Influence of diaphragm modelling on the dynamic performance of a reinforced concrete high-rise building

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Abstract
The present study investigated the dynamic behaviour of an irregular 32-storey reinforced concrete, Vancouver-style, high-rise, podium tower by assessing the influence of the floor diaphragm modelling stiffness assumptions on the structure's modal properties. The state-of-the-art diaphragm modelling approaches and their effect on structures' dynamic performance are currently related to symmetrical buildings with multiple shear walls. The influence of horizontal floor diaphragms on the dynamic behaviour of irregular, high-rise buildings with concrete core has not been sufficiently investigated either on a numerical or experimental basis. In this context, the primary objectives of the current study are: (a) to enhance the comprehension of the horizontal floor diaphragm behaviour of a high-rise building with a concrete core and, (b) to quantify the effect of varying diaphragms modelling approaches on the dynamic properties of the structure.

1 Introduction

Most existing civil engineering structures as buildings, bridges and other infrastructures are unique structures and the real dynamic performance capacity of these structures are relatively uncertain. For designers' convenience and saving computational power, the code-confronted design of structural systems is commonly subjected to several simplifications and crude assumptions that may not reflect the actual static and, primarily, dynamic performance of structural systems. The latter is more profound for complex structures that deviate from symmetrical configurations and typical structural systems with regular distribution of the mass and stiffness both in plan and elevation. Today engineers are challenged more than never by the request of designing irregular and code-breaking structures, where the fundamental engineering principles are deficient as the design basis. Therefore, it is of high importance to address the accuracy of those assumptions thoroughly and investigate their effect on the structural response evaluation that, in turn, constitutes the concrete basis of any decision needs to be made by the relevant stakeholders for repairing or retrofitting of existing structures. Future design of structures and infrastructure systems can benefit and be optimised by revising the primary design considerations, and quantifying their impact on the actual structural performance. Designers frequently treat the horizontal plates at each storey level, as rigid diaphragms and this simplification, though a convenient approach regarding modelling perspective, dominates the current design practice. Such an infinite stiffness assumption for the diaphragm modelling may be suitable for somewhat regular and symmetrical reinforced concrete (RC) buildings. For complex structures, the validity of this assumption is questionable, in which the diaphragms response affects the lateral performance of the entire structural system. The engineering society is missing, how accurate is the real diaphragm performance captured both in experimental and numerical investigations, to identify a refined modelling approach for the diaphragms.

Current research efforts describe the suitability of the rigid diaphragms modelling approach, evaluated for different structural systems. For example, Saffarini and Qudaimat [1], investigated 37 reinforced concrete structures of varying height, structural configuration and dimensions for the storeys as well as the columns and the shear walls. They concluded that the rigid-floor assumption captures less accurately the dynamic response of buildings with shear walls compared to structural systems that the beam-column frames have been designed to resist the lateral forces. Moreover, Ju and Lin [2] undertook a thorough response analysis of 520 buildings with varying configurations, i.e., T-shaped, U-shaped and rectangular shaped buildings, and they highlighted that the rigid diaphragm assumption was found to be associated with the less accurate calculation of the building response compared to the flexible diaphragm approach. A study to assess the influence of floor diaphragm flexibility on the dynamic response of torsional-sensitive asymmetric buildings was carried out by Basu and Jain [3], and they found the flexible diaphragms modelling approach is somewhat deficient in capturing the accuracy of lateral performance of buildings with torsional sensitivity. Additionally, the lateral performance of three existing building systems, modelled with both flexible and rigid floor diaphragms respectively, was comparatively assessed by Tena-Colunga and Abrams [4] revealing higher deformations (i.e., accelerations and displacements) calculated for the building systems with the flexible diaphragms. The torsional response was also found reduced by increasing the diaphragms flexibility that, in turn, affected the dynamic properties of the building by elongating their natural building period.

Moeini and Rafezy [5], assessed the diaphragm modelling relation to the code provision, who performed modal response spectrum analysis of RC buildings with T-shaped, U-shaped and rectangular structural configuration. The study divides existing building codes and standards into two categories regarding the recommended diaphragm modelling assumptions. Primarily, according to the Eurocode 8 (EN1998-1), the New Zealand code (NZS4203) and the Chinese one (GSC-2000) the decision regarding the diaphragms modelling is based on qualitative criteria associated with the shape of the floor diaphragm. On the other hand, the Iranian 2800 code as well as the UBC 97, SEAOC-90 and FEMA-273, prescribe quantitative criteria related to the in-plane deformation of the plates to define whether the diaphragm should be treated as fixed or flexible. However, the latter approach was concluded deficient due to the deformation of the diaphragm is dependent on the acting force. Moeini and Rafezy [5] were in favour of modelling the horizontal floor plated with shell elements rather than beam finite elements, widely used in the engineering practice.

The studies mentioned above addressed the influence of different diaphragms modelling approaches on the dynamic properties (i.e., Eigen frequency and mode shapes) and the numerically-calculated response of various structures, missing the investigation of typical RC high-rise podium towers with irregular distribution of mass and stiffness along the height and in-plane. Therefore, the City Crest Tower, being an emblematic tall building from early 1990's located in downtown Vancouver (British Columbia, Canada), was chosen as the testbed to investigate whether the rigid or flexible diaphragm modelling approach respectively captured more accurately the actual dynamic response. To this end, the measured response data sets were used to identify the modal properties of the building that, in turn, enabled the refinement of a detailed finite element model increasing, eventually, the reliability and accuracy of the investigation results. The primary objectives of the current study are summarised below:

- Identification of the benchmark structure's modal properties based on the measured response data that was obtained during the ambient vibration tests.
- Development of the structure's finite element model, which is refined further by experimentally, identified dynamic properties.
- Investigation of the suitability of the different diaphragms modelling approaches and the related assumptions (i.e., rigid or flexible diaphragm) for the specific structural system accounting for the identified dynamics properties.
- Assessment of the effect the diaphragm modelling assumptions have on the dynamic response for the high-rise RC building presented in this study.

2 Constraints of Diaphragms and Numerical Modelling Aspects

Floor diaphragms/slabs in multi-storey buildings are often modelled as shells elements and discretisation of the diaphragms can be considered massive computational power-wise. To avoid massive computational models kinematic constraints are applied, and the constraints are often "master-slave" nodes for many automated structural analysis programs. A floor system in three dimensions with the shape of a rectangle and has four corner nodes. Each node has six degrees of freedom three translations and three rotations ones before any constraints are applied. If a structural element as columns or beams is attached to the floor system, this introduces additional six degrees of freedom per node. For large structures, the in-plane definition of the lower floors is often considered to be small compared with the horizontal inter-storey drift, and the applied master-slave approach will provide safe computation power, [6]. Reducing the degrees of freedom has disadvantages regarding the response of the floor system. Simplifying the behaviour of the floor leads to significant errors and non-conservative estimations, [3]. Figure 1, illustrates the master-slave approach for the floor system, the centre of mass is located at the point O, and a random point i is considered. By assuming the kinematic constraint, the diaphragm is rigid; the displacement of any node can be expressed as a linear function of the master node, in the point O. The figure shows displacements in the x-direction is illustrated as $u_x^{(O)}$, the corresponding y-direction is $u_y^{(O)}$ and the rotational displacement is illustrated as $u_{\theta z}^{(O)}$, [7].

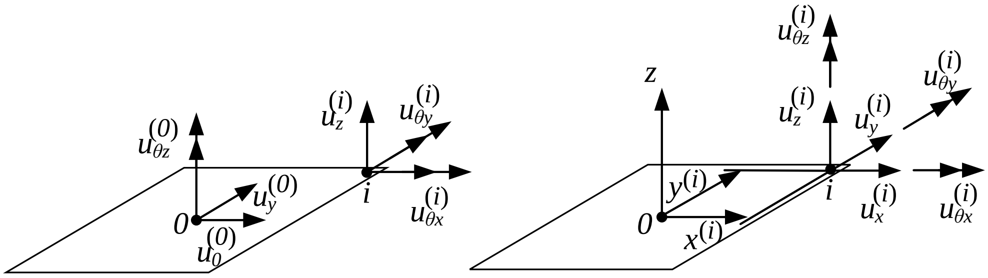


Figure 1: Master-slave diaphragm assumption. Left: 6 degrees of freedom node. Right: slave node with 3 degrees of freedom.

The compatibility equations yields, the displacement of any slave node is

$$\begin{aligned} u_x^{(i)} &= u_x^{(O)} - y^{(i)} u_{\theta z}^{(O)} \\ u_y^{(i)} &= u_x^{(O)} + x^{(i)} u_{\theta z}^{(O)} \\ u_\theta^{(i)} &= u_{\theta z}^{(O)} \end{aligned} \tag{1}$$

The kinematic constraints further yields any external force $\mathrm{f}_x^{(Oi)}$ associated with the rigid displacements, estimated with the equation above, is transform with respect to the master node. By static equilibrium, the slave node force is transformed to the master node, which yields, [7].

$$\begin{aligned} \mathrm{f}_x^{(Oi)} &= \mathrm{f}_x^{(i)} \\ \mathrm{f}_y^{(Oi)} &= \mathrm{f}_y^{(i)} \\ \mathrm{f}_{\theta z}^{(Oi)} &= \mathrm{f}_{\theta z}^{(Oi)} - y^{(i)} \mathrm{f}_x^{(i)} + x^{(i)} \mathrm{f}_y^{(i)} \end{aligned} \tag{2}$$

3 The City Crest Tower

The building adopted for investigation by the current study is the City Crest Tower (Figure 2), which is a Vancouver style high-rise 32-storey RC building reaching 83.2 m in height. The high-rise podium was constructed from autumn 1992 to January 1994 and is located in downtown Vancouver (British Columbia, Canada). The structure is representative of high-rise podium towers in Vancouver and was designed according to Vancouver Buildings By-law 6134, CAN/CSA-S413-87 for the parking structure and the Canadian RC concrete design standard CSA CAN3-A23.3-M84. The basement of the tower and a pair of walkway bridges constructed at the second level are used to facilitate the connection between the high-rise building and a two-storey townhouse. However, the latter is not a central part of the primary structural system of the tower and hence, is not further considered in this study. The typical storey height is 2.6 m except for the ground floor, designed deliberately higher, i.e., 4.0 m, to facilitate commercial activities.


Figure 2: South-East (left) and South-West (right) side of the City Crest Tower.

The main seismic force resisting system of the high-rise tower is the concrete core, located in the centre of each floor above the ground and designed to resist lateral forces. Additionally, the columns are a part of the load-bearing gravity system. The core and edge columns are continuous at each floor to support the in situ-cast floor slabs. From the 30th level and above, the columns at the perimeter are replaced with walls of 204 mm thickness. Regarding the concrete core per se, it measures 10.3 m by 7.6 m in the plan (Figure 3) with a varying wall thickness from 457 mm at the basement to 356 mm at the top floors. The core houses mechanical and electrical conduits, stairwells and elevator shafts. Figure 3 also illustrates the plan layouts for different floor levels. The lower floors (3-10) measure 24.7 m in East-West and 22.4 m in North-South direction respectively while the floor area is reduced thought out the height of the tower, from 600 m^2 for levels three to ten, 500 m^2 for levels 11 to 25, and the floor area is further reduced from levels 25 to 32. Additionally, the floor slabs are typically 191 mm thick with varying reinforcement, concentrated mainly around the columns and the concrete core while diaphragm reinforcement is placed along the slab edges. Significant variation regarding size, geometry and material properties (Table 1) are found for the columns, the shear walls and the beams designed for the entire building stemming from its basement up to higher elevations.

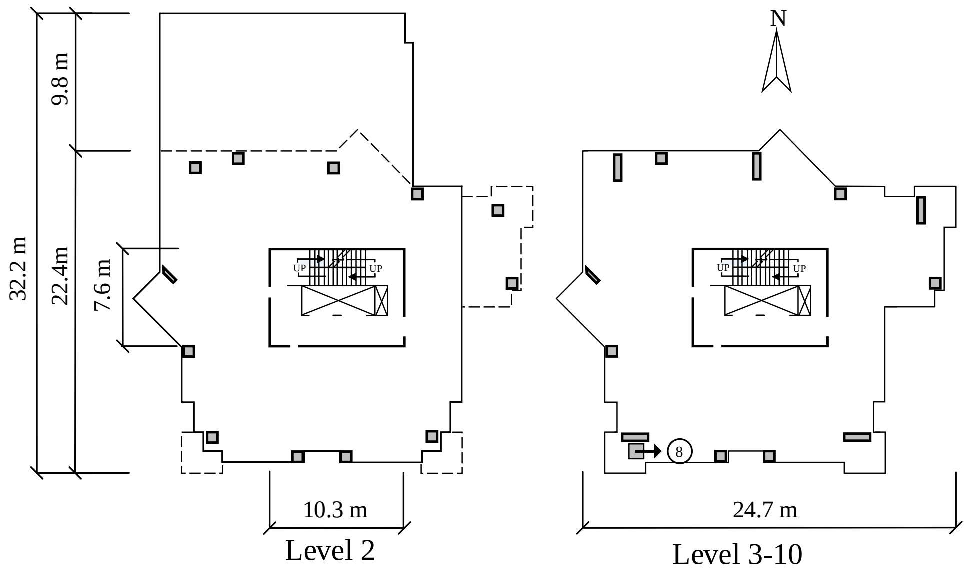


Figure 3: Level 2 (left) and level 3-10 (right) floor layout of the City Crest Tower.

Structural element/description	Strength [MPa]
Footings and walls	25
Tower core walls	
Level P6 – 5	25
Level 6 - 32	30
Suspended slabs and beams and retaining walls	30
Columns	
Level P6 – 4	40
Level 5 – 9	35
Level 10 – 19	30
Level 20 - 32	25
Exterior slabs, driveways, and other surfaces subject to de-icing salts (Class C-1 exposure)	35
All other concrete	20
Reinforcing steel	Grade 400

Table 1: Material properties adopted for designing the City Crest Tower [8].

An extensive setback, found from level 10 at the Southeast corner of the City Crest Tower (Figure 2), reduces the number of the perimeter columns introducing, in such a way, non-uniform mass and stiffness distribution along the height of the structure's height. The latter, commonly considered as a primary source of structural irregularity, is heightened by additional, smaller though, setbacks that can be found for the higher floors of the tower. The extension of the second floor by 9.8 m, being connected to the tower with a monolithic slab connection, contributes further to the asymmetric configuration of the high-rise tower increasing its structural irregularity. It is notable that the slab of the extended second floor has been designed with a varying thickness between 178 mm and 229 mm while the slab thickness of the second floor's central area is 216 mm.

Regarding the basement of the podium tower, six staggered levels were constructed to accommodate parking areas measuring 62.3 m in the East-West direction and 35.8 m in the North-South direction while the sixth parking level is located 9.4 m below the ground surface. Different types of slab bands with varying thickness, i.e., from 406 mm to 457 mm, were applied for the parking-related levels. Finally, both strip and footing foundations were used to support the entire building and designed to undertake loads of the columns. The footing foundations are varying significantly regarding their size since the smaller ones measure 2.1 m by

2.4 m in plan and 0.7 m thick while the most massive footing column foundations were calculated to be square of 4.1 m length and 1.2 m thickness. The concrete core was also supported by a stiff mat foundation measuring 12.8 m by 16.2 m with a thickness of 3.1 m. The line foundations varying in width (0.6 m up to 1.8 m) and thickness (0.3 m up to 0.5 m) were used to support the shear walls.

4 Ambient Vibration Tests and Operational Modal Analysis

The ambient vibration test of the City Crest Tower was performed in the summer of 1993 with eight Kinematic Inc. force balance sensors, i.e., the FBA-11 accelerometer, connected with the signal conditioner by using cables of 90 m up to 300 m length. The signal conditioner filtered and amplified the raw FBA signals that, afterwards, were converted from analogue to digital format. The sensor locations, as illustrated in Figure 4, were carefully chosen to capture the dynamic characteristics of the building, i.e., the natural frequencies and mode shapes efficiently. Primarily, the guidelines of the California Strong Motion Instrumentation Program, [9], were followed to place the sensors and ensure adequate isolation of the translational and rotational vibrations at each storey level in both the North-South and East-West direction respectively. It is worth mentioning that during the ambient vibration tests, the City Crest Tower was under construction and partition walls had been installed only for the first five floors while the building's roof had been cast two weeks prior the test. The sensors were mounted with anchor bolts to the concrete floor slabs. The ambient vibrations experimental campaign included 19 different test setups, with four roving sensors and four reference sensors each. The latter was placed at the 29th floor at antinodes of the vibration modes of interest. The sampling rate was set to 40 Hz, ensuring a Nyquist frequency of 20 Hz while the acceleration measurements were filtered onsite with a high pass filter of 0.1 Hz and a low pass filter of 12.5 Hz. The duration of each measured acceleration signal was 13 minutes and 39.2 seconds, corresponding to 32768 data points.

The measured response of the City Crest tower during the ambient vibration tests was used to identify the dynamic properties of the building by applying the Operational Modal Analysis (OMA) is a widely used method for system identification [10] of civil engineering structures. Figure 5 (left) illustrates the geometrical model of the high-rise structure created by using the commercial software ARTeMIS Pro Modal (v4) [11], applied to perform the OMA. The blue arrows represent the data channels being associated with the reference sensors placed on the 29th floor while the pink and the green arrows are the projection and data channels respectively. Furthermore, the singular values of the spectral densities, calculated for calculated for all measurement setups by using ARTeMIS Pro Modal (v4), were plotted by Figure 5 showing well identified (tapered bell shape) natural frequencies of the high-rise structure within the frequency range of 0-12 Hz. It is notable that the spectral density estimation/segment size used was taken equal to 1024 while no additional filters were applied to derive the singular values.

A pair of OMA-based identification techniques, namely the Enhanced Frequency Domain Decomposition (EFDD) and the Stochastic Subspace Identification – Unweight Principal Component (SSI - UPC) technique respectively, was used to estimate the dynamic properties of the City Crest Tower including the natural frequencies and the modal damping as well as the mode shapes. Table 2 lists the identified properties for the first eight translational and first four rotational modes respectively ranging, regarding natural frequencies, from 0.55 Hz (NS – first translational mode, EFDD) to 10.27 Hz (fourth rotational mode, SSI-UPC). Furthermore, the first two translational modes along the North-South and East-West directions were identified as closely spaced modes since the EFDD identification technique resulted in 0.55Hz and 0.65 Hz respectively. The first rotational mode was identified with a natural frequency of 1.29 Hz. As can be seen, by Table 2, the two identification techniques, i.e., the EFDD and the SSI – UPC, led to estimating somewhat similar natural frequencies for the 12 modes investigated with the highest difference being of 4.4%. On the other hand, the EFDD and SSI-UPC techniques resulted in slightly different estimates for the modal damping ratios that were found to be in the range of 0.8% to 3.53% with a maximum difference of 130.1%. The latter is mainly attributed to the increased uncertainty, generally related to the damping estimation. It is to the best knowledge of the authors that the SSI-UPC technique usually provides estimates of modal damping ratios with higher accuracy compared to the EFDD-based estimates especially when the duration of the acceleration signals is shorter than 20 minutes [12]. Finally, the first four translational (NS1, EW1, NS2 and EW2) modes and the first two torsional modes (T1 and T2) determined with the SSI – UPC method, are illustrated at Figure 6.

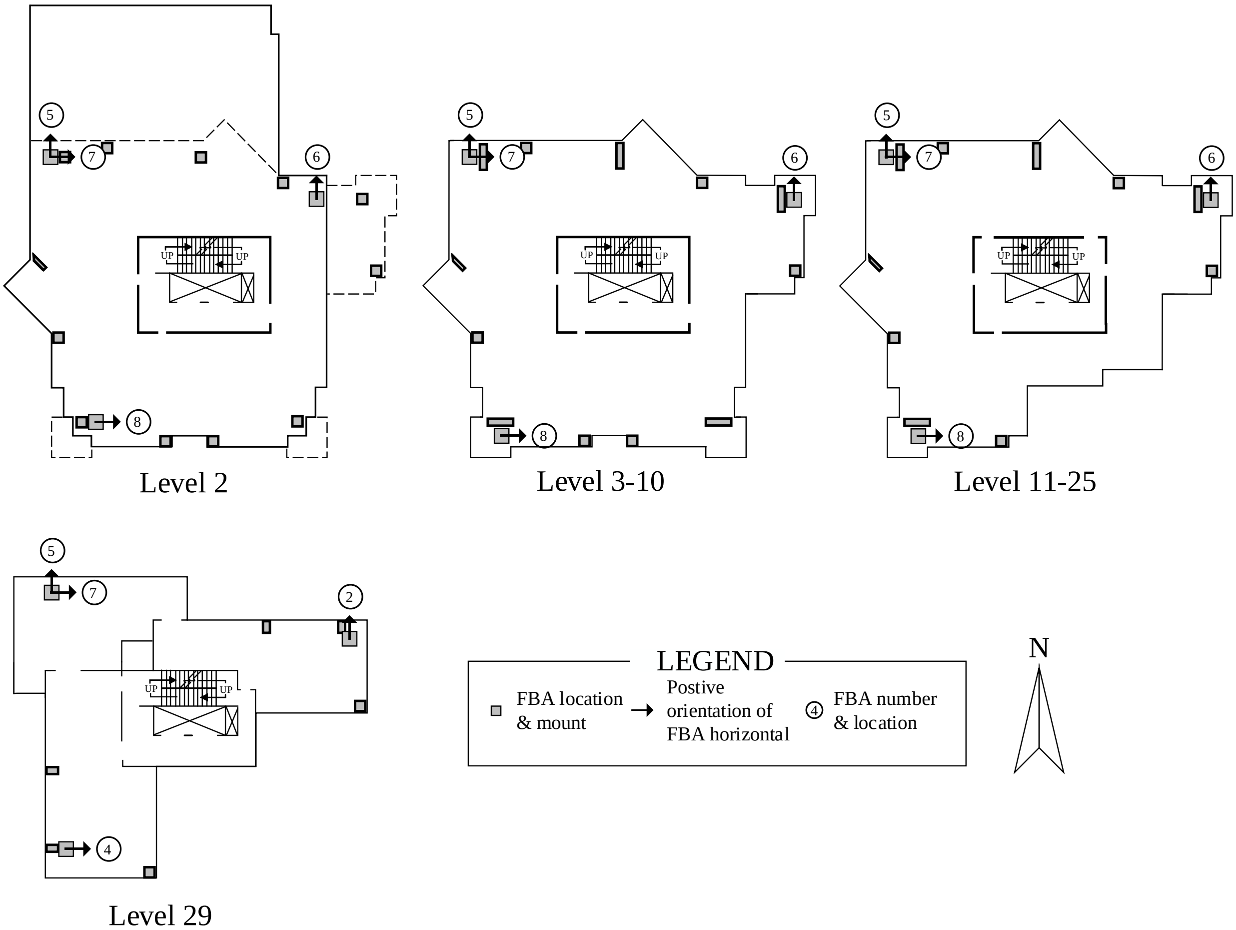


Figure 4: Sensors layout at different levels

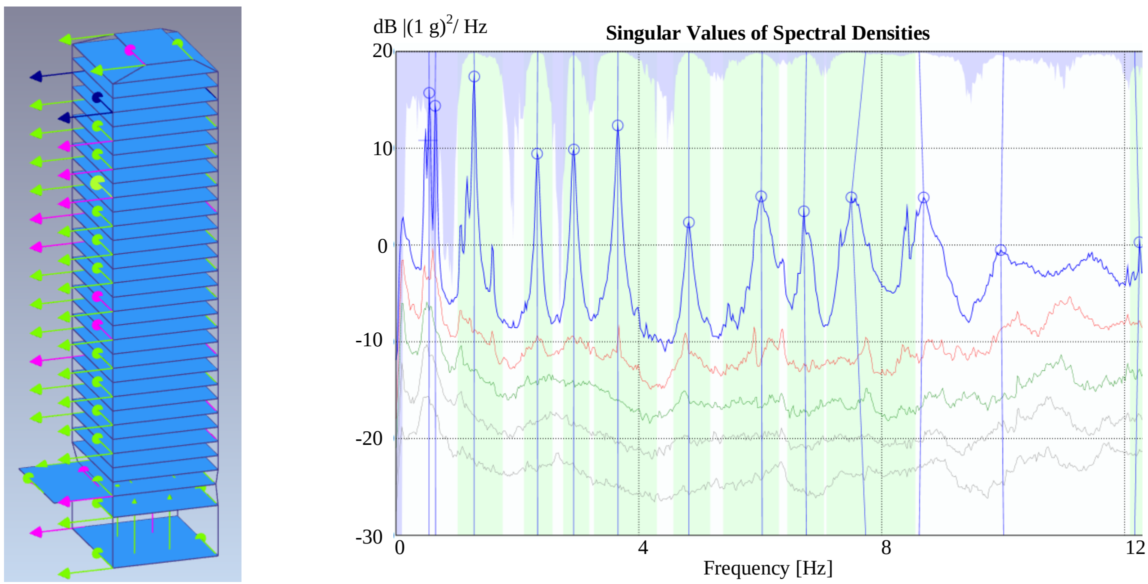


Figure 5: Computer model in ARTeMIS for the Operational Modal Analysis (left) and the singular values of spectral densities (right).

Method Mode	EFDD Frequency [Hz]	SSI - UPC Frequency [Hz]	Frequency Difference (%)	EFDD Damping (%)	SSI - UPC Damping (%)	Damping Difference (%)
NS 1	0.55	0.57	4.4	3.53	2.88	-18.4
EW 1	0.65	0.64	-2.0	2.09	3.41	63.7
T 1	1.29	1.26	-2.1	1.04	2.31	121.8
NS 2	2.33	2.35	0.8	1.05	1.71	62.4
EW 2	2.92	2.93	0.2	1.04	0.92	-11.8
T 2	3.66	3.55	-2.8	0.80	1.42	77.6
NS 3	4.82	4.83	0.2	1.17	1.61	37.7
EW 3	6.03	6.03	0.0	1.62	1.94	19.9
T 3	6.76	6.80	0.6	1.20	1.91	59.3
NS 4	7.74	7.49	-3.2	1.67	1.93	16.0
EW 4	8.62	8.79	2.1	1.71	2.06	20.5
T 4	10.01	10.27	2.6	1.35	3.11	130.1

Table 2: Natural frequencies and damping ratios identified from the ambient vibration test

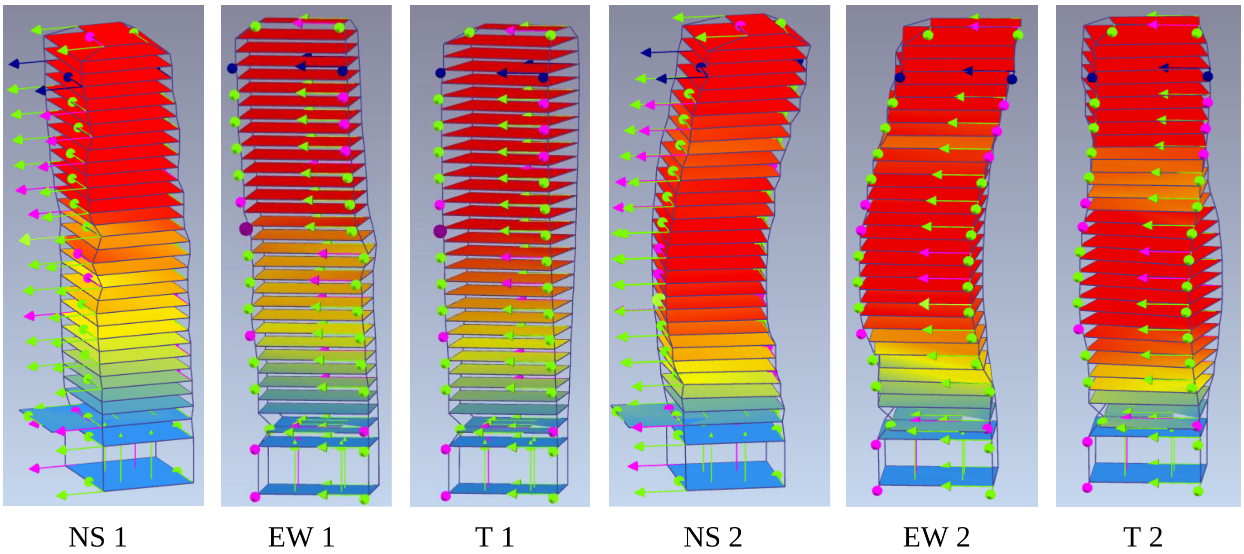
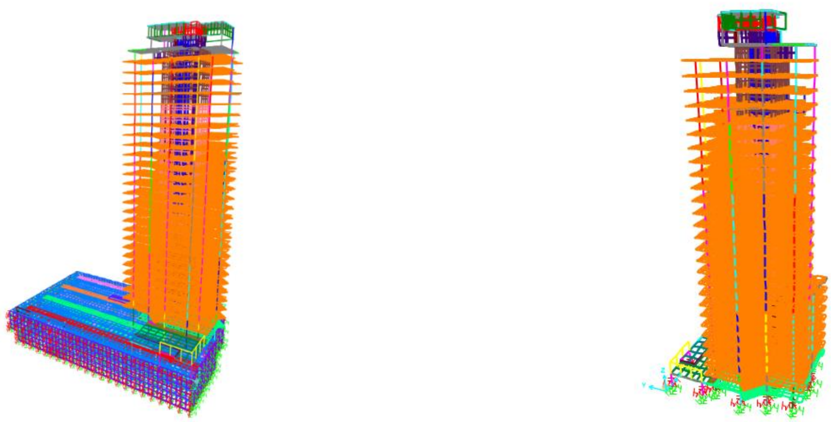


Figure 6: The first six mode shapes from the ambient vibration test.

5 Finite Element model

A linear Finite Element (FE) model of the City Crest Tower was developed to investigate the effect of different diaphragms modelling approaches on the dynamic performance of the high-rise structure. By strictly following the building's drawings being available to the authors, a high level of details were applied for the development FE model, with beams, columns, shear walls and slabs of varying size and complexity, found at each floor level, was modelled by the use of SAP2000 [13]. To utilise the OMA-based identification of the dynamic properties of the structure, for the refinement of the finite element model, the FE model should reflect precisely the structure, for which the ambient tests were performed. Therefore, under normal operating conditions of a building the opening and closing of microcracks in the elements of a building is very small and that the assumption of an "uncracked" section is justifiable. The detailed modelling of the reinforcement was outside the scope of the current study. Hence, the applied concrete mass density was adopted equal to 2500 kg/m^3 accounting, in such a way, for the additional mass density related to the reinforcement steel bars [14]. Moreover, the different strength used for the concrete material (Table 1) led to different values for Young's modulus that was applied for the structural members modelled as either beam elements (columns and beams) or shell elements (floor slabs, shear walls and basement walls). The shell elements enable modelling of the transverse shear deformation by the Mindlin/Reissner formulation, while the bending shell stiffness is also considered [15].

Regarding the diaphragms considered at each story level, three modelling approaches were adopted to investigate their influence on the structure's global dynamic response, captured already during the ambient vibration test campaign. Along these lines, the diaphragms were modelled by the current study as flexible, semi-flexible and rigid ones. For the latter case, being the most common chosen for designers and engineers due to the computation power savings and simplicity, a constraint is applied to the nodes of the slab and the total mass is concentrated at the centre of the diaphragm condensing, in such a way, the associated stiffness matrix. Infinite in-plane stiffness properties are also related to the rigid diaphragm while, for the earthquake-induced lateral loads, the accidental eccentricity is applied to the centre of mass of the diaphragm [15]. On the other hand, no constraints are assigned for the diaphragms flexible modelling approach, and the floor-associated masses are not lumped while no modification is applied to both the flexural and shear stiffness of the shell elements, used to model the diaphragm. The semi-rigid diaphragm assumption, which lies between the rigid and the flexible assumptions, accounts for the actual in-plan stiffness of the slab. The accidental eccentricity, generally considered in seismic analysis, is applied to each node of the shell elements.



FE-Model with basement FE-Model without basement

Figure 7: The two FE-models with and without the basement.

As it concerns the RC tower's six-story basement, an investigation was carried out to identify the effect that such a rigid RC block underneath the soil surface may have on the dynamic performance of the high-rise building studied. To this end, two numerical models were created with and without the multi-storey basement (Figure 7). For both FE models, fixed boundary conditions were considered due to the firm soil found on the structure's site, i.e., Soil Class C according to NBCC 2015 with average shear wave velocity at the upper 30 m of the soil profile (v_{s30}) equal to 450 m/s [16] that limits the soil-structure interaction phenomena [17]. The results of the eigenvalue analysis, carried out by using SAP2000, are presented by Table 3 listing the modal frequencies for both the FE models, i.e., with and without the basement. For the 12 modes considered, the numerical model of the high-rise structure without the basement led to estimating natural frequencies that are in better agreement with the experimentally derived ones in section four. Notably, the average difference between the FE model and the experimental frequencies for the first three modes was found to be equal to 11.0 % and 9.1 % for the model with and without the basement respectively. Therefore, the authors chose to disregard the FE model of the high-rise structure with the basement and continue the investigation by using the FE model without the basement since its associated frequencies were found to be in closer agreement with the experimentally derived modal frequencies. The investigation concerning the effect of the diaphragms modelling approaches on the dynamic response of the City Crest

Tower was solely based on the non-basement numerical model. The latter was found to be associated with mode shapes (Figure 8) that are in agreement with ones estimated using the ambient vibration analysis results (Figure 6).

	OMA (SSI)	**Without basement**		**With basement**	
Mode	**Frequency [Hz]**	**Frequency [Hz]**	**Difference (%)**	**Frequency [Hz]**	**Difference (%)**
NS 1	0.57	0.54	-5.3	0.49	-14.0
EW 1	0.64	0.68	6.3	0.61	-4.7
T 1	1.26	1.46	15.9	1.44	14.3
NS 2	2.36	2.50	5.9	2.30	-2.5
EW 2	2.93	3.18	8.5	2.91	-0.7
T 2	3.55	4.33	22.0	4.25	19.7
NS 3	4.83	5.38	11.4	5.08	5.2
EW 3	6.03	6.82	13.1	6.49	7.6
T 3	6.80	7.31	7.5	7.08	4.1
NS 4	7.49	8.38	11.9	8.07	7.7
EW 4	8.79	9.32	6.0	9.10	3.5
T 4	10.27	10.58	3.0	10.01	-2.5

Table 3: Frequencies and the modal mass participation ratios for both FE models.

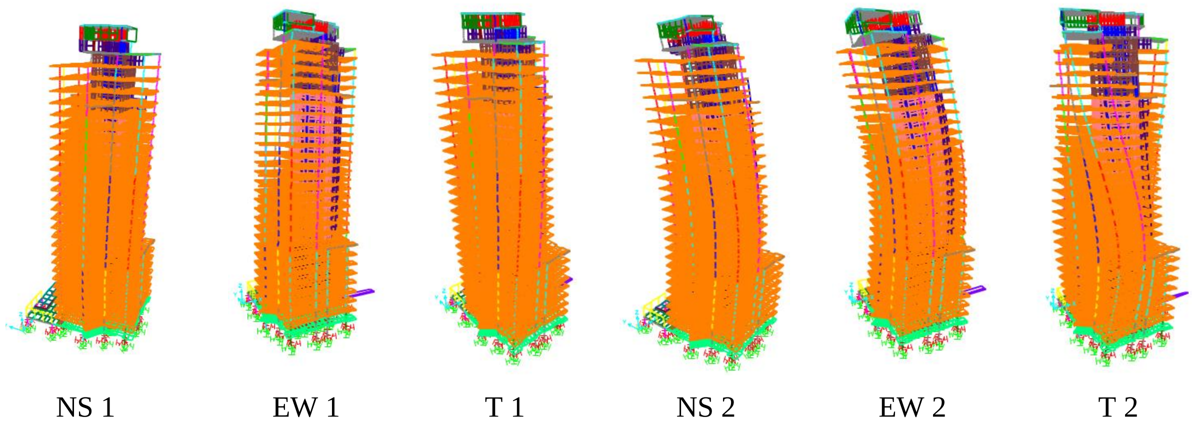


Figure 8: The first six mode shapes from the FE model without the basement

6 Effect of diaphragms assumptions on dynamic performance

Engineers use analysis to verify the design of a structure, and need to make simplifications and implement modelling assumptions that can affect significantly the reliability of the analysis results, and could potentially lead to erroneous conclusions about the expected behaviour of the structure. Along these lines, the choice of the most appropriate diaphragm modelling approach is not a trivial task, and hence, the rigid, semi-rigid and flexible diaphragm's numerical modelling approaches considered enabled investigating their influence on the estimated dynamic properties of the City Crest Tower. To this end, Table 4 provides the basis for a thorough comparative assessment modal behaviour (i.e., vibration frequencies), estimated by performing eigenvalue analysis of the FE model of the high-rise building with different diaphragm's modelling approaches. The suitability of the diaphragm above's assumptions is assessed by comparing the

FE model-estimated modal frequencies with the ones identified by performing OMA on the measured vibration responses. Comparing the first four translational (NS1, EW1, NS2 and EW2) and the first two torsional FE model-related modes (T1 and T2) with their experimental counterparts obtained by using the SSI-UPC technique, it is seen that the rigid diaphragm modelling approach led to natural frequencies that deviate significantly (i.e., the average difference is equal to 35.1 %) from the OMA-identified ones. On the other hand, both the semi-rigid and the flexible diaphragm modelling approaches were found to result in modal frequencies being significantly closer to the ones identified by using the measured vibration responses, i.e., the average relative difference was calculated to be equal to 9.9 % and 8.7 % for the semi-rigid and flexible diaphragms modelling approach respectively. Indeed, for the first two translational modes along the main directions of the high-rise building (NS, EW), the latter two approaches led to quite accurate prediction of the modal frequencies since the relative difference from the OMA-identified ones were found to be within the range of 5.3 % and 8.5 %. Contrarily, the rigid diaphragm modelling approach led to translational frequencies deviating from the experimental ones with relative difference higher than 54%. Slightly lower, though still excessive, deviation (i.e., relative difference up to 44%) was calculated for the first two rotational modes-related frequencies, which were estimated almost identically by applying the semi-rigid and flexible diaphragm's modelling approaches leading to a maximum relative difference of 22.0% from the OMA-identified frequencies.

Mode	OMA (SSI)	Flexible		Semi-rigid		Rigid	
	Frequency [Hz]	**Frequency [Hz]**	**Difference (%)**	**Frequency [Hz]**	**Difference (%)**	**Frequency [Hz]**	**Difference (%)**
NS 1	0.57	0.54	-5.3	0.54	-5.3	0.88	54.4
EW 1	0.64	0.68	6.3	0.68	6.3	0.97	51.6
T 1	1.26	1.46	15.9	1.45	15.1	1.79	42.1
NS 2	2.35	2.50	6.4	2.50	6.4	2.61	11.1
EW 2	2.93	3.18	8.5	3.17	8.2	3.14	7.2
T 2	3.55	4.33	22.0	4.31	21.4	5.12	44.2

Table 4: Experimental and FE model-based natural frequencies of the benchmark building accounting for the three different diaphragm modelling approaches considered.

Based on the results presented by Table 4 and briefly discussed above, the rigid diaphragm assumption, widely adopted by building designers, was found to result in natural frequencies significantly different than the experimental ones. The latter, estimated significantly higher than the corresponding OMA-identified natural frequencies, can be attributed to the artificially high floor stiffness, being related to the simplified modelling approach of rigid diaphragms. On the other hand, such a deviating modal behaviour was not detected for the semi-rigid, and flexible diaphragms are modelling approaches since the corresponding FE model-related estimates for the natural frequencies are in good agreement with those experimentally identified from the actual vibration response of the 32-storey RC building. It is observed that the first NS translational natural frequency estimated by the FE model accounting for either the semi-rigid or the flexible diaphragms, was found slightly lower than the OMA-identified one while, for the rest five frequencies presented (Table 4), the FE-model based estimates were lower than the experimental ones. This peculiar result, already reported by Schuster [18] by OMA identification results for the City Crest Tower, may be attributed to deviations between the designed structural members and the ones eventually constructed at the building's site. For example, construction imperfections regarding the appropriate reinforcement of the tower's walls can modify the stiffness distribution along the height of the structure that, in turn, affects the vibration frequencies. A thorough on-site investigation is necessary to elaborate on this peculiar result identification result.

In addition to evaluating the effects of diaphragm flexibility on the modal frequencies, these effects were also investigated by comparing the modes shapes obtained from the FE-models and those obtained experimentally. This was done by using the Modal Assurance Criterion (MAC) to compare the vectors of the mode shapes. Notably, the MAC values calculated higher than 91% (Table 5 for the semi-rigid and the flexible diaphragm modelling approaches corroborate their superiority in predicting with increased reliability the modal behaviour of City Crest Tower. The less accurate prediction of the building's mode shapes was found to be associated with the rigid diaphragm since an average MAC value of 73.0 % was calculated for the first translational and two rotational modes considered.

Mode Shapes	Flexible MAC (%)	Semi-rigid MAC (%)	Rigid MAC (%)
NS 1	95.3	95.3	85.8
EW 1	97	97.0	76.1
T 1	98.2	98.2	88.3
NS 2	95.3	95.3	60.4
EW 2	91.9	91.9	58.2
T 2	92.8	92.7	69.3

Table 5: Modal Assurance Criterion values for the three different diaphragm-modelling approaches.

7 Conclusion

The outcome of the current study carried out for an existing RC high-rise irregular building located in Vancouver, Canada, highlights the effect of three different, widely adopted though, floor diaphragms modelling approaches on the reliable prediction of the global modal behaviour of the structure. Along these lines, the natural frequencies of the City Crest Tower, being a single RC core structural system with irregular stiffness distribution both in-plane and elevation, were experimentally identified by applying OMA techniques. These modal estimates were compared with the natural frequencies found on FE model basis accounting for three diaphragm's modelling approach: flexible, semi-flexible and rigid. The latter approach led to natural frequencies deviating significantly from the experimentally obtained ones (i.e., the relative difference was calculated, on average, equal to 35.1 % considering the first four translational and the first two rotational modes). On the contrary, both the flexible and the semi-rigid diaphragm modelling approaches resulted in natural frequencies that deviate, on average, from the experimental ones with less than 10 % considering the first four translational and first two rotational modes respectively. The assumptions of flexible and the semi-rigid diaphragm in the FE model led to closer modal results (frequencies and mode shapes) with those obtained experimentally than those obtained based on a rigid-floor assumption. MAC values higher than 91% were calculated for the FE models with the semi-rigid and flexible diaphragms while, on the other hand, the rigid diaphragm affected the estimation of the mode adversely shapes adversely since the associated, on average, MAC value for the six modes considered was calculated equal to 73.0%.

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