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Analysis and construction of transfer structures: a case study

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Abstract

A transfer structure of a building is a structure that alters the load path of the gravity loads, shifting the line of thrust laterally to a different vertical alignment. They usually represent major elements of a structure and their impact on building cost and construction time can be substantial. The design for deflection control is frequently the primary consideration when defining the geometry of a transfer element, given their usual long spans and the high magnitude of the forces involved. Considerations of robustness and disproportionate collapse may also be key for the design of transfer structures as these are often regarded as critical elements for the overall stability of the building. Furthermore, the sequence of construction should be modelled through a construction-staged analysis, as it greatly affects deflections as well as final forces distribution. This paper provides guidance and highlights the key aspects that are likely to determine the structural design of transfer structures, as well as typical construction methods and challenges. It also presents a comparative study between two different types of transfer structures for the same building: an alternative composite scheme for the post-tensioned concrete transfer grid of the St. Gabriel Tower (Lisbon) was designed and the most relevant factors are highlighted.

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Keywords: Type your keywords here, separated by semicolons ;

1. Introduction

Functional, aesthetic, or planning needs predicate changes and discontinuities in the vertical load-bearing system of a building. These demands are often outside the boundaries of normal commercial development and create special and interesting engineering problems that are usually solved with some form of transfer structure.

Transfer structures provide a means of redirecting gravity loadings when a vertical supporting member has to be interrupted and a direct load path to the foundations is not possible. There are several reasons for which discontinuities in the supporting system are desired. For example, mixed-use high-rise buildings that provide for two or more types of occupancies require a different arrangement of the supporting structure for each functionality. In densely populated cities, large column-free spaces for lobbies or shopping areas are also required at the lower levels of tall buildings, and the construction almost invariably involves working within severe site constraints (Zunz and Wise 1988).

2. Design and construction considerations

The design for deflection control is frequently the primary consideration when defining the dimensions of a transfer element, as transfer structures are usually prone to large deflections due to long spans and the high magnitude of the forces involved (CEN 2004). As a guiding principle, a transfer structure should ideally lead to deflections similar to those that would occur if the vertical elements were continuous (or, in other words, if there was no transfer structure). This may be achieved through the use of post-tensioning in concrete structures or pre-cambering in steel structures. Additionally, the great variability in vertical stiffness and mobilized mass makes some transfer structures extremely susceptible to vertical excitations. Therefore, the effects of human-induced vibrations must be controlled, and the vertical component of the seismic action gains additional importance, which is not common in building structures (Willford and Young 2006).

The conceptual design of buildings with global transfer structures in seismic regions must be carefully planned and ensure that the transfer structure does not jeopardize or impair the seismic design of the building. Regarding the seismic design of the transfer structure, code prescriptive design procedures may not be appropriate for complex structural systems. Non-linear time-history analyses as part of a Performance Based Design approach are recommended, which can demonstrate adequate behavior of the transfer structure by showing it remains elastic even under large seismic events (CTBUH 2012). As an alternative, the capacity design philosophy may be applied to the design of the transfer structure to ensure elastic behavior under all seismic loadings.

The sequence of construction greatly affects the total deflections and the final forces distributions and should be properly accounted for in the design stages through a construction-staged analysis. Considerations of robustness and disproportionate collapse may also be key for the design of transfer structures as these are often regarded as critical elements for the overall stability of the building (Gilbert et al. 2015). Finally, connections design might be challenging in transfer structures due to very high forces and complex geometry involved, and the general seismic design principle of capacity based design should be followed.

Transfer structures usually create logistical construction challenges related to sequencing and erection of heavy elements, as well as formwork complications. Therefore, temporary support systems play an important role in the construction process and may have a considerable influence on the overall benefit of a solution (Taranath 2010). Transfer structures should always utilize the highest strength of materials that are locally available in order for these elements to be the most effective and constructible - this includes using high-strength concrete (HSC), high-strength steel rebar (HSR), and high-strength structural steelwork (HSS) (VSL 1992).

Regarding the construction of concrete transfer structures, the casting of large concrete elements, usually termed mass concrete, might be challenging as their thermal behavior is considerably different from ordinary concrete works, and high-temperature differentials between the core and the surface may originate early-age cracking. On the other hand, steel transfer structures usually require a high degree of dimensional control during fabrication and erection and the definition of a complex system of tolerances.

3. Case study: Saint Gabriel Tower

3.1. General context and description of the building

Saint Gabriel Tower (Figure 1 (a)) was constructed in the year 2000 in Lisbon, Portugal, and is a 110m-tall residential building. The building comprises a post-tensioned concrete grid system that transfers the loads from 25

residential storeys to the two RC cores, which are the only vertical elements that go all the way to the foundations (Figure 1 (b)). For architectural reasons, it was requested that the transfer grid be in the shape of a ship's hull, with ellipsoidal geometry. Figure 1 (c) shows the transfer floor plan. The areas between the cores and in the cantilevers will be henceforth referred to as Zone 1 and Zone 2, respectively (Almeida et al. 2004). In this paper, an alternative transfer structure in the form of a composite steel and concrete truss system is designed for the Saint Gabriel Tower.

3.2. Transfer systems: post-tensioned concrete original solution and alternative composite scheme

Architectural requirements dictate that the transfer system is completely within the first-floor structure. It is shaped like a ship's hull, having an ellipsoidal geometry with a maximum depth of 5.6 m at the center and a minimum depth of 1.5 m at the edges. Due to the ellipsoidal geometry, the depth of the structure at the supports (approximately 4 m) is smaller than at the span (center), which is not the most efficient layout since the highest bending moments and shear forces are at the supports. There are three main longitudinal elements, which span 25 m between the cores and cantilever 14 m beyond them. In order to ensure continuity and a fixed support for these elements at the building's cores, their position is determined by the location of the elevators, stairs and vertical openings (Almeida et al. 2004). Therefore, the central element receives load directly from a total of nine columns whereas the lateral elements could not be placed right beneath the columns at the alignments B and D and are positioned at alignments B' and D' instead (Figure 1 (c)). To transfer the load arising from the columns for the lateral longitudinal elements, transverse elements at alignments 3, 5, 8, 9, 10, 13 and 15 are required. Additionally, also due to the vertical services in the cores, the width of the three main longitudinal elements is limited to 1.2 m.

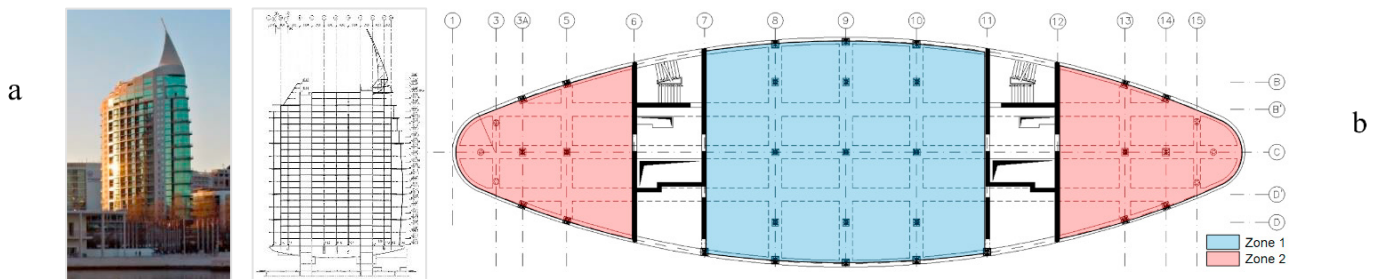


Fig. 1. Transfer floor plan of Saint Gabriel Tower

The concrete solution receives the load from all the interrupted columns of the upper structure, and is supported in the two cores of the building, with a central span of about 25 m and cantilevering approximately 15 m long beyond the cores. The transfer grid is composed of a set of orthogonal post-tensioned beams 1.2 m wide with variable depth: the main beams, arranged longitudinally, are 1.5 m deep at the edges and approximately 5.6 m deep at the center, whereas the transversal beams have a depth of 1.0 m at the edges that increases linearly towards the center. The materials used were: C45/55 concrete; A500 steel grade for the ordinary reinforcement; and prestressing steel A1670/1780 for the tendons (Almeida et al. 2004).

The alternative composite scheme comprises steel truss elements that work compositely with concrete slabs at the top and the bottom of the hull. The materials employed were: S355 for the steel framework and C45/55 for the concrete elements. The main elements are the longitudinal trusses that span between the concrete cores and cantilever beyond them. Their configuration is represented in Figure 2, where the arrows represent the discontinued columns that transmit the forces from the upper floors to the trusses. A Pratt truss layout was adopted in order to have the diagonal members (which are the longest unbraced elements) in tension for the gravity load effects. This type of truss is usually the most efficient design under static, vertical loading.

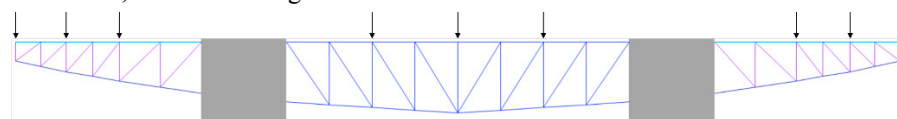


Fig. 2. Longitudinal trusses configuration

3.3. Modeling and analysis

Modelling of the variant composite structure was conducted using the software ETABS 2016. The majority of the analysis was carried out with a local model of the transfer structure (Figure 3(a)), whereas a global model (Figure 3(b)) was produced to evaluate the dynamic properties of the building, to undertake an analysis of the vibrations, and to assess the effects of the seismic action on the transfer structure. The three first natural modes of vibration of the building are: (i) translation in the longitudinal direction, with a frequency of 0.44 Hz; (ii) translation in the transverse direction, with a frequency of 0.59 Hz; and (iii) torsion, with a frequency of 0.98 Hz. The long-term load distribution was obtained through a staged construction analysis using a global model with accurate properties and dimensions of the elements to account for eventual load redistributions.

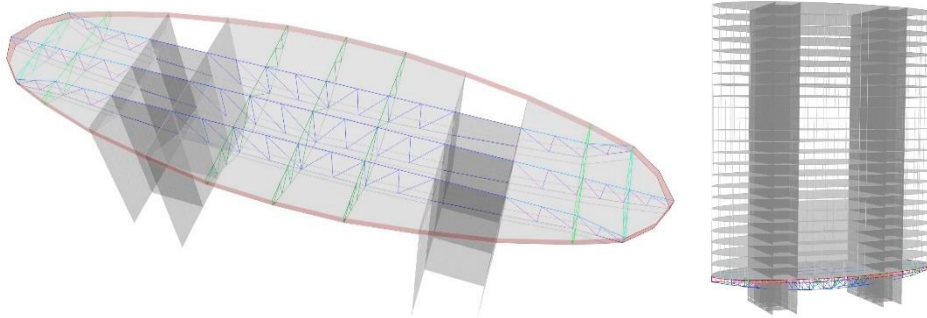


Fig. 3. (a) global model of the transfer structure; (b) global model of the building

3.4. Loadings

The gravity loads from the 25 residential storeys that need transferring are the dead loads (structural self-weight), super-dead loads, and live loads, which gives a combined total of 8 kN/m². In addition, the self-weight of the transfer structure, as well as the weight of mechanical equipment that is stored at the transfer level, also need to be taken into account. The aforementioned loads are the result of a comprehensive construction staged analysis (Almeida et al. 2004).

The performance criteria adopted for the seismic design of the transfer structure was that the structure must remain elastic under the EN 1998-1 design earthquake (Taranath 2011). Regarding lateral loading, the conceptual design of the building is appropriate as the transferred columns are completely secondary. Additionally, the transfer structure has the beneficial effect of creating a frame behavior together with the lower portion of the core walls. A response spectrum analysis was used, with a conservative behavior factor of 2.0 and it was observed that the magnitude of the seismic forces generated on the transfer structure was about half the corresponding values for the ULS fundamental load combination.

The effects of the vertical component of the seismic action were taken into account by following the provisions from the Eurocode. The natural frequencies of the main vertical modes of vibration are around 2 Hz and 3 Hz, thus avoiding the usual exciting frequencies (greater than 7 Hz) of the vertical component of the earthquake. Hence, the structure is not very sensitive to vertical ground motion. Nevertheless, a simulation of the building under the vertical component of the design earthquake (EN 1998-1) was performed to ensure that the structure complies with the code requirements. As expected, this action is not important as the magnitude of the forces induced in the transfer structure is even lower than for the horizontal component. A unitary behavior factor was used in the response spectrum analysis for the vertical earthquake.

It was concluded that the design of the transfer structure is determined by gravity loads rather than by seismic loading. Moreover, the lateral-load resisting system and the transfer system itself are extremely stiff, whereby the imposed seismic deformations do not impair the load-carrying capacity of the transfer structure.

3.5. Disproportionate collapse analysis

The ability to provide alternative load paths for the vertical load is the most adequate way to design against disproportionate collapse, and it is always preferable over the key element design approach (Ribeiro 2018). As the transfer system of the SGT has an intrinsic level of redundancy (existence of three longitudinal trusses and several transverse trusses), it is possible to design the truss members for extreme situations in which other members fail.

Thus, four scenarios were considered involving the sudden failure of different critical members, as illustrated in Figure 4. These are: (i) failure of the upper chord, lower chord, and diagonal of the central longitudinal truss in the section where the truss connects directly to the concrete walls in Zone 2; (ii) failure of the upper chord, lower chord, and diagonal of the lateral longitudinal truss in the section where the truss connects directly to the concrete walls in Zone 2; (iii) failure of the upper chord, lower chord, and diagonal of the central longitudinal truss in the section where the truss connects directly to the concrete walls in Zone 1; (iv) failure of the upper chord, lower chord, and diagonal of the lateral longitudinal truss in the section where the truss connects directly to the concrete walls in Zone 1. The four scenarios considered represent the most adverse situations concerning the loss of structural elements.

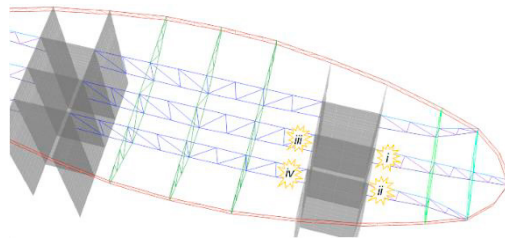


Fig. 4. Schematic representation of the four scenarios considered in the disproportionate collapse analysis

A linear elastic dynamic analysis was carried out, and a local finite element model of the transfer structure was used to reduce the computational demands. The load combination used in the analysis ($1.2[DL + SDL] + 0.5LL$) was derived from the recommendations present in the ASCE 7-10. A damping coefficient of 2% was used. The analysis requires a model of the structure without the elements that are assumed to fail. A first linear static load case is settled comprising the applied loads and an additional set of loads that consist in the internal reaction forces of the lost elements on the remaining structure. This load case consists in a state of the structure “at rest”, representing the whole structure static behavior. The removal of the elements is simulated by creating a linear time-history load case that starts at the end of the previous load case, in which a new set of forces with the same magnitude and opposite to the aforementioned internal reaction forces is applied using a ramp function. Ramp functions go linearly from the value 0 to 1 in a small fraction of time, denominated t_{rise} , which is the time interval necessary to disable the members assumed to fail.

The alternate load path method is considered to be “threat-independent” as explicit consideration of the initial loading event is not needed. However, the value of t_{rise} adopted in the analysis might have implications on the dynamic effects of the sudden removal of an element. Therefore, in order to assess the influence of this factor and to substantiate its choice, a sensitivity analysis of this parameter was undertaken. The results are presented in Figure 5, which shows the dynamic amplification factor associated with the removal of the elements for each of the scenarios considered and for different values of t_{rise} .

Results show that, for values of t_{rise} greater than 0.5 seconds, the dynamic effects are significantly reduced, as the amplification coefficient does not exceed 1.05. Furthermore, there do not seem to exist significant resonance effects in the cases where the t_{rise} is a multiple of the natural period of the vertical modes of vibration (around 0.5 seconds). The time interval necessary to disable the members that fail (t_{rise}) was assumed to be 0.1 seconds, as their failure is very sudden, but it is not completely instantaneous. Thus, the dynamic amplification factor ranges from 1.05 to 1.25, which means that the design forces of elements adjacent to the removed members should be 5% to 25% higher than the ones obtained with a linear elastic static analysis of the model with the damaged structure.

To illustrate the dynamic effect of the sudden loss of an element, Figure 6 shows the time-history of the axial force in selected member and the displacement of the elements adjacent to the failure, for different values of t_{rise} . A sampling

frequency of 100 Hz was used in the numerical simulations. The frequency of the responses is around 2.4 Hz, which is approximately the vertical natural frequency of the damaged structure. Additionally, results show that the maximum deflection for scenario (iv) is equal to 58.2 mm (maximum deflections for the other scenarios are similar), which is very close to the established SLS limits. Thus, the imposed deformations on the upper floors would not have any damaging consequences.

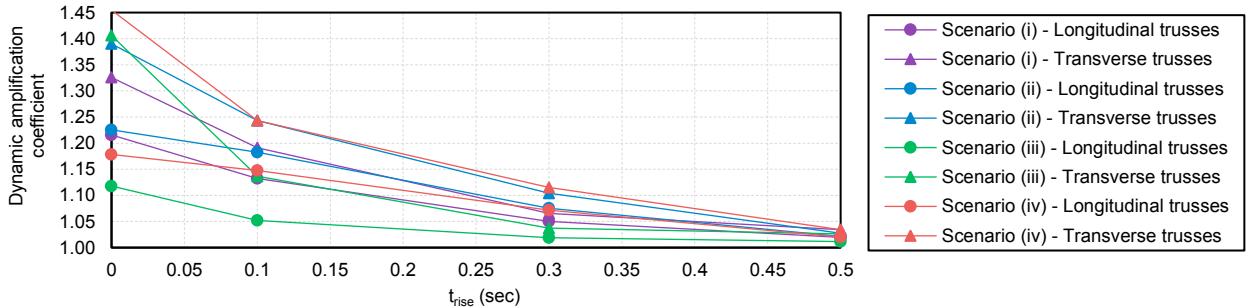


Fig. 5. Dynamic amplification coefficients for the sudden loss of a structural member and sensitivity analysis of the t_{rise}

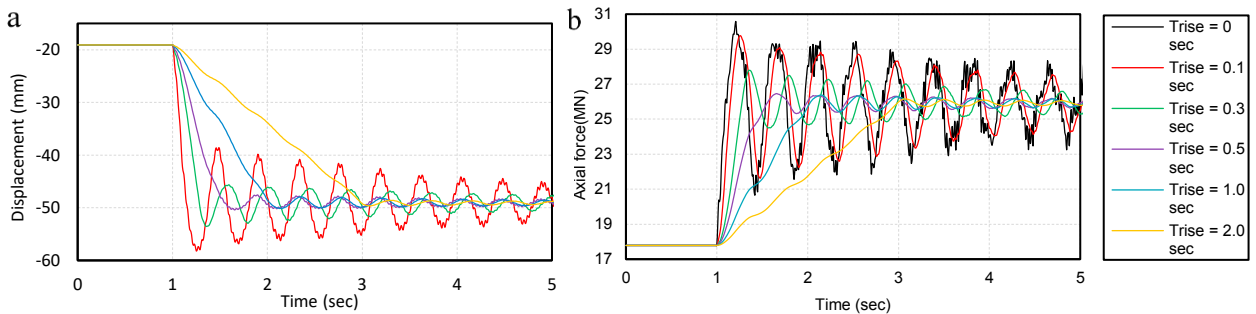


Fig. 6. (a) time-history of the vertical displacement of the elements adjacent to the failure when scenario (iv) occurs; (b) time-history of the axial force in the diagonal of the central longitudinal truss adjacent to the core, in Zone 1, when scenario (iv) occurs

3.6. Members and connections' design

The overall design of the transfer system was determined by the serviceability behavior with respect to deflections. The design of the members of the transfer structure for the ULS was based on the provisions of EN 1993-1-1 and EN 1994-1-1. Regarding the longitudinal trusses, the chords have the same cross-section throughout their entire length, whereas the dimensions of the web members (diagonals and verticals) were optimized according to their internal forces. Also, for the simplicity of the connections, it was decided that the web member widths must be lower than the chord widths. The chords are embedded in RC slabs, which provides fire protection and composite behavior, and the bond between the steel members and the concrete is achieved by headed studs. The connections of the steel trusses to the RC cores are critical elements of the system. They must be sufficiently strong and rigid to ensure a fixed support condition at the RC core walls. Several options were considered, but the one that was found most advantageous was to connect both sides of the core where the trusses meet through a set of post-tensioned Diwidag bars.

3.7. Deflection control

The design criteria adopted were the following: (i) long-term deflection must not exceed $L/250$ for spans and $L/125$ for the cantilevers, and (ii) the deformation imposed on non-structural elements must not exceed $L/500$ for spans and $L/250$ for the cantilevers, where L represents the length of the span or cantilever. The characteristic load combination, rather than the quasi-permanent one, was used to calculate the deflections, in order to ensure that the building does not have excessive deformations even under rare loading scenarios.

3.8. Vibration analysis

A dynamic analysis of the structure resulted in the estimated natural frequencies for the two first vertical modes of vibration, represented in Figure 7, of 1.95 Hz and 2.69 Hz. These modes of vibration are susceptible to excitations due to normal actions such as walking, running, dancing or jumping. Thus, since the whole building depends on the transfer floor, it is important to evaluate the serviceability behavior of the structure under human-induced vibrations, in order to assure the comfort of the users.

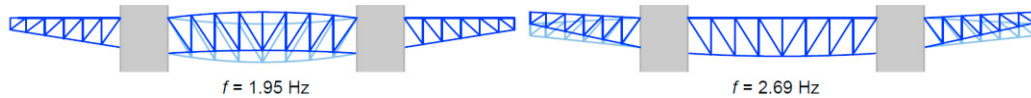


Fig. 7. Natural vertical modes of vibration of the transfer structure and respective frequencies

A vibration analysis was carried out based on the International Standards ISO 2631 and ISO 10137. The forces were placed at the anti-nodes for the relevant modes and four dynamic actions were considered, namely: (i) 125 people walking, simultaneously, on all the storeys (approximately 5 people per floor) in the center of Zone 1; (ii) 80 people dancing in the top 2 floors, in Zone 2 (simulating a party); (iii) 45 people running in the center of Zone 1, at 9 different storeys; (iv) Synchronized jumping of 20 people on the transfer floor in the center of Zone 1. The actions are described by a Fourier series of the form (Equation (1)):

$$F_v(t) = G \left(1 + \sum_{n=1}^k \alpha_n \sin(2\pi nft + \varphi_n) \right) \quad (1)$$

where $F_v(t)$ represents the forcing function, t is the time, G is the static load of the participating person, f is the excitation frequency in Hz, α_n is the Fourier coefficient of the n th harmonic, φ_n is the phase angle of the n th harmonic, n is the integer designating harmonic components, and k is the total number of contributing harmonics. The average weight of a person, G , was assumed to be 0.8 kN (approximately corresponding to 80 kg). A modal damping ratio of 4% was used, which accounts for structural damping, damping due to furniture and damping due to finishings.

Figure 8 (a) represents the force functions for one person corresponding to each of the actions considered. Each action was simulated with a duration of 10 seconds. The movements of large groups of people on different floors were considered uncoordinated and a coordination factor, C , given by equation $C(N)=1/N^{1/2}$, was applied for the walking, dancing, and running actions, where N is the number of people. This factor multiplies for the forcing function to obtain the effective dynamic action.

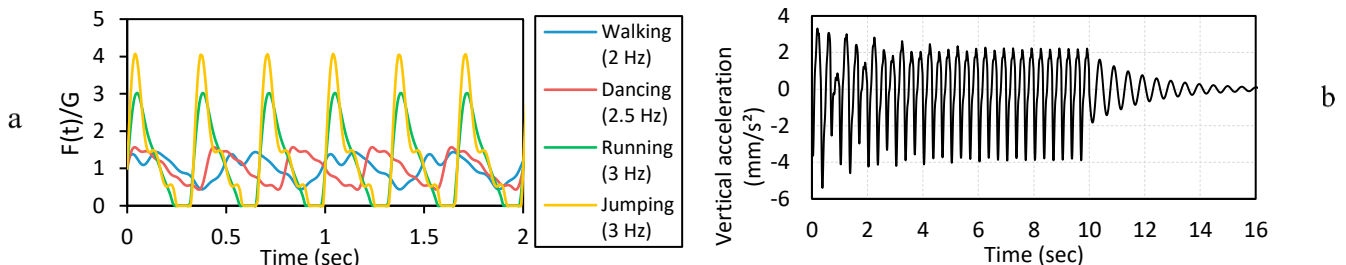


Fig. 8. (a) force functions for one person corresponding to walking, dancing, running and jumping; (b) Vertical acceleration time series for action (iv), measured at the center of Zone 1 on the transfer floor

The movements of large groups of people on different floors were considered uncoordinated and a coordination factor, C , given by Equation $C(N)=1/N^{1/2}$, was applied for the walking, dancing, and running actions, where N is the number of people. This factor multiplies for the forcing function to obtain the effective dynamic action. Figure 8 (b) presents the structure's response in terms of the vertical acceleration for each dynamic action. The measurements are related to the nodes with the highest accelerations in each case. It was observed that the top floors experience much higher vibrations than the lower ones, due to the effect of vibration propagation by the vertical elements (columns).

The evaluation of vibration according to ISO 2631-2 involves measurements of the weighted root-mean-square (r.m.s.) acceleration, in accordance with Equation (2):

$$a_{w,rms} = \sqrt{\frac{1}{T} \int_0^T a_w^2(t) dt} \quad (2)$$

where a_w is the weighted acceleration as a function of time, in meters per seconds squared (m/s^2), and T is the duration of the measurement, in seconds. To take into account the frequency dependency of human comfort and perception of vibration, it is necessary to apply weighting factors to the accelerations. The frequency weighting W_m was used, as recommended in ISO 2631-2 for buildings and vibration measurements within the frequency range 1 Hz to 80 Hz. The results for the weighted r.m.s. accelerations are shown in Table 1.

Table 1. Weighted r.m.s. accelerations for each action

Action	(i) Walking	(ii) Dancing	(iii) Running	(iv) Jumping
Weighted r.m.s., a_w (m/s^2)	0.007435	0.006534	0.002963	0.001723

The acceptance criteria were derived from ISO 2631-1, which establishes acceleration (r.m.s.) limits from which users may experience discomfort. The limits prescribed are dependent on the frequency of the exciting action and on the exposure times. Actions i), iii) and iv) are not likely to last longer than a few minutes, therefore the resulting limit for reduced comfort range from 0.79 m/s^2 to 0.98 m/s^2 , for exciting frequencies between 2 Hz and 3 Hz and a 15 minute-long excitation. Action ii), on the contrary, may have longer exposure times. However, even for an eight-hour duration, which is very unlikely to take place, the acceleration limit is 0.13 m/s^2 . The weighted r.m.s. accelerations shown in Table 1 are much lower than the former limit values, meaning that it is unlikely that residents of the building will experience discomfort or even perceive the vibrations under any of the actions considered.

4. Concluding remarks

An alternative composite transfer structure was designed for the Saint Gabriel Tower and the most relevant aspects of its development are highlighted. Regarding the serviceability behavior, the design essentially addressed the control of the vibrations and deflections. A comprehensive vibration analysis was undertaken as the transfer structure is susceptible to human excitations – the natural frequencies of the main vertical modes of vibration range from 2 Hz to 3 Hz. Results showed that none of the actions considered induces excessive vibrations on the building. In fact, the accelerations obtained are well below code limits. Notwithstanding, the design of the transfer structure was ultimately determined by the control of deflections. The ULS design of the structural elements accounted for the gravity loads, the seismic action and an accidental action through the disproportionate collapse analysis. It was concluded that the latter was determinant in the design of the truss members. Neither the horizontal nor the vertical components of the seismic action were determinant, and the transfer structure remains elastic in the event of a moderate earthquake.

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