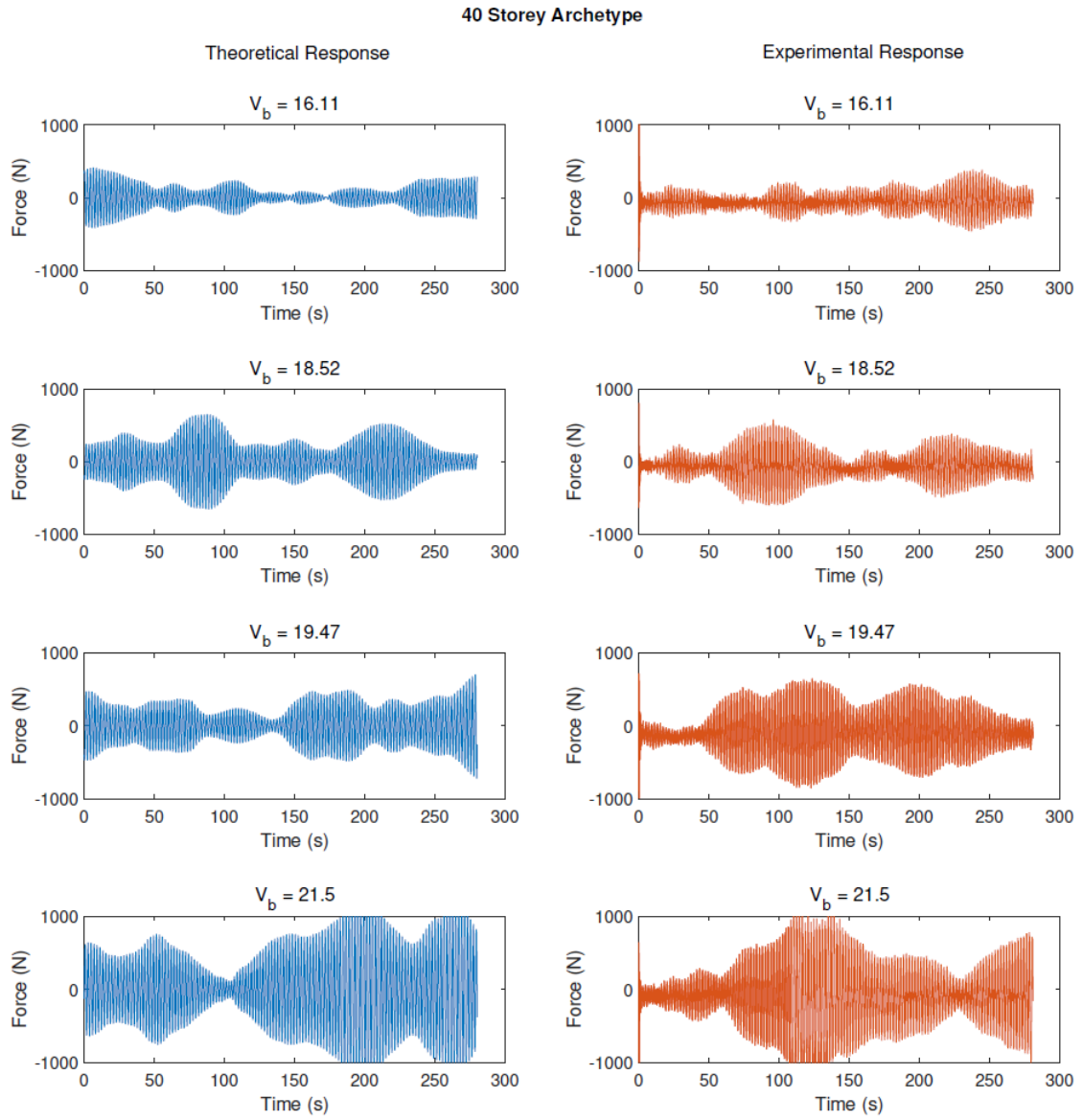


Figure 7.10: Theoretical vs experimental TLD force response of a 40 storey archetype under wind loading

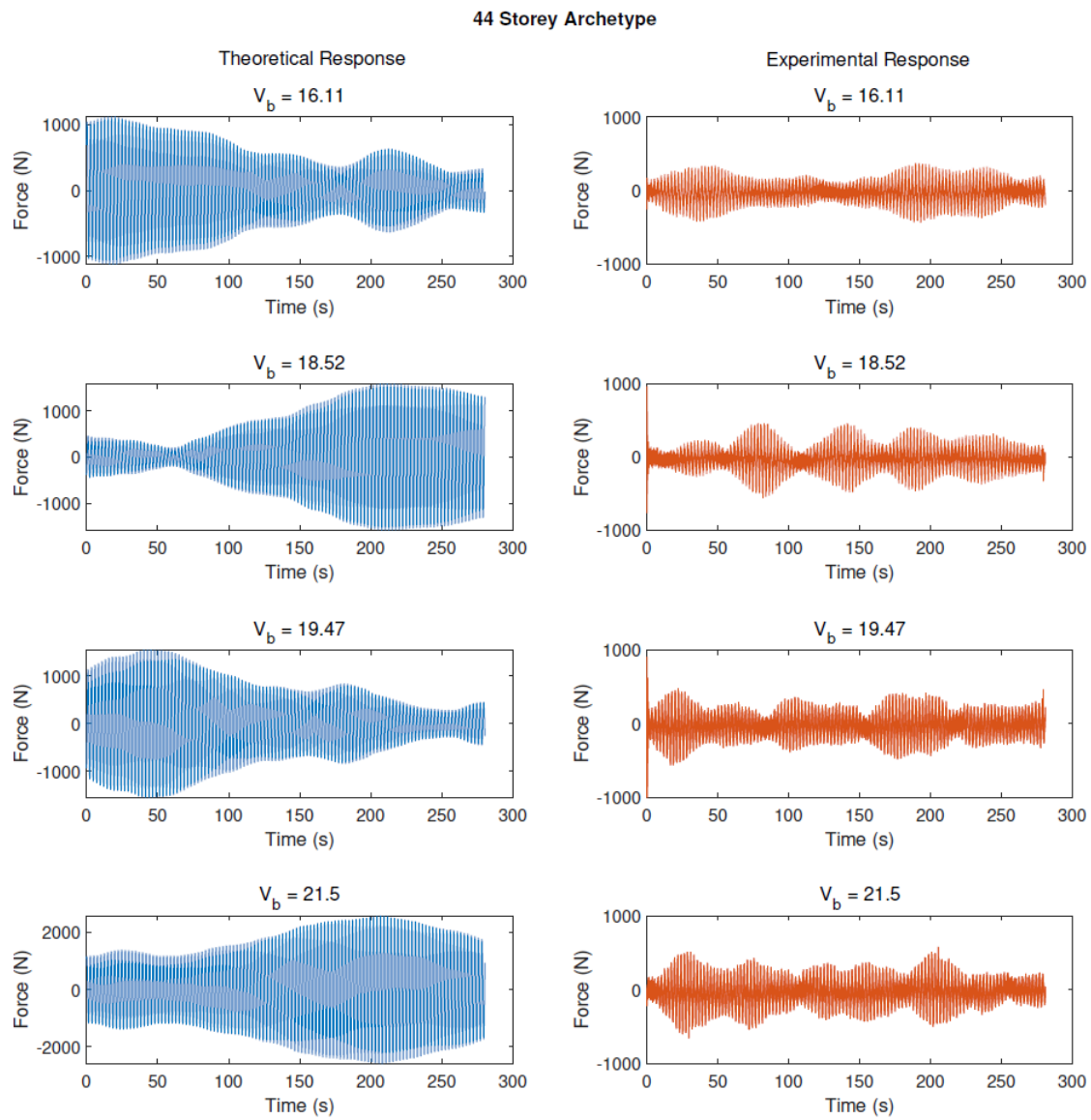


Figure 7.11: Theoretical vs experimental TLD force response of a 44 storey archetype under wind loading

7.4. Validating the equivalent TMD Model of a TLD

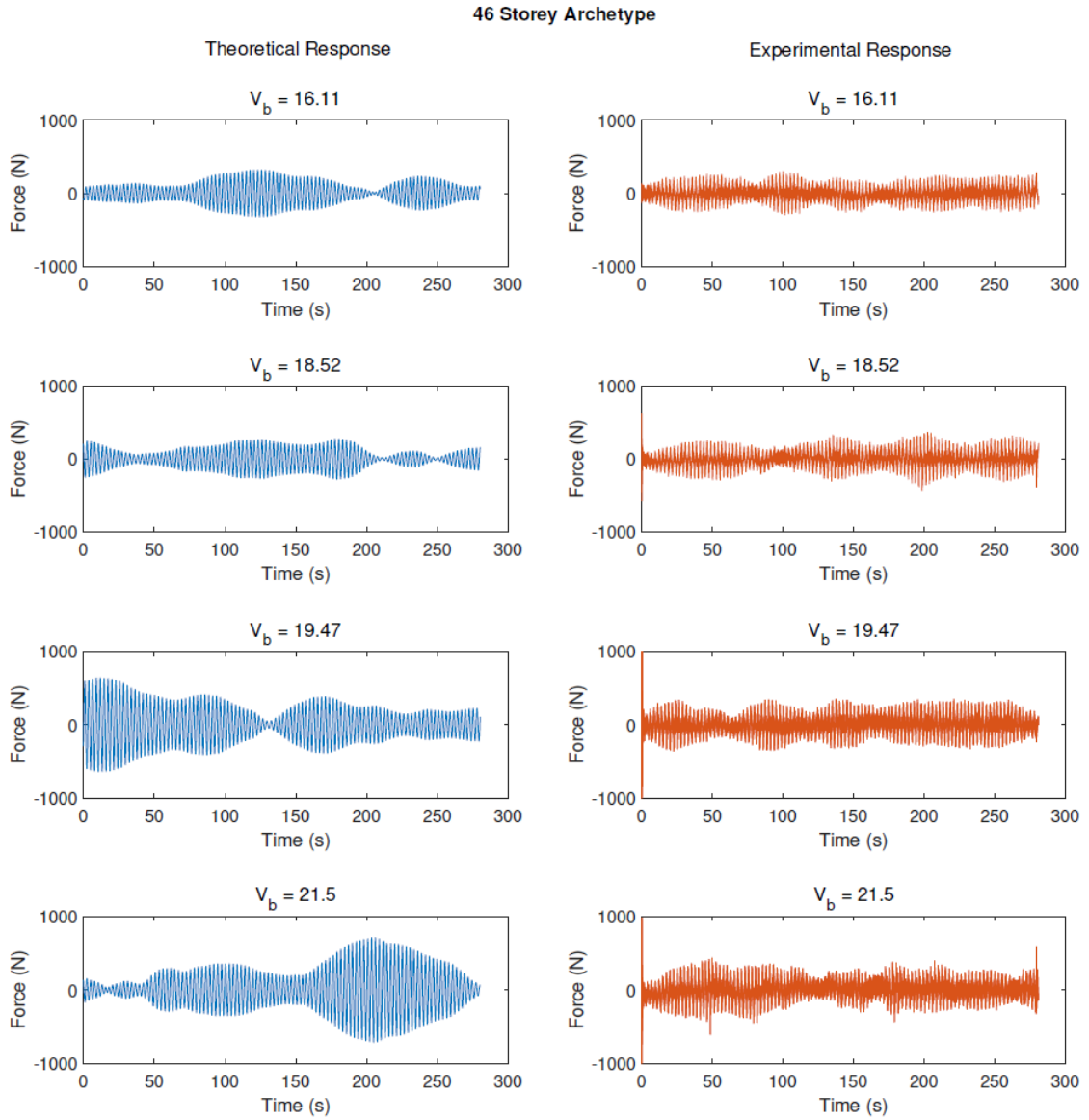


Figure 7.12: Theoretical vs experimental TLD force response of a 46 storey archetype under wind loading

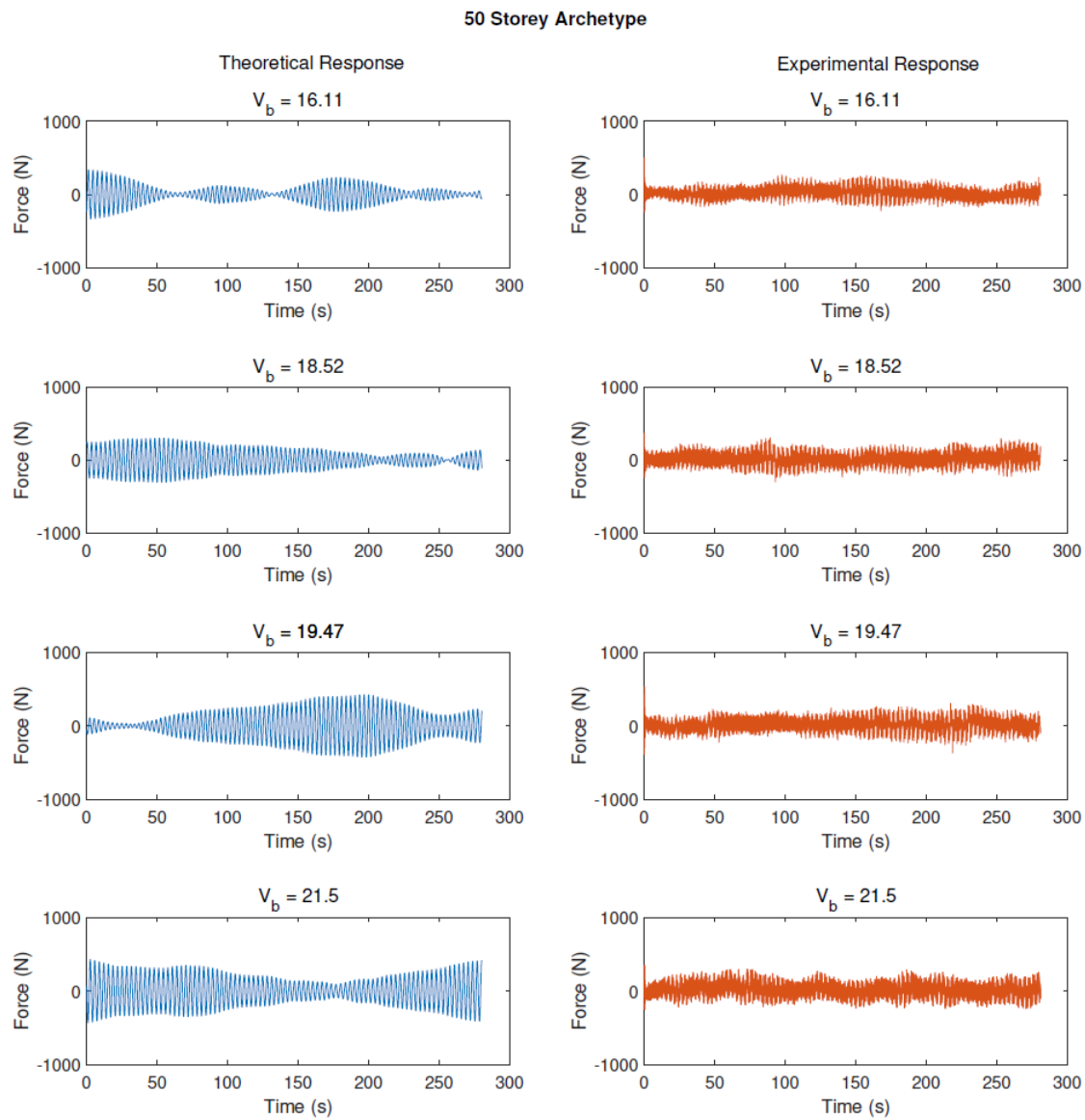


Figure 7.13: Theoretical vs experimental TLD force response of a 50 storey archetype under wind loading

7.4. Validating the equivalent TMD Model of a TLD

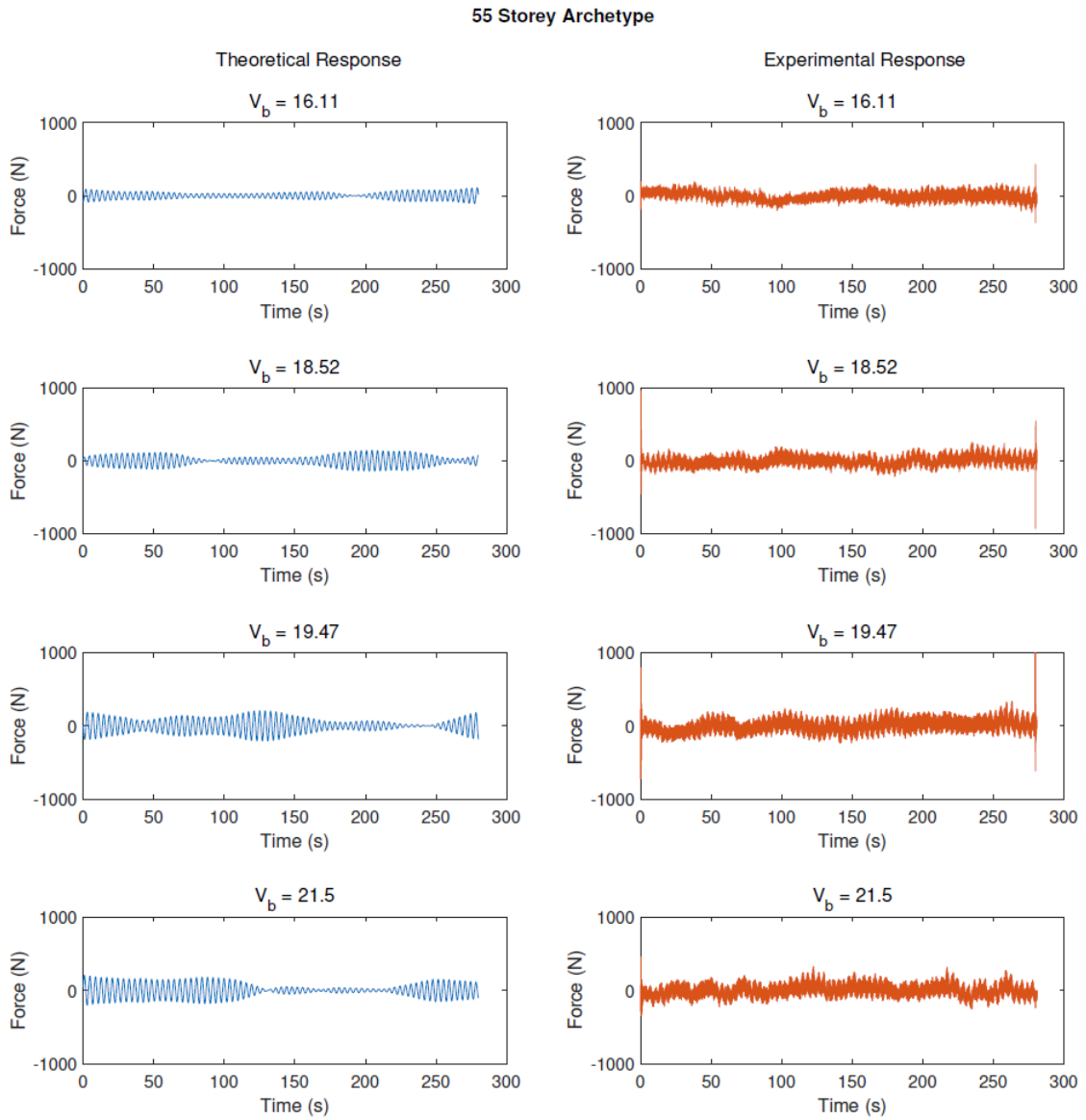


Figure 7.14: Theoretical vs experimental TLD force response of a 55 storey archetype under wind loading

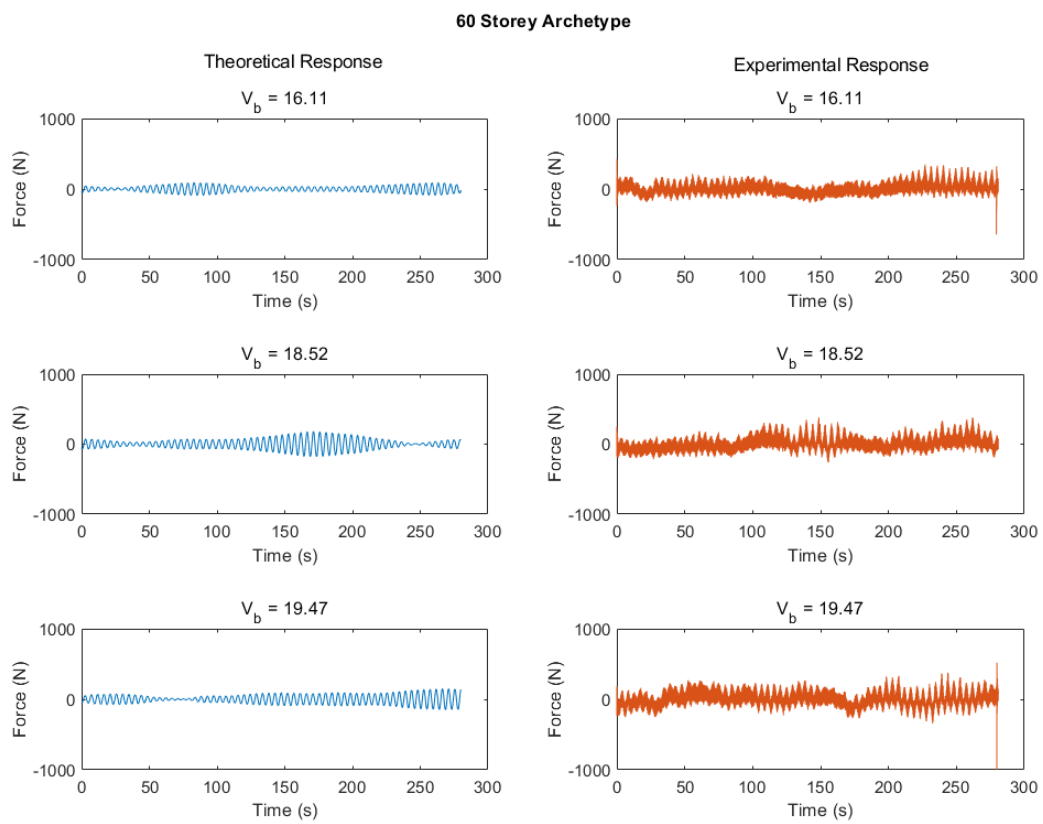


Figure 7.15: Theoretical vs experimental TLD force response of a 60 storey archetype under wind loading

There are a number of methods that can be used to compare time histories. According to Kavrakov et al. [248] comparing the Root Mean Square (RMS) and the Power Spectral Density (PSD) of the responses is one suitable measure. Table 7.6 shows the RMS of the response of both the theoretical and experimental TLDs. It can be seen that in a clear majority of cases, the two are within 50% of each other. However, for the 44 storey archetype, the theoretical TLD model provides large overestimates of the RMS force response. This is likely due to the fact that the natural frequency of the theoretical TLD, calculated using the equivalent TMD model, provides a tuning ratio, $\Omega = \omega_{TLD}/\omega_s = 0.9940$, which is the value closest to 1 out of all the theoretical TLDs. The equivalent TMD model was developed to favour high tuning ratios as these result in more significant mass mobilisation of the liquid and hence a larger force response [155]. The next highest tuning ratio of any of the archetypes is that of the 46 storey ($\Omega = 0.9834$), for which the theoretical TLD response also overestimates the experimental force response. Considering Figure 7.16, the relationship between the tuning ratio and the force response of the theoretical TLD is apparent. The lower tuning ratios of the 50, 55 and 60 storey archetypes resulted in lower RMS force estimates. However, this relationship is not reflected in the experimental TLD and the stronger relationship appears to be between the depth of water in the TLD, and hence mass of the TLD, and the RMS force response.

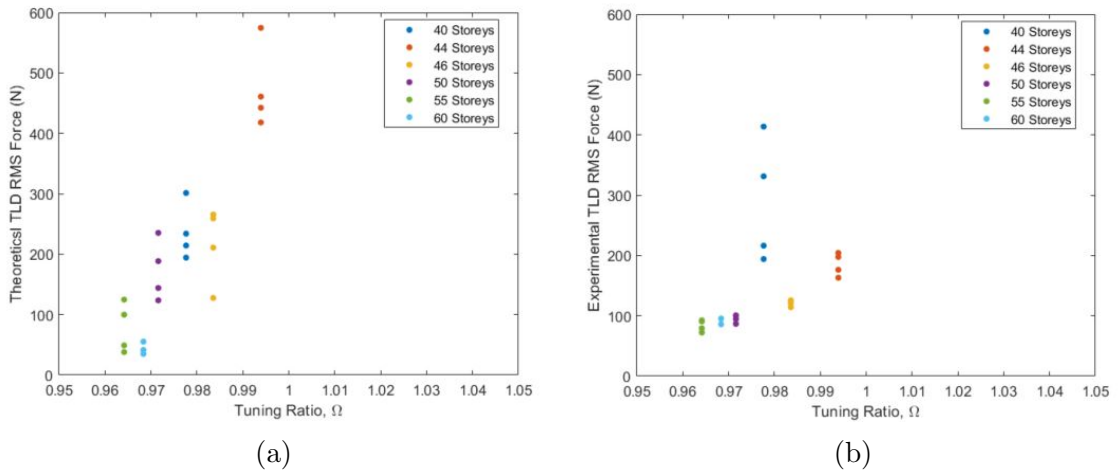


Figure 7.16: RMS force response vs tuning ratio for (a) Theoretical TLD and (b) Experimental TLD

Table 7.6: Maximum and RMS values of the theoretical and experimental force response

Archetype (Storeys)	h/L	Tuning Ratio, Ω	Return Period (years)	Theoretical RMS Force Response of TLD (N)	Experimental RMS Force Response of TLD (N)
40	0.2021	0.9777	1	194.26	194.17
			5	233.91	331.263
			10	214.38	216.255
			50	301.30	413.69
44	0.1399	0.9940	1	460.76	163.14
			5	418.05	176.25
			10	442.42	197.50
			50	574.59	204.27
46	0.1088	0.9834	1	127.51	114.17
			5	210.83	120.62
			10	265.54	121.84
			50	259.19	125.71
50	0.0777	0.9715	1	144.00	86.56
			5	188.45	95.64
			10	235.25	100.78
			50	123.58	100.06
55	0.0518	0.9639	1	37.93	72.17
			5	48.99	79.33
			10	99.90	90.70
			50	124.71	92.54
60	0.0363	0.9650	1	41.4	85.94
			5	55.28	95.47
			10	66.41	93.67

The ratio of the depth of liquid in the TLD to the length of the TLD, h/L , also has an impact on the behaviour of the liquid, and hence, the response of the TLD. Figure 7.17 shows the relationship between h/L and the force response for both the theoretical and experimental TLDs, which appear to be quite different. The theoretical TLD shows an increase in RMS force as h/L increases, before reducing again for the deeper TLD of the 40 storey archetype. The experimental TLD shows a much slower increase in force response as h/L increases, with the TLD with the largest mass of water resulting in the largest RMS force response. This is likely a result of the equivalent TMD model being a simplified model that does not account for theory relating to deep and shallow TLDs. It is well established that as h/L increases, the liquid at the base of the tank does not become involved and standing waves occur which are not as effective as the wave breaking which occurs when $h/L < 0.1$ [155, 166]. Wave breaking in shallower TLDs results in higher damping values as the breaking waves impact the walls of the TLD. As the horizontal dimensions of the experimental TLD are fixed, the mass of the TLD is proportional to water depth and is different for each archetype. Without controlling for mass, it is difficult to see the effect of h/L on the RMS Force response of the experimental TLD. However, it can be seen that where h/L is less than 0.1, particularly for the 55 and 60 storey TLDs, the equivalent TMD model of the TLD underestimates the force response of the experimental TLD. This is likely due to the increased wave breaking occurring which is not captured entirely by the equivalent TMD model. Despite this, the magnitude of the TLD RMS forces from both the theoretical and experimental TLDs are similar magnitude across most of the tests, suggesting that the equivalent TMD model is suitable for the preliminary design of a TLD.

Another method for comparing the theoretical and experimental models of a TLD is to compare the power spectral densities of the responses. Considering the power spectral densities shown in Figures 7.18 - 7.23 for each archetype, the power content of the theoretical and experimental force responses can be observed to be quite similar. The frequency and amplitude of the strongest peaks in the theoretical and experimental PSDs show strong agreement. As expected, the experimental PSDs are noisier, and pick up on responses in more than one mode, which the equivalent TMD theoretical model is not capable. Moreover, the experimental PSDs of the shorter archetypes which had TLDs with larger h/L ratios exhibit less noise. Nevertheless, the peak of power at the natural frequency of both the theoretical model and experimental TLD remain clear.

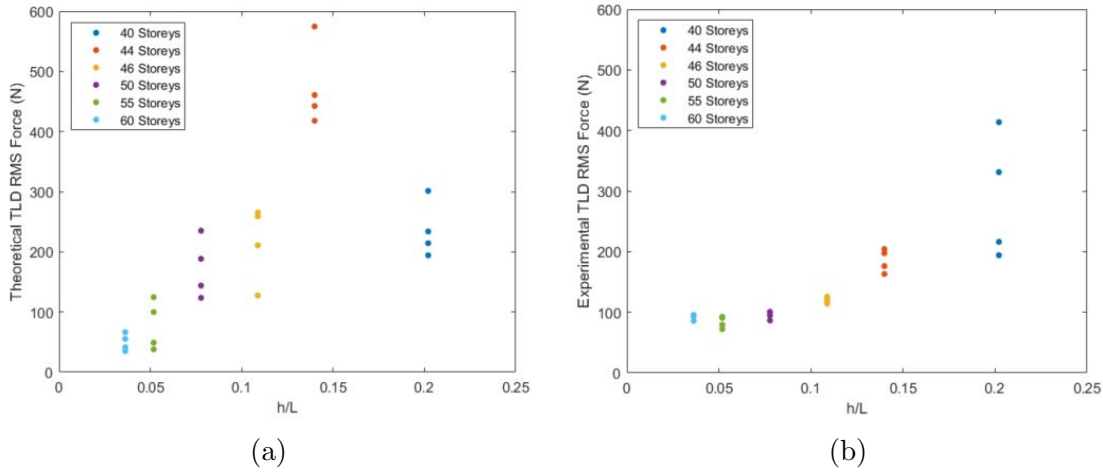


Figure 7.17: RMS force response vs h/L for (a) Theoretical TLD and (b) Experimental TLD

The similarity of the PSDs further validates the equivalent TMD as a suitable preliminary model of a TLD. However, this model is not without limitations. As noted, the simplified equivalent TMD model does not capture the complexity of the behaviour of liquid within a TLD. In particular, where tuning ratios are further from 1 and at lower h/L ratios. This study is limited by the dimensions of the full-scale TLD which was available. As an area of future work, this study could be repeated using multiple TLDs of dimensions optimised for each archetype modular building, enabling optimal tuning ratios and h/L ratios to be examined. Despite the limitations, through the validation performed, considering both the magnitudes of the force response and PSD of the responses, it is clear that the equivalent TMD model proposed by Tait [155], integrated into the data-informed MDOF surrogate model, is appropriate to use in the parametric assessment of vertically distributed TLDs for the control of high-rise modular buildings.

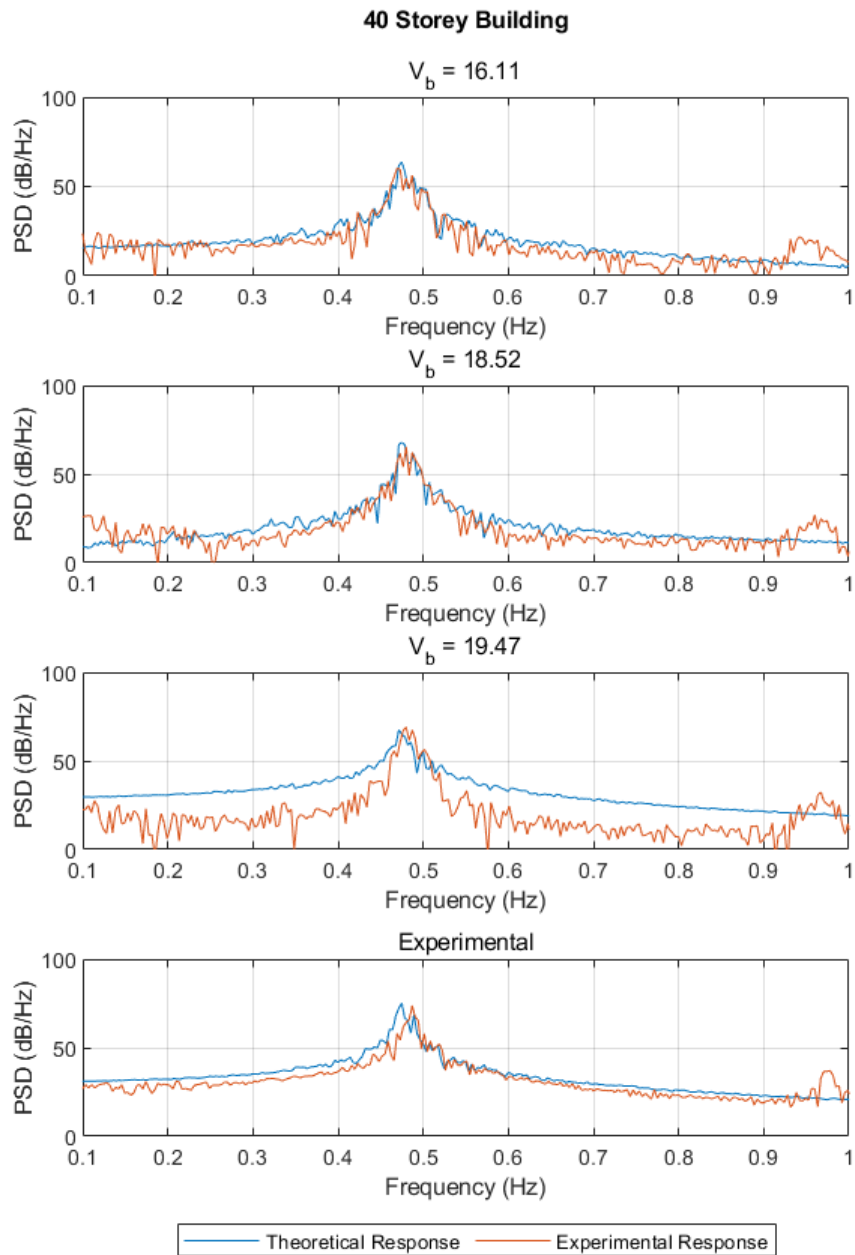


Figure 7.18: Theoretical vs experimental Power Spectral Density of Force response of a 40 storey archetype under wind loading

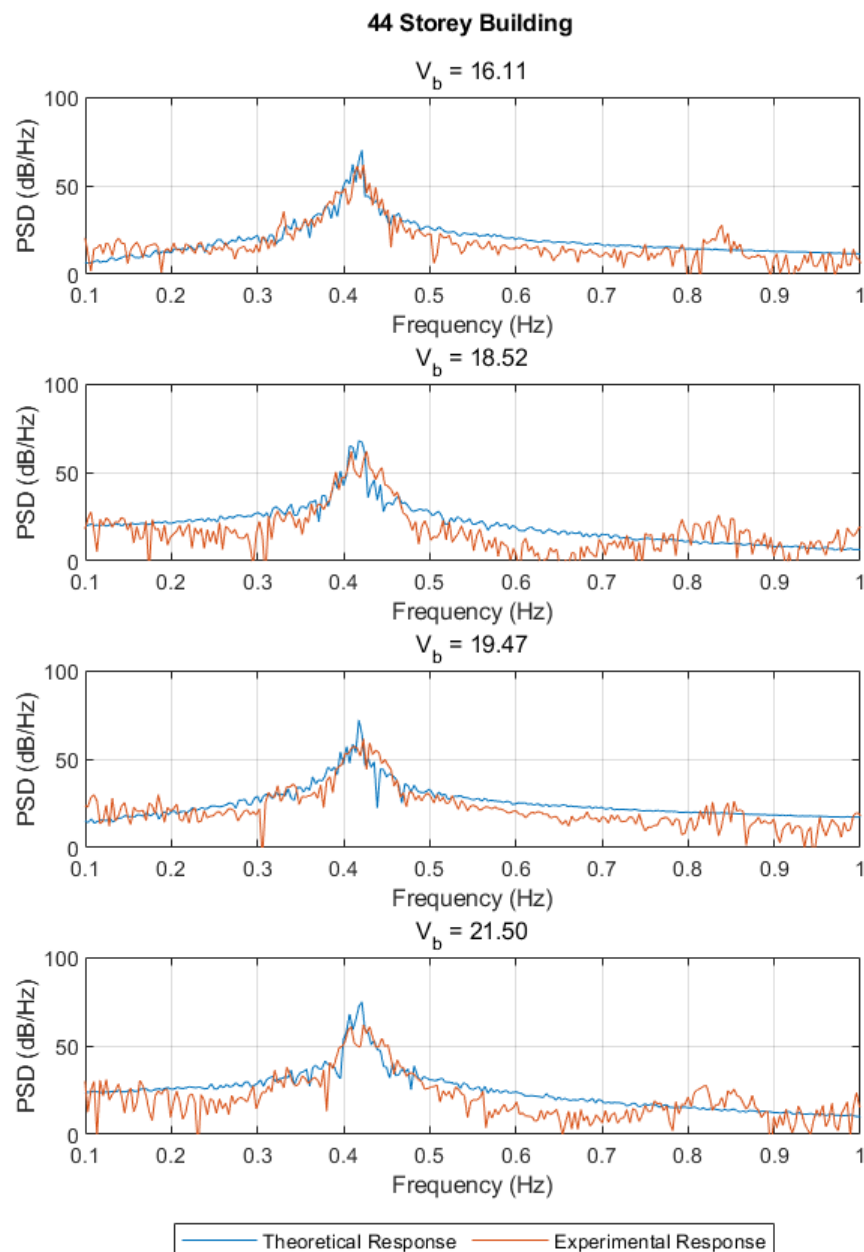


Figure 7.19: Theoretical vs experimental Power Spectral Density of Force response of a 44 storey archetype under wind loading

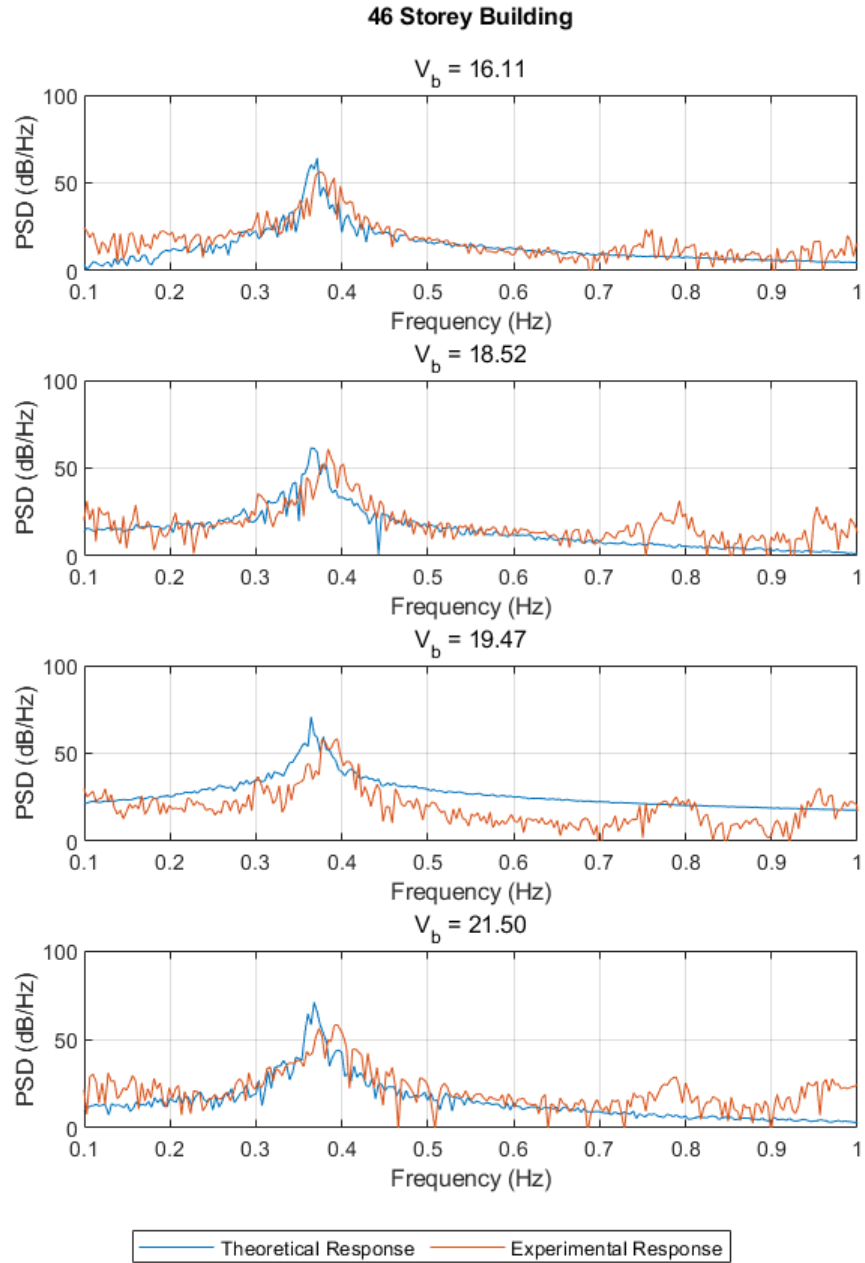


Figure 7.20: Theoretical vs experimental Power Spectral Density of Force response of a 46 storey archetype under wind loading

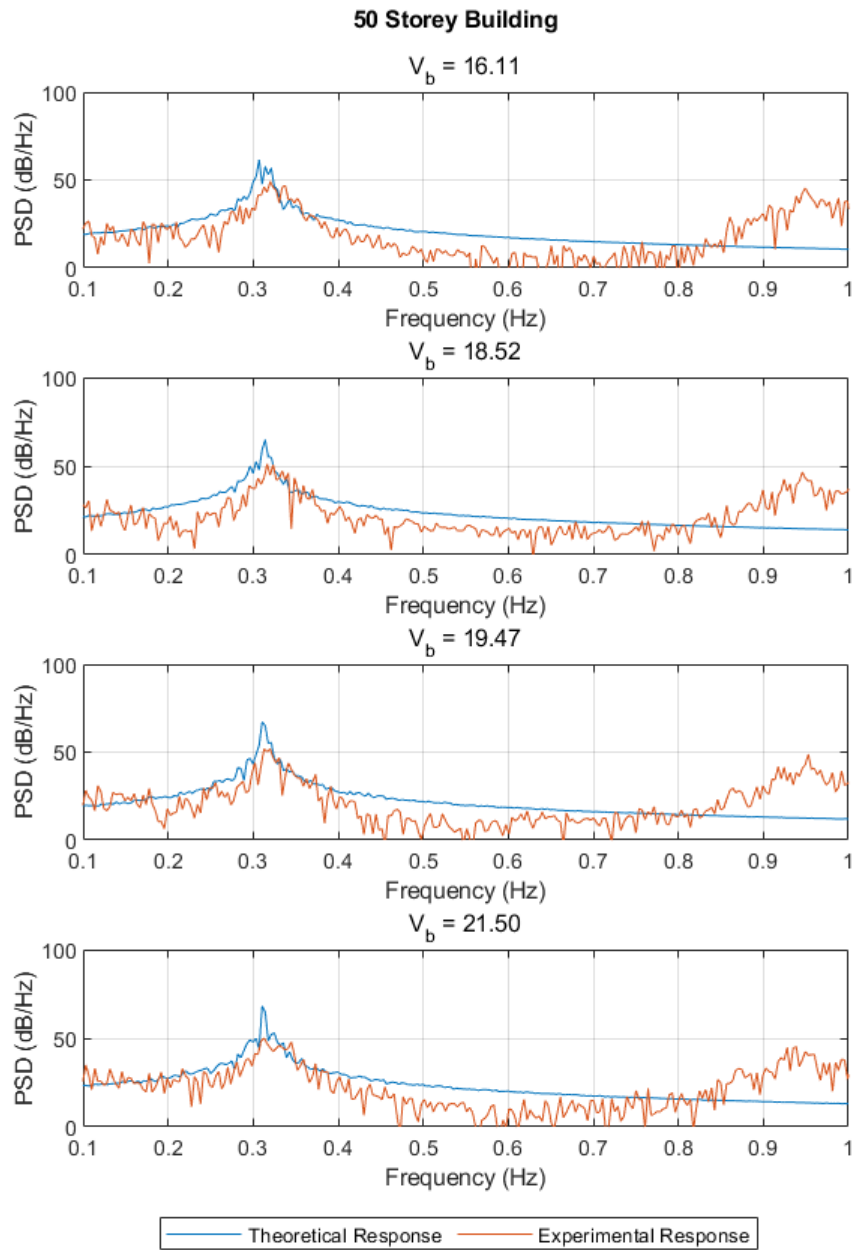


Figure 7.21: Theoretical vs experimental Power Spectral Density of Force response of a 50 storey archetype under wind loading

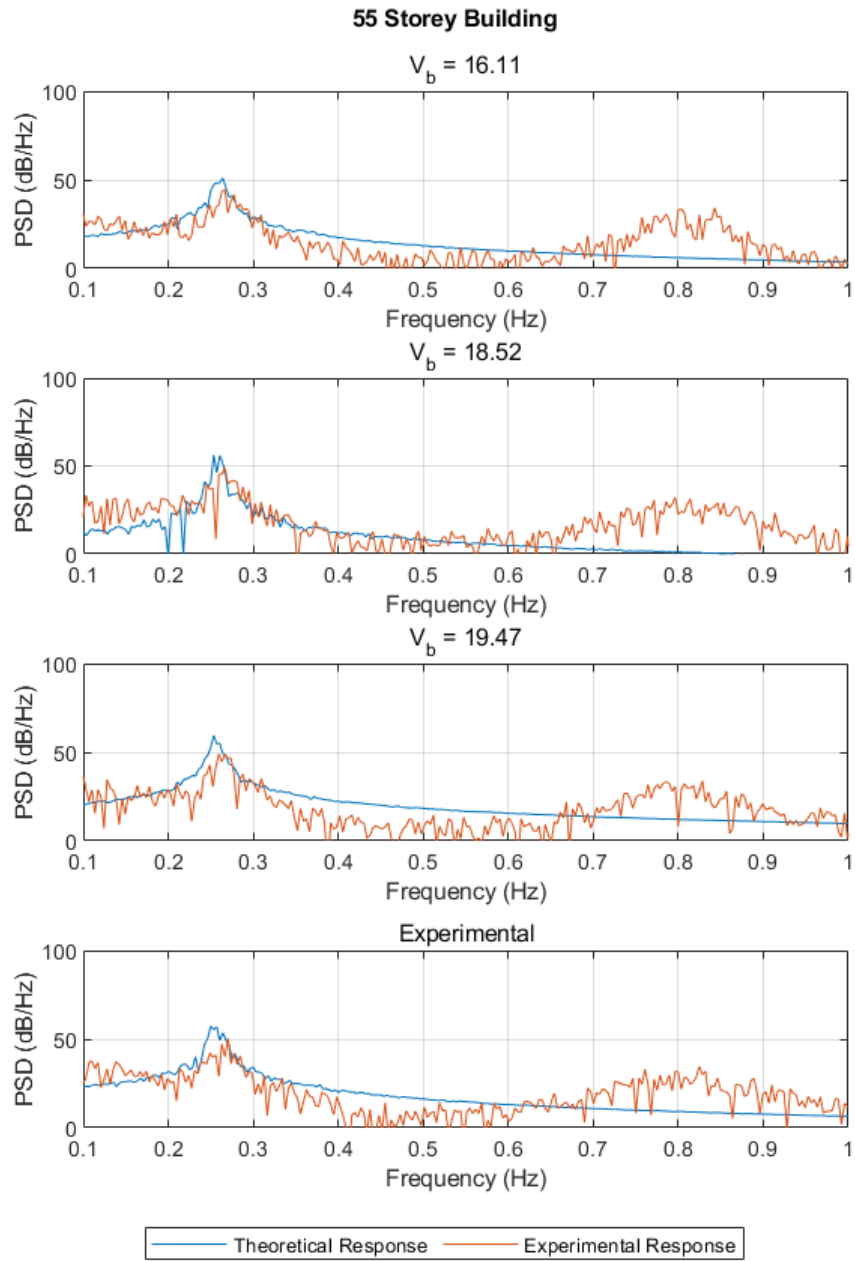


Figure 7.22: Theoretical vs experimental Power Spectral Density of Force response of a 55 storey archetype under wind loading

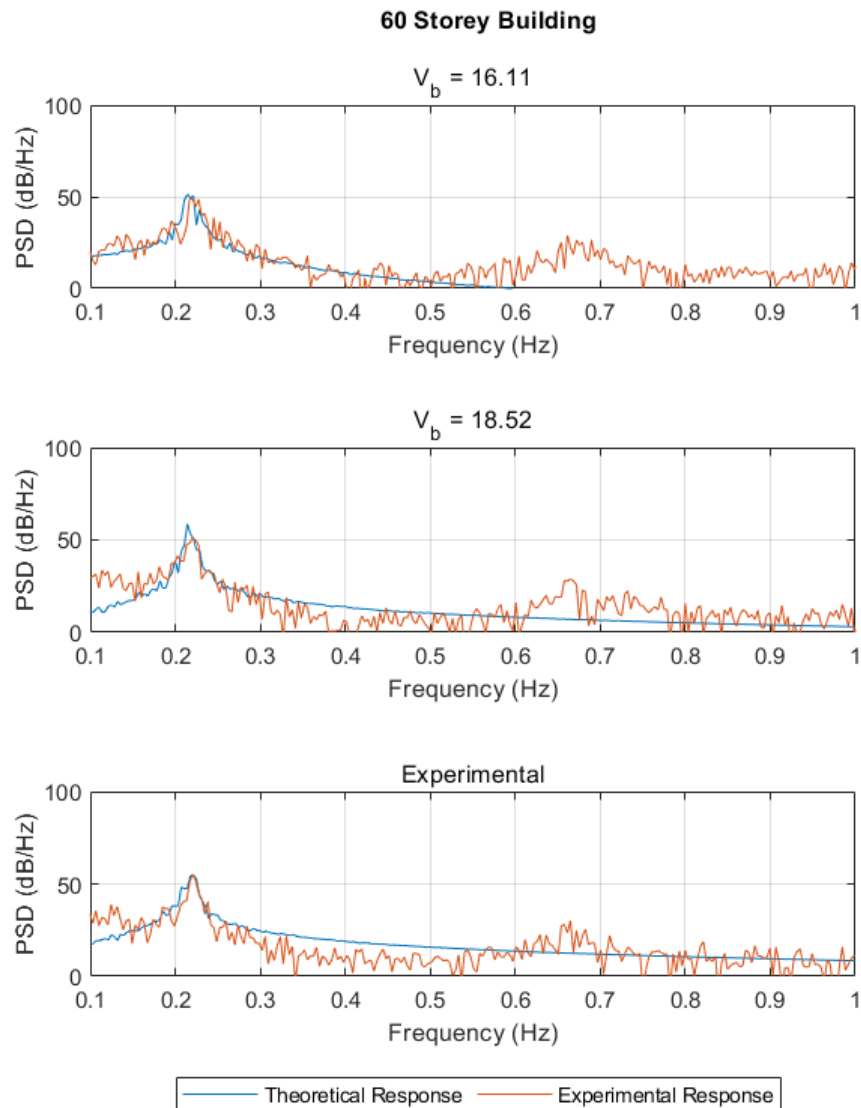


Figure 7.23: Theoretical vs experimental Power Spectral Density of Force response of a 60 storey archetype under wind loading

7.5 Parametric Study of MVDTLDS

The equivalent TMD model proposed by Tait [155], as implemented in the data-informed MDOF surrogate model, is used to parametrically assess Multiple Vertically Distributed Tuned Liquid Dampers (MVDTLDS) and their capacity to reduce the wind-induced response of modular high-rise buildings. The validation performed in Section 7.4 supports findings that the equivalent TMD model is suitable in the preliminary assessment of TLDs [160, 165, 172, 174, 185, 195, 250]. The individual volumetric modules used to create a modular building offer a unique opportunity to integrate small TLDs in each module during the off-site manufacturing process. In theory, the TLDs could become another ‘part’ of the modules added in a high-quality factory environment. Alternatively, individual modules could house a number of stacked TLDs. This would negate the need to occupy large floor space at the upper storeys of the modular building where real-estate values are often significantly higher [251]. The concept of MVDTLDS suits the building block nature of modular construction and demonstrates an approach which capitalises on the characteristics of modular buildings by adapting traditional control methods to suit the novel construction technology rather than forcing modular construction to conform to traditional approaches.

The 50 storey archetype modular building described in Section 7.4 is used for this parametric study. Wind forces corresponding to return periods of 1, 5, 10 and 50 years, as calculated previously, are applied to the MDOF surrogate model of the 50 storey modular building and a number of different vertical distributions of TLDs across the 50 storeys and a range of mass ratios, μ , are investigated for their ability to reduce the response of the structure. For consistency, and to allow for further laboratory testing, a TLD of the same dimensions described in Section 7.4 (1.93 m in Length, 0.59 m in width and 1.59 m in height) is used and multiples of this TLD are distributed at different storeys of the structure to achieve the specified mass ratios.

A single TLD of these dimensions will not provide enough auxiliary damping to cause meaningful change in the dynamic response of a 50 storey archetype as the equivalent mass of the TLD is not significant in comparison to that of the structure. A typical module in the two structures studied: *Ten Degrees* and *College Road* is approximately 5.5 m long, 3 m wide and 2.875 m tall. Given this configuration,

a maximum of 8 of the specified TLDs can reasonably occupy the floor space of a single module allowing for adequate space for maintenance. If stacked on top of each other, a single module can accommodate 16 TLDs. Alternatively, a number of modules per storey can have a single TLD installed. For purposes of this analysis, the number of TLDs assessed was kept to multiples of 8 in order to achieve different values of mass ratio, μ , calculated as in eq. 7.42. The mass ratios studied lie between $0.005 \leq \mu < 0.04$ as anything greater becomes unfeasible and this is the typical range of mass ratios studied in literature [166, 186, 250]. Table 7.7 shows the combination of parameters investigated. Each combination of parameters is considered, resulting in 25 different distributions of TLDs being investigated under 4 different wind loading conditions. The total number of TLDs installed is divided evenly amongst the number of storeys in which the TLDs are vertically distributed over.

$$\mu = \frac{\text{Total Mass of TLDs}}{\text{Total Mass of Structure}} = \frac{\sum_{n=1}^n m_{eq}}{M_{structure} + \rho b h L - \sum_{n=1}^n m_{eq}} \quad (7.42)$$

Table 7.7: Range of parameters investigated in assessment of response reduction using MVDTLs

Parameter	Values
Structure Storeys	50
Wind Speed Return Period	1 year (16.11 m/s), 5 year (18.52 m/s), 10 year (19.47 m/s) and 50 years (21.50 m/s)
Number of Storeys TLDs Installed	1, 6, 15, 30 and 40 storeys
Total Number of TLDs installed	120, 240, 480, 720, 960
Mass Ratios , μ	0.005, 0.01, 0.019, 0.029, 0.038

The effectiveness of the MVDTLs in each scenario in the parametric assessment is compared back to the uncontrolled response of the 50 storey archetype. The response reduction is assessed by considering a number of performance criteria defined using the RMS responses of the structure as proposed by Yang et al. [120]. The criteria implemented in this study are defined in eqs. 7.43 - 7.48.

The first evaluation criterion considers the reduction in the maximum RMS acceleration as a result of the MVDTLDS.

$$J_1 = \frac{\max(\sigma_{\ddot{x}1}, \dots, \sigma_{\ddot{x}i})}{\sigma_{\ddot{x}50o}} \quad (7.43)$$

where $\sigma_{\ddot{x}i}$ is the RMS acceleration response at the i^{th} storey in the translational DOF of the controlled structure and $\sigma_{\ddot{x}50o}$ is the RMS acceleration response of the 50^{th} storey in the translational DOF of the uncontrolled structure. The second criteria considers the average reduction in the RMS acceleration response of the structure

$$J_2 = \sum_i \left(\frac{\sigma_{\ddot{x}i}}{\sigma_{\ddot{x}io}} \right), \text{ for } i = 1, 2, \dots, 50 \quad (7.44)$$

The third and fourth criteria evaluate the ability of the MVDTLDS to reduce the RMS displacement response of the structure. The third criterion considers the reduction in the maximum RMS displacement response, such that

$$J_3 = \frac{\max(\sigma_{x1}, \dots, \sigma_{xi})}{\sigma_{x50o}} \quad (7.45)$$

where σ_{xi} is the RMS displacement response at the i^{th} storey in the translational DOF of the controlled structure and σ_{x50o} is the RMS displacement response of the 50^{th} storey in the translational DOF of the uncontrolled structure.

The fourth evaluation criterion assesses the reduction in the average RMS displacement response as a result of the MVDTLDS

$$J_4 = \sum_i \left(\frac{\sigma_{xi}}{\sigma_{xio}} \right), \text{ for } i = 1, 2, \dots, 50 \quad (7.46)$$

The reduction in the peak acceleration and displacement response of the structure is also considered using criteria J7 and J9.

$$J_7 = \frac{\max(\ddot{x}_{p1}, \dots, \ddot{x}_{pi})}{\ddot{x}_{p50o}} \quad (7.47)$$

where $\ddot{x}_{pi}$ is the peak acceleration response at the i^{th} storey in the translational DOF of the controlled structure and $\ddot{x}_{p50o}$ is the peak acceleration response at the 50^{th} storey of the uncontrolled structure.

$$J_9 = \frac{\max(x_{p1}, \dots, x_{pi})}{x_{p50o}} \quad (7.48)$$

where x_{pi} is the peak displacement response at the i^{th} storey in the translational DOF of the controlled structure and x_{p50o} is the peak displacement response at the 50^{th} storey of the uncontrolled structure.

The data-informed MDOF surrogate model was used to perform the parametric analysis of a 50 storey modular building with MVDTLs using the parameters outlined in Table 7.7. Each combination of parameters was used to build an MDOF model of the 50 storey structure as described in Section 7.3, resulting in 25 different MDOF models of 50 storey modular buildings with different configurations of TLDs. Wind forces with 1, 5, 10 and 50 year return periods as stated in Table 7.7 were used as the exciting forces for each of the 25 MDOF systems with MVDTLs. Each MDOF model was then solved to obtain time histories of the dynamic response of the structure including the displacement (X_N), velocity ($\dot{X}_N$), and the acceleration ($\ddot{X}_N$) responses at each DOF. The RMS acceleration and displacement responses of each of the 25 MDOF models with MVDTLs under the four different wind loading conditions are shown in Figure 7.24.

Examining Figure 7.24, it can be seen that the RMS response is generally lower when all TLDs are distributed at the top storey of the structure. However, the difference in the response is not significant when TLDs are distributed over 1, 6 or 15 storeys for larger values of the mass ratio, μ , particularly in the case of lower wind forces (i.e. shorter return periods). For the return periods of 5 years, it is observed that for the two largest mass ratios studied, $\mu = 0.029$ and 0.038 , the response appears to reduce as the MVDTLs are distributed over the top 15 storeys of the structure, prior to increasing as the number of stories the TLDs are distributed over increases. For the 10 year return period, this relationship is observed for $\mu = 0.029$ but appears to be true for the TLDs distributed over the top 30 storeys for $\mu = 0.038$. Given the MVDTLs have been investigated for control of a 50 storey modular building in the first mode only, where the typical mode shape

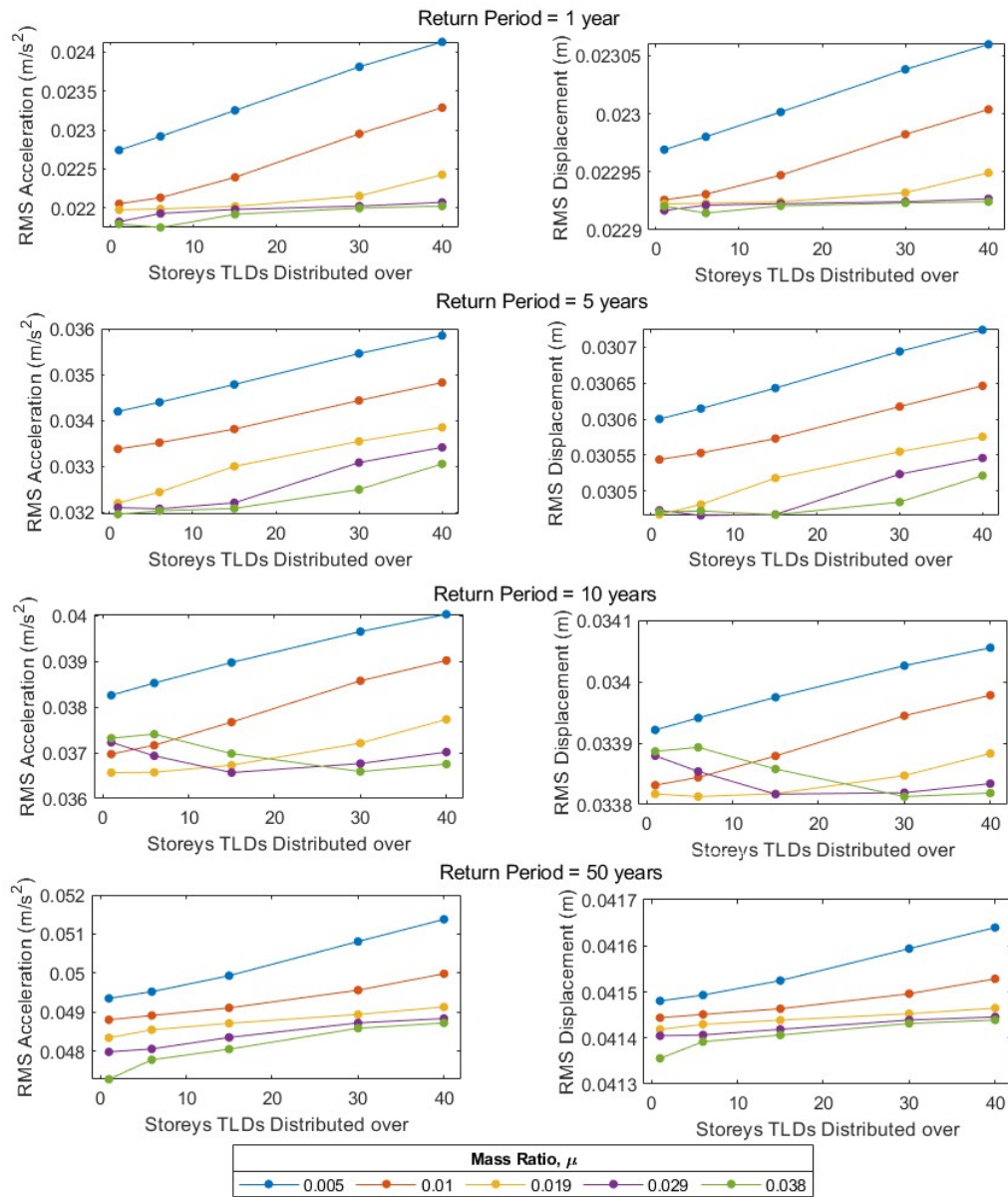


Figure 7.24: RMS acceleration and displacement responses from parametric analysis of MVDTLDS

results in maximum displacements occurring in the top third of the structure, it is a sensible observation that TLDs distributed across approximately the top third of the structure (15 storeys) results in the most effective response reduction. This suggests that there is an optimum mass ratio and vertical distribution of TLDs which relates to the mode shape of the frequency the TLDs have been tuned to control.

Figures 7.25 - 7.30 compare the response of the different configurations of MVDTLs with respect to the evaluation criteria J1-J4, J7 and J9 as outlined in eqs. 7.43 - 7.48. From these figures, it is apparent that the MVDTLs reduce both the RMS acceleration and displacement response of a 50 storey modular structure, regardless of mass ratio, return period or the number of storeys the TLDs are distributed over. However, it is clear that distributing the TLDs over the top 1, 6 and 15 storeys resulted in the most significant reduction in the RMS response, particularly for the higher mass ratios. Again, for $\mu = 0.029$, 0.038 and return periods of 5 and 10 years, it can be seen that distributing the same number of TLDs over the top 15 storeys of a structure can be more effective in controlling the wind-induced response of a high-rise modular building than lumping all TLDs at the top storey. Furthermore, across all return periods and mass ratios, the difference in the RMS response reduction between distributing the TLDs over 1, 6 and 15 storeys is quite small. There is more impact on the response observed for larger mass ratios i.e. when a more significant number of TLDs are implemented than whether the same number of TLDs are located at the top storey vs. over the top 15 storeys.

Considering criteria J7 and J9 shown in Figures 7.29 and 7.30, there is much less of a clear trend. J7 relates to the reduction in the peak acceleration response and it is difficult to identify any strong relationship between wind speed, mass ratio or the storeys TLDs have been distributed over. However, it is clear that the peak acceleration response is reduced in all cases. This is not true for J9 which considers the peak acceleration response. For higher wind speeds the displacement response is seen to increase when TLDs are distributed over more than 15 storeys in the archetype. However, it is difficult to note any other strong relationship. The peak acceleration and displacement response of a building are somewhat random. Without running the same parametric analysis for multiples of the same scenarios and finding the mean of the maximum response it is difficult to draw conclusions about the effects of MVDTLs in the peak response of a modular high-rise structure.

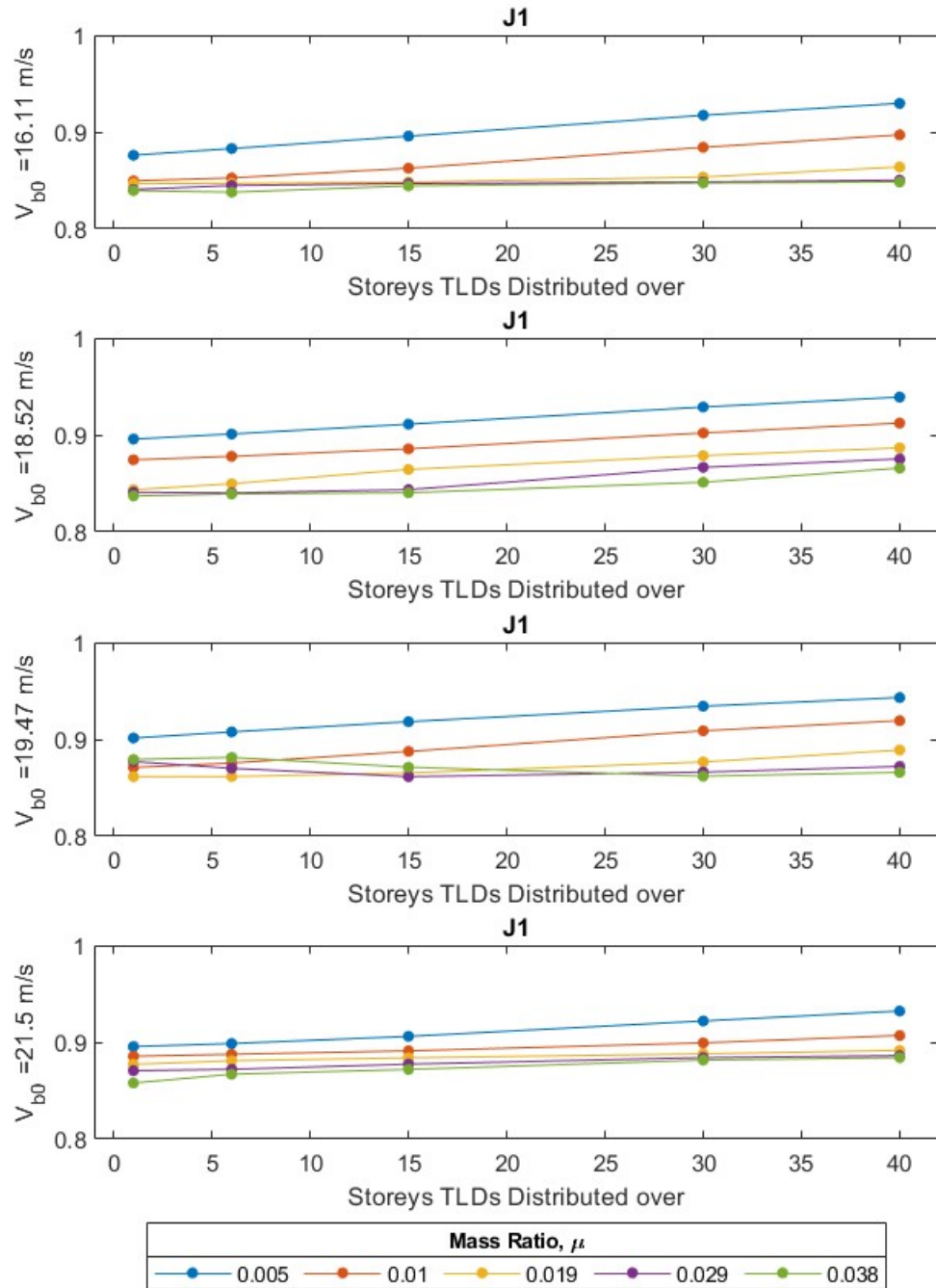


Figure 7.25: Comparison of MVDTLDS considering evaluation criterion J1

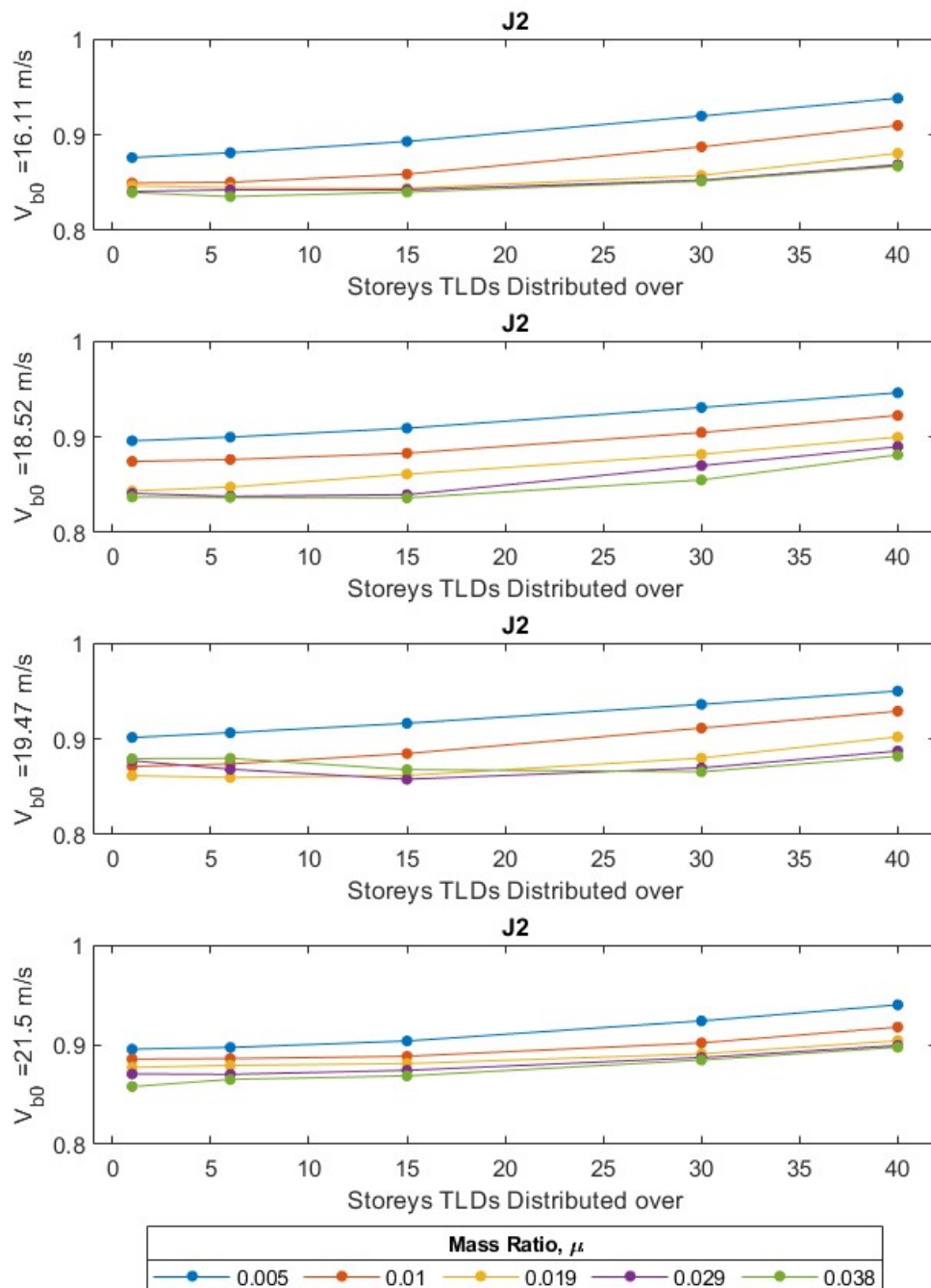


Figure 7.26: Comparison of MVDTLDS considering evaluation criterion J2

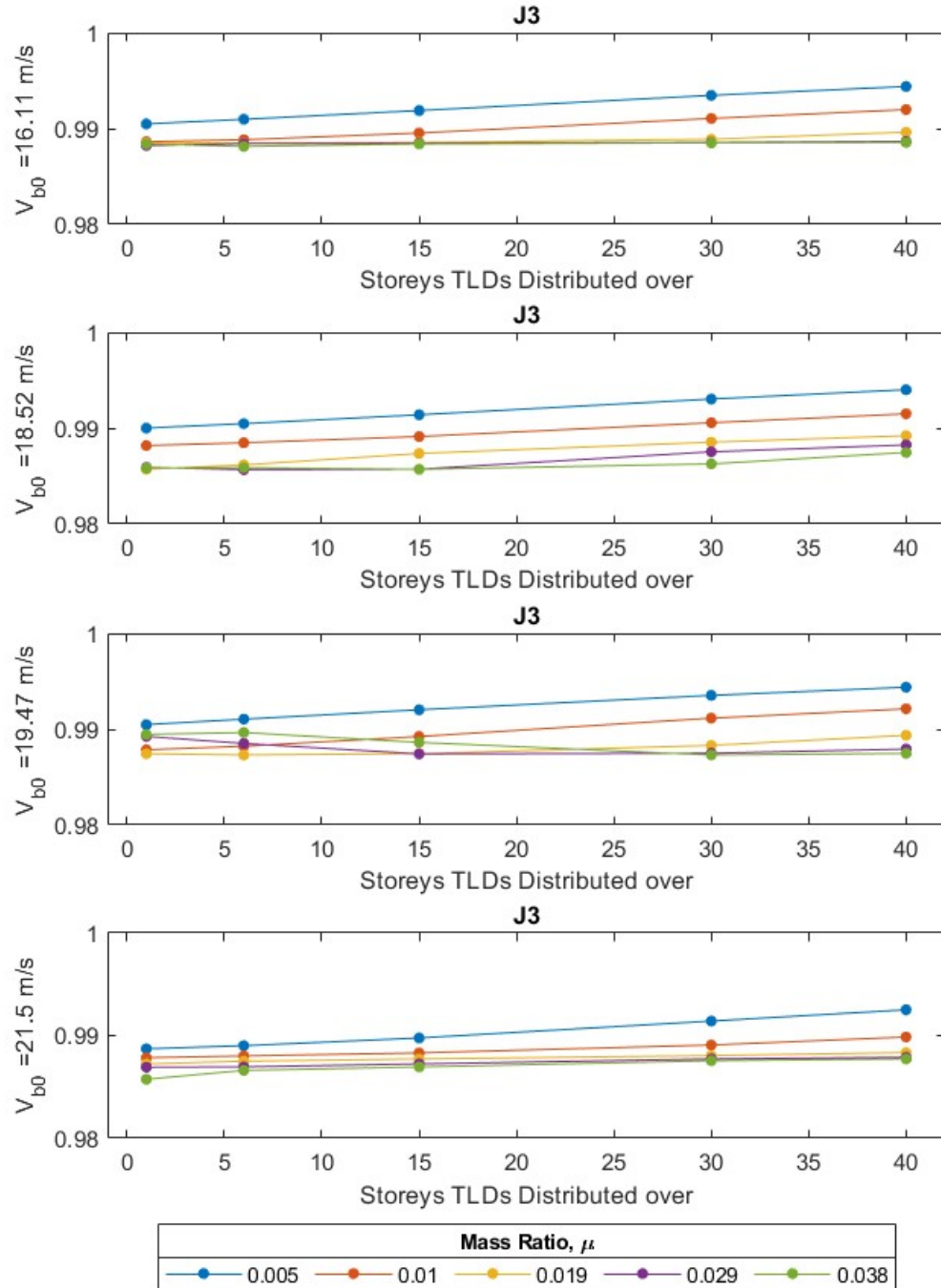


Figure 7.27: Comparison of MVDTLDS considering evaluation criterion J3

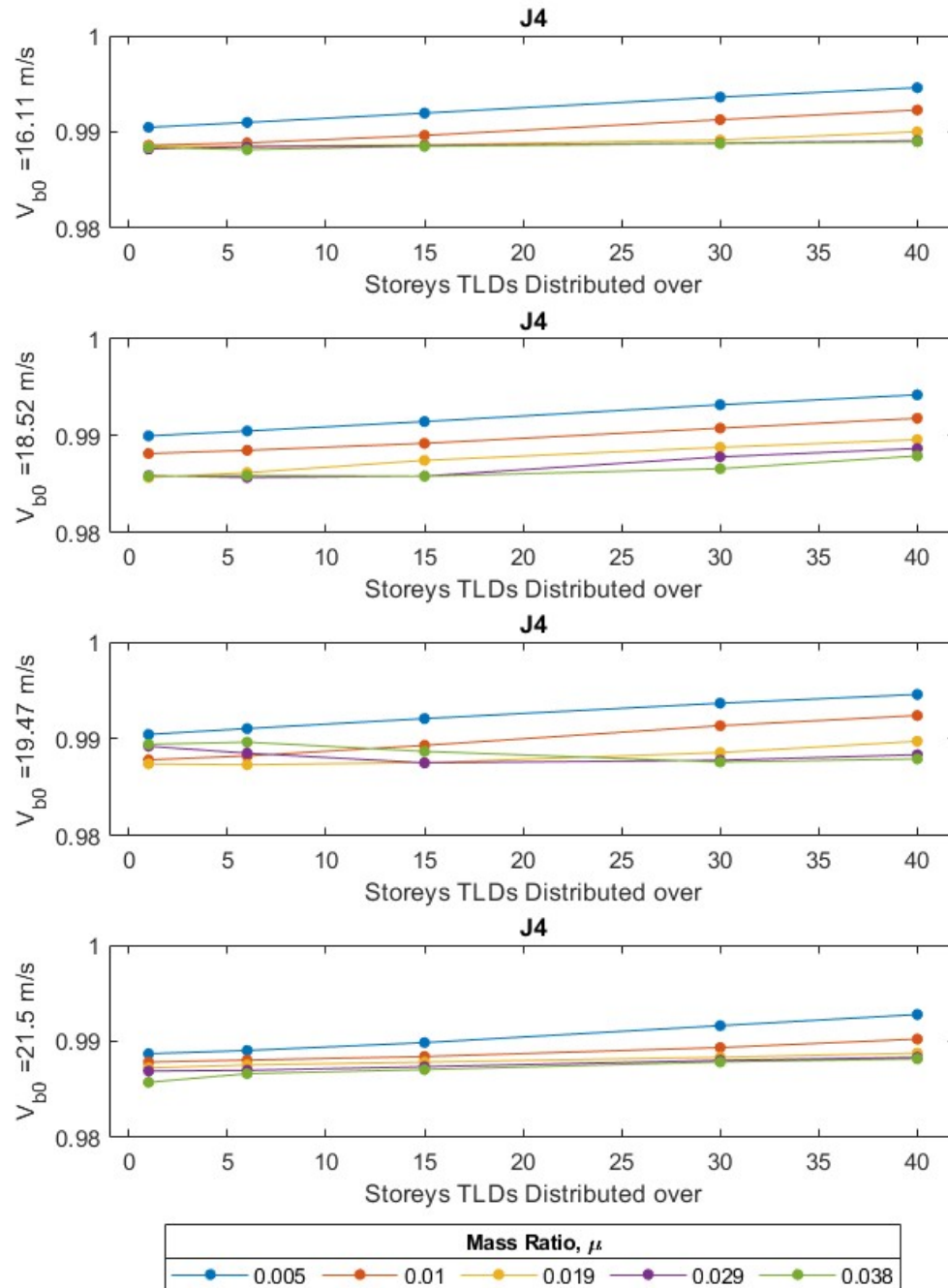


Figure 7.28: Comparison of MVDTLDS considering evaluation criterion J4

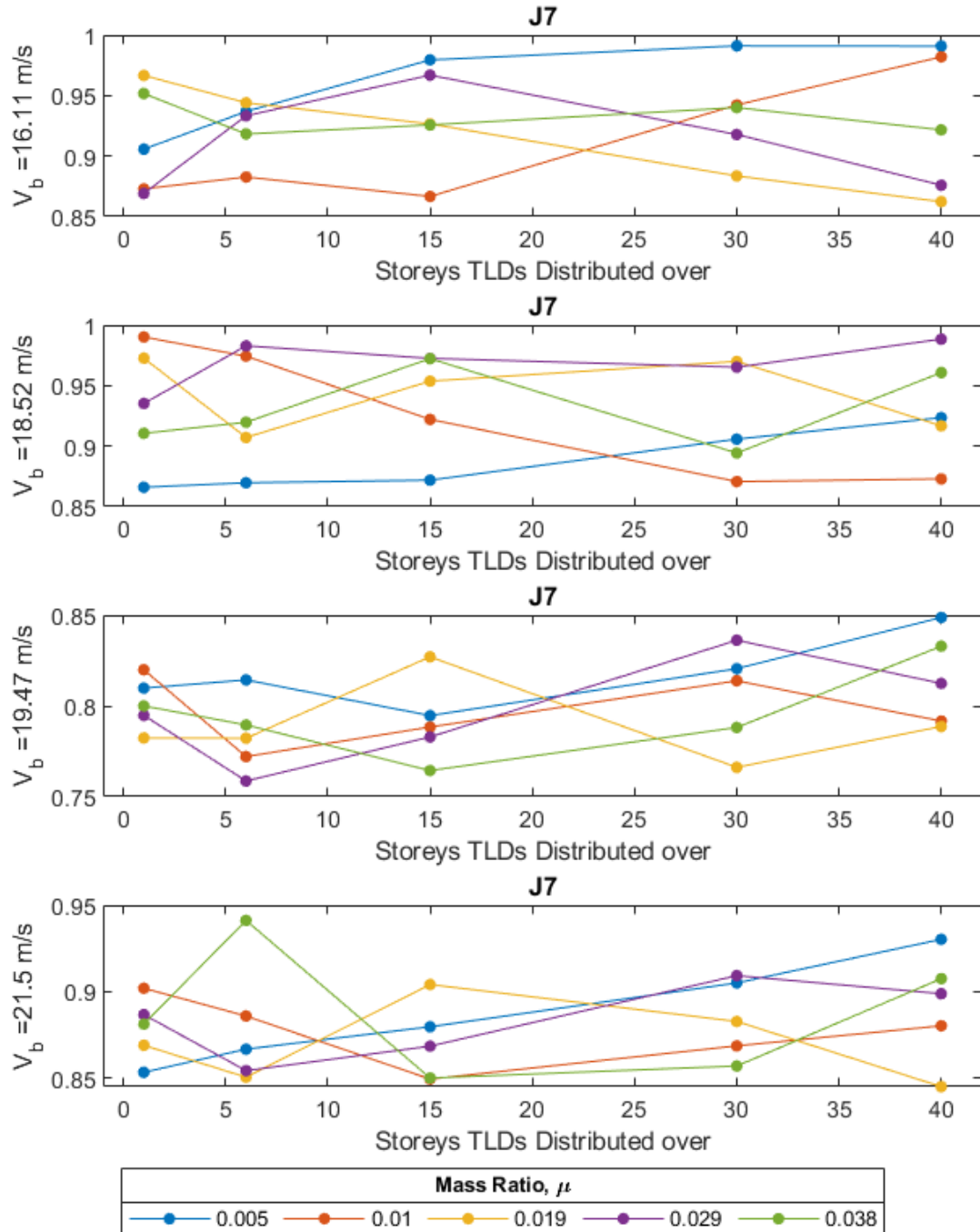


Figure 7.29: Comparison of MVDTLDS considering evaluation criterion J7

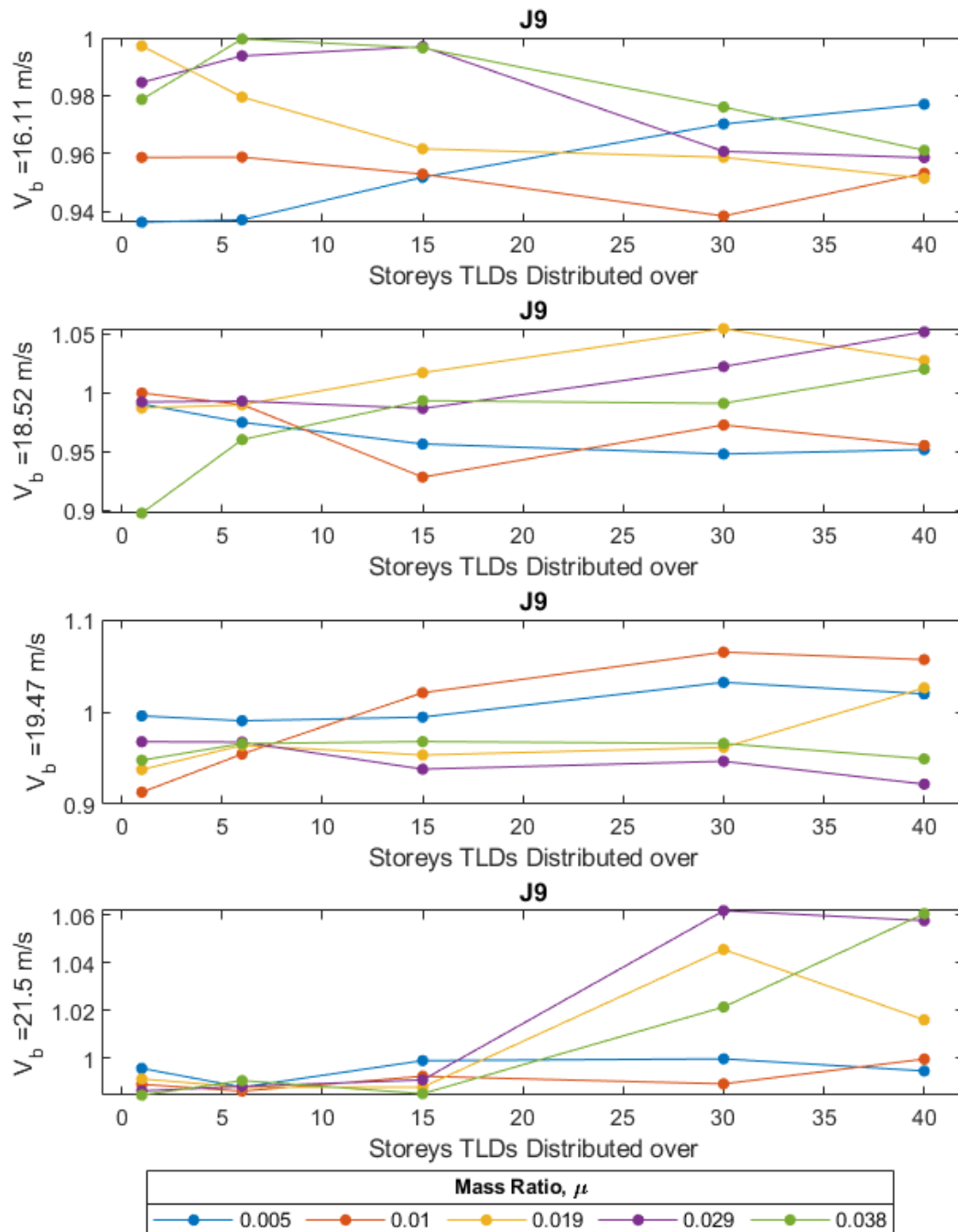


Figure 7.30: Comparison of MVDTLDS considering evaluation criterion J9

The clear trends noted in the RMS response reduction furthers the argument for capitalising on the discretised nature of modular construction in which individual TLDs can be installed in a number of modules distributed throughout a structure rather than occupying valuable real estate at the top storey of a structure. A possible further area of study lies in expanding the parametric assessment to a greater number of archetypes and considering the optimum mass ratio and number of storeys the TLDs are distributed over. This could include distributing TLDs to control multiple modes of vibration and would provide an interesting direction for future research into the control of wind-induced vibrations of modular high-rise buildings.

This parametric analysis was performed using an equivalent TMD model of a TLD suitable for preliminary assessment. Therefore, MVDTLDS are further investigated through Real Time Hybrid Tests (RTHT) performed using a full scale TLD. This is discussed further in Chapter 8. Still, it remains important to note that this parametric assessment has shown that through addition of supplementary damping devices, the response of a modular high-rise building can be reduced. The importance of such a finding indicates that the addition of MVDTLDS can provide leeway for the uncertainty surrounding the inherent properties of modular buildings which influence the dynamic response. If the response reduction provided by auxiliary damping devices, such as MVDTLDS, outweighs the range of possible responses arising from uncertainty in the inherent stiffness or damping ratio of a modular structure, then adherence to habitability requirements can be guaranteed without the need to remove the uncertainty surrounding inherent properties, which in reality will always exist, even for more traditional structures built using steel or concrete. This concept is investigated further in the following section where fragility curves are developed for modular buildings with and without MVDTLDS considering the uncertainty in the stiffness and damping ratio values which have been identified in this thesis.

Figure 7.31 shows one example of the extent of the response of the structure controlled using MVDTLDS versus the uncontrolled structure. The example shown is for 960 TLDs in total; 64 TLDs located at each of the top 15 storeys of the 50 storey modular building, resulting in a mass ratio of 0.038. The force applied to the MDOF surrogate model of the structure with MVDTLDS was equivalent to a wind load with a return period of 1 year and a wind speed of 16.11 m/s.

Considering the ISO RMS acceleration limits for habitability of residential buildings under wind loads of a 1 year return period, for this 50 storey structure with a natural frequency of 0.3203 Hz has a limit of 0.0664 m/s^2 . The RMS acceleration of both the uncontrolled (0.026 m/s^2) and controlled structure (0.022 m/s^2) are well within these limits. Even for the uncontrolled 50 storey structure under wind loading with an equivalent return period of 50 years, the RMS response is within ISO limits. However, it is likely that taller modular buildings will be constructed in the future where this is not the case, and a suitable control strategy will be required. Furthermore, to reiterate, the stiffness values and damping ratios assumed in the MDOF surrogate model used here are not without error. The MVDTLs help to reduce the response effectively, allowing for more certainty that the response of a tall modular building will satisfy habitability requirements.

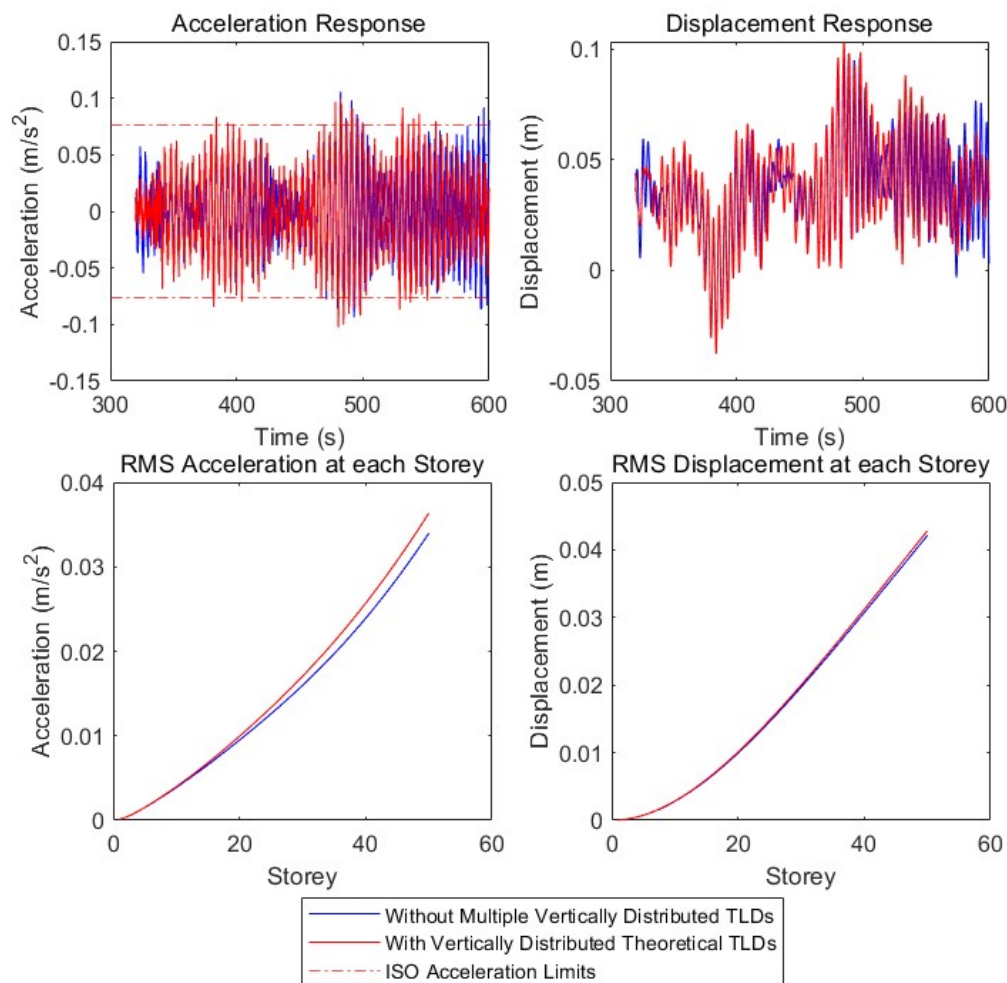


Figure 7.31: Acceleration and displacement response of 50 storey modular archetype with 960 TLDs installed over the top 15 storeys

7.6 Fragility Curves for Modular High-Rise Buildings with Multiple Vertically Distributed Tuned Liquid Dampers under Wind Loading

The data-informed MDOF surrogate model was developed using mass, stiffness and damping values inherent to modular buildings. The mass value used was obtained from the FE models of *Ten Degrees* and *College Road* and it is fair to assume that these estimates of mass are reasonably accurate as mass is a well defined parameter. However, this is not the case for stiffness and damping estimates, in particular for modular buildings. The bending stiffness values used in the data-informed MDOF surrogate model were obtained using results from in situ monitoring of the two case study buildings and a stepped-beam numerical modelling approach as described in Section 6.3.2. However, these estimates of the bending stiffness are not without margins of error which could impact the wind-induced response predicted using the MDOF surrogate model. Similarly, the damping ratios used in the MDOF surrogate model were calculated using OMA methods and the measured in situ response of the two case study structures as described in Chapter 5. The damping ratio of a structure has a significant effect on the dynamic response, and uncertainty in the damping value can lead to questions during the design process of high-rise structures relating to habitability requirements as was seen in the design of *Ten Degrees* and is discussed in Chapter 4.

The margin of error of the estimates of the bending stiffness have not been quantified. In reality, a much more significant data set including more case study structures is required to further characterise the bending stiffness of modular buildings and obtain a reasonable quantitative measure of the error associated with estimates. The OMA methods applied to the measured response of the case study structures not only provided values for the damping ratio, but also provided margins of error of the predicted damping ratios. These margins of error allow for a more informed assessment of the dynamic response of high-rise modular buildings. However, there will always be some level of uncertainty in the wind-induced dynamic response, whether from uncertainty in the bending stiffness and/or inherent damping values, which can not be mitigated. When this uncertainty threatens the performance of high-rise buildings and the adherence to habitability requirements, the most suitable solution is to introduce supplemental damping devices [10, 252].

The added damping which can be provided by auxiliary damping devices, often twice that of the inherent damping of the structure, far outweighs the uncertainty in the inherent damping and dynamic response of a structure.

TLDs are an economical, low maintenance and effective auxiliary damping device [155, 166, 174]. As discussed, the form of volumetric modular construction suits the concept of MVDTLs, and in the previous section a parametric analysis using the data-informed MDOF surrogate model showed that distributing TLDs over the top 1, 6 or 15 storeys of a 50 storey modular building effectively controlled the dynamic response of the high-rise modular building subject to along-wind loading. There are few studies which have investigated the reliability enhancement of structural response with the use of auxiliary damping devices [252]. This is even more apparent for SLS design and improving the reliability of the dynamic performance to meet habitability requirements. With this in mind, fragility curves have been developed for a 50 storey modular building which compare the probability of exceeding ISO 10137 [84] habitability criteria with uncertain levels of bending stiffness and damping ratio with and without MVDTLs. The data-informed MDOF surrogate model is used to assess the dynamic response of the structure both in its uncontrolled state without TLDs and in its controlled state with MVDTLs. The bending stiffness values identified for *Ten Degrees* are varied by $\pm 50\%$ in the MDOF model to achieve different fragility curves. Similarly, the damping ratio value identified as 1.10% for *Ten Degrees* is varied by the identified CV of the estimate of $\pm 2\%$ in the MDOF model. For the controlled structure, 960 TLDs are distributed over the top 6 storeys of the 50 storey modular building, achieving a mass ratio, μ , of 0.038. Details of how the fragility curves were developed are provided below.

7.6.1 Development of Fragility Curves

Fragility curves represent a cumulative probability distribution function of the capacity of a structure to satisfy a particular damage state. In this case, the damage state is the ISO 10137 peak acceleration limits set out for residential buildings. For the 50 storey modular structure which is being studied, this limit is $0.068m/s^2$. The capacity of the 50 storey modular building to satisfy this criteria is investigated for a range of basic wind speeds, vb_0 , between 0 and 60 m/s . This is achieved by applying equivalent wind forces using the data-informed MDOF surrogate model of the 50 storey structure. Each wind speed is simulated 30 times. This process is

7.6. Fragility Curves for Modular High-Rise Buildings with Multiple Vertically Distributed Tuned Liquid Dampers under Wind Loading

carried out for the same 50 storey structure with and without MVDTLs and with bending stiffness $\pm 50\%$ and for the damping ratio $\pm 2\%$. The purpose of this is to demonstrate the improved reliability in structural response when MVDTLs are implemented as discussed.

The fragility function relates the structural response and the limit and provides the exceedance probability of the structure reaching the specified limit at a given hazard intensity measure (IM), which in this case is a basic wind speed. The fragility can be expressed as:

$$F_i(IM) = \Phi \left[\frac{\ln(\widehat{DP}_i) - \ln(dt_i)}{\sigma_{\ln DP, j}} \right] \quad (7.49)$$

where ϕ is the cumulative distribution function, $\widehat{DP}_i$ is the median of the structural response, which can be expressed as a function of IM , dt_i is the demand threshold which in this case is the ISO 10137 acceleration limit and $\sigma_{\ln DP, j}$ is the standard deviation of the natural logarithm of the structural response and can be calculated using eq. 7.50. Eq. 7.49 makes the assumption that the structural response follows a log-normal distribution. This has been shown to be a reasonable assumption by a number of studies [252, 253, 254, 255] and is also assumed true here.

$$\sigma_{\ln DP, j} = \sqrt{\frac{\sum_{i=1}^N [\ln(dt_i) - \ln(\widehat{DP}_i)]^2}{N - 1}} \quad (7.50)$$

where N is the number of simulations carried out.

Figures 7.32 and 7.33 present the calculated fragility curves. Figure 7.32 shows the curve for the uncontrolled and controlled (160 TLDs at each of the top 6 storeys) 50 storey modular structure with the bending stiffness, EI , varied by 50%. It can be seen that the curve for the uncontrolled structure with $EI - 50\%$ have the highest chance of exceeding ISO 10137 acceleration limits. However, the curve for $EI - 50\%$ with MVDTLs shows probabilities of exceedance similar to the uncontrolled structure without modified EI values. This is true across all cases. The controlled modular buildings have similar fragility curves to the uncontrolled

structures with 50% more bending stiffness. This shows the impact that control methods such as MVDTLDS can have on the reliability of a structure conforming to habitability requirements, especially when inherent properties which influence the dynamic response are uncertain.

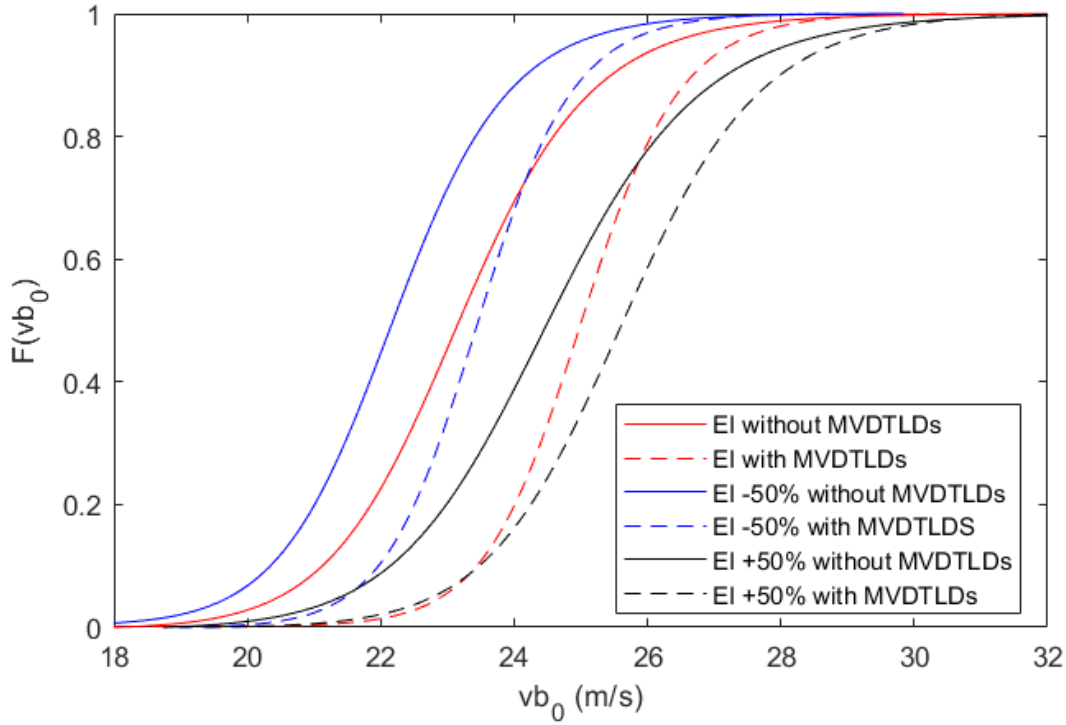


Figure 7.32: Fragility curve for the controlled and uncontrolled 50 storey modular building with varied bending stiffness, EI

Considering Figure 7.33, which shows the fragility curve for the same controlled and uncontrolled modular building with the damping ratio varied by 2%, it can be seen that the curves for the uncontrolled structure lie close together as the variation in damping ratio is not significant. The 2% variation was chosen as it was the error margin determined in Chapter 5 where OMA techniques were used to identify the damping ratio of the case study structures. However, it is evident that the controlled structure with MVDTLDS reduces the probability of failure of the structure regardless of the damping ratio used, hence improving the reliability of the structure. This reemphasises the benefit of including MVDTLDS in modular buildings.

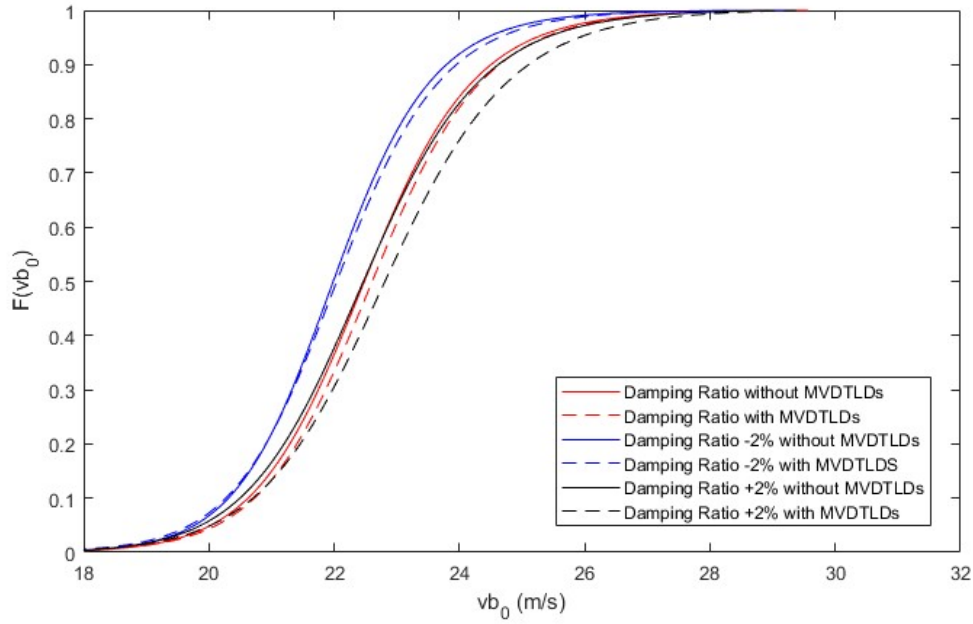


Figure 7.33: Fragility curve for the controlled and uncontrolled 50 storey modular building with varied damping ratio, ξ

As previously stated, empirical studies on the dynamic behaviour of high-rise modular structures are limited. Whilst this thesis has focused on identifying the inherent properties of this novel form of construction, there is still further research and in situ studies required to improve confidence in the identified inherent properties and to better understand the global dynamic response of modular high-rise buildings. However, what is evident is that through the addition of properly tuned MVDTLDS, the uncertainty surrounding the inherent properties and their influence on the dynamic response can be mitigated against. Furthermore, implementing MVDTLDS can improve confidence in the adherence of a high-rise modular building to habitability requirements, allowing this form of construction to continue to be employed in increasingly taller structures.

7.7 Conclusions

The primary focus of this chapter has been to investigate the suitability of MVDTLDS in controlling the wind-induced response of modular high-rise buildings. An equivalent TMD model of a TLD proposed by Tait [155] was incorporated in the data-informed MDOF surrogate model developed and validated in Chapter 6 in or-

der to study the response reduction and reliability enhancement achieved using MVDTLDS. The integrated equivalent TMD MDOF surrogate model was validated using six archetype modular buildings and laboratory tests on a full-scale TLD under both cyclic and random wind-loading conditions. It was found that the equivalent TMD captured the dynamic properties of the full-scale TLD satisfactorily.

Upon validation, the integrated equivalent TMD MDOF surrogate model was used to parametrically study the response reduction achievable using MVDTLDS. A 50 storey archetype modular building was modelled using the MDOF surrogate model and different combinations of MVDTLDS distributed over 1, 6, 15, 30 and 40 storeys were investigated using performance criteria suggested by Yang et al. [120]. In all cases the MVDTLDS reduced the wind-induced acceleration response of the 50 storey archetype modular building when compared to the uncontrolled response. It was found that for the same mass ratio, distributing the TLDs over 1, 6 or 15 storeys did not greatly affect the response reduction and in some cases, for the higher mass ratios, the response reduction was actually improved by distributing the TLDs over 15 storeys. An interesting area for further research could be to identify an optimum mass ratio and distribution of MVDTLDS for controlling the wind-induced response of modular buildings.

MVDTLDS take advantage of the discretised nature of modular construction and offer the opportunity for the further development of a control strategy well suited to modular high-rise buildings which does not involve occupying large quantities of space at the top storey of the structure. Furthermore, including a control strategy like MVDTLDS offers certainty in the dynamic response of a modular building and reliability that habitability requirements will be achieved. Through the use of fragility curves which have been developed considering variation in the inherent properties of modular buildings where uncertainty remains, such as the bending stiffness and the damping ratio, the supplementary damping provided by the MVDTLDS has been shown to outweigh the uncertainty and provide a reliable method for ensuring high-rise modular buildings conform to habitability requirements.

There are some limitations to the results presented in this chapter which must also be discussed. Firstly, the MDOF model incorporating equivalent TMDs representing TLDs proved to underestimate the force response of the system under cyclic loading but overestimate the response under simulated wind loading. This

is likely a result of the equivalent TMD model of the TLD not fully capturing the behaviour of the liquid sloshing within a TLD. Furthermore, the same size full-scale TLD was used for all experimental tests for each archetype. Only the depth of liquid within the TLD was altered to tune the TLD to the respective archetype. Further work could consider multiple full-scale TLDs with optimum dimensions for each archetype building. Additionally, as modular buildings become increasingly taller, they will soon surpass the tallest 60 storey archetype investigated. This study could be expanded to include a larger range of archetype modular buildings as an area for future research.

The findings of the research presented in this chapter show that MVDTLs are an effective method in controlling the wind-induced acceleration response of modular high-rise buildings. This control method is a suitable strategy which complements the discretised nature of modular buildings and allows for multiple TLDs to be dispersed throughout the structure in individual modules rather than occupying valuable floor space at the top of the building. This control strategy is aligned with this modern method of construction, ensuring habitability requirements are satisfied whilst also providing a less obtrusive manner.

Chapter 8

Real Time Hybrid Testing

8.1 Introduction

It would be a lengthy and large scale project to parametrically study the control of high-rise modular buildings controlled by TLDs using full scale tests. Furthermore, to date, there have been no modular buildings with supplementary damping devices constructed. Another method for experimentally performing similar parametric assessment to that carried out in Chapter 7 would be to create many scaled models of the modular structures and TLDs and to perform laboratory tests on a shake table or in a wind tunnel. This would be uneconomical, time-consuming and the scaled models, particularly those of the TLDs, would likely suffer from scaling errors. Real Time Hybrid Testing (RTHT) offers a unique opportunity to test a structure-TLD system, using a full-scale model of a TLD and without the need to create a scaled model of the structure. This provides an alternative option for experimental evaluation and parametric assessment of the ability of TLDs to control the wind-induced vibration response of high-rise modular structures. Moreover, RTHT provides an efficient method for assessing the dynamic response of a modular building equipped with a TLD by using the data-informed MDOF surrogate model developed and validated using results from in situ testing of full-scale modular structures. Essentially, using RTHT, results from the full-scale monitoring of high-rise modular buildings can be combined with the experimental response of a full-scale TLD without the need to physically install multiple TLDs in the modular structures and perform further in situ monitoring.

Many studies, discussed in more detail in Chapter 2, have considered RTHT of structure-TLD systems where the structure is numerically modelled and a physical model of a TLD is used [160, 181, 195, 198, 200, 202, 203]. The motivation for the studies stems from the complexity of the behaviour of the fluid within a TLD and the difficulty in accurately modelling this behaviour. Through RTHT, a full-scale experimental model of a TLD can be created and tested, capturing the actual dynamic response of the supplementary damping device. This dynamic response can be incorporated within a numerical model of a structure using a time-stepping procedure to simulate the controlled response of a structure-TLD system. Hence, allowing for the actual behaviour of a full-scale TLD to be captured without the difficulty which comes with creating scaled models of structures. This concept is known as substructuring and is used to split a system into parts, a part whose behaviour can be modelled numerically with reasonable confidence, and a part with more complex behaviour which requires physical testing.

In this chapter, RTHT has been used to experimentally investigate the ability of multiple TLDs to control the wind-induced acceleration response of a modular high-rise building. At present, the implementation of the RTHT procedure at Trinity College Dublin is limited to updating the force response at a single location in a structure. Therefore, the 50 storey modular building parametrically assessed in Chapter 7 is investigated considering different mass ratios for multiple TLDs all located at the top storey of the modular structure. As with all RTHT, the system is separated into two substructures: a numerical substructure and a physical substructure. The data-informed MDOF surrogate model which was developed in Chapter 6 is used to form the numerical model of the system and a full-scale physical TLD which was tested experimentally in the RTHT testing frame forms the physical substructure. The results from the RTHT are compared to the dynamic response estimated using the MDOF surrogate model and the equivalent TMD model which was discussed in detail in Chapter 7.

8.2 RTHT Procedure and Setup

8.2.1 RTHT Background

The challenges of full scale structural testing have led to the concept of hybrid simulation, which refers to a family of testing techniques including pseudo dynamic

tests [199], substructured tests [256], real time [257] and distributed hybrid tests [258], hybrid shake table testing [259] and effective force tests [260], all of which involve coupled physical-computational simulations of seismic response, in which a part of the structural behaviour is tested physically while other terms are modelled numerically [261, 262].

The Pseudo-Dynamic (PsD) testing method was first implemented by Takahashi et al. [199] and has evolved as an experimental testing method over the last 40 years, the reader is referred to McCrum and Williams for more details [262]. The method involves numerically calculating the dynamic response of a structure via step-wise time integration of the equations of motion, which for a nonlinear system can be expressed as:

$$M\ddot{x}(t_{i+1}) + C\dot{x}(t_{i+1}) + R_{i+1} = F(t_{i+1}) \quad (8.1)$$

where M and C are the mass and damping matrices, $\ddot{x}(t_{i+1})$ and $\dot{x}(t_{i+1})$ are the time dependant acceleration and velocity response vectors, R_{i+1} is the vector of nodal restoring forces and $F(t_{i+1})$ is the applied load vector.

In hybrid simulation, the calculated displacement at each time step are then applied to the test specimen using a displacement-controlled actuator. The experimental force required to impose these displacements, R , are measured and fed back to the test simulation computer for use in the calculation of the displacements in the next time step. Therefore, a hybrid simulation can be thought of as a numerical testing method where the resistance of a structure is measured experimentally [263]. This testing approach overcomes both the difficulty in creating accurate models of complex structural components and enables full-scale testing without the costs and scaling issues associated with shake table testing of complete structural models [262].

The earliest pseudo-dynamic tests were performed in a ramp and hold manner, which is sometimes termed conventional PsD testing. A development of the Conventional PsD method is the Continuous PsD [264] test which sees loads applied smoothly, without the starts and stops associated with conventional PsD testing. PsD tests are performed at an extended timescale, however given sufficient computational power, real time tests are now possible. The first RTHT was performed by Nakashima et al. [257]. These tests differ from the PsD method as the inertia and

damping response of the physical substructure, as well as its stiffness, are measured during the test.

A key development of hybrid simulation is the concept of substructuring, which was first introduced by Dermitzakis and Mahin [256] and has since become a widely used experimental technique (e.g. [265, 266, 267, 268]). In substructured hybrid simulation some parts of the structure, typically critical elements whose behaviour is not easily modelled, are tested experimentally using the hybrid approach, whilst components with more predictable behaviour are modelled numerically. The part of the structure that is physically tested is referred to as the experimental substructure, with the remainder termed the numerical substructure. This substructuring approach allows large structures to be tested physically in a laboratory, and is especially useful when the response of a particular element is difficult to model. The equation of motion for a substructured PsD test can be written with the restoring force split into two components as follows:

$$M\ddot{x}(t_{i+1}) + C\dot{x}(t_{i+1}) + R_{E,i+1} + R_{N,i+1} = F(t_{i+1}) \quad (8.2)$$

where $R_{E,i+1}$ is the restoring force vector for the physical substructure, which is measured experimentally, and $R_{N,i+1}$ is the restoring force vector for the numerical substructure, which is modelled numerically. In the case where the numerical substructure is assumed to behave linearly, eq. 8.2 can be written as:

$$M\ddot{x}(t_{i+1}) + C\dot{x}(t_{i+1}) + R_{E,i+1} + K_N x(t_{i+1}) = F(t_{i+1}) \quad (8.3)$$

Previous research in the field of RTHT has focused on the development of the RTHT procedure. However, more recently the methodology is becoming increasingly popular as a developed research tool in its own right, such as it has been applied in this research.

8.2.2 RTHT Facilities at Trinity College Dublin

The fundamental RTHT testing procedure followed is a state-of-the-art testing procedure as described above. The hybrid testing facilities in the Structural Engineering

Laboratory at Trinity College Dublin were used to perform the RTHT described in this chapter. A number of studies have been performed previously using this testing system, including research which considered steel bracing members [267, 269, 270] and research which investigated the ability of the same full scale TLD tested in this chapter to control the response of wind turbines [173, 198, 200, 207]. The setup at Trinity College Dublin comprises a MTS real-time hybrid test system and is shown in Figure 8.1. The hardware consists of a Series 111 MTS Accumulator and a high speed linear hydraulic actuator with a capacity of 150 kN and a stroke of 250 mm (± 125 mm). The full-scale TLD being tested is suspended from the top of the testing frame and the actuator is attached to the side of the TLD. This forms the physical substructure of the RTHT. The actuator is controlled by the Structural Test System (STS) servo-controller which is operated using a Test PC. The numerical model of the structure is created in Simulink/Matlab using the data-informed MDOF surrogate model of a 50 storey modular structure. This model is compiled using a Simulation PC and then downloaded to the Target PC where real-time simulations are run in the Mathworks xPC Target Environment, forming the numerical substructure of the RTHT. The communication between the Target PC and the MTS controller is facilitated through the Shared Common Random Access Memory Network (SCRAMNet), which is a local high-speed network ring which drastically reduces delays, making real-time simulations possible. SCRAMNet is also used to manage communication between the MTS controller and the actuator.

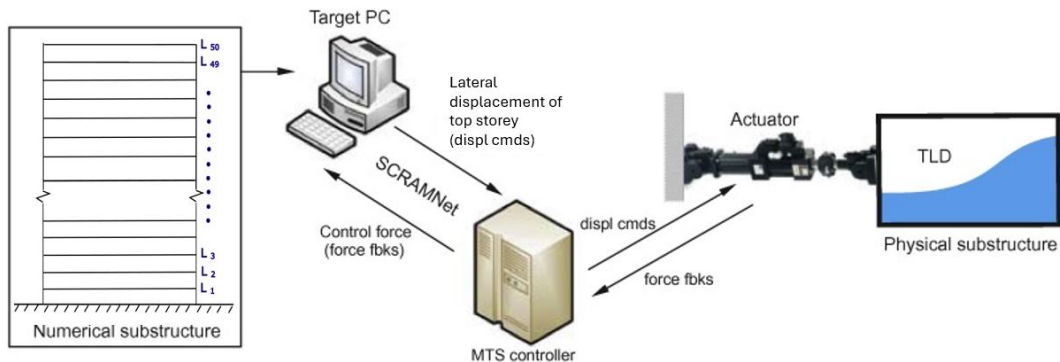


Figure 8.1: RTHT setup at Trinity College Dublin

8.2.3 RTHT procedure

The RTHT procedure relies on the time step method to solve the equation of motion as presented in eqs. 8.1 - 8.3. At each timestep, the numerical substructure is used to calculate the structure displacement. Calculations are performed by the Target PC and sent to the MTS controller. In turn, the MTS controller generates an appropriate signal which is then communicated with the physical substructure via the servo-valve which attempts to move the actuator to the commanded position. The resulting force and displacement of the TLD is measured and passed back to the numerical subsystem, i.e. the Target PC, through the MTS controller. This force response is used in calculating the displacement of the modular building at the next time step and the process is repeated. The integration time step of the numerical substructure is set to 1/1024 seconds in order to equal the sampling time of the MTS controller, enabling synchronisation.

The data-informed MDOF surrogate model described in Chapter 6 has been converted into a Simulink model and is used in calculating the displacement response of the structure at each timestep. The force which is applied to the structure is a time history of a wind force which corresponds to a basic wind speed, $v_{b,0}$ of 16.11 m/s. This wind speed has a typical return period of 1 year for the location of the case study structures in London, United Kingdom. The wind force time history is calculated using the method outlined in Section 7.4.4 with a time step of 1/100 seconds. The Simulink model which represents the 50 storey modular building in the RTHT is shown in Figure 8.2.

The full-scale TLD which is tested during the RTHT is the same as that described in Chapter 7 and is 1.93 m long, 0.59 m wide and has a water depth of 0.15 m. The water depth within the TLD was chosen so as the frequency of the water sloshing in the TLD will match that of the 50 storey modular structure as closely as possible to achieve a tuning ratio, Ω , close to 1. The natural frequency of the 50 storey modular building, obtained using the data-informed MDOF surrogate model, is 0.3203 Hz. The natural frequency of the TLD with a 0.15 m depth of water, calculated using the equivalent TMD formulae proposed by Tait [155] outlined in Section 7.2.2, is 0.3112 Hz, resulting in a tuning ratio of 0.9715. A single TLD of these dimensions would not achieve a sufficient mass ratio, μ , to result in a significant reduction in the dynamic response of a 50 storey modular building. Therefore, multiples of this TLD are used, similar to the parametric analysis of MVDTLDS

performed in Chapter 7. However, for these tests all TLDs are considered to be co-located at the top storey of the modular building. Whilst only one full-scale TLD is physically tested, the measured response of this single TLD is factored by the number of TLDs to be installed at the top storey of the structure to obtain the overall damper force transferred to the primary modular structure. This is done in the Simulink model prior to the next displacement command in the time stepping process being calculated. This demonstrates a significant advantage of RTHT of TLDs; only one full-scale element requires testing, even if multiples of this element are used in the numerical model. A variety of different numbers of TLDs installed at the top storey of the modular structure are investigated in order to achieve a range of mass ratios. The different mass ratios and the number of TLDs required which are investigated by the RTHT are shown in Table 8.1.

Table 8.1: The number of TLDs required to achieve each mass ratio investigated by the RTHT

Mass Ratio, μ	Number of TLDs
0.001	24
0.005	120
0.01	240
0.019	480
0.029	720
0.038	960

8.2.4 Integration Schemes in RTHT

Numerical time stepping techniques are generally used in RTHT. These time stepping techniques rely on integration algorithms to solve the equation of motion representing the test structure at each sequential step. A number of integration methods exist, however, only some are suitable for use in RTHT. Generally, integration algorithms can be separated into two categories; explicit and implicit. Explicit algorithms are those where the solution is dependent only on the response value at the previous step, whereas implicit algorithms calculate the next response value based on the current and previous response values. Early implementations of RTHT favoured explicit integration algorithms as they do not require iterations and are

therefore more computationally efficient. However, explicit integration algorithms are conditionally stable which results in small time steps being required if a structure has a high modal frequency. The size of the time step impacts computational efficiency, with smaller time steps requiring more computational power [271].

Implicit integration algorithms are typically unconditionally stable and this is their primary advantage over explicit algorithms. However, the high number of iterations that is often required to solve nonlinear equations of motion makes this form of algorithm difficult to implement in RTHT. Some iterations may result in load cases which rapidly change direction and may damage the specimen or testing equipment [271]. Typically implicit algorithms continue to iterate until convergence criterion are met as part of a convergence test which can require significant computational demand. However, by specifying a fixed number of iterations per time step this issue can be prevented. Addressing the possibility of rapid changes in loading conditions, Lagrange interpolation can be used to calculate the displacement increments resulting in more regulated changes between time steps [269].

Algorithms have been proposed which split a numerical integration method into implicit and explicit components. These algorithms are known as operator splitting techniques and are unconditionally stable methods which do not require iterations. Operator splitting techniques are implicit for the linear part of the response and explicit for the nonlinear part of the response. However, a disadvantage of these methods is that they do not account for geometric non-linearities in an appropriate manner [271].

The integration method applied in the RTHT reported on in this Chapter is the Backward Euler Method. It is an implicit method derived from the backward difference expression of a derivative. RTHT first emerged for seismic applications, where consideration of strong non-linearities is necessary. However for the study of wind loaded structures, it is generally acceptable to assume linear behaviour. Hence, it is preferred to use an implicit algorithm for RTHT. This guarantees stability without encountering challenges with excessive iteration. The Backward Euler Method solves the differential equation $f(t, y) = \frac{dy}{dt}$ at each timestep as shown in eq. 8.4.

$$y_{k+1} = y_k + hf(t_{k+1}, y_{k+1}) \quad (8.4)$$

where y is the unknown function which depends on the real variable t , and h is the step size. This method was previously used in RTHT on the control of wind turbines using the same prototype full-scale TLD in the Structural Engineering Laboratory at Trinity College Dublin by Zhang et al. [173, 198, 200, 207]. More details on this method are provided by Biswas et al. [272].

8.2.5 Errors in RTHT

There are three main sources of error in RTHT which have been identified by McCrum and Williams [262]; structural idealisations, approximations in numerical integration and experimental errors. Structural idealisation errors are rooted in the lumped mass assumptions which are made in the simplified numerical models necessary to perform RTHT. The actuators in the RTHT setup receive displacement commands for the lumped masses at the DOF of the test specimen. However, this assumes a discrete system and in reality, a structure is a continuous system. Therefore, it is important to ensure the laboratory setup reflects the actual dynamic system being studied in the RTHT. Furthermore, it is necessary to accurately reflect the boundary conditions at the interface of the numerical and physical substructure [262]. Considering the numerical model and structural idealisation, it is important to note that errors may arise in how the structure is represented by the model used in the RTHT. As discussed throughout this thesis, the global dynamic behaviour of modular buildings is incompletely understood. Therefore, despite the work which has been performed to create an accurate data-informed MDOF surrogate model, it is likely that the numerical model still does not completely capture the properties of a modular building, particularly its stiffness and damping. However, the validation which has been performed on the MDOF surrogate model suggests the numerical model offers a fair representation of a modular building similar to the two case study structures, *Ten Degrees* and *College Road*.

Numerical integration is only capable of providing an approximate solution, regardless of which algorithm, time step or computational power is used. However, whilst errors at each time step may be small, they can accumulate to result in a large total error. Therefore, it is important to choose an appropriate integration

algorithm and time step for RTHT.

Experimental errors result from control of the physical substructure. Errors can originate from the displacement control of the hydraulic actuators, force measurement, force relaxation or strain rate effects due to slow rates of testing, calibration errors and from noise generated in the instrumentation and the passing of signals through analogue to digital converters [273].

Another source of error in RTHT occurs when the time step of the numerical substructure is larger than that of the physical substructure. The speed of the numerical substructure is limited by computational power, whereas servo-controllers can run at sampling rates in the order of 1 kHz or above [269]. The discrepancy between the two can be minimised through the use of a predictor-corrector algorithm acting as an intermediate between the two processes [269, 274]. This method generates the displacement command at the rate of the physical substructure by extrapolating between time steps as the numerical substructure completes calculations for the next time step. Once the next time step is calculated the displacement command is passed to the actuator, which moves smoothly to reach the target displacement. The predictor-corrector method ensures the physical substructure receives a constant displacement and the actuator moves smoothly during RTHT. The natural period of the 50 storey modular structure being tested is relatively long compared to other RTHT which have been performed [266, 269, 275]. Therefore, the time steps and test speeds at which the RTHT are required to run at to adequately test the system are not large, giving the numerical model sufficient time to perform iterations. Hence, there is not expected to be large error related to the speed at which the tests can be run.

The most significant error which is likely to be seen from the RTHT performed is related to the accuracy and noise in the force measurements recorded by the load cell. The load cell used for running the RTHT has a capacity of 150kN. The force response expected from the TLD in the RTHT performed are less than 1kN. Therefore, excessive noise or inaccuracies in the recorded measurement are likely to cause error in the results of the RTHT.

8.3 Results from RTHT

RTHT was performed for 15 minutes for each mass ratio in Table 8.1. The first 5 minutes of each test is excluded from analysis to ensure the transient response does not contaminate the results. The acceleration and displacement responses of the controlled and uncontrolled structure are presented in Figure 8.3.

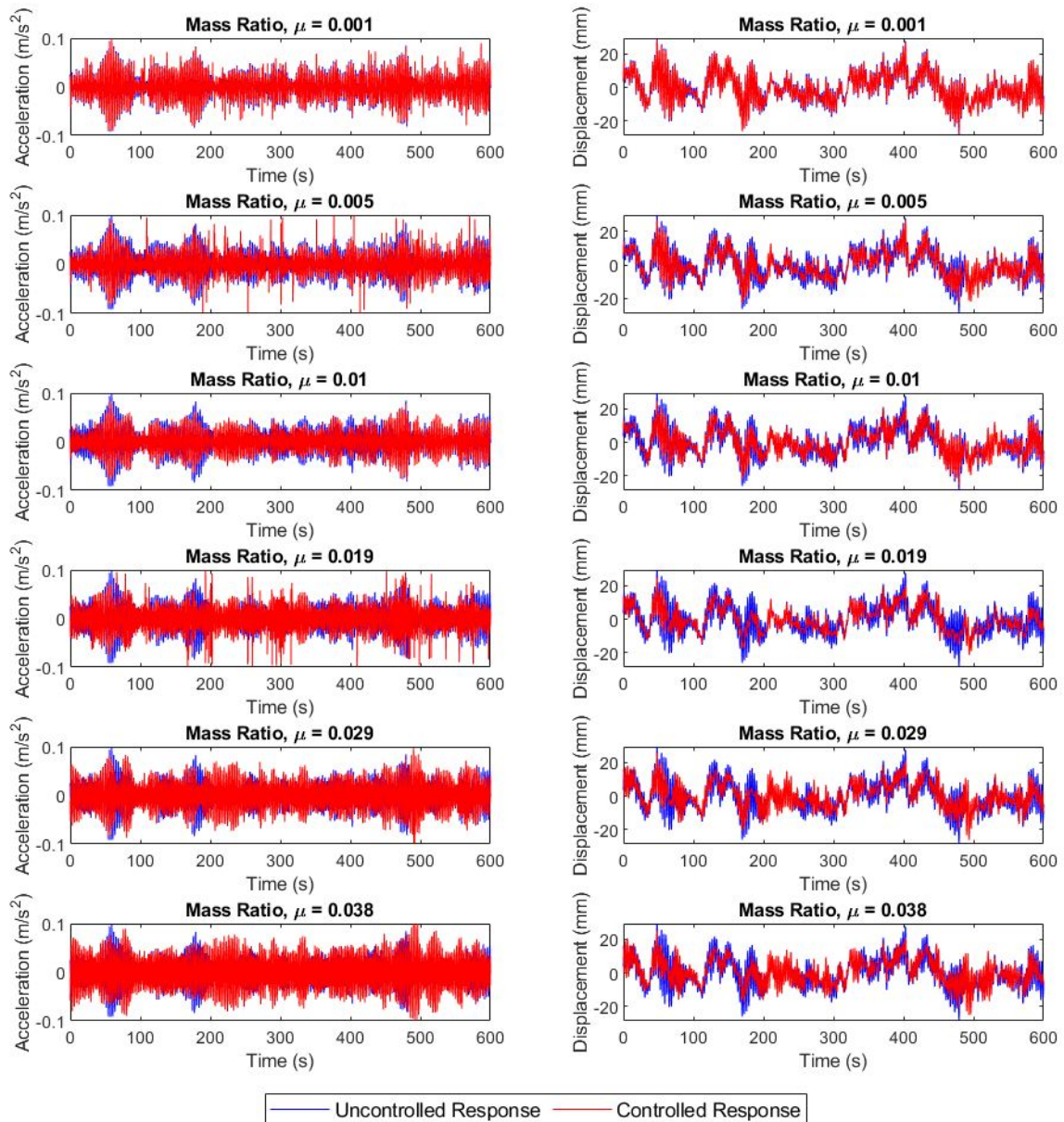


Figure 8.3: Acceleration and displacement responses of the uncontrolled and controlled modular building

From Figure 8.3, it is apparent that across all mass ratios there is a difference in the magnitudes of the controlled and uncontrolled responses. The acceleration response exhibits more noise in the response than the displacement response, particularly for a mass ratio of 0.005 and 0.019. This noise may be due to the displacement response being measured directly whereas the acceleration response was calculated through differentiation of the displacement response.

Visually, it appears that the most significant displacement response reduction was for a mass ratio of 0.019. However, this can be better assessed using the performance criteria J1, J3, J7 and J9 developed by Yang et al. [120] and as outlined in Section 7.5. Criteria J1 and J3 relate to the reduction in the RMS acceleration and displacement responses respectively. Criteria J7 and J9 relate to the reductions in the peak acceleration and displacement response respectively. Figure 8.4 shows the performance criteria for each mass ratio. With regards to the criteria relating to the acceleration response, it can be seen that mass ratios of 0.005, 0.01 and 0.019 gave the largest reductions in RMS response, with reductions of between 20 and 30 percent. A mass ratio of 0.01 gave the largest reduction in peak acceleration response; the reduction was approximately 20%. For both the RMS and peak acceleration response, the reduction in the response reduced for the two largest mass ratios of 0.029 and 0.038.

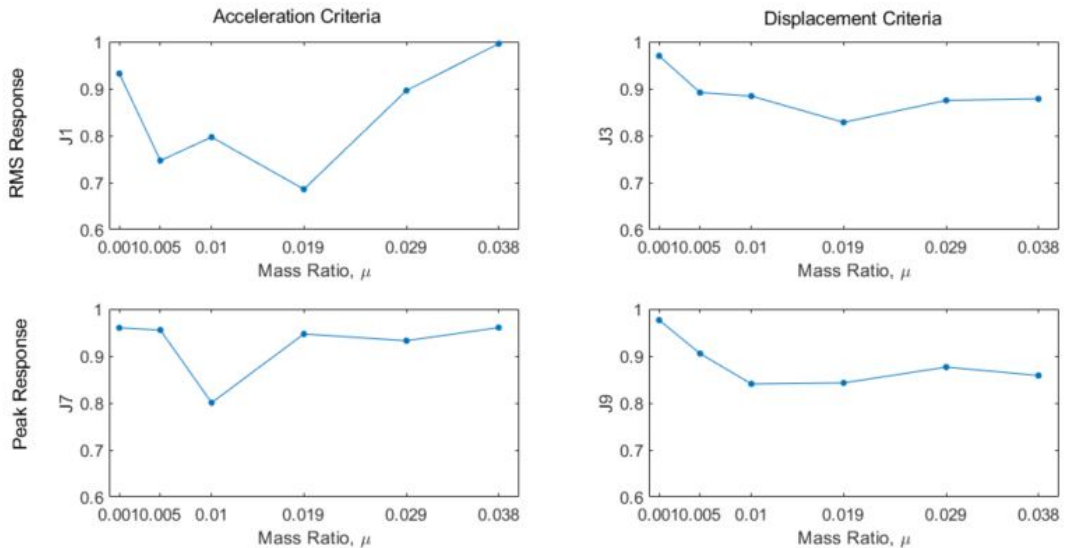


Figure 8.4: Performance criteria for each mass ratio investigated

This trend is more nuanced when the displacement response is considered. For both the peak and RMS displacement response, the reduction in the response of

the controlled structure again appears to be most significant for mass ratios of 0.01, 0.019 and 0.029. The RMS displacement response is reduced most for a mass ratio of 0.019 and then the reduction appears to decrease slightly. For the peak displacement response, the reduction appears to decrease slightly for mass ratios above 0.01. In both cases, reductions in the range of 10 - 15 % are typically observed.

Whilst the results from these RTHT appear to show a mass ratio of 0.01 - 0.02 having the largest impact on response reduction when compared to the uncontrolled structure, further tests and parametric analysis are required to verify this and is an area identified for possible future research. Some of the large differences between mass ratios are likely to be within the overall experimental error. However, what can be concluded from the RTHT is that multiple TLDs installed at the top storey of a modular building appear to reduce the RMS and peak acceleration and displacement response of the structure. Further RTHT considering the control of modular structures of increased heights with uncontrolled responses exceeding ISO 10137 guidelines could provide more evidence that TLDs are a suitable control method for modular buildings.

The force output of the full-scale TLD measured in the RTHTs can also be compared to the force response obtained from the data-informed MDOF surrogate model of a 50 storey modular building with the same number of TLDs installed at the top storey. This comparison is made for each mass ratio for which a RTHT was performed. The force response of the TLD from the MDOF surrogate model is calculated as outlined in Section 7.4.3 using eq. 7.33. The force response measured in the RTHTs from the single full-scale TLD is multiplied by the number of TLDs used to achieve each mass ratio. Figure 8.5 compares the experimental and theoretical force response of the TLDs.

It is apparent from Figure 8.5 that both the theoretical and RTHT force responses of the TLDs are greater for the larger mass ratios, as expected. For lower mass ratios, the magnitudes of the theoretical and experimental force responses are similar, while as the mass ratio increases past $\mu = 0.01$, the theoretical force response of the TLDs begins to exceed the TLD force response from the RTHT. This trend is heightened with each increasing mass ratio. This suggests that there is a difference between the theoretical force response and the experimental force response of the TLD which is amplified when the TLD response is multiplied to represent a significant number of TLDs and achieve a larger mass ratio.

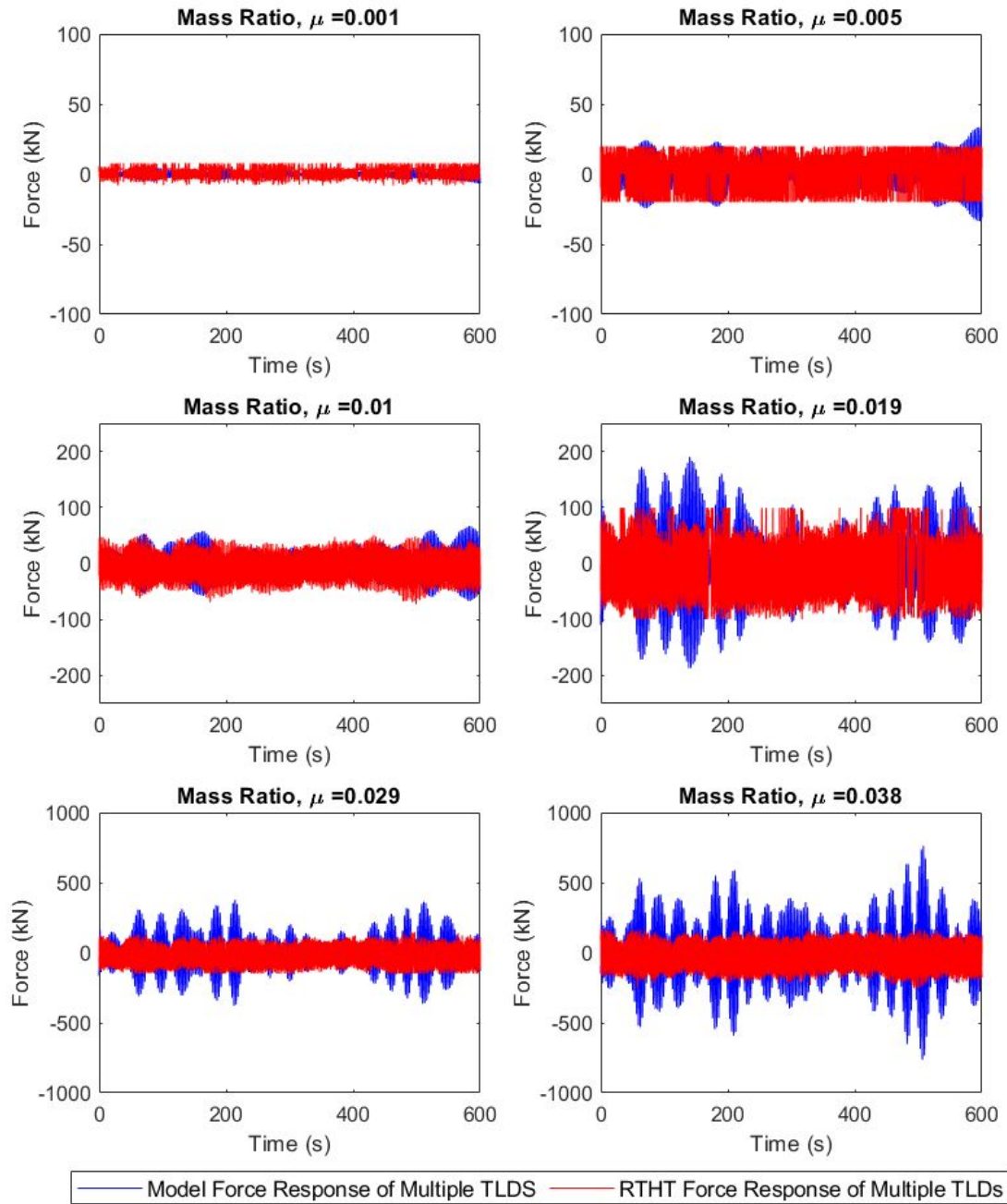


Figure 8.5: Comparison of numerical model and RTHT force responses of multiple TLDs

Figure 8.6 shows the power spectral density of the force response of the TLDs calculated from the theoretical model results and the RTHT measurements for each mass ratio investigated. Reasonable agreement between the theoretical and experimental response is observed, however, a similar trend to Figure 8.5 is identified. The magnitude of the PSD of the TLD force obtained through RTHT appears to drop below the theoretical force response as the mass ratio increases. This disparity is again amplified by the multiplication of the response of a single TLD in both the theoretical model and the RTHT. Furthermore, the PSD of the theoretical response is considerably smoother than that of the experimental response. This suggests that there is noise in the measured experimental response. The noise is likely a result of the non-modal components of the liquid motion. The theoretical equivalent TMD model of a TLD only considers the response of the TLD in one mode, when in reality, the sloshing of the TLD under wind-induced loads will be influenced by other modes. The difference between the theoretical and experimental PSDs demonstrates that the RTHT captures the influence of the non-ideal behaviour of a full-scale TLD which is not encompassed in the theoretical response.

The force response of the single full-scale experimental TLD was recorded during the RTHT at each experimental mass ratio. The equivalent force response of a single TLD in the MDOF surrogate model can be calculated by dividing the total force response obtained for the multiple TLDs by the number of TLDs at each mass ratio. Therefore, a comparison can be made between the experimental and numerical force response of a single TLD for each mass ratio. Figure 8.7 shows this comparison. The response of the single TLD obtained through RTHT is consistent, as is expected. However, the force response of a single TLD appears to increase when calculated using the theoretical model. This suggests an inaccuracy of how the multiple TLDs are modelled in the MDOF surrogate model. It appears that the force response, and therefore the reduction in the dynamic response of a structure estimated using this model is overestimated when a significant number of TLDs are lumped at one DOF. An area of further research would be to investigate if this is also true when TLDs are vertically distributed over a number of storeys in the structure, therefore distributing the TLDs over a number of DOFs rather than lumping them at one.

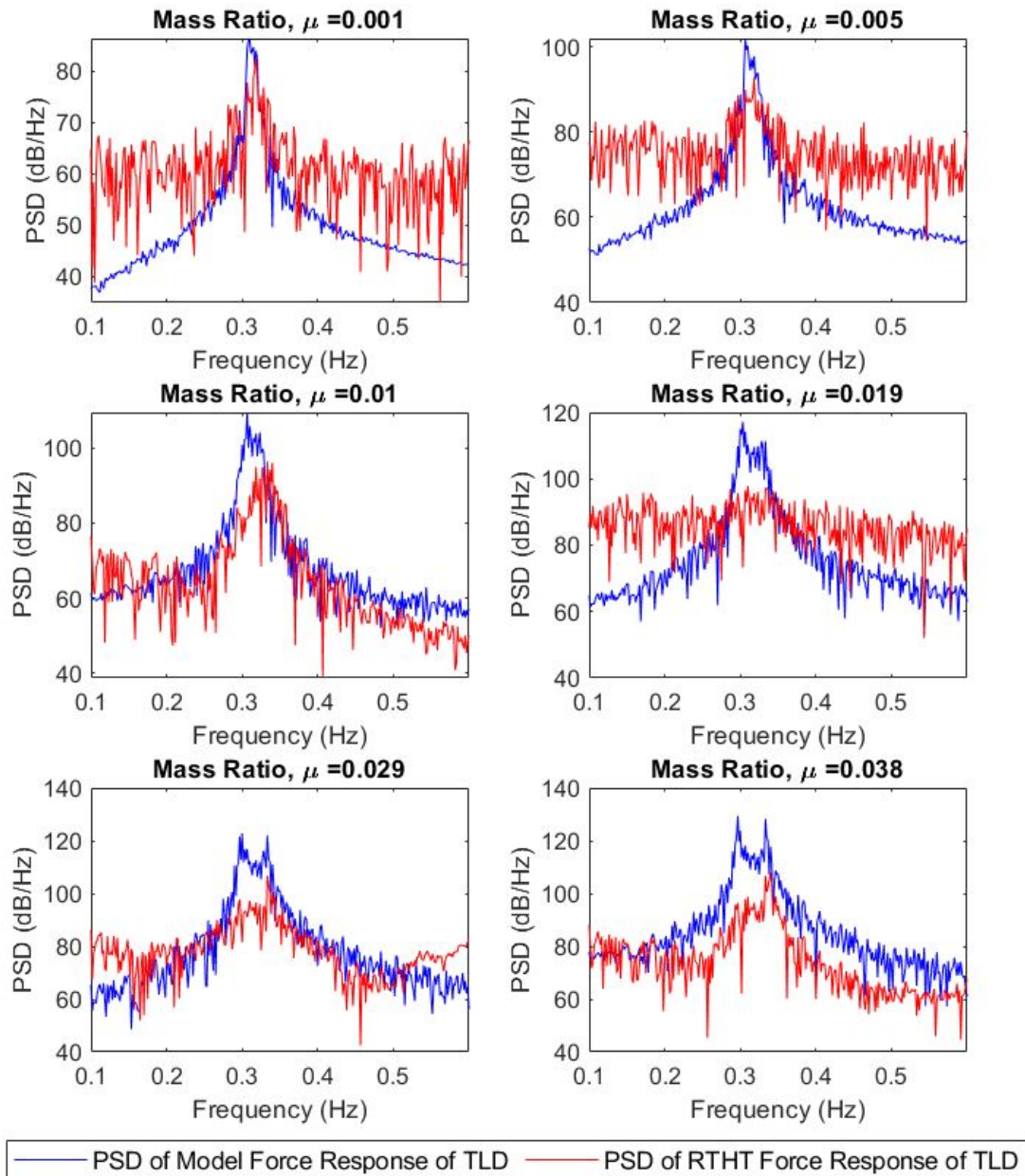


Figure 8.6: PSD of the TLD force calculated using MDOF surrogate model results and RTHT measurements

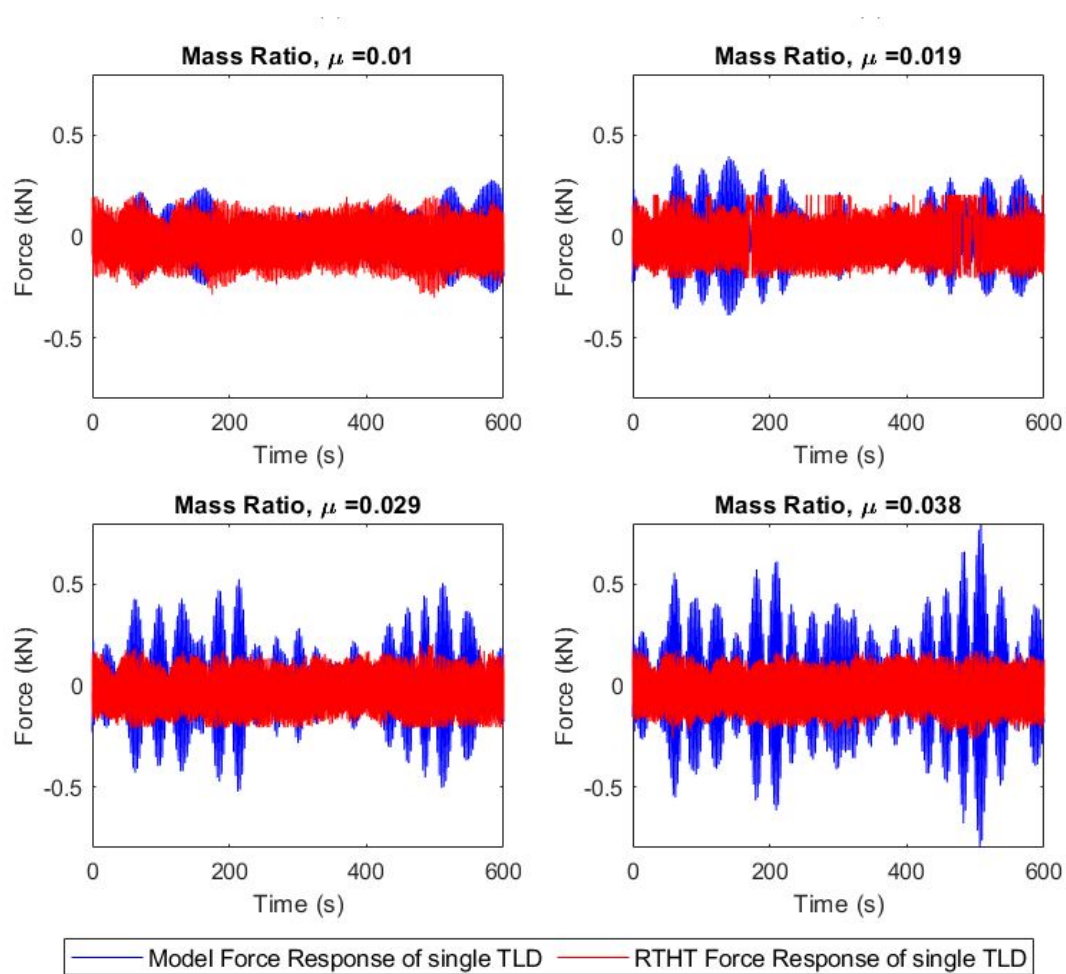


Figure 8.7: Comparison of numerical model and RTHT force response of a single TLD

8.4 Conclusions

This chapter has described a series of RTHT aimed at capturing the real response of a TLD and further assessing if TLDs are a suitable control measure for wind-induced accelerations in tall modular buildings. In order to perform the RTHT, the data-informed MDOF surrogate model developed in Chapter 6 and a full-scale prototype TLD were combined in the RTHT facilities at Trinity College Dublin. Details of the RTHT facilities, procedure and implementation are outlined. The RTHTs were performed considering a 50 storey modular building with multiples of the full-scale TLD employed to achieve six different mass ratios.

The controlled acceleration and displacement response for each mass ratio, obtained from RTHT, has been compared to the uncontrolled response obtained from the MDOF surrogate model. Performance criteria relating to the RMS and peak acceleration and displacements were calculated for each mass ratio. These displayed a trend that using multiple TLDs to achieve mass ratios between 0.01 and 0.02 resulted in the greatest reduction in the response of the modular building when performance criteria are considered.

The force response of the theoretical TLDs calculated using the MDOF surrogate model have also been compared to the force response of the TLDs obtained through RTHT. For lower mass ratios the theoretical and experimental forces appear to be in good agreement. However, as the mass ratio increases the MDOF surrogate model using the equivalent TMD model of a TLD appears to increasingly overestimate the force response of the multiple TLDs. This is also evident in the plots of the power spectral density of the force response. When the theoretical and experimental responses of a single TLD are compared, the experimental force response from the RTHT remains constant, however, the theoretical response of a single TLD is seen to increase in structures with a larger mass ratio. This suggests that the application of the equivalent TMD model of a TLD in the surrogate model is not capturing the force response of multiple TLDs effectively.

A number of areas of further research have been identified in this chapter. Firstly, to investigate the response reduction for a larger number of mass ratios for a range of building heights to identify an optimum mass ratio for controlling the wind-induced response. A significant benefit of RTHT is that these parameters can be easily changed in the numerical model and the only change required to the

physical model being the tuning of the sloshing frequency of the water in the TLD to the natural frequency of the structure by changing the water level.

Another area for further research is to experimentally investigate the vertical distribution of TLDs over a number of storeys. Again, this requires changes to only the numerical substructure, with the same full-scale TLD being used to update each DOF where a TLD is included in the numerical model. If different heights of modular buildings are considered, the depth of water in the TLD can be modified for tuning. This work would help to further validate the parametric assessment of the control of modular structures using MVDTLs outlined in Chapter 7. Furthermore, it may help with identifying the cause of the overestimation of the TLD force response calculated using the equivalent TMD model within the MDOF surrogate model. This discrepancy deserves further consideration in order to ensure that the MDOF surrogate model can be used to examine wind-induced acceleration response control using MVDTLs.

Chapter 9

Conclusions

9.1 Summary of Completed Work

The objective of this research has been to address the knowledge gap surrounding the global dynamic behaviour of high-rise modular buildings and their inherent properties, in order to provide greater confidence that habitability requirements are met by future high-rise structures employing volumetric modular construction. The first step taken was to perform an initial parametric analysis of the wind-induced response of 6125 archetype high-rise modular buildings with RC cores and to investigate the dependency of the response on core and module mass, core stiffness, and inherent damping. The effect of increasing the core breadth and depth on the dynamic response was compared to the effect of installing a TLD. The addition of a TLD was found to cause a more significant reduction in the wind-induced response. However, significant uncertainty around the inherent properties of modular buildings remained and a more in depth analysis was deemed necessary to improve confidence in response predictions

The ambient acceleration responses of two case study modular high-rise structures, *Ten Degrees* and *College Road*, obtained from full-scale monitoring of the structures were analysed in order to identify their inherent properties. The acceleration responses were captured at different stages of construction of the two structures, in total 10 sets of measurements from *Ten Degrees* and eleven sets of measurements from *College Road* were studied. Four different OMA techniques were applied to each set of measurements in order to identify the natural frequency and

damping ratio of the structure. The OMA methods used were the BFFT, RDT, AMD-RDT and a novel bootstrapped AMD-RDT method capable of identifying closely spaced modes and providing a measure of uncertainty of the estimated parameters. This work presented the first estimates of the damping ratios of high-rise modular structures from in-situ testing.

The natural frequencies of the two case study structures estimated using FE models were compared to the actual values calculated from the measured acceleration response of the full-scale structures. It was found that the FE models significantly underestimated the measured natural frequency of the buildings and it was determined that an alternative model for predicting the dynamic response of modular structures was required. Using the natural frequencies at the varying stages of construction of the case study structures and applying a numerical stepped-beam model, an estimate of the global bending stiffness of the modular structures was calculated. The hybrid module-RC core system was found to be up to three times as stiff as the RC core alone, invalidating previous design assumptions in design that the modules provide no additional stiffness. The mass of the modular structures was taken from the FE models of the structures as this is unlikely to differ significantly for the real structures. Using the identified damping ratios, bending stiffness and mass, a data-informed MDOF surrogate model of a modular building was developed which can be used to predict the dynamic response of a modular building of any height, dimensions or stage of construction. This model was validated using the results from full-scale monitoring of the modular buildings.

The data-informed MDOF surrogate model represents a critical tool in the assessment of the dynamic response of a modular building which can be used during the design process to ensure compliance with habitability requirements. However, some level of uncertainty will always surround the inherent properties of a modular building, especially until further full-scale testing and analysis is performed. In order to mitigate this uncertainty, and given the previous parametric assessment which proved the effectiveness of supplementary damping devices, the applicability of TLDs in controlling the wind-induced response of high-rise modular structures was further investigated. To do this, an equivalent TMD model of a TLD proposed by Tait [155] was incorporated in the data-informed MDOF surrogate model, enabling the analysis of any number of any size of TLD be assessed at any DOF of a modular building. The addition of the equivalent TMD model to the MDOF

surrogate model was validated using laboratory tests of a full-scale prototype TLD. Given the factory environment in which volumetric modules are constructed and their itemised nature, it was conceived that a small TLD could be included within the design of a number of modules distributed over a number of storeys of a high-rise structure, rather than occupying significant floor space at the top storey of a building. A parametric assessment was carried out to investigate the effectiveness of Multiple Vertically Distributed Tuned Liquid Dampers (MVDTLs) for wind-induced vibration control of modular buildings considering a 50 storey building, mass ratios ranging from 0.005 to 0.04 and TLDs distributed over 1, 6, 15, 30 and 40 storeys. Distributing TLDs over the top third of the structure was found to offer a noticeable reduction in the wind-induced response. Fragility curves were developed to demonstrate that by providing auxiliary damping, the uncertainty in the inherent properties of a modular building could be overcome to ensure occupant comfort criteria are met.

The sloshing of liquid within TLDs exhibits complex behaviour making modelling of the true force output, and hence vibration control, difficult to achieve. Real Time Hybrid Testing (RTHT) offers the ability to combine the results from full-scale testing of a modular building, by way of the numerical data-informed MDOF surrogate model, with the actual physical response of a prototype full-scale TLD. A number of RTHTs were performed considering multiples of the full-scale TLD to achieve mass ratios ranging from 0.001 - 0.04. The RTHT confirmed the effectiveness of TLDs in controlling the wind-induced response of modular buildings, however, more tests are necessary to investigate a wider range of parameters including MVDTLs.

The work presented by this Thesis offers an initial insight into the global behaviour and inherent properties of high-rise modular buildings. The findings contribute essential knowledge for the future design of modular buildings to occupant comfort criteria, particularly those of increasing height. Furthermore, the assessment of MVDTLs demonstrates how uncertainty in inherent properties can be overcome and how wind-induced vibrations can be controlled effectively whilst considering the nature of modular construction.

9.2 Main Findings

The research outcomes of this thesis can be categorised into four subgroups of main findings:

1. Natural Frequency and Stiffness of Modular Buildings

- OMA of the ambient acceleration responses of *Ten Degrees* showed that the completed structure has a natural frequency of 0.314 Hz in the East-West direction and 0.370 Hz in the North-South Direction. These measured natural frequencies of the structure are significantly larger than estimates from performing modal analysis of FE models in ETABs software.
- OMA of the ambient acceleration responses of *College Road* calculated the natural frequency of the modular building as 0.248 Hz in the East-West direction and 0.350 Hz in the North-South direction. These values were larger than estimates from an FE model of the structure in ETABs software.
- Considering the natural frequencies of *Ten Degrees* measured as the project progressed through construction from 32 storeys to 44 storeys of modules installed, the natural frequency was observed to decrease. The opposite trend was observed for *College Road* where OMA of the acceleration response of the structure showed an increase in natural frequency as the number of storeys with modules installed rose from 0 to 24. These trends indicate that the stiffness contribution of the modules to the global dynamic response is significant, particularly for modules added at lower storeys. However, at higher storeys, the additional mass provided by modules is more significant than their contribution to bending stiffness.
- Using the natural frequencies of the case study structures measured at different stages of completion and a stepped beam numerical model, the bending stiffnesses of the RC core and the hybrid module-RC core system was calculated for both structures. For *Ten Degrees* the RC core was found to have a bending stiffness ($E_c I_c$) of $7.98 \times 10^{12} \text{ Nm}^2$ in the East-West direction and $1.29 \times 10^{13} \text{ Nm}^2$ in the North-South direction. The equivalent bending stiffness of the combined modules-RC core system ($E_{cm} I_{cm}$) was calculated as $1.99 \times 10^{13} \text{ Nm}^2$ and $3.80 \times 10^{13} \text{ Nm}^2$ in the East-West and North-South directions respectively. In the East-West direction, the module-RC core system was

2.9 times as stiff as the RC core alone. This value was 3.4 in the North-South direction. Similarly, for *College Road* $E_c I_c$ was calculated as $1.69 \times 10^{13} \text{ Nm}^2$ in the East-West direction and $2.93 \times 10^{13} \text{ Nm}^2$ in the North-South direction. $E_{cm} I_{cm}$ was calculated as $2.72 \times 10^{13} \text{ Nm}^2$ and $1.05 \times 10^{14} \text{ Nm}^2$ in the East-West and North-South directions. This results in the ratio of the bending stiffness of the combined module-RC core system to that of the RC core alone being 1.5 in the East-West direction and 3.5 in the North-South direction. The variation in the bending stiffness in the two directions is likely related to the dimensions of the breadth and depth of the two buildings.

2. Damping Ratio of Modular Buildings

- Furthermore, reports of full-scale in-situ testing of modular buildings are limited and to the best of the author's knowledge, there have been no previously reported values of the inherent damping of modular structures.
- Through OMA, the damping ratio of *Ten Degrees* in its first mode was estimated as approximately $1.10\% \pm 0.02$ in the East-West direction and $1.14\% \pm 0.03$ in the North South Direction. These estimates are close to the Eurocode 1991-1-4 recommended value for damping in composite structures of 1.27%.
- The damping ratio of *College Road* in its first mode was estimated as $0.72\% \pm 0.05$ in the East-West direction and $1.11\% \pm 0.05$ in the North South direction from in situ testing and OMA techniques.

3. Modelling of Modular Buildings

- Through literature review it was discovered that design and analysis of modular structures relies on FE software developed for traditional forms of construction. No specific software exists for modular construction, resulting in difficulty modelling the bespoke connections and global behaviour of modular buildings.
- Modular buildings contain significantly more members than structures designed using traditional methods. This results in large, cumbersome and computationally expensive FE models which take significant amounts of time to perform modal analysis.

- The natural frequencies of the two case study structures were underestimated by FE models of the structures created in ETABs software. Estimates of the natural frequency of *Ten Degrees* from modal analysis using the ETABs model were 0.18 Hz and 0.19 Hz in the East-West and North-South direction respectively. Similar estimates for *College Road* were 0.18 Hz in the East-West direction and 0.20 Hz in the North-South direction. The error in the estimates using the FE models ranged from 27% to 49%.
- The data-informed MDOF surrogate model developed using the identified damping ratios and bending stiffness values as stated above and the mass calculated using the FE models was found to accurately predict the natural frequencies of the two case study modular buildings. The MDOF surrogate model calculates the natural frequency of a modular building with similar dimensions and height to *Ten Degrees* as 0.297 Hz and 0.410 Hz in East-West and North-South directions. For a structure similar to the height and dimensions of *College Road* these values are 0.272 Hz and 0.376 Hz respectively. These estimates are significantly closer to the measured values noted above than FE models of the two structures.

4. Acceleration Response Control of Modular Buildings

- Through parametric analysis of the Eurocode 1991-1-4 calculated acceleration response of 6,125 archetype modular buildings with RC cores it was found that increasing the dimension of the depth of the RC core by 1000 mm will achieve an approximate 10% reduction in the peak acceleration response.
- This parametric assessment also showed that increasing the depth of the RC core by 2100 mm has the same effect on acceleration response as a 1% increase in damping ratio
- This indicated that inclusion of a supplementary damping device, such as a TLD, was more effective in reducing the peak wind-induced acceleration response of the archetype modular structures than altering the dimensions of the RC core.
- Laboratory tests of a prototype full-scale TLD validated an equivalent TMD model of a TLD which was included in the data-informed MDOF surrogate

model for the assessment of response control of a modular building using TLDs.

- Parametric analysis of a 50 storey modular building with MVDTLs evaluated using performance criteria showed that the vertical distribution of TLDs over the top 15 storeys of the structure could reduce the RMS acceleration response by as much as 15%. Distributing TLDs over more than the top third of the structure was found to have adverse effects on the acceleration and displacement response for each mass ratio and wind speed studied.
- Fragility curves were used to demonstrate that supplementary damping, in the form of MVDTLs, could overcome challenges presented by uncertainty in the inherent properties of modular buildings in ensuring that habitability requirements are met. A 50 storey modular building with MVDTLs was shown to have a fragility curve with performance similar to the same modular building with a bending stiffness of +50% with no MVDTLs. Similarly, a modular building with MVDTLs was shown to perform to the same extent as the same structure with a 2% increase in inherent damping and without MVDTLs.
- RTHT using the data-informed MDOF surrogate model as a numerical representation of a 50 storey modular building and a prototype full-scale TLD showed that multiple TLDs located at the top storey of the structure were capable of reducing the wind-induced response of the structure for a range of mass ratios.
- The results from the RTHT showed that the equivalent TMD model of the TLD overestimated the force output of the TLD more significantly when a larger number of multiple TLDs were used. This suggests propagation of error in the MDOF surrogate model employing the equivalent TMD model.

9.3 Suggestions for Future Work

OMA of the measured ambient acceleration responses of the two case study structures *Ten Degrees* and *College Road* provided valuable insight into the dynamic response and inherent properties of high-rise volumetric modular buildings with RC cores. Whilst ten time segments of measured data each 12 hours in length were available for *Ten Degrees*, two five month periods of near continuous recordings were available for *College Road*. The eleven most significant one hour long wind events from these measurements were processed using OMA techniques, however, considering the significant amount of acceleration responses recorded it would be worthwhile to process more and longer segments of the response. Furthermore, only the first mode in the East-West and North-South directions were identified from the acceleration responses. Future work could benefit from assessing more modes of vibration, particularly torsional modes and their contribution to the dynamic response.

The stepped beam numerical model was applied to the natural frequencies calculated using OMA of the earliest and latest captured responses of the modular buildings only. This was due to the significant time and computational power required to solve the numerical model. An area of future research could address further validation of the identified bending stiffness values by applying the stepped beam model to more of the identified natural frequencies of the modular buildings at differing stages of completion. Moreover, further application of the stepped beam model to more results could provide an insight into the level of uncertainty surrounding the estimates of the bending stiffness presented.

The insight into the dynamic behaviour of modular buildings provided by this thesis is an initial step towards addressing the lack of research surrounding the inherent properties of modular buildings. Recommended values of damping ratios and bending stiffnesses for other forms of construction are based on large databases of empirical tests and results. Further testing and in situ monitoring of many more modular buildings is necessary to build a suitable database of the inherent properties of this novel form of construction. Moreover, *Ten Degrees* and *College Road* were designed and built using the same structural system by the same team of architects, engineers, module fabricators and contractors. This represents only one form of volumetric modular construction and connection systems when in reality

many forms exist. It is necessary to study a range of modular building systems in order to provide more insight into the dynamic performance.

The data-informed MDOF surrogate model provides an estimate of the along wind-induced response of modular buildings considering different dimensions, heights and stages of completion. The acceleration response is calculated using the Eurocode 1991-1-4 recommended Kaimal Spectrum for along-wind excitation of structures. However, it is often the across-wind response of tall, flexible structures which induces a more significant vibration response. Including an across-wind spectrum and forces in the MDOF surrogate model and performing further validation and parametric assessment of the conformity of modular buildings to habitability criteria would provide a suitable direction for further research.

The parametric assessment of the control of modular buildings using MVDTLs considered a 50 storey modular structure, similar to *College Road*. The acceleration response of the 50 storey building to a wind load with a return period of 1 year calculated using the MDOF surrogate model met ISO 10137 criteria for occupant comfort of residential structures. Future research could consider a taller modular building which is at risk of exceeding the habitability criteria and the degree to which MVDTLs can minimise the response to conform to ISO 10137.

Additionally, the fragility curves presented could be expanded to include a greater range of building heights and more varied levels of uncertainty. The fragility curves are a simple preliminary tool which can be used in the early stages of structural design to determine how susceptible a structure is to wind-induced vibrations and the exceedance of habitability requirements, enabling design engineers to make more informed decisions earlier in the design process.

The number of RTHTs performed in this thesis is small and only considered multiple TLDs distributed over one storey. There is significant scope to expand the number of mass ratios investigated by varying the number of TLDs in the numerical model. Furthermore, the Simulink numerical model could be modified to allow for the measured response of the full-scale TLD tested to represent the response of multiple TLDs included over a number of storeys. This would further the assessment of MVDTLs in the control of the wind-induced response of modular structures.

Lastly, the equivalent TMD model of a TLD was found to overestimate the response of a TLD increasingly when multiple TLDs were considered by the MDOF surrogate model. The error in this model and its shortcomings should be considered and improved upon to enable more accurate assessment of the control of wind-induced acceleration responses of modular buildings using the data-informed MDOF surrogate model.

Appendix A

Validation of Identified Bending Stiffness Contribution of Modules

It is important that the identified bending stiffness values of the modular buildings are validated prior to their use in further models. This can be achieved through estimating a range of possible values for $E_{cm}I_{cm}$ and E_cI_c given approximate calculations of the second moment of area, I , of the structures and the typical Young's Modulus, E , of steel and concrete.

The second moment of area of a rectangular section can be calculated using eq. A.1.

$$I = \frac{bd^3}{12} \quad (\text{A.1})$$

For *Ten Degrees* the two RC cores measure 8 x 7 m and 9 x 8 m with walls 450 mm deep. Therefore in the X-X direction, I_c can be roughly estimated as:

$$(I_c)_{XX} = \left(\frac{8 \times 7^3}{12} - \frac{7.55 \times 6.55^3}{12} \right) + \left(\frac{9 \times 8^3}{12} - \frac{8.55 \times 7.55^3}{12} \right) = 129m^4 = 129 \times 10^{12}mm^4 \quad (\text{A.2})$$

I_c in the Y-Y direction is:

$$(I_c)_{YY} = \left(\frac{7 \times 8^3}{12} - \frac{6.55 \times 7.55^3}{12} \right) + \left(\frac{8 \times 9^3}{12} - \frac{7.55 \times 8.55^3}{12} \right) = 156.5m^4 = 156.5 \times 10^{12}mm^4 \quad (A.3)$$

Taking a Young's modulus of 33,000 MPa, typical of highly reinforced cracked concrete, $(E_c I_c)_{XX}$ becomes 4.26×10^{12} and $(E_c I_c)_{YY}$ becomes 5.16×10^{12} . These values are to the same order of magnitude as those estimated using the stepped beam numerical model and presented in Table 6.9. The values estimated from the stepped beam model are slightly higher, this is likely due to the fact that the values calculated here are approximations which do not account for internal core walls and stair wells within the RC cores.

Considering the entire completed structure of *Ten Degrees*, I_{cm} can also be approximated in the X-X and Y-Y directions using eq. A.1, the estimated value of I_c and the dimensions of a typical column section used in the modules. A SHHF 200x200x12.5 column section with $I_{XX} = I_{YY} = 4.98 \times 10^7 mm^4$ is the typical section size representative of most columns in the volumetric modules. There are four corner post columns in each module and 38 modules per storey. Therefore the calculation of I_{cm} becomes:

$$(I_{cm})_{XX} = 38 \times 4 \times 4.98 \times 10^7 mm^4 + 129 \times 10^{12}mm^4 = 1.29 \times 10^{14}mm^4 \quad (A.4)$$

$$(I_{cm})_{YY} = 38 \times 4 \times 4.98 \times 10^7 mm^4 + 156.5 \times 10^{12}mm^4 = 1.56 \times 10^{14}mm^4 \quad (A.5)$$

The aim of the stepped beam numerical model is to quantify $E_{cm} I_{cm}$, it is an inherent property of modular buildings which has yet to be reported. Therefore, to approximate $E_{cm} I_{cm}$, a Young's modulus typical of RC concrete, 33,000 MPa, and steel, 215 GPa may be considered. Using these values and multiplying them by the I value of the steel and concrete sections which comprise $E_{cm} I_{cm}$, values of $(E_{cm} I_{cm})_{XX} = 4.46 \times 10^{15}$ and $(E_{cm} I_{cm})_{YY} = 6.79 \times 10^{15}$ are calculated. These

estimates are slightly larger than those approximated using the stepped beam numerical method. However, the order of magnitude of the stepped beam estimates are sufficiently similar to these approximations to validate the suitability of the stepped beam numerical model in identifying the stiffness contribution of the modules to the global stiffness of a modular building.

A similar process can be carried out to approximate the $E_c I_c$ and $E_{cm} I_{cm}$ for *College Road*. A SHHF 200x200x12.5 with $I_{XX} = I_{YY} = 4.98 \times 10^7 \text{ mm}^4$ is again the typical section size representative of most columns in the volumetric modules of *College Road* also. However, there are 43 modules per storey in *College Road*, each with four corner post columns. The approximations of $E_c I_c$ and $E_{cm} I_{cm}$ are provided in Table A.1.

Table A.1: Approximated $E_c I_c$ and $E_{cm} I_{cm}$ values for *College Road*

	X-X	Y-Y
$E_c I_c$	8.24×10^{12}	8.75×10^{12}
$E_{cm} I_{cm}$	1.84×10^{15}	1.84×10^{15}

The values of $E_c I_c$ and $E_{cm} I_{cm}$ approximated here for both *Ten Degrees* and *College Road* are of the same order of magnitude as the values obtained using the stepped beam numerical model. Therefore, it is reasonable to assume that the stepped beam numerical model estimates of the bending stiffness of the RC core and completed volumetric modular building are validated. The bending stiffness values identified using the Stepped Beam Model for *Ten Degrees* and *College Road* provide the first quantification of the stiffness contribution of volumetric modules to the global structure of an RC core supported high-rise modular structure using in situ measurements. These values can be used in the development of data-informed surrogate models of modular buildings which allow for improved assessment of the wind-induced dynamic response of modular buildings.

Bibliography

- [1] Rajagopal, Balakrishnan. *Adequate housing as a component of the right to an adequate standard of living, and on the right to non-discrimination in this context*. Report. United Nations, 2023.
- [2] Lawson, Mark, Ogden, Ray, and Goodier, Chris. *Design in modular construction*. CRC Press, 2014. ISBN: 0415554500.
- [3] Generalova, Elena M., Generalov, Viktor P., and Kuznetsova, Anna A. “Modular Buildings in Modern Construction”. In: *25th Polish - Russian - Slovak Seminar Theoretical Foundation of Civil Engineering, 2016, July 11, 2016 - July 16, 2016*. Vol. 153. Procedia Engineering. Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20165203164869 3D reinforced concrete Block construction Construction productivity Eco-friendly housing High rise building High-rise construction Modular buildings Prefabricated blocks. Elsevier Ltd, pp. 167–172. ISBN: 18777058. DOI: 10.1016/j.proeng.2016.08.098. URL: <http://dx.doi.org/10.1016/j.proeng.2016.08.098>.
- [4] Ferdous, Wahid, Bai, Yu, Ngo, Tuan Duc, Manalo, Allan, and Mendis, Priyan. “New advancements, challenges and opportunities of multi-storey modular buildings – A state-of-the-art review”. In: *Engineering Structures* 183 (2019), pp. 883–893. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2019.01.061>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029618334175>.
- [5] Thai, Huu-Tai, Ngo, Tuan, and Uy, Brian. “A review on modular construction for high-rise buildings”. In: *Structures* 28 (2020), pp. 1265–1290. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2020.09.070>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012420305476>.
- [6] Construction, Tide. *George Street, Croydon*. Web Page. 2022. URL: <https://tideconstruction.co.uk/projects/george-street-croydon/>.
- [7] Jafari, Mohammad and Alipour, Alice. “Methodologies to mitigate wind-induced vibration of tall buildings: A state-of-the-art review”. In: *Journal of Building Engineering* 33 (2021), p. 101582. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobbe.2020.101582>. URL: <http://www.sciencedirect.com/science/article/pii/S2352710220305106>.
- [8] Kwok, Kenny C. S., Hitchcock, Peter A., and Burton, Melissa D. “Perception of vibration and occupant comfort in wind-excited tall buildings”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 97.7 (2009), pp. 368–380. ISSN: 0167-6105. DOI: <https://doi.org/10.1016/j.jweia.2009.05.006>. URL: <https://www.sciencedirect.com/science/article/pii/S0167610509000373>.

- [9] Tamura, Yukio. “Application of damping devices to suppress wind-induced responses of buildings”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 74-76 (1998), pp. 49–72. ISSN: 0167-6105. DOI: [https://doi.org/10.1016/S0167-6105\(98\)00006-3](https://doi.org/10.1016/S0167-6105(98)00006-3). URL: <https://www.sciencedirect.com/science/article/pii/S0167610598000063>.
- [10] Tamura, Yukio and Yoshida, Akihito. “Amplitude Dependency of Damping in Buildings”. In: *Structures Congress 2008* (2012). doi:10.1061/41000(315)39, pp. 1–10. ISSN: 9780784410004. DOI: doi:10.1061/41000(315)3910.1061/41000(315)39. URL: [https://doi.org/10.1061/41000\(315\)39](https://doi.org/10.1061/41000(315)39).
- [11] Smith, R., Merello, R., and Willford, M. “Intrinsic and supplementary damping in tall buildings”. In: *Proceedings of the Institution of Civil Engineers - Structures and Buildings* 163.2 (2010), pp. 111–118. DOI: 10.1680/stbu.2010.163.2.111. URL: <https://www.icevirtuallibrary.com/doi/abs/10.1680/stbu.2010.163.2.111>.
- [12] Satake, Naoki, Suda, Ken-ichi, Arakawa, Toshiharu, Sasaki, Atsushi, and Tamura, Yukio. “Damping Evaluation Using Full-Scale Data of Buildings in Japan”. In: *Journal of Structural Engineering* 129.4 (2003). doi: 10.1061/(ASCE)0733-9445(2003)129:4(470), pp. 470–477. DOI: 10.1061/(ASCE)0733-9445(2003)129:4(470). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(2003\)129:4\(470\)](https://doi.org/10.1061/(ASCE)0733-9445(2003)129:4(470)).
- [13] Kijewski-Correa, Tracy et al. “Validating wind-induced response of tall buildings: Synopsis of the Chicago full-scale monitoring program”. In: *Journal of Structural Engineering* 132.10 (2006), pp. 1509–1523. ISSN: 0733-9445.
- [14] Lamb, S., Kwok, K. C. S., and Walton, D. “Occupant comfort in wind-excited tall buildings: Motion sickness, compensatory behaviours and complaint”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 119 (2013), pp. 1–12. ISSN: 0167-6105. DOI: <https://doi.org/10.1016/j.jweia.2013.05.004>. URL: <http://www.sciencedirect.com/science/article/pii/S0167610513000871%20https://www.sciencedirect.com/science/article/pii/S0167610513000871?via%5C%3Dihub>.
- [15] Kareem, Ahsan, Kijewski, Tracy, and Tamura, Yukio. “Mitigation of motions of tall buildings with specific examples of recent applications”. In: *Wind and structures* 2.3 (1999), pp. 201–251. ISSN: 1226-6116.
- [16] Kavyashree, B. G., Patil, Shantharam, and Rao, Vidya S. “Review on vibration control in tall buildings: from the perspective of devices and applications”. In: *International Journal of Dynamics and Control* 9.3 (2021), pp. 1316–1331. ISSN: 2195-2698. DOI: 10.1007/s40435-020-00728-6. URL: <https://doi.org/10.1007/s40435-020-00728-6>.
- [17] Chua, Y. S., Liew, J. Y. Richard, and Pang, S. D. “Modelling of connections and lateral behavior of high-rise modular steel buildings”. In: *Journal of Constructional Steel Research* 166 (2020), p. 105901. ISSN: 0143-974X. DOI: <https://doi.org/10.1016/j.jcsr.2019.105901>. URL: <https://www.sciencedirect.com/science/article/pii/S0143974X19302044>.
- [18] Lacey, Andrew William, Chen, Wensu, Hao, Hong, and Bi, Kaiming. “Lateral behaviour of modular steel building with simplified models of new inter-module connections”. In: *Engineering Structures* 236 (2021), p. 112103. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2021.112103>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029621002534>.

- [19] Gunawardena, Tharaka, Ngo, Tuan, Mendis, Priyan, and Alfano, Jose. “Innovative Flexible Structural System Using Prefabricated Modules”. In: *Journal of Architectural Engineering* 22.4 (2016), p. 05016003. DOI: 10.1061/(ASCE)AE.1943-5568.0000214. URL: [https://doi.org/10.1061/\(ASCE\)AE.1943-5568.0000214](https://doi.org/10.1061/(ASCE)AE.1943-5568.0000214).
- [20] Hickey, John, Broderick, Brian, Fitzgerald, Breiffni, and Moore, Hollie. “Mitigation of wind induced accelerations in tall modular buildings”. In: *Structures* 37 (2022), pp. 576–587. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2022.01.037>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012422000376>.
- [21] Chen, Zhihua, Khan, Kashan, Khan, Arsalan, Javed, Khadija, and Liu, Jiadi. “Exploration of the multidirectional stability and response of prefabricated volumetric modular steel structures”. In: *Journal of Constructional Steel Research* 184 (2021), p. 106826. ISSN: 0143-974X. DOI: <https://doi.org/10.1016/j.jcsr.2021.106826>. URL: <https://www.sciencedirect.com/science/article/pii/S0143974X21003084>.
- [22] Pan, Wei, Yang, Yi, and Pan, Mi. “Implementing modular integrated construction in high-rise high-density cities: perspectives in Hong Kong”. In: *Building Research Information* (2022). doi: 10.1080/09613218.2022.2113024, pp. 1–15. ISSN: 0961-3218. DOI: 10.1080/09613218.2022.2113024. URL: <https://doi.org/10.1080/09613218.2022.2113024>.
- [23] Ministry of Housing, Communities Local Government Cross Industry Working Group. *Modern Methods of Construction: introducing the MMC definition framework*. Report. 2019. URL: <https://www.gov.uk/government/publications/modern-methods-of-construction-working-group-developing-a-definition-framework>.
- [24] FurturArc. *Avenue South Residence: World’s tallest PPVC residential building*. Web Page. 2022. URL: <https://www.futurarc.com/project/avenue-south-residence-worlds-tallest-ppvc-residential-building/>.
- [25] Construction, Tide. *College Road*. Web Page. 20/02/2024 2024. URL: <https://tideconstruction.co.uk/projects/college-road/>.
- [26] Ali, Ali Hassan, Kineber, Ahmed Farouk, Elyamany, Ahmed, Hussein Ibrahim, Ahmed, and Daoud, Ahmed Osama. “Exploring stationary and major modular construction challenges in developing countries: a case study of Egypt”. In: *Journal of Engineering, Design and Technology* ahead-of-print.ahead-of-print (2023). ISSN: 1726-0531. DOI: 10.1108/JEDT-03-2023-0099. URL: <https://doi.org/10.1108/JEDT-03-2023-0099>.
- [27] Liew, J. Y. Richard and Chua, Y. S. “Innovative modular systems for high-rise buildings”. In: *Engineering Structures* 323 (2025), p. 119270. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2024.119270>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029624018327>.
- [28] Hough, Michael J and Lawson, R Mark. “Design and construction of high-rise modular buildings based on recent projects”. In: *Proceedings of the Institution of Civil Engineers-Civil Engineering*. Vol. 172. Thomas Telford Ltd, pp. 37–44. ISBN: 0965-089X.

- [29] Lawson, R. M., Ogden, R. G., and Bergin, R. “Application of modular construction in high-rise buildings”. In: *Journal of Architectural Engineering* 18.2 (2012). Cited By :126 Export Date: 16 October 2020, pp. 148–154. DOI: 10.1061/(ASCE)AE.1943-5568.0000057. URL: <https://www.scopus.com/inward/record.uri?eid=2-s2.0-84862557157&doi=10.1061%5C%2f%5C%28ASCE%5C%29AE.1943-5568.0000057&partnerID=40&md5=725be4d7740199cad4df69c1bb8b3550>.
- [30] Liew, J. Y. R., Chua, Y. S., and Dai, Z. “Steel concrete composite systems for modular construction of high-rise buildings”. In: *Structures* 21 (2019), pp. 135–149. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2019.02.010>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012419300347>.
- [31] Lim, Robert Z. C., Looi, Daniel T. W., Tsang, Hing-Ho, and Wilson, John L. “A preliminary design tool for volumetric buildings”. In: *The Structural Design of Tall and Special Buildings* 31.13 (2022). <https://doi.org/10.1002/tal.1946>, e1946. ISSN: 1541-7794. DOI: <https://doi.org/10.1002/tal.1946>. URL: <https://doi.org/10.1002/tal.1946>.
- [32] Pan, Wei, Wang, Zhen, and Zhang, Yang. “Module equivalent frame method for structural design of concrete high-rise modular buildings”. In: *Journal of Building Engineering* (2021), p. 103214. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobe.2021.103214>. URL: <https://www.sciencedirect.com/science/article/pii/S235271022101072X>.
- [33] Sun, Yanhui, Wang, Jun, Wu, Jeremy, Shi, Weixiang, Ji, Dazhi, Wang, Xiangyu, and Zhao, Xianzhong. “Constraints Hindering the Development of High-Rise Modular Buildings”. In: *Applied Sciences* 10.20 (2020). ISSN: 2076-3417. DOI: 10.3390/app10207159.
- [34] Cao, X., Li, X., Zhu, Y., and Zhang, Z. “A comparative study of environmental performance between prefabricated and traditional residential buildings in China”. In: *Journal of Cleaner Production* 109 (2015). Cited By :84 Export Date: 16 October 2020, pp. 131–143. DOI: 10.1016/j.jclepro.2015.04.120. URL: <https://www.scopus.com/inward/record.uri?eid=2-s2.0-84929224246&doi=10.1016%5C%2fj.jclepro.2015.04.120&partnerID=40&md5=824c90781fdaeb20046d72fb28f7abcb>.
- [35] Forman, Tim, Pomponi, Francesco, and Saint, Ruth. *A Streamlined Assessment of Whole Life Embodied Carbon in The Valentine, Gants Hill, Redbridge, UK Ten Degrees, George Street, Croydon, UK*. Report. 2021. URL: <chrome-extension://efaidnbmnnnibpcajpcglclefindmkaj/https://tideconstruction.co.uk/wp-content/uploads/2023/07/Tide-Extended-Abstract-Valentine-Ten-Degrees.pdf>.
- [36] Shan, Sidi and Pan, Wei. “Structural design of high-rise buildings using steel-framed modules: A case study in Hong Kong”. In: (2020). Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20203008960503 Design solutions Earthquake load High rise building Inter-story drift ratios Lateral displacements Modular buildings Rise structures Structural behaviors. ISSN: 15417794. DOI: 10.1002/tal.1788. URL: <http://dx.doi.org/10.1002/tal.1788%20https://onlinelibrary.wiley.com/doi/abs/10.1002/tal.1788>.

- [37] Pan, Wei, Yang, Yi, and Yang, Lin. "High-Rise Modular Building: Ten-Year Journey and Future Development". In: *Construction Research Congress 2018: Sustainable Design and Construction and Education, CRC 2018, April 2, 2018 - April 4, 2018*. Vol. 2018-April. Construction Research Congress 2018: Sustainable Design and Construction and Education - Selected Papers from the Construction Research Congress 2018. Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20182505344248 Concrete and composites Connection details Construction period Construction wastes Industry stakeholders Operational savings Structural stabilities Wind load resistances. American Society of Civil Engineers (ASCE), pp. 523–532. DOI: 10.1061/9780784481301.052. URL: <http://dx.doi.org/10.1061/9780784481301.052>. <https://ascelibrary.org/doi/10.1061/9780784481301.052>.
- [38] Andy Prabowo, Pho. "Multi-storey Modular Cold-Formed Steel Building in Hong Kong: Challenges Opportunities". In: *1st International Conference of Construction, Infrastructure, and Materials, ICCIM 2019, July 16, 2019 - July 17, 2019*. Vol. 650. IOP Conference Series: Materials Science and Engineering. Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20194707701683 Cold-formed steel Commercial building Construction method High rise building Low-rise buildings Manufacturing process Modular buildings Structural component. Institute of Physics Publishing, BCA Senantiasadi Sisi Anda, et al., Indocement Heidelberg Cement Group, Mitra10, Total Bagun Persada, Waskita Precast. ISBN: 17578981. DOI: 10.1088/1757-899X/650/1/012033. URL: <http://dx.doi.org/10.1088/1757-899X/650/1/012033>. <https://iopscience.iop.org/article/10.1088/1757-899X/650/1/012033>.
- [39] Yang, Yi, Pan, Mi, Pan, Wei, and Zhang, Zhiqian. "Sources of Uncertainties in Offsite Logistics of Modular Construction for High-Rise Building Projects". In: *Journal of Management in Engineering* 37.3 (2021), p. 04021011. ISSN: 0742-597X.
- [40] Lacey, Andrew William, Chen, Wensu, Hao, Hong, and Bi, Kaiming. "Structural response of modular buildings – An overview". In: *Journal of Building Engineering* 16 (2018), pp. 45–56. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobe.2017.12.008>. URL: <http://www.sciencedirect.com/science/article/pii/S2352710217303650>.
- [41] Zhang, Yidan, Pan, Mi, Pan, Wei, Yang, Yi, and Wu, Jiewen. "Enhancing modular construction supply chain: Drivers, opportunities, constraints, concerns, strategies, and measures". In: *Developments in the Built Environment* 18 (2024), p. 100408. ISSN: 2666-1659. DOI: <https://doi.org/10.1016/j.dibe.2024.100408>. URL: <https://www.sciencedirect.com/science/article/pii/S2666165924000899>.
- [42] Munmulla, Thisari, Navaratnam, Satheeskumar, Hidallana-Gamage, H. D., Tushar, Quddus, Ponnampalam, Thusiyanthan, Zhang, Guomin, and Jayasinghe, M. T. R. "Sustainable approaches to improve the resilience of modular buildings under wind loads". In: *Journal of Constructional Steel Research* 211 (2023), p. 108124. ISSN: 0143-974X. DOI: <https://doi.org/10.1016/j.jcsr.2023.108124>. URL: <https://www.sciencedirect.com/science/article/pii/S0143974X23003516>.
- [43] Mark Lawson, Robert and Richards, Jane. "Modular design for high-rise buildings". In: vol. 163. Proceedings of the Institution of Civil Engineers: Structures and Buildings. Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20104413336300 Buckling resistance Compression resistance Design resistance High rise building Notional Horizontal Forces Residential

- building Second order effect Structural frameworks. ICE Publishing Ltd, pp. 151–164. ISBN: 09650911. DOI: 10.1680/stbu.2010.163.3.151. URL: <http://dx.doi.org/10.1680/stbu.2010.163.3.151><https://www.icevirtuallibrary.com/doi/pdf/10.1680/stbu.2010.163.3.151>.
- [44] Patil, VB and Jangid, RS. “Response of wind-excited benchmark building installed with dampers”. In: *The Structural Design of Tall and Special Buildings* 20.4 (2011), pp. 497–514. ISSN: 1541-7794.
- [45] Heng, Si Hwa, Hyland, David, Hough, Michael, and McCrum, Daniel. “Numerical Assessment of The Robustness of a Corner-supported Modular Steel Building”. In: *ce/papers* 6.3-4 (2023), pp. 2151–2156. ISSN: 2509-7075. DOI: <https://doi.org/10.1002/cepa.2732>. URL: <https://doi.org/10.1002/cepa.2732>.
- [46] Lawson, P. M., Byfield, M. P., Popo-Ola, S. O., and Grubb, P. J. “Robustness of light steel frames and modular construction”. In: *Proceedings of the Institution of Civil Engineers - Structures and Buildings* 161.1 (2008), pp. 3–16. DOI: 10.1680/stbu.2008.161.1.3. URL: <https://www.icevirtuallibrary.com/doi/abs/10.1680/stbu.2008.161.1.3>.
- [47] Munmulla, Thisari, Navaratnam, Satheeskumar, Thamboo, Julian, Ponnampalam, Thusiyanthan, Damruwan, Hidallana-Gamage Hasitha, Tsavdaridis, Konstantinos Daniel, and Zang, Guomin. “Analyses of Structural Robustness of Prefabricated Modular Buildings: A Case Study on Mid-Rise Building Configurations”. In: *Buildings* 12.8 (2022), p. 1289. ISSN: 2075-5309.
- [48] Tusnin, Alexander R., Alekseytsev, Anatoly V., and Tusnina, Olga. *Using Machine Learning Technologies to Design Modular Buildings*. Electronic Article. 2024. DOI: 10.3390/buildings14072213.
- [49] Denoon, RO and Strobel, KR. “PRACTICAL CONSIDERATIONS IN THE ASSESSMENT OF BUILDING MOTION ACCEPTABILITY”. In: ().
- [50] Ye, Zhihang, Giriunas, Kevin, Sezen, Halil, Wu, Gang, and Feng, De-Cheng. “State-of-the-art review and investigation of structural stability in multi-story modular buildings”. In: *Journal of Building Engineering* 33 (2021), p. 101844. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobbe.2020.101844>. URL: <https://www.sciencedirect.com/science/article/pii/S235271022033477X>.
- [51] Chen, Zhihua, Liu, Yang, Zhong, Xu, and Liu, Jiadi. “Rotational stiffness of inter-module connection in mid-rise modular steel buildings”. In: *Engineering Structures* 196 (2019), p. 109273. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2019.06.009>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029619309484><https://www.sciencedirect.com/science/article/abs/pii/S0141029619309484?via%5C%3Dihub>.
- [52] Srisangeerthan, Srisankanthan, Hashemi, M. Javad, Rajeev, Pathmanathan, Gad, Emad, and Fernando, Saman. “Review of performance requirements for inter-module connections in multi-story modular buildings”. In: *Journal of Building Engineering* 28 (2020), p. 101087. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobbe.2019.101087>. URL: <https://www.sciencedirect.com/science/article/pii/S2352710218315651>.

- [53] Lacey, Andrew William, Chen, Wensu, Hao, Hong, and Bi, Kaiming. “Review of bolted inter-module connections in modular steel buildings”. In: *Journal of Building Engineering* 23 (2019), pp. 207–219. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobe.2019.01.035>. URL: <http://www.sciencedirect.com/science/article/pii/S2352710218310866>.
- [54] Sendanayake, S. V., Thambiratnam, D. P., Perera, N. J., Chan, T. H. T., and Aghdamy, S. “Enhancing the lateral performance of modular buildings through innovative inter-modular connections”. In: *Structures* 29 (2021), pp. 167–184. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2020.10.047>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012420306032>.
- [55] Sharafi, Pezhman, Mortazavi, Mina, Samali, Bijan, and Ronagh, Hamid. “Interlocking system for enhancing the integrity of multi-storey modular buildings”. In: *Automation in Construction* 85 (2018), pp. 263–272. ISSN: 0926-5805. DOI: <https://doi.org/10.1016/j.autcon.2017.10.023>. URL: <http://www.sciencedirect.com/science/article/pii/S092658051730256X>.
- [56] Zhang, Ge, Xu, Long-He, and Li, Zhong-Xian. “Theoretical and parametric studies of a self-centering modular steel structure connection”. In: *Engineering Structures* 247 (2021), p. 113146. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2021.113146>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029621012797>.
- [57] Chen, Zhihua, Li, Hongxing, Chen, Aoyi, Yu, Yujie, and Wang, Hui. “Research on pretensioned modular frame test and simulations”. In: *Engineering Structures* 151 (2017), pp. 774–787. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2017.08.019>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029617310301>.
- [58] Bowron, J., Gulliford, J., Churchill, E., Cerone, J., and Mallie, J. Patent. 2017.
- [59] Mao, L. “Research on joint mechanical behavior of container architecture”. In: *Tongji University, Master of Science Degree* (2015).
- [60] Lee, Seungjae, Park, Jaeseong, Kwak, Euishin, Shon, Sudeok, Kang, Changhoon, and Choi, Hosoon. “Verification of the Seismic Performance of a Rigidly Connected Modular System Depending on the Shape and Size of the Ceiling Bracket”. In: *Materials* 10.3 (2017), p. 263. ISSN: 1996-1944. DOI: [10.3390/ma10030263](https://doi.org/10.3390/ma10030263). URL: <https://dx.doi.org/10.3390/ma10030263>.
- [61] Chen, Zhihua, Liu, Jiadi, and Yu, Yujie. “Experimental study on interior connections in modular steel buildings”. In: *Engineering Structures* 147 (2017), pp. 625–638. ISSN: 0141-0296.
- [62] Deng, En-Feng, Yan, Jia-Bao, Ding, Yang, Zong, Liang, Li, Zhong-Xian, and Dai, Xiao-Meng. “Analytical and numerical studies on steel columns with novel connections in modular construction”. In: *International Journal of Steel Structures* 17 (2017), pp. 1613–1626. ISSN: 1598-2351.
- [63] Wang, Zhen and Pan, Wei. “A hybrid coupled wall system with replaceable steel coupling beams for high-rise modular buildings”. In: *Journal of Building Engineering* 31 (2020). Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20201308365327 Hybrid coupled walls Modular buildings Non-linear finite elements Precast concrete walls Reinforced concrete coupled walls Seismic Performance Steel coupling beam Structural performance. ISSN: 23527102. DOI: [10.1016/j.jobe.2020.101355](https://doi.org/10.1016/j.jobe.2020.101355). URL: <http://dx.doi.org/10.1016/j.jobe.2020.101355>.

- org/10.1016/j.jobe.2020.101355%20https://www.sciencedirect.com/science/article/pii/S2352710219326154?via%5C%3Dihub.
- [64] Pan, Tianyao Ping Wei and Mou, Ben. “An innovative type of module-to-core wall connections for high-rise steel modular buildings”. In: *Journal of Building Engineering* (2022), p. 105425. ISSN: 2352-7102.
- [65] Lane, Thomas. “The sky’s the limit: See the world’s tallest modular tower in Croydon”. In: *Building* (2019). URL: <https://www.building.co.uk/buildings/the-skys-the-limit-see-the-worlds-tallest-modular-tower-in-croydon/5101741.article>.
- [66] Pan, Wei, Yang, Yi, and Yang, Lin. “High-Rise Modular Building: Ten-Year Journey and Future Development”. In: *Construction Research Congress 2018* (2018). doi:10.1061/9780784481301.052, pp. 523–532. DOI: doi:10.1061/9780784481301.05210.1061/9780784481301.052. URL: <https://doi.org/10.1061/9780784481301.052>.
- [67] Peng, Jiahao, Hou, Chao, and Shen, Luming. “Numerical analysis of corner-supported composite modular buildings under wind actions”. In: *Journal of Constructional Steel Research* 187 (2021), p. 106942. ISSN: 0143-974X. DOI: <https://doi.org/10.1016/j.jcsr.2021.106942>. URL: <https://www.sciencedirect.com/science/article/pii/S0143974X21004247>.
- [68] Kareem, A. and Gurley, K. “Damping in structures: its evaluation and treatment of uncertainty”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 59.2 (1996), pp. 131–157. ISSN: 0167-6105. DOI: [https://doi.org/10.1016/0167-6105\(96\)00004-9](https://doi.org/10.1016/0167-6105(96)00004-9). URL: <https://www.sciencedirect.com/science/article/pii/0167610596000049>.
- [69] Bi, Jiabin, Zong, Liang, Si, Qi, Ding, Yang, Lou, Ni, and Huang, Yun. “Field measurement and numerical analysis on wind-induced performance of modular structure with concrete cores”. In: *Engineering Structures* 220 (2020), p. 110969. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2020.110969>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029620310580>.
- [70] Zhou, Kang and Li, Qiu-Sheng. “Reliability analysis of damping estimation by random decrement technique for high-rise buildings”. In: (2020). Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20204809555275 Acceleration response Confidence interval Damping estimations Field measurement Free-vibration tests High rise building Inherent limitations Random decrement technique. ISSN: 00988847. DOI: 10.1002/eqe.3396. URL: <http://dx.doi.org/10.1002/eqe.3396>.
- [71] Feng, Zhouquan, Zhao, B., Hua, Xugang, Chen, Z., and Asce, M. “Enhanced EMD-RDT Method for Output-Only Ambient Modal Identification of Structures”. In: *Journal of Aerospace Engineering* 32 (2019), p. 04019046. DOI: 10.1061/(ASCE)AS.1943-5525.0001034.
- [72] Ivanovic, Sanja S, Trifunac, Mihailo D, and Todorovska, MI. “Ambient vibration tests of structures-a review”. In: *ISET Journal of earthquake Technology* 37.4 (2000), pp. 165–197.
- [73] Stadarization, European Committee for. *Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions*. 2004.
- [74] Civil Engineers (ASCE), American Society of. *Minimum Design Loads for Buildings and Other Structures*. Standard. 2010.

-
- [75] Standardization (ISO), The International Organization for. *ISO 4354:2009 Wind Actions on Structures*. Standard. 2009.
 - [76] Ohkuma, T., Marukawa, H., Niihori, Y., and Kato, N. “Full-scale measurement of wind pressures and response accelerations of a high-rise building”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 38.2 (1991), pp. 185–196. ISSN: 0167-6105. DOI: [https://doi.org/10.1016/0167-6105\(91\)90040-4](https://doi.org/10.1016/0167-6105(91)90040-4). URL: <https://www.sciencedirect.com/science/article/pii/0167610591900404>.
 - [77] Gonzalez-Fernandez, D, Macdonalda, JHG, Titurus, B, Margnellic, A, Grecoc, L, Gkoktsic, K, Leeksc, M, Janssenc, J, and Zanchetta, MA. “Full-scale identification of aeroelastic effects on a tall building”. In: *Proceedings of the 15th International Conf. on Wind Engineering*, pp. 129–130.
 - [78] Fu, Jiyang, Zheng, Qingxing, Huang, Youqin, Wu, Jiurong, Pi, Yonglin, and Liu, Qionxiang. “Design optimization on high-rise buildings considering occupant comfort reliability and joint distribution of wind speed and direction”. In: *Engineering Structures* 156 (2018), pp. 460–471. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2017.11.041>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029617319880?via%5C%3Dihub>.
 - [79] Burton, Melissa D, Kwok, Kenny CS, and Abdelrazaq, A. “Wind-induced motion of tall buildings: designing for occupant comfort”. In: *International journal of high-rise buildings* 4.1 (2015), pp. 1–8. ISSN: 2234-7224.
 - [80] Chen Peter, W. and Robertson Leslie, E. “Human Perception Thresholds of Horizontal Motion”. In: *Journal of the Structural Division* 98.8 (1972), pp. 1681–1695. DOI: 10.1061/JSDEAG.0003297. URL: <https://doi.org/10.1061/JSDEAG.0003297>.
 - [81] Canada. Associate Committee on the National Building Code, National Research Council of. *National Building Code of Canada: 1975*. On inside cover: First Edition 1941. Second Edition 1953. Third Edition 1960. Fourth Edition 1965. Fifth Edition 1970. Sixth Edition 1975. National Research Council of Canada. Associate Committee on the National Building Code, 1975. DOI: 10.4224/40001677.
 - [82] ISO. *Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0,063–1 Hz)*. (International Organization for Standardization). 1984.
 - [83] Japan, Architectural Institute of. *Guidelines for the Evaluation of Habitability to Building Vibration*. 1991.
 - [84] *ISO 10137: Basis for Design Structures : Serviceability of Buildings and Walkways Against Vibrations*. British Standard, 2007. URL: <https://books.google.ie/books?id=nYWzzQEACAAJ>.
 - [85] Avci, Onur, Alkhamis, Khalid, Abdeljaber, Osama, Alsharo, Anas, and Hussein, Mohammed. “Operational modal analysis and finite element model updating of a 230 m tall tower”. In: *Structures* 37 (2022), pp. 154–167. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2021.12.078>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012421011966>.

- [86] Reynders, Edwin. "System Identification Methods for (Operational) Modal Analysis: Review and Comparison". In: *Archives of Computational Methods in Engineering* 19.1 (2012), pp. 51–124. ISSN: 1134-3060. DOI: 10.1007/s11831-012-9069-x. URL: <https://dx.doi.org/10.1007/s11831-012-9069-x>.
- [87] Rainieri, Carlo and Fabbrocino, Giovanni. "Operational modal analysis: overview and applications". In: *Strategies for reduction of the seismic risk* (2008), pp. 29–44.
- [88] Zahid, Fahad Bin, Ong, Zhi Chao, and Khoo, Shin Yee. "A review of operational modal analysis techniques for in-service modal identification". In: *Journal of the Brazilian Society of Mechanical Sciences and Engineering* 42.8 (2020). ISSN: 1678-5878. DOI: 10.1007/s40430-020-02470-8. URL: <https://dx.doi.org/10.1007/s40430-020-02470-8>.
- [89] Hu, Jiaxing, Li, Zhengnong, and Zhao, Zhefei. "Field Measurement on Translational and Torsional Wind-Induced Response of a High-Rise Building during Typhoon Khanun". In: *Buildings* 12.10 (2022), p. 1698. ISSN: 2075-5309.
- [90] Au, Siu-Kui. "Fast Bayesian ambient modal identification in the frequency domain, Part I: Posterior most probable value". In: *Mechanical Systems and Signal Processing* 26 (2012), pp. 60–75. ISSN: 0888-3270. DOI: <https://doi.org/10.1016/j.ymssp.2011.06.017>. URL: <https://www.sciencedirect.com/science/article/pii/S0888327011002391>.
- [91] Zhang, JW and Li, QS. "Wind tunnel test and field measurement study of wind effects on a 600-m-high super-tall building". In: *The Structural Design of Tall and Special Buildings* 26.17 (2017), e1385. ISSN: 1541-7794.
- [92] Reynders, Edwin. "System Identification Methods for (Operational) Modal Analysis: Review and Comparison". In: *Archives of Computational Methods in Engineering* 19.1 (2012), pp. 51–124. ISSN: 1134-3060. DOI: 10.1007/s11831-012-9069-x. URL: <https://dx.doi.org/10.1007/s11831-012-9069-x>.
- [93] Cole, H. "On-line failure detection and damping measurement of aerospace structures by random decrement signatures". In:
- [94] Kijewski, Tracey and Kareem, Ahsan. "Reliability of random decrement technique for estimates of structural damping". In: *8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability*, pp. 294–299.
- [95] Tamura, Yukio and Suganuma, Shin-ya. "Evaluation of amplitude-dependent damping and natural frequency of buildings during strong winds". In: *Journal of Wind Engineering and Industrial Aerodynamics* 59.2 (1996), pp. 115–130. ISSN: 0167-6105. DOI: [https://doi.org/10.1016/0167-6105\(96\)00003-7](https://doi.org/10.1016/0167-6105(96)00003-7). URL: <https://www.sciencedirect.com/science/article/pii/0167610596000037>.
- [96] Ibrahim, SR. "Random decrement technique for modal identification of structures". In: *Journal of Spacecraft and Rockets* 14.11 (1977), pp. 696–700. ISSN: 0022-4650.
- [97] Chen, Genda and Wang, Zuocai. "A signal decomposition theorem with Hilbert transform and its application to narrowband time series with closely spaced frequency components". In: *Mechanical Systems and Signal Processing* 28 (2012), pp. 258–279. ISSN: 0888-3270. DOI: <https://doi.org/10.1016/j.ymssp.2011.02.002>. URL: <https://www.sciencedirect.com/science/article/pii/S0888327011000641>.

- [98] Wang, Zuo-Cai and Chen, Gen-Da. “Analytical mode decomposition of time series with decaying amplitudes and overlapping instantaneous frequencies”. In: *Smart materials and structures* 22.9 (2013), p. 095003. ISSN: 0964-1726.
- [99] Wang, Zuocai and Chen, Genda. “Analytical mode decomposition with Hilbert transform for modal parameter identification of buildings under ambient vibration”. In: *Engineering Structures* 59 (2014), pp. 173–184. ISSN: 0141-0296.
- [100] Kijewski, T. and Kareem, A. “On the reliability of a class of system identification techniques: insights from bootstrap theory”. In: *Structural Safety* 24.2 (2002), pp. 261–280. ISSN: 0167-4730. DOI: [https://doi.org/10.1016/S0167-4730\(02\)00028-0](https://doi.org/10.1016/S0167-4730(02)00028-0). URL: <https://www.sciencedirect.com/science/article/pii/S0167473002000280>.
- [101] Russotto, Salvatore, Di Matteo, Alberto, and Pirrotta, Antonina. *An Innovative Structural Dynamic Identification Procedure Combining Time Domain OMA Technique and GA*. Electronic Article. 2022. DOI: 10.3390/buildings12070963.
- [102] James, George, Carne, Thomas, Lauffer, J., and Nord, A. “Modal testing using natural excitation”. In: *Proceedings of the 10th International Modal Analysis Conference* 2 (1992).
- [103] Van Overschee, Peter and De Moor, Bart. “Subspace algorithms for the stochastic identification problem”. In: *Automatica* 29.3 (1993), pp. 649–660. ISSN: 0005-1098. DOI: [https://doi.org/10.1016/0005-1098\(93\)90061-W](https://doi.org/10.1016/0005-1098(93)90061-W). URL: <https://www.sciencedirect.com/science/article/pii/S000510989390061W>.
- [104] Van Overschee, Peter and De Moor, Bart. “Subspace identification for linear systems. Theory, implementation, applications. Incl. 1 disk”. In: vol. xiv. 1996, pp. xiv + 254. ISBN: 0-7923-9717-7. DOI: 10.1007/978-1-4613-0465-4.
- [105] Peeters, Bart and De Roeck, Guido. “Stochastic system identification for operational modal analysis: a review”. In: *J. Dyn. Sys., Meas., Control* 123.4 (2001), pp. 659–667. ISSN: 0022-0434.
- [106] Wen, Q, Hua, XG, Chen, ZQ, Niu, HW, and Wang, XY. “AMD-based random decrement technique for modal identification of structures with close modes”. In: *Journal of Aerospace Engineering* 31.5 (2018), p. 04018057. ISSN: 0893-1321.
- [107] Hwang, Jae-Seung, Noh, Jung-Tae, Lee, Sang-Hyun, and Kareem, Ahsan. “Experimental Verification of Modal Identification of a High-rise Building Using Independent Component Analysis”. In: *International Journal of Concrete Structures and Materials* 13.1 (2019), p. 4. ISSN: 2234-1315. DOI: 10.1186/s40069-018-0319-7. URL: <https://doi.org/10.1186/s40069-018-0319-7>.
- [108] Yuen, Ka-Veng and Katafygiotis, Lambros S. “Bayesian fast Fourier transform approach for modal updating using ambient data”. In: *Advances in Structural Engineering* 6.2 (2003), pp. 81–95. ISSN: 1369-4332.
- [109] Yuen, Ka-Veng and Katafygiotis, Lambros S. “Bayesian time-domain approach for modal updating using ambient data”. In: *Probabilistic Engineering Mechanics* 16.3 (2001), pp. 219–231. ISSN: 0266-8920. DOI: [https://doi.org/10.1016/S0266-8920\(01\)00004-2](https://doi.org/10.1016/S0266-8920(01)00004-2). URL: <https://www.sciencedirect.com/science/article/pii/S0266892001000042>.

- [110] Au, Siu-Kui. “Fast Bayesian FFT Method for Ambient Modal Identification with Separated Modes”. In: *Journal of Engineering Mechanics - ASCE* 137.3 (2011). 10.1061/(ASCE)EM.1943-7889.0000213, pp. 214–226. ISSN: 0733-9399. DOI: 10.1061/(ASCE)EM.1943-7889.0000213.
- [111] Ni, Yan-Chun and Zhang, Feng-Liang. “Fast Bayesian frequency domain modal identification from seismic response data”. In: *Computers and Structures* 212 (2019), pp. 225–235. ISSN: 0045-7949. DOI: <https://doi.org/10.1016/j.compstruc.2018.08.018>. URL: <https://www.sciencedirect.com/science/article/pii/S0045794917314128>.
- [112] Kareem, Ahsan. “Emerging frontiers in wind engineering: Computing, stochastics, machine learning and beyond”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 206 (2020), p. 104320. ISSN: 0167-6105. DOI: <https://doi.org/10.1016/j.jweia.2020.104320>. URL: <http://www.sciencedirect.com/science/article/pii/S0167610520302300>.
- [113] Chan, Chun Man, Huang, MF, and Kwok, Kenny CS. “Stiffness optimization for wind-induced dynamic serviceability design of tall buildings”. In: *Journal of structural engineering* 135.8 (2009), pp. 985–997. ISSN: 0733-9445.
- [114] Asghari Mooneghi, Maryam and Kargarmoakhar, Ramtin. “Aerodynamic Mitigation and Shape Optimization of Buildings: Review”. In: *Journal of Building Engineering* 6 (2016), pp. 225–235. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobbe.2016.01.009>. URL: <https://www.sciencedirect.com/science/article/pii/S2352710216300092>.
- [115] Collins, Ruth. “Vibration control of structures using active and semi-active tuned mass dampers”. Thesis. 2005. URL: <http://hdl.handle.net/2262/86680>.
- [116] Wang, Qinhua, Tian, Huarui, Qiao, Haoshuai, Tiwari, Nayan Deep, and Wang, Quan. “Wind-induced vibration control and parametric optimization of connected high-rise buildings with tuned liquid-column-damper-inerter”. In: *Engineering Structures* 226 (2021), p. 111352. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2020.111352>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029620339535>.
- [117] Aydin, Ersin, Öztürk, Baki, and Dutkiewicz, Maciej. “Analysis of efficiency of passive dampers in multistorey buildings”. In: *Journal of Sound and Vibration* 439 (2019), pp. 17–28. ISSN: 0022-460X. DOI: <https://doi.org/10.1016/j.jsv.2018.09.031>. URL: <https://www.sciencedirect.com/science/article/pii/S0022460X18306163>.
- [118] Liu, Yanhui, Wang, Kan, Mercan, Oya, Chen, Huating, and Tan, Ping. “Experimental and numerical studies on the optimal design of tuned mass dampers for vibration control of high-rise structures”. In: *Engineering Structures* 211 (2020), p. 110486. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2020.110486>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029619340301>.
- [119] Yang, Fan, Sedaghati, Ramin, and Esmailzadeh, Ebrahim. “Vibration suppression of structures using tuned mass damper technology: A state-of-the-art review”. In: *Journal of Vibration and Control* 28.7-8 (2021). doi: 10.1177/1077546320984305, pp. 812–836. ISSN: 1077-5463. DOI: 10.1177/1077546320984305. URL: <https://doi.org/10.1177/1077546320984305>.

-
- [120] Yang, Jann N, Agrawal, Anil K, Samali, Bijan, and Wu, Jong-Cheng. “Benchmark problem for response control of wind-excited tall buildings”. In: *Journal of engineering mechanics* 130.4 (2004), pp. 437–446. ISSN: 0733-9399.
 - [121] Jaisee, Sujit, Yue, Feng, and Ooi, Yi Hao. “A state-of-the-art review on passive friction dampers and their applications”. In: *Engineering Structures* 235 (2021), p. 112022. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2021.112022>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029621001723>.
 - [122] Saaed, Tarek Edrees, Nikolakopoulos, George, Jonasson, Jan-Erik, and Hedlund, Hans. “A state-of-the-art review of structural control systems”. In: *Journal of Vibration and Control* 21.5 (2013). doi: 10.1177/1077546313478294, pp. 919–937. ISSN: 1077-5463. DOI: 10.1177/1077546313478294. URL: <https://doi.org/10.1177/1077546313478294>.
 - [123] Koutsoloukas, Lefteris, Nikitas, Nikolaos, and Aristidou, Petros. “Passive, semi-active, active and hybrid mass dampers: A literature review with associated applications on building-like structures”. In: *Developments in the Built Environment* 12 (2022), p. 100094. ISSN: 2666-1659. DOI: 10.1016/j.dibe.2022.100094. URL: <https://dx.doi.org/10.1016/j.dibe.2022.100094>.
 - [124] Symans, Michael D. and Constantinou, Michael C. “Semi-active control systems for seismic protection of structures: a state-of-the-art review”. In: *Engineering Structures* 21.6 (1999), pp. 469–487. ISSN: 0141-0296. DOI: [https://doi.org/10.1016/S0141-0296\(97\)00225-3](https://doi.org/10.1016/S0141-0296(97)00225-3). URL: <https://www.sciencedirect.com/science/article/pii/S0141029697002253>.
 - [125] Casciati, Fabio, Rodellar, José, and Yildirim, Umut. “Active and semi-active control of structures – theory and applications: A review of recent advances”. In: *Journal of Intelligent Material Systems and Structures* 23.11 (2012). doi: 10.1177/1045389X12445029, pp. 1181–1195. ISSN: 1045-389X. DOI: 10.1177/1045389X12445029. URL: <https://doi.org/10.1177/1045389X12445029>.
 - [126] Stanikzai Mohammad, H., Elias, Said, and Chae, Yunbyeong. “Recent Advances in Hybrid Vibration-Control Systems”. In: *Practice Periodical on Structural Design and Construction* 27.3 (2022). doi: 10.1061/(ASCE)SC.1943-5576.0000685, p. 03122003. DOI: 10.1061/(ASCE)SC.1943-5576.0000685. URL: [https://doi.org/10.1061/\(ASCE\)SC.1943-5576.0000685](https://doi.org/10.1061/(ASCE)SC.1943-5576.0000685).
 - [127] Fisco, N. R. and Adeli, H. “Smart structures: Part II — Hybrid control systems and control strategies”. In: *Scientia Iranica* 18.3 (2011), pp. 285–295. ISSN: 1026-3098. DOI: <https://doi.org/10.1016/j.scient.2011.05.035>. URL: <https://www.sciencedirect.com/science/article/pii/S1026309811001003>.
 - [128] Arvind, R. and Santhi, M. Helen. “A State of Art Review on Hybrid Passive Energy Dissipating Devices”. In: *Journal of Vibration Engineering and Technologies* 10.5 (2022), pp. 1931–1954. ISSN: 2523-3920. DOI: 10.1007/s42417-022-00492-1. URL: <https://dx.doi.org/10.1007/s42417-022-00492-1>.
 - [129] Ghosh, Aparna and Konar, Tanmoy. “Popular passive dampers for structural control: A review”. In: *Journal of Structural Engineering (Madras)* 50 (2023), pp. 24–38.
 - [130] Lago, A, Wood, A, and Trabucco, D. *Damping technologies for tall buildings: New trends in comfort and safety*. Generic. 2015.

- [131] Zhang, Ri-Hui and Soong, T. T. “Seismic Design of Viscoelastic Dampers for Structural Applications”. In: *Journal of Structural Engineering* 118.5 (1992). doi: 10.1061/(ASCE)0733-9445(1992)118:5(1375), pp. 1375–1392. DOI: 10 . 1061/(ASCE)0733-9445(1992)118:5(1375). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(1992\)118:5\(1375\)](https://doi.org/10.1061/(ASCE)0733-9445(1992)118:5(1375)).
- [132] Shen, K. L. and Soong, T. T. “Modeling of Viscoelastic Dampers for Structural Applications”. In: *Journal of Engineering Mechanics* 121.6 (1995). doi: 10.1061/(ASCE)0733-9399(1995)121:6(694), pp. 694–701. DOI: 10 . 1061 / (ASCE) 0733-9399(1995)121:6(694). URL: [https://doi.org/10.1061/\(ASCE\)0733-9399\(1995\)121:6\(694\)](https://doi.org/10.1061/(ASCE)0733-9399(1995)121:6(694)).
- [133] Tsai, C. S. and Lee, H. H. “Applications of Viscoelastic Dampers to High-Rise Buildings”. In: *Journal of Structural Engineering* 119.4 (1993). doi: 10.1061/(ASCE)0733-9445(1993)119:4(1222), pp. 1222–1233. DOI: 10.1061/(ASCE)0733-9445(1993)119:4(1222). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(1993\)119:4\(1222\)](https://doi.org/10.1061/(ASCE)0733-9445(1993)119:4(1222)).
- [134] Xu, Zhao-Dong, Wang, Deng-Xiang, and Shi, Chun-Fang. “Model, tests and application design for viscoelastic dampers”. In: *Journal of Vibration and Control* 17.9 (2010). doi: 10.1177/1077546310373617, pp. 1359–1370. ISSN: 1077-5463. DOI: 10.1177/1077546310373617. URL: <https://doi.org/10.1177/1077546310373617>.
- [135] Park, S. W. “Analytical modeling of viscoelastic dampers for structural and vibration control”. In: *International Journal of Solids and Structures* 38.44 (2001), pp. 8065–8092. ISSN: 0020-7683. DOI: [https://doi.org/10.1016/S0020-7683\(01\)00026-9](https://doi.org/10.1016/S0020-7683(01)00026-9). URL: <https://www.sciencedirect.com/science/article/pii/S0020768301000269>.
- [136] Vatanshenas, Ali, Rohanimanesh, Mohammad, and Mohammadiha, Ehsan. “Investigating the Performance of Viscoelastic Dampers (VED) Under Nearfield Earthquakes with Directivity Feature”. In: *Civil and Environmental Engineering* 14 (2018). DOI: 10.2478/cee-2018-0003.
- [137] McNamara, Robert J. and Taylor, Douglas P. “Fluid viscous dampers for high-rise buildings”. In: *The Structural Design of Tall and Special Buildings* 12.2 (2003), pp. 145–154. ISSN: 1541-7794. DOI: 10 . 1002 / tal . 218. URL: <https://dx.doi.org/10.1002/tal.218>.
- [138] Hou, Chien-Yuan. “Fluid Dynamics and Behavior of Nonlinear Viscous Fluid Dampers”. In: *Journal of Structural Engineering* 134.1 (2008). doi: 10.1061/(ASCE)0733-9445(2008)134:1(56), pp. 56–63. DOI: 10 . 1061 / (ASCE) 0733-9445(2008) 134:1(56). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(2008\)134:1\(56\)](https://doi.org/10.1061/(ASCE)0733-9445(2008)134:1(56)).
- [139] Taylor, Dauglas P and Constantinou, Michlael C. “Fluid dampers for applications of seismic energy dissipation and seismic isolation”. In: *página web www.taylordevices.com/dampers.htm* (1996).
- [140] Rama Raju, K., Ansu, M., and Iyer, Nagesh R. “A methodology of design for seismic performance enhancement of buildings using viscous fluid dampers”. In: *Structural Control and Health Monitoring* 21.3 (2014), pp. 342–355. ISSN: 1545-2255. DOI: 10.1002/stc.1568. URL: <https://dx.doi.org/10.1002/stc.1568>.

- [141] Bhaskararao, A. V. and Jangid, R. S. "Seismic analysis of structures connected with friction dampers". In: *Engineering Structures* 28.5 (2006), pp. 690–703. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2005.09.020>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029605003469>.
- [142] Cherry, S. and Filiatrault, A. "Seismic Response Control of Buildings Using Friction Dampers". In: *Earthquake Spectra* 9.3 (1993). doi: 10.1193/1.1585724, pp. 447–466. ISSN: 8755-2930. DOI: 10.1193/1.1585724. URL: <https://doi.org/10.1193/1.1585724>.
- [143] López, I., Busturia, J. M., and Nijmeijer, H. "Energy dissipation of a friction damper". In: *Journal of Sound and Vibration* 278.3 (2004), pp. 539–561. ISSN: 0022-460X. DOI: <https://doi.org/10.1016/j.jsv.2003.10.051>. URL: <https://www.sciencedirect.com/science/article/pii/S0022460X03013282>.
- [144] Caudana Quintana, H. and Petkovski, M. "Optimum performance of structural control with friction dampers". In: *Engineering Structures* 172 (2018), pp. 154–162. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2018.06.017>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029617321843>.
- [145] Gutierrez Soto, Mariantonieta and Adeli, Hojjat. "Tuned Mass Dampers". In: *Archives of Computational Methods in Engineering* 20.4 (2013), pp. 419–431. ISSN: 1886-1784. DOI: 10.1007/s11831-013-9091-7. URL: <https://doi.org/10.1007/s11831-013-9091-7>.
- [146] Elias, Said and Matsagar, Vasant. "Research developments in vibration control of structures using passive tuned mass dampers". In: *Annual Reviews in Control* 44 (2017), pp. 129–156. ISSN: 1367-5788. DOI: <https://doi.org/10.1016/j.arcontrol.2017.09.015>. URL: <https://www.sciencedirect.com/science/article/pii/S1367578817301372>.
- [147] Chen, Zhaowei, Fang, Hui, Han, Zhaoling, and Sun, Shizheng. "Influence of bridge-based designed TMD on running trains". In: *Journal of Vibration and Control* 25.1 (2019), pp. 182–193. ISSN: 1077-5463. DOI: 10.1177/1077546318773022. URL: <https://dx.doi.org/10.1177/1077546318773022>.
- [148] Warburton, G. B. "Optimum absorber parameters for minimizing vibration response". In: *Earthquake Engineering Structural Dynamics* 9.3 (1981), pp. 251–262. ISSN: 0098-8847. DOI: <https://doi.org/10.1002/eqe.4290090306>. URL: <https://doi.org/10.1002/eqe.4290090306>.
- [149] Farghaly, Ahmed Abdelraheem and Salem Ahmed, Mahmoud. "Optimum Design of TMD System for Tall Buildings". In: *ISRN Civil Engineering* 2012 (2012), pp. 1–13. ISSN: 2090-5114. DOI: 10.5402/2012/716469. URL: <https://dx.doi.org/10.5402/2012/716469>.
- [150] Lee, Chien-Liang, Chen, Yung-Tsang, Chung, Lap-Loi, and Wang, Yen-Po. "Optimal design theories and applications of tuned mass dampers". In: *Engineering Structures* 28.1 (2006), pp. 43–53. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2005.06.023>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029605002610>.
- [151] Gerges, Rafik R. and Vickery, Barry J. "Optimum design of pendulum-type tuned mass dampers". In: *The Structural Design of Tall and Special Buildings* 14.4 (2005), pp. 353–368. ISSN: 1541-7794. DOI: 10.1002/tal.273. URL: <https://dx.doi.org/10.1002/tal.273>.

- [152] Frahm, Hermann. *Device for damping vibrations of bodies*. Patent. 1909.
- [153] Den Hartog, J.P. *Mechanical Vibrations*. 3rd Edition. McGraw-Hill Book Company ., 1947. ISBN: 9781443725361. URL: <https://books.google.ie/books?id=fcxJOHlgnqEC>.
- [154] Poon, Dennis, Shieh, Shaw-song, Joseph, LM, and Chang, C. “Structural design of Taipei 101, the world’s tallest building”. In: *Proceedings of the CTBUH 2004 Seoul Conference, Seoul, Korea*. sn, pp. 271–278.
- [155] Tait, M. J. “Modelling and preliminary design of a structure-TLD system”. In: *Engineering Structures* 30.10 (2008), pp. 2644–2655. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2008.02.017>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029608000667>.
- [156] Konar, Tanmoy and Ghosh, Aparna Dey. “Flow Damping Devices in Tuned Liquid Damper for Structural Vibration Control: A Review”. In: *Archives of Computational Methods in Engineering* 28.4 (2021), pp. 2195–2207. ISSN: 1886-1784. DOI: 10.1007/s11831-020-09450-0. URL: <https://doi.org/10.1007/s11831-020-09450-0>.
- [157] Li, Yi, Liu, Xiang-Long, Yin, Jie-ting, Chen, Fu-Bin, and Han, Yan. “Parameter study of tuned liquid damper (TLD) on wind-induced vibration control of high-rise buildings”. In: *Structures* 64 (2024), p. 106475. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2024.106475>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012424006271>.
- [158] Sun, Hao-ding, He, Hao-xiang, Cheng, Yang, and Cao, Qing. “Theoretical and experimental research on vibration control of the tuned liquid damper with damping net and sloped-bottom”. In: *Journal of Building Engineering* 81 (2024), p. 108170. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobbe.2023.108170>. URL: <https://www.sciencedirect.com/science/article/pii/S2352710223023501>.
- [159] Suthar, Sameer J. and Jangid, Radhey Shyam. “Design of tuned liquid sloshing dampers using nonlinear constraint optimization for across-wind response control of benchmark tall building”. In: *Structures* 33 (2021), pp. 2675–2688. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2021.05.059>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012421004628>.
- [160] Malekghasemi, Hadi, Ashasi-Sorkhabi, Ali, Ghaemmaghami, Amir Reza, and Mercan, Oya. “Experimental and numerical investigations of the dynamic interaction of tuned liquid damper–structure systems”. In: *Journal of Vibration and Control* 21.14 (2013). doi: 10.1177/1077546313514759, pp. 2707–2720. ISSN: 1077-5463. DOI: 10.1177/1077546313514759. URL: <https://doi.org/10.1177/1077546313514759>.
- [161] Kareem, A. and Sun, W. J. “Stochastic response of structures with fluid-containing appendages”. In: *Journal of Sound and Vibration* 119.3 (1987), pp. 389–408. ISSN: 0022-460X. DOI: [https://doi.org/10.1016/0022-460X\(87\)90405-6](https://doi.org/10.1016/0022-460X(87)90405-6). URL: <https://www.sciencedirect.com/science/article/pii/0022460X87904056>.

-
- [162] Fujino, Yozo, Sun, Limin, Pacheco Benito, M., and Chaiseri, Piyawat. “Tuned Liquid Damper (TLD) for Suppressing Horizontal Motion of Structures”. In: *Journal of Engineering Mechanics* 118.10 (1992). doi: 10.1061/(ASCE)0733-9399(1992)118:10(2017), pp. 2017–2030. DOI: 10.1061/(ASCE)0733-9399(1992)118:10(2017). URL: [https://doi.org/10.1061/\(ASCE\)0733-9399\(1992\)118:10\(2017\)](https://doi.org/10.1061/(ASCE)0733-9399(1992)118:10(2017)).
 - [163] Jin, Heng, Calabrese, Andrea, and Liu, Yong. “Effects of different damping baffle configurations on the dynamic response of a liquid tank under seismic excitation”. In: *Engineering Structures* 229 (2021), p. 111652. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2020.111652>. URL: <http://www.sciencedirect.com/science/article/pii/S014102962034253X>.
 - [164] Love, J. S. and Tait, M. J. “Parametric depth ratio study on tuned liquid dampers: Fluid modelling and experimental work”. In: *Computers Fluids* 79 (2013), pp. 13–26. ISSN: 0045-7930. DOI: <https://doi.org/10.1016/j.compfluid.2013.03.004>. URL: <https://www.sciencedirect.com/science/article/pii/S0045793013000935>.
 - [165] Soliman, Islam M., Tait, Michael J., and El Damatty, Ashraf A. “Development and Validation of Finite Element Structure-Tuned Liquid Damper System Models”. In: *Journal of Dynamic Systems, Measurement, and Control* 137.11 (2015). ISSN: 0022-0434. DOI: 10.1115/1.4030866. URL: <https://doi.org/10.1115/1.4030866>.
 - [166] Konar, Tanmoy and Ghosh, Aparna. “A review on various configurations of the passive tuned liquid damper”. In: *Journal of Vibration and Control* 29.9-10 (2022). (Dey) doi: 10.1177/10775463221074077, pp. 1945–1980. ISSN: 1077-5463. DOI: 10.1177/10775463221074077. URL: <https://doi.org/10.1177/10775463221074077>.
 - [167] Saha, Supradip and Debbarma, Rama. “An experimental study on response control of structures using multiple tuned liquid dampers under dynamic loading”. In: *International Journal of Advanced Structural Engineering* 9.1 (2017), pp. 27–35. ISSN: 2008-6695. DOI: 10.1007/s40091-016-0146-5. URL: <https://doi.org/10.1007/s40091-016-0146-5>.
 - [168] Awad, Bishoy N., Tait, Michael J., and Love, J. Shayne. “Numerical modelling of dual function tanks for fire suppression and tuned liquid damper applications”. In: *Engineering Structures* 299 (2024), p. 117080. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2023.117080>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029623014955>.
 - [169] Watts, P. *The use of water-chambers for reducing the rolling of ships at sea*. Conference Paper. Mar. 1885.
 - [170] Vandiver, J. Kim and Mitome, Shuhei. “The Effect of Liquid Storage Tanks on the Dynamic Response of Offshore Platforms”. In: *Journal of Petroleum Technology* 31.10 (1979), pp. 1231–1240. ISSN: 0149-2136. DOI: 10.2118/7285-PA. URL: <https://doi.org/10.2118/7285-PA>.
 - [171] Tait, M. J. and Deng, X. “The performance of structure-tuned liquid damper systems with different tank geometries”. In: *Structural Control and Health Monitoring* (2008), n/a–n/a. ISSN: 1545-2255. DOI: 10.1002/stc.286. URL: <https://dx.doi.org/10.1002/stc.286>.

- [172] Love, J. S., Morava, B., and Smith, A. W. “Monitoring of a Tall Building Equipped with an Efficient Multiple-Tuned Sloshing Damper System”. In: *Practice Periodical on Structural Design and Construction* 25.3 (2020), p. 05020003. ISSN: 1084-0680. DOI: [10.1061/\(asce\)sc.1943-5576.0000481](https://doi.org/10.1061/(asce)sc.1943-5576.0000481). URL: [https://dx.doi.org/10.1061/\(asce\)sc.1943-5576.0000481](https://dx.doi.org/10.1061/(asce)sc.1943-5576.0000481).
- [173] Zhang, Zili. “Numerical and experimental investigations of the sloshing modal properties of sloped-bottom tuned liquid dampers for structural vibration control”. In: *Engineering Structures* 204 (2020), p. 110042. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2019.110042>. URL: <http://www.sciencedirect.com/science/article/pii/S0141029619325453>.
- [174] Kashani, Armin Hajighasem, Halabian, Amir Mahdi, and Asghari, Keyvan. “A numerical study of tuned liquid damper based on incompressible SPH method combined with TMD analogy”. In: *Journal of Fluids and Structures* 82 (2018), pp. 394–411. ISSN: 0889-9746. DOI: <https://doi.org/10.1016/j.jfluidstructs.2018.07.013>. URL: <https://www.sciencedirect.com/science/article/pii/S088997461730734X>.
- [175] Love, J. S. and Tait, M. J. “Nonlinear simulation of a tuned liquid damper with damping screens using a modal expansion technique”. In: *Journal of Fluids and Structures* 26.7 (2010), pp. 1058–1077. ISSN: 0889-9746. DOI: <https://doi.org/10.1016/j.jfluidstructs.2010.07.004>. URL: <https://www.sciencedirect.com/science/article/pii/S0889974610000897>.
- [176] Warnitchai, P. and Pinkaew, T. “Modelling of liquid sloshing in rectangular tanks with flow-dampening devices”. In: *Engineering Structures* 20.7 (1998), pp. 593–600. ISSN: 0141-0296. DOI: [https://doi.org/10.1016/S0141-0296\(97\)00068-0](https://doi.org/10.1016/S0141-0296(97)00068-0). URL: <https://www.sciencedirect.com/science/article/pii/S0141029697000680>.
- [177] Wu, Jong-Cheng, Shih, Ming-Hsiang, Lin, Yuh-Yi, and Shen, Ying-Chang. “Design guidelines for tuned liquid column damper for structures responding to wind”. In: *Engineering Structures* 27.13 (2005), pp. 1893–1905. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2005.05.009>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029605002002>.
- [178] Gao, H., Kwok, K. C. S., and Samali, B. “Optimization of tuned liquid column dampers”. In: *Engineering Structures* 19.6 (1997), pp. 476–486. ISSN: 0141-0296. DOI: [https://doi.org/10.1016/S0141-0296\(96\)00099-5](https://doi.org/10.1016/S0141-0296(96)00099-5). URL: <https://www.sciencedirect.com/science/article/pii/S0141029696000995>.
- [179] Yalla Swaroop, K. and Kareem, Ahsan. “Optimum Absorber Parameters for Tuned Liquid Column Dampers”. In: *Journal of Structural Engineering* 126.8 (2000). doi: 10.1061/(ASCE)0733-9445(2000)126:8(906), pp. 906–915. DOI: 10.1061/(ASCE)0733-9445(2000)126:8(906). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(2000\)126:8\(906\)](https://doi.org/10.1061/(ASCE)0733-9445(2000)126:8(906)).
- [180] Gao, H., Kwok, K. S. C., and Samali, B. “Characteristics of multiple tuned liquid column dampers in suppressing structural vibration”. In: *Engineering Structures* 21.4 (1999), pp. 316–331. ISSN: 0141-0296. DOI: [https://doi.org/10.1016/S0141-0296\(97\)00183-1](https://doi.org/10.1016/S0141-0296(97)00183-1). URL: <https://www.sciencedirect.com/science/article/pii/S0141029697001831>.

-
- [181] Zhu, Fei, Wang, Jin-Ting, Jin, Feng, and Lu, Li-Qiao. "Real-time hybrid simulation of full-scale tuned liquid column dampers to control multi-order modal responses of structures". In: *Engineering Structures* 138 (2017), pp. 74–90. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2017.02.004>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029617303619>.
- [182] McNamara, K. P., Love, J. S., and Tait, M. J. "Nonlinear Modeling of Series-Type Pendulum Tuned Mass Damper-Tuned Liquid Damper". In: *Journal of Vibration and Acoustics* 144.4 (2022). ISSN: 1048-9002. DOI: 10.1115/1.4053636. URL: <https://doi.org/10.1115/1.4053636>.
- [183] Moon, Kyoung Sun. "Vertically distributed multiple tuned mass dampers in tall buildings: performance analysis and preliminary design". In: *The Structural Design of Tall and Special Buildings* 19.3 (2010), pp. 347–366. ISSN: 1541-7794. DOI: <https://doi.org/10.1002/tal.499>. URL: <https://doi.org/10.1002/tal.499>.
- [184] Tait Michael, J., Isyumov, Nicholas, and Damatty Ashraf, A. "Tuned Liquid Dampers to Mitigate Wind-Induced Motions of Buildings". In: *Structures Congress 2006* (2012). doi:10.1061/40889(201)101, pp. 1–10. ISSN: 9780784408896. DOI: doi:10.1061/40889(201)10110.1061/40889(201)101. URL: [https://doi.org/10.1061/40889\(201\)101](https://doi.org/10.1061/40889(201)101).
- [185] Tait, MJ and Deng, X. "The performance of structure-tuned liquid damper systems with different tank geometries". In: *Structural Control and Health Monitoring: The Official Journal of the International Association for Structural Control and Monitoring and of the European Association for the Control of Structures* 17.3 (2010), pp. 254–277. ISSN: 1545-2255.
- [186] Sun, L. M., Fujino, Y., Chaiseri, P., and Pacheco, B. M. "The properties of tuned liquid dampers using a TMD analogy". In: *Earthquake Engineering Structural Dynamics* 24.7 (1995), pp. 967–976. ISSN: 0098-8847. DOI: <https://doi.org/10.1002/eqe.4290240704>. URL: <https://doi.org/10.1002/eqe.4290240704>.
- [187] Tait Michael, J. "The performance of 1-D and 2-D tuned liquid dampers". Thesis. 2004.
- [188] Tait, M. J., Isyumov, N., and El Damatty, A. A. "Performance of Tuned Liquid Dampers". In: *Journal of Engineering Mechanics* 134.5 (2008). doi: 10.1061/(ASCE)0733-9399(2008)134:5(417), pp. 417–427. DOI: 10.1061/(ASCE)0733-9399(2008)134:5(417). URL: [https://doi.org/10.1061/\(ASCE\)0733-9399\(2008\)134:5\(417\)](https://doi.org/10.1061/(ASCE)0733-9399(2008)134:5(417)).
- [189] Love, J. S. and Tait, M. J. "Multiple Tuned Liquid Dampers for Efficient and Robust Structural Control". In: *Journal of Structural Engineering* 141.12 (2015). doi: 10.1061/(ASCE)ST.1943-541X.0001290, p. 04015045. DOI: 10.1061/(ASCE)ST.1943-541X.0001290. URL: [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001290](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001290).
- [190] Fujino, Y. and Sun, L. M. "Vibration Control by Multiple Tuned Liquid Dampers (MTLDs)". In: *Journal of Structural Engineering* 119.12 (1993). doi: 10.1061/(ASCE)0733-9445(1993)119:12(3482), pp. 3482–3502. DOI: 10.1061/(ASCE)0733-9445(1993)119:12(3482). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(1993\)119:12\(3482\)](https://doi.org/10.1061/(ASCE)0733-9445(1993)119:12(3482)).

- [191] Igusa, T. and Xu, K. “Vibration Control Using Multiple Tuned Mass Dampers”. In: *Journal of Sound and Vibration* 175.4 (1994), pp. 491–503. ISSN: 0022-460X. DOI: <https://doi.org/10.1006/jsvi.1994.1341>. URL: <https://www.sciencedirect.com/science/article/pii/S0022460X84713411>.
- [192] Kareem, Ahsan and Kline, Samuel. “Performance of Multiple Mass Dampers under Random Loading”. In: *Journal of Structural Engineering* 121.2 (1995). doi: 10.1061/(ASCE)0733-9445(1995)121:2(348), pp. 348–361. DOI: 10.1061/(ASCE)0733-9445(1995)121:2(348). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(1995\)121:2\(348\)](https://doi.org/10.1061/(ASCE)0733-9445(1995)121:2(348)).
- [193] Elias, Said, Matsagar, Vasant, and Datta, T. K. “Dynamic Response Control of a Wind-Excited Tall Building with Distributed Multiple Tuned Mass Dampers”. In: *International Journal of Structural Stability and Dynamics* 19.06 (2019). doi: 10.1142/S0219455419500597, p. 1950059. ISSN: 0219-4554. DOI: 10.1142/S0219455419500597. URL: <https://doi.org/10.1142/S0219455419500597>.
- [194] Ozturk, Baki, Cetin, Huseyin, and Aydin, Ersin. “Optimum vertical location and design of multiple tuned mass dampers under seismic excitations”. In: *Structures* 41 (2022), pp. 1141–1163. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2022.05.014>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012422003757>.
- [195] Kwon, Dae Kun and Kareem, Ahsan. “Hybrid simulation of a tall building with a double-decker tuned sloshing damper system under wind loads”. In: *The Structural Design of Tall and Special Buildings* 29.15 (2020), e1790. ISSN: 1541-7794.
- [196] Elias, Said and Matsagar, Vasant. “Distributed Multiple Tuned Mass Dampers for Wind Vibration Response Control of High-Rise Building”. In: *Journal of Engineering* 2014 (2014), p. 198719. ISSN: 2314-4904. DOI: 10.1155/2014/198719. URL: <https://doi.org/10.1155/2014/198719>.
- [197] Elias, Said and Matsagar, Vasant. “Wind response control of 76-storey benchmark building with distributed multiple tuned mass dampers”. In: *Journal of Wind and Engineering* 11.2 (2014), pp. 37–49.
- [198] Roy, A., Staino, A., Ghosh, A., Basu, B., and Chatterjee, S. “Seismic Vibration Control of Elevated Water Tank by TLD and Validation of Full-Scale TLD Model through Real-Time-Hybrid-Testing”. In: *Journal of Physics: Conference Series* 744.1 (2016). (D), p. 012042. ISSN: 1742-6596 1742-6588. DOI: 10.1088/1742-6596/744/1/012042. URL: <https://dx.doi.org/10.1088/1742-6596/744/1/012042>.
- [199] Takanashi, Koichi. *Non-linear Earthquake Response Analysis of Structures by a Computer-actuator On-line System (detail of the System)*. 1975.
- [200] Zhang, Zili, Staino, Andrea, Basu, Biswajit, and Nielsen, Søren R. K. “Performance evaluation of full-scale tuned liquid dampers (TLDs) for vibration control of large wind turbines using real-time hybrid testing”. In: *Engineering Structures* 126 (2016), pp. 417–431. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2016.07.008>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029616303364>.

-
- [201] Banerji, Pradipta, Murudi, Mohan, Shah, Arvind H., and Popplewell, Neil. “Tuned liquid dampers for controlling earthquake response of structures”. In: *Earthquake Engineering and Structural Dynamics* 29.5 (2000), pp. 587–602. ISSN: 0098-8847. DOI: 10.1002/(sici)1096-9845(200005)29:5<587::aid-eqe926>3.0.co;2-i. URL: [https://dx.doi.org/10.1002/\(sici\)1096-9845\(200005\)29:5%3C587::aid-eqe926%3E3.0.co;2-i](https://dx.doi.org/10.1002/(sici)1096-9845(200005)29:5%3C587::aid-eqe926%3E3.0.co;2-i).
 - [202] Wang, Jin-Ting, Gui, Yao, Zhu, Fei, Jin, Feng, and Zhou, Meng-Xia. “Real-time hybrid simulation of multi-story structures installed with tuned liquid damper”. In: *Structural Control and Health Monitoring* 23.7 (2016), pp. 1015–1031. ISSN: 1545-2255. DOI: <https://doi.org/10.1002/stc.1822>. URL: <https://doi.org/10.1002/stc.1822>.
 - [203] Lee, Sung-Kyung, Park, Eun Churn, Min, Kyung-Won, Lee, Sang-Hyun, Chung, Lan, and Park, Ji-Hun. “Real-time hybrid shaking table testing method for the performance evaluation of a tuned liquid damper controlling seismic response of building structures”. In: *Journal of Sound and Vibration* 302.3 (2007), pp. 596–612. ISSN: 0022-460X. DOI: <https://doi.org/10.1016/j.jsv.2006.12.006>. URL: <https://www.sciencedirect.com/science/article/pii/S0022460X06008832%20https://www.sciencedirect.com/science/article/abs/pii/S0022460X06008832?via%5C%3Dihub>.
 - [204] Sun, L. M., Fujino, Y., Pacheco, B. M., and Chaiseri, P. “Modelling of tuned liquid damper (TLD)”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 43.1 (1992), pp. 1883–1894. ISSN: 0167-6105. DOI: [https://doi.org/10.1016/0167-6105\(92\)90609-E](https://doi.org/10.1016/0167-6105(92)90609-E). URL: <https://www.sciencedirect.com/science/article/pii/016761059290609E>.
 - [205] Yu, Jin-Kyu, Wakahara, Toshihiro, and Reed, Dorothy A. “A non-linear numerical model of the tuned liquid damper”. In: *Earthquake Engineering and Structural Dynamics* 28.6 (1999), pp. 671–686. ISSN: 0098-8847. DOI: 10.1002/(sici)1096-9845(199906)28:6<671::aid-eqe835>3.0.co;2-x. URL: [https://dx.doi.org/10.1002/\(sici\)1096-9845\(199906\)28:6%3C671::aid-eqe835%3E3.0.co;2-x](https://dx.doi.org/10.1002/(sici)1096-9845(199906)28:6%3C671::aid-eqe835%3E3.0.co;2-x).
 - [206] Xin, Yuxiang, Chen, Genda, and Lou, Menglin. “Seismic response control with density-variable tuned liquid dampers”. In: *Earthquake Engineering and Engineering Vibration* 8.4 (2009), pp. 537–546. ISSN: 1671-3664. DOI: 10.1007/s11803-009-9111-7. URL: <https://dx.doi.org/10.1007/s11803-009-9111-7>.
 - [207] Zhang, Zili. “Understanding and exploiting the nonlinear behavior of tuned liquid dampers (TLDs) for structural vibration control by means of a non-linear reduced-order model (ROM)”. In: *Engineering Structures* 251 (2022), p. 113524. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2021.113524>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029621016205>.
 - [208] Bianchi, Giorgio and Vatti, Katerina. “Opportunities and challenges of building taller with modular construction”. In: *Council on Tall Buildings and Urban Habitat 10th World Congress - 50 Forward — 50 Back: The Recent History and Essential Future of Sustainable Cities, CTBUH 2019, October 28, 2019 - November 2, 2019*. 50 Forward 50 Back: The Recent History and Essential Future of Sustainable Cities - Proceedings of the CTBUH 10th World Congress. Compilation and indexing terms, Copyright 2020 Elsevier Inc. 20200708179327 Advanced manufacturing DFMA Digital designs High rise

- High-rise construction International projects Volumetric Wind-induced acceleration. Council on Tall Buildings and Urban Habitat, CTBUH, pp. 289–295.
- [209] Soong, T. T. and Spencer, B. F. “Supplemental energy dissipation: state-of-the-art and state-of-the-practice”. In: *Engineering Structures* 24.3 (2002), pp. 243–259. ISSN: 0141-0296. DOI: [https://doi.org/10.1016/S0141-0296\(01\)00092-X](https://doi.org/10.1016/S0141-0296(01)00092-X). URL: <http://www.sciencedirect.com/science/article/pii/S014102960100092X>.
- [210] Banks, Colin, BurrIDGE, Jenny, Cammelli, Stefano, and Chiorino, Mario A.. *Tall Buildings-Structural Design of Concrete Buildings Up to 300 M Tall: A Working Group of The Concrete Centre and Fib Task Group 1.6*. MPA The Concrete Centre and Fédération internationale du béton (fib), 2014. ISBN: 1908257148.
- [211] Davenport, Alan G. “Gust loading factors”. In: *Journal of the Structural Division* 93.3 (1967), pp. 11–34.
- [212] Steenbergen, RDJM, Vrouwenvelder, ACWM, and Geurts, CPW. “Dynamics of tall buildings under stochastic wind load: applicability of Eurocode EN 1991-1-4 procedures 1 and 2”. In: *5th European and African Conference on Wind Engineering (EACWE 2009)*, pp. 1–18.
- [213] “Big Borough: Average House Price In London By Floor Space”. In: *Attic Self Storage* 2020.13 August 2019 (2019). URL: <https://www.atticstorage.co.uk/blog/big-borough>.
- [214] Warburton, Geoffrey B. “Optimum absorber parameters for various combinations of response and excitation parameters”. In: *Earthquake Engineering Structural Dynamics* 10 (1982), pp. 381–401.
- [215] Tall Buildings, Council of and Habitats, Urban. *CTBUH Conference, 2022*. Web Page. 2022. URL: <https://ctbuhconference.com/about/award-of-excellence-winners/>.
- [216] Construction, Tide. *George Street, Croydon*. Web Page. 2022. URL: <https://tideconstruction.co.uk/projects/george-street-croydon/>.
- [217] Heng, Si H., Hyland, David, Hough, Michael, and McCrum, Daniel. *Computational Modelling of Intra-Module Connections and Their Influence on the Robustness of a Steel Corner-Supported Volumetric Module*. Electronic Article. 2024. DOI: 10.3390/modelling5010021.
- [218] Mechanics, BMT Fluid. *George Street, Croydon, UK: Overall Wind Loading Study*. Report. 2017.
- [219] Engineers, RWDI Consulting. *College Road, Croydon, UK: Wind Induced Structural Responses*. Report. 2020.
- [220] Au, Siu-Kui and Zhang, Feng-Liang. “Ambient modal identification of a primary-secondary structure by Fast Bayesian FFT method”. In: *Mechanical Systems and Signal Processing* 28 (2012), pp. 280–296. ISSN: 0888-3270. DOI: <https://doi.org/10.1016/j.ymssp.2011.07.007>. URL: <https://www.sciencedirect.com/science/article/pii/S0888327011002962>.
- [221] Au, Siu-Kui, Zhang, Feng-Liang, and Ni, Yan-Chun. “Bayesian operational modal analysis: Theory, computation, practice”. In: *Computers and Structures* 126 (2013), pp. 3–14. ISSN: 0045-7949. DOI: <https://doi.org/10.1016/j.compstruc.2012.12.015>. URL: <https://www.sciencedirect.com/science/article/pii/S0045794912003288>.

-
- [222] Qing, Xu, Huang, M. F., Lou, W., and Ni, Y. “Modal identification of Canton Tower using the fast Bayesian FFT method”. In.
 - [223] Yuen, Ka-Veng. *Bayesian methods for structural dynamics and civil engineering*. John Wiley and Sons, 2010. ISBN: 0470824557.
 - [224] Li, Qiu-Sheng, Zhou, Kang, and Li, Xiao. “Damping estimation of high-rise buildings considering structural modal directions”. In: *Earthquake Engineering and Structural Dynamics* 49.6 (2020), pp. 543–566. ISSN: 0098-8847.
 - [225] Yang, Jann N, Lei, Ying, Lin, Silian, and Huang, Norden. “Identification of natural frequencies and dampings of in situ tall buildings using ambient wind vibration data”. In: *Journal of engineering mechanics* 130.5 (2004), pp. 570–577. ISSN: 0733-9399.
 - [226] Tamura, Y., Yoshida, A., Zhang, L., Ito, T., Nakata, S., and Sato, K. “Examples of modal identification of structures in Japan by FDD and MRD techniques”. In: *Proceedings of the 1st International Operational Modal Analysis Conference, IOMAC 2005*. Export Date: 10 July 2024; Cited By: 21. 2005. URL: <https://www.scopus.com/inward/record.uri?eid=2-s2.0-84906070988&partnerID=40&md5=367748f0a79cd4291ce3f867d383e677>.
 - [227] Tamura, Yukio, Zhang, Lingmi, Yoshida, Akihito, Nakata, Shinji, and Itoh, Takayoshi. “Ambient vibration tests and modal identification of structures by FDD and 2DOF-RD technique”. In: *The Proceedings of the Symposium on the Motion and Vibration Control* 2003.8 (2002). DOI: 10.1299/jsmemovic.2003.8.601.
 - [228] Efron, B. and Tibshirani, R. “Bootstrap Methods for Standard Errors, Confidence Intervals, and Other Measures of Statistical Accuracy”. In: *Statistical Science* 1.1 (1986), pp. 54–75. ISSN: 08834237. URL: <http://www.jstor.org/stable/2245500>.
 - [229] Office, The Met. Web Page. URL: <https://www.metoffice.gov.uk/>.
 - [230] Fu, J. Y., Li, Q. S., Wu, J. R., Xiao, Y. Q., and Song, L. L. “Field measurements of boundary layer wind characteristics and wind-induced responses of super-tall buildings”. In: *Journal of Wind Engineering and Industrial Aerodynamics* 96.8 (2008), pp. 1332–1358. ISSN: 0167-6105. DOI: <https://doi.org/10.1016/j.jweia.2008.03.004>. URL: <https://www.sciencedirect.com/science/article/pii/S0167610508000962>.
 - [231] He, Yinghou, Liu, Yujie, Wu, Ming, Fu, Jiyang, and He, Yuncheng. “Amplitude dependence of natural frequency and damping ratio for 5 supertall buildings with moderate-to-strong typhoon-induced vibrations”. In: *Journal of Building Engineering* 78 (2023), p. 107589. ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobe.2023.107589>. URL: <https://www.sciencedirect.com/science/article/pii/S2352710223017692>.
 - [232] Ramaji, Issa J and Memari, Ali M. “Extending the current model view definition standards to support multi-storey modular building projects”. In: *Architectural Engineering and Design Management* 14.1-2 (2018), pp. 158–176. ISSN: 1745-2007.
 - [233] Wang, Zixiao and Tsavdaridis, Konstantinos Daniel. “Optimality criteria-based minimum-weight design method for modular building systems subjected to generalised stiffness constraints: A comparative study”. In: *Engineering Structures* 251 (2022), p. 113472. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2021.113472>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029621015765>.

- [234] Annan, C. D., Youssef, M. A., and El Naggar, M. H. “Experimental evaluation of the seismic performance of modular steel-braced frames”. In: *Engineering Structures* 31.7 (2009), pp. 1435–1446. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2009.02.024>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029609000935>.
- [235] Benjamin, Sanchez, Christopher, Rausch, and Carl, Haas. “Feature modeling for configurable and adaptable modular buildings”. In: *Advanced Engineering Informatics* 51 (2022), p. 101514. ISSN: 1474-0346. DOI: <https://doi.org/10.1016/j.aei.2021.101514>. URL: <https://www.sciencedirect.com/science/article/pii/S1474034621002627>.
- [236] Shan, Sidi, Looi, Daniel, Cai, Yancheng, Ma, Peng, Chen, Man-Tai, Su, Ray, Young, Ben, and Pan, Wei. “Engineering modular integrated construction for high-rise building: A case study in Hong Kong”. In: *Proceedings of the Institution of Civil Engineers-Civil Engineering*. Vol. 172. Thomas Telford Ltd, pp. 51–57. ISBN: 0965-089X.
- [237] Shirokov, Vyacheslav S., Kholopov, Igor S., and Solovejv, Aleksey V. “Determination of the Frequency of Natural Vibrations of a Modular Building”. In: *Procedia Engineering* 153 (2016), pp. 655–661. ISSN: 1877-7058. DOI: <https://doi.org/10.1016/j.proeng.2016.08.218>. URL: <https://www.sciencedirect.com/science/article/pii/S1877705816323669>.
- [238] Hong, Sung-Gul, Cho, Bong-Ho, Chung, Kyung-Soo, and Moon, Ju-hyuk. “Behavior of framed modular building system with double skin steel panels”. In: *Journal of Constructional Steel Research* 67.6 (2011), pp. 936–946. ISSN: 0143-974X. DOI: <https://doi.org/10.1016/j.jcsr.2011.02.002>. URL: <https://www.sciencedirect.com/science/article/pii/S0143974X11000411>.
- [239] Alembagheri, M., Sharafi, P., Rashidi, M., Bigdeli, A., and Farajian, M. “Natural dynamic characteristics of volumetric steel modules with gypsum sheathed LSF walls: Experimental study”. In: *Structures* 33 (2021), pp. 272–282. ISSN: 2352-0124. DOI: <https://doi.org/10.1016/j.istruc.2021.04.068>. URL: <https://www.sciencedirect.com/science/article/pii/S2352012421003672>.
- [240] Rashidi, Maria, Sharafi, Pejman, Alembagheri, Mohammad, Bigdeli, Ali, and Samali, Bijan. “Operational Modal Analysis, Testing and Modelling of Prefabricated Steel Modules with Different LSF Composite Walls”. In: *Materials* 13.24 (2020), p. 5816. ISSN: 1996-1944. DOI: 10.3390/ma13245816. URL: <https://dx.doi.org/10.3390/ma13245816>.
- [241] Stadarization, European Committee for. “Eurocode 2: Design of concrete structures - Part 1-1 : General rules and rules for buildings”. In: (2004).
- [242] Mao, Qibo and Pietrzko, Stanislaw. “Free vibration analysis of stepped beams by using Adomian decomposition method”. In: *Applied Mathematics and Computation* 217.7 (2010), pp. 3429–3441. ISSN: 0096-3003. DOI: <https://doi.org/10.1016/j.amc.2010.09.010>. URL: <https://www.sciencedirect.com/science/article/pii/S0096300310009574>.
- [243] Kumar, Mukesh and Umesh. “Recent Development of Adomian Decomposition Method for Ordinary and Partial Differential Equations”. In: *International Journal of Applied and Computational Mathematics* 8.2 (2022). ISSN: 2349-5103. DOI: 10.1007/s40819-022-01285-6. URL: <https://dx.doi.org/10.1007/s40819-022-01285-6>.

-
- [244] Adomian, G. “A review of the decomposition method and some recent results for nonlinear equations”. In: *Computers Mathematics with Applications* 21.5 (1991), pp. 101–127. ISSN: 0898-1221. DOI: [https://doi.org/10.1016/0898-1221\(91\)90220-X](https://doi.org/10.1016/0898-1221(91)90220-X). URL: <https://www.sciencedirect.com/science/article/pii/089812219190220X>.
 - [245] Hosseini, M. M. and Nasabzadeh, H. “On the convergence of Adomian decomposition method”. In: *Applied Mathematics and Computation* 182.1 (2006), pp. 536–543. ISSN: 0096-3003. DOI: <https://doi.org/10.1016/j.amc.2006.04.015>. URL: <https://www.sciencedirect.com/science/article/pii/S0096300306003614>.
 - [246] Adomian, George. *Solving frontier problems of physics: the decomposition method*. Vol. 60. Springer Science Business Media, 2013. ISBN: 9401582890.
 - [247] Wazwaz, Abdul-Majid. “A reliable modification of Adomian decomposition method”. In: *Applied Mathematics and Computation* 102.1 (1999), pp. 77–86. ISSN: 0096-3003. DOI: [https://doi.org/10.1016/S0096-3003\(98\)10024-3](https://doi.org/10.1016/S0096-3003(98)10024-3). URL: <https://www.sciencedirect.com/science/article/pii/S0096300398100243>.
 - [248] Kavrakov, Igor, Kareem, Ahsan, and Morgenthal, Guido. “Comparison Metrics for Time-Histories: Application to Bridge Aerodynamics”. In: *Journal of Engineering Mechanics* 146.9 (2020). doi: 10.1061/(ASCE)EM.1943-7889.0001811, p. 04020093. DOI: 10.1061/(ASCE)EM.1943-7889.0001811. URL: [https://doi.org/10.1061/\(ASCE\)EM.1943-7889.0001811](https://doi.org/10.1061/(ASCE)EM.1943-7889.0001811).
 - [249] Cavalagli, Nicola, Biscarini, Chiara, Facci, Andrea L., Ubertaini, Filippo, and Ubertaini, Stefano. “Experimental and numerical analysis of energy dissipation in a sloshing absorber”. In: *Journal of Fluids and Structures* 68 (2017), pp. 466–481. ISSN: 0889-9746. DOI: <https://doi.org/10.1016/j.jfluidstructs.2016.11.020>. URL: <https://www.sciencedirect.com/science/article/pii/S0889974616303681>.
 - [250] Love, J. S., Morava, B., Robinson, J. K., and Haskett, T. C. “Tuned Sloshing Dampers in Tall Buildings: A Practical Performance-Based Design Approach”. In: *Practice Periodical on Structural Design and Construction* 26.3 (2021). ISSN: 1084-0680. DOI: 10.1061/(asce)sc.1943-5576.0000582. URL: [https://dx.doi.org/10.1061/\(asce\)sc.1943-5576.0000582](https://dx.doi.org/10.1061/(asce)sc.1943-5576.0000582).
 - [251] Barnes, June, Beharrell, Andrew, Robinson, Dickon, and Scanlon, Kath. *What is the Future of High-Rise Housing? Examining the Long-Term Social and Financial Impacts of Residential Towers*. Report. 2023.
 - [252] Nava-González, R., Pozos-Estrada, A., and López-Ibarra, A. “Development of Fragility Curves for Tall Buildings with Tuned Liquid Dampers for Vibration Control Under Wind Loading”. In: *International Journal of Civil Engineering* (2024). ISSN: 1735-0522. DOI: 10.1007/s40999-024-01020-w. URL: <https://dx.doi.org/10.1007/s40999-024-01020-w>.
 - [253] Fitzgerald, Breiffni, Sarkar, Saptarshi, and Staino, Andrea. “Improved reliability of wind turbine towers with active tuned mass dampers (ATMDs)”. In: *Journal of Sound and Vibration* 419 (2018), pp. 103–122. ISSN: 0022-460X. DOI: <https://doi.org/10.1016/j.jsv.2017.12.026>. URL: <https://www.sciencedirect.com/science/article/pii/S0022460X17308854>.

- [254] Ciampoli, M., Petrini, F., and Augusti, G. “Performance-Based Wind Engineering: Towards a general procedure”. In: *Structural Safety* 33.6 (2011), pp. 367–378. ISSN: 0167-4730. DOI: <https://doi.org/10.1016/j.strusafe.2011.07.001>. URL: <https://www.sciencedirect.com/science/article/pii/S0167473011000579>.
- [255] Martín del Campo, J. Osvaldo, Pozos-Estrada, Adrián, and Pozos-Estrada, Oscar. “Development of fragility curves of land-based wind turbines with tuned mass dampers under cyclone and seismic loading”. In: *Wind Energy* 24.7 (2021), pp. 737–753. ISSN: 1095-4244. DOI: <https://doi.org/10.1002/we.2600>. URL: <https://doi.org/10.1002/we.2600>.
- [256] Dermitzakis, Stavros N and Mahin, Stephen A. “Development of substructuring techniques for on-line computer controlled seismic performance testing”. Thesis. 1985.
- [257] Nakashima, Masayoshi, Kato, Hiroto, and Takaoka, Eiji. “Development of real-time pseudo dynamic testing”. In: *Earthquake Engineering Structural Dynamics* 21.1 (1992), pp. 79–92. ISSN: 0098-8847. DOI: <https://doi.org/10.1002/eqe.4290210106>. URL: <https://doi.org/10.1002/eqe.4290210106>.
- [258] Takahashi, Yoshikazu and Fenves, Gregory L. “Software framework for distributed experimental–computational simulation of structural systems”. In: *Earthquake Engineering Structural Dynamics* 35.3 (2006), pp. 267–291. ISSN: 0098-8847. DOI: [10.1002/eqe.518](https://doi.org/10.1002/eqe.518). URL: <https://dx.doi.org/10.1002/eqe.518>.
- [259] Neild, S. A., Stoten, D. P., Drury, D., and Wagg, D. J. “Control issues relating to real-time substructuring experiments using a shaking table”. In: *Earthquake Engineering Structural Dynamics* 34.9 (2005), pp. 1171–1192. ISSN: 0098-8847. DOI: [10.1002/eqe.473](https://doi.org/10.1002/eqe.473). URL: <https://dx.doi.org/10.1002/eqe.473>.
- [260] Dimig, J., Shield, C., French, C., Bailey, F., and Clark, A. “Effective Force Testing: A Method of Seismic Simulation for Structural Testing”. In: *Journal of Structural Engineering* 125.9 (1999). doi: [10.1061/\(ASCE\)0733-9445\(1999\)125:9\(1028\)](https://doi.org/10.1061/(ASCE)0733-9445(1999)125:9(1028)), pp. 1028–1037. DOI: [10.1061/\(ASCE\)0733-9445\(1999\)125:9\(1028\)](https://doi.org/10.1061/(ASCE)0733-9445(1999)125:9(1028)). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(1999\)125:9\(1028\)](https://doi.org/10.1061/(ASCE)0733-9445(1999)125:9(1028)).
- [261] Shao, Xiaoyun and Griffith, Chelsea. “An overview of hybrid simulation implementations in NEES projects”. In: *Engineering Structures* 56 (2013), pp. 1439–1451. ISSN: 0141-0296. DOI: <https://doi.org/10.1016/j.engstruct.2013.07.008>. URL: <https://www.sciencedirect.com/science/article/pii/S0141029613003325>.
- [262] McCrum, D. P. and Williams, M. S. “An overview of seismic hybrid testing of engineering structures”. In: *Engineering Structures* 118 (2016), pp. 240–261. ISSN: 0141-0296. DOI: [10.1016/j.engstruct.2016.03.039](https://doi.org/10.1016/j.engstruct.2016.03.039). URL: <https://dx.doi.org/10.1016/j.engstruct.2016.03.039>.
- [263] Bonnet, P and Bonnet, P. A. “The development of multi-axis real-time substructure testing”. Thesis. 2006.
- [264] Magonette, Georges. “Development and application of large-scale continuous pseudo-dynamic testing techniques”. In: *Philosophical Transactions of the Royal Society of London. Series A: Mathematical, Physical and Engineering Sciences* 359 (2001), pp. 1771–1799.

-
- [265] Pegon, P. and Pinto, A. V. “Pseudo-dynamic testing with substructuring at the ELSA Laboratory”. In: *Earthquake Engineering Structural Dynamics* 29.7 (2000), pp. 905–925. ISSN: 0098-8847. DOI: 10.1002/1096-9845(200007)29:7<905::aid-eqe941>3.0.co;2-p. URL: [https://dx.doi.org/10.1002/1096-9845\(200007\)29:7%3C905::aid-eqe941%3E3.0.co;2-p](https://dx.doi.org/10.1002/1096-9845(200007)29:7%3C905::aid-eqe941%3E3.0.co;2-p).
 - [266] Pinto, A. V., Pegon, P., Magonette, G., and Tsionis, G. “Pseudo-dynamic testing of bridges using non-linear substructuring”. In: *Earthquake Engineering Structural Dynamics* 33.11 (2004), pp. 1125–1146. ISSN: 0098-8847. DOI: 10.1002/eqe.393. URL: <https://dx.doi.org/10.1002/eqe.393>.
 - [267] McCrum, D. P. and Broderick, B. M. “Evaluation of a substructured soft-real time hybrid test for performing seismic analysis of complex structural systems”. In: *Computers Structures* 129 (2013), pp. 111–119. ISSN: 0045-7949. DOI: <https://doi.org/10.1016/j.compstruc.2013.02.009>. URL: <https://www.sciencedirect.com/science/article/pii/S0045794913000576>.
 - [268] Bertrand, D., Grange, S., and Charrié, J. B. In: (2022). ISSN: 2352-7102. DOI: <https://doi.org/10.1016/j.jobe.2022.104420>. URL: <https://www.sciencedirect.com/science/article/pii/S2352710222004338>.
 - [269] Hickey, John. “Numerical and Experimental Investigation of the Seismic Performance of Steel Concentrically Braced Frame Structures”. Thesis. 2019.
 - [270] McCrum, Daniel Patrick, Simon, Jordan, Grimes, Michael, Broderick, Brian Michael, Wrzesien, Andrzej, and Lim, James Boon Piang. “Hybrid seismic testing of cold-formed steel moment-resisting frames”. In: *Proceedings of the Institution of Civil Engineers - Structures and Buildings* 173.2 (2020), pp. 88–98. DOI: 10.1680/jstbu.18.00017. URL: <https://www.icevirtuallibrary.com/doi/abs/10.1680/jstbu.18.00017>.
 - [271] Schellenberg, A. H., Mahin, S. A., and Fenves, G. L. *Advanced Implementation of Hybrid Simulation, PEER Report 2009-104*. Report. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, 2009.
 - [272] Biswas, BN, Chatterjee, Somnath, Mukherjee, SP, and Pal, Subhradeep. “A discussion on Euler method: A review”. In: *Electronic Journal of Mathematical Analysis and Applications* 1.2 (2013), pp. 2090–2792.
 - [273] Mosqueda, Gilberto, Stojadinovic, Bozidar, and Mahin Stephen, A. “Real-Time Error Monitoring for Hybrid Simulation. Part I: Methodology and Experimental Verification”. In: *Journal of Structural Engineering* 133.8 (2007). doi: 10.1061/(ASCE)0733-9445(2007)133:8(1100), pp. 1100–1108. DOI: 10.1061/(ASCE)0733-9445(2007)133:8(1100). URL: [https://doi.org/10.1061/\(ASCE\)0733-9445\(2007\)133:8\(1100\)](https://doi.org/10.1061/(ASCE)0733-9445(2007)133:8(1100)).
 - [274] Zhang, Zili. “Passive and Active Vibration Control of Renewable Energy Structures”. Thesis. 2015. DOI: 10.5278/vbn.phd.engsci.00024.
 - [275] Dong, Xiaohui, Tang, Zhenyun, and Du, Xiuli. “State of the art and development trends in numerical simulation for real-time hybrid simulation”. In: *Earthquake Engineering and Resilience* n/a.n/a (2022). <https://doi.org/10.1002/eer2.25>. ISSN: 2770-5706. DOI: <https://doi.org/10.1002/eer2.25>. URL: <https://doi.org/10.1002/eer2.25>.