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StructuralAwards2019

Shortlisted: Award for Tall or Slender Structures

Manhattan Loft Gardens, London: balancing design and constructability

↙FIGURE 1: Completed tower

Project credits

Owner	Manhattan Loft Corporation
Structural engineer	Skidmore Owings & Merrill (Europe), LLP
Architect	Skidmore Owings & Merrill (Europe), LLP
General contractor	Bouygues UK
MEP and fire protection engineering	Hoare Lea
Landscape architect	Martha Schwartz, Inc.
Lighting design	Paul Nulty Lighting Design
Sustainable design	Greengage

SYNOPSIS

Manhattan Loft Gardens is a new residential tower in London that opened in the spring of 2019. The tower presents an arresting silhouette on the skyline as a result of three, triple-height, sky- [illegible]

Introduction

Manhattan Loft Gardens (Figure 1) is a 143m tall, 42-storey, residential tower located in Stratford (east London), adjacent to existing high-speed rail tracks, just north of Westfield Stratford City retail centre (Figure 2). The tower was designed by Skidmore Owings & Merrill (Europe) LLP (SOM) architects and structural engineers, and built by Bouygues UK for Manhattan Loft Corporation. VSL International was the post-tensioning specialist contractor.

The tower structure is formed integrally with a seven-storey podium structure that houses a 248-key hotel and amenities. The tower accommodates a mix of one-and-a-half-storey loft and single-storey apartment layouts, but is perhaps most distinguished by the three sky gardens that take the form of completely open notches cutting through the building at levels 7–10, again at levels 25–28, and from level 36 to the roof.

The architectural form of the tower was arrived at by removing three triangular volumes from an otherwise straight rectangular extrusion (Figure 3). Each notch eliminates half of the perimeter columns on two sides of the tower, giving it a seemingly improbable and structurally precarious silhouette.

The dramatic tower form and notches are made possible by a unique system of hybrid steel perimeter trusses and post-tensioned concrete outrigger transfer cantilever structures, incorporated into the floors immediately above the notches (Figures 4 and 5).

The architecture is organised around the idea of a vertical community and is intended to promote social interactions among the residents. The double-cantilevered tower design celebrates the three sky gardens in which residents can meet. These gardens provide a range of shared open spaces that provide spectacular views of the London skyline.

Overall stability of the tower is provided primarily by the central reinforced core wall, with additional stiffness resulting from the interaction between the post-tensioned outriggers, belt trusses and perimeter columns.

Above each sky garden, a one-storey-deep, steel, perimeter belt truss carries the gravity load of the columns from up to 15 storeys above (up to the next notch level or roof). The belt trusses are, in turn, carried by four pairs of post-tensioned concrete outriggers that emanate from the central core (Figures 5–7). The asymmetry of the notch arrangement and the resulting unbalanced load had the potential to cause the building to drift laterally during construction, and the multiple load paths caused by the outriggers result in a redundant, albeit highly indeterminate structure.

To minimise adverse lateral gravity drifts and to ensure a predictable load path, a specific construction sequence was developed as part of the structural design. This ensured gravity loads were introduced in a balanced manner to each set of outriggers and provided a high level of determinacy by ensuring that most of the self-weight gravity load was carried directly through the outriggers back to the core.

These structural design choices, which related to and – to some degree – dictated the construction sequence, eliminated potentially complex corrections and adjustments during construction and allowed the tower to be constructed in a simple and largely conventional manner.

This article primarily presents a description of the tower's structural design, with a focus on the design of the unusual post-tensioned outrigger and construction sequence.

Site and history

The building is located immediately north of Westfield Stratford City and directly to the east of Stratford International rail and underground station, in the heart of one of London's fastest growing districts. Since the 2012 Olympics, this area has seen marked growth in residential, retail and office developments, as well as major transport connections to central London.

Before the development of the high-speed rail network, the site was largely industrial, with a profusion of rail sidings and engine sheds occupying the space where the Manhattan Loft Gardens tower and surrounding buildings are now situated (Figure 8).

With the land subdivided as part of a large-scale outline planning application that consisted of several dozen plots earmarked for a range of developments, with density and height limits defined, the Olympic Village construction project commenced in 2007. Following the Olympics, the apartment blocks for athletes were sold as private residential units and the district became known as East Village. The Manhattan Loft Gardens is the tallest tower in the district, sitting just under the +150m AOD (above ordnance datum) level dictated by the proximity of London City Airport's control zone.

Manhattan Loft Corporation, with designs prepared by SOM, won planning approval for the tower in October 2010. RIBA Stage D design work began and completed in 2011. Further design work stalled for a short period once it became clear that security access restrictions introduced for the Olympics would prohibit any significant site activities or construction until after the games.

The project was tendered on Stage E information in October 2013, after which the preferred contractor's proposals were incorporated into the final design. Bouygues UK won the project and started construction in February 2015, with the continuous flight auger (CFA) bearing pile construction and the installation of the temporary perimeter sheet piled retaining wall and propping works.

The tower and hotel superstructures topped out in December 2016, and the residential component reached practical completion in April 2019, with the hotel opening in May 2019.

Foundations and substructure

The foundation system consists of a 350mm thick, fully reinforced basement slab formed over individual pile caps and ground beams within the hotel podium area to the west of the main tower. The basement walls are 300mm thick reinforced concrete, formed inside temporary sheet piling, to retain approx. 3.5m of earth below ground level. A Grade 3 basement, in accordance with BS 8102[1], was achieved with a combination of gas and waterproof membranes, waterproof concrete, and by limiting concrete crack widths.

The 42-storey tower is founded on a piled raft, which measures approx. 30m by 30m in plan, and is 2500mm thick to directly support the central core and buttress walls. The raft distributes the loads from the tower walls and columns to the piles. The top of the raft is set about 1.5m below finished ground level, to limit excavation depths adjacent to a multistorey car park, which has its piled foundation and above-grade framing virtually abutting the eastern end of the site.

Given the nature of the structure, where many of the columns have been removed, the primary lateral stability element is the central core, which at 11m by 10m in plan is very small for a 143m tall building. Buttress walls were added through the seven floors of the hotel, to stiffen the bottom section of the core. This simple system reduced lateral drift during construction and reduces wind drift in service. It also distributes the tower's vertical loads at the extremities of the piled raft, so that 'dishing' (the tendency of the centre of the raft, under the core, to deflect more than the perimeter) of the raft is limited.

There is no net tension at the base of the buttress or primary core walls and the connection between the walls and the raft consists only of standard compression reinforcement dowel bars.

There are approx. 34 piles of 750mm or 900mm diameter below the hotel, and 88 piles of 1200mm diameter below the piled raft. The capacities for each pile diameter were provided by the specialist geotechnical engineer and all piles were constructed using the CFA method, in accordance with the detailed specification prepared by High-Point Rendell (HPR) and to the ICE piling specification[2]. At this site, the London clay layer is quite thin and is underlain with Thanet sands, so all piles were taken roughly 1–2m into this stratum. The piles were designed to utilise both skin-friction (in clay) and end-bearing (in Thanet sands).

Another major consideration of the foundation design was the proximity of the Channel Tunnel Rail Link Stratford Box, which is 10m away and parallel to the southern edge of the tower. The 1500mm thick retaining walls cantilever vertically for approx. 12m above the rail base, and Network Rail/High Speed 1, as the owner of both the multistorey car park and the Stratford Box structures, was very concerned about the impact the new tower would have on the condition and performance of these assets.

Initial geotechnical studies were carried out by Coffey Geotechnics, with additional design validation and temporary works design completed by HPR, which completed a range of additional studies and reporting to verify the impact on the adjacent structures.

Initial two-dimensional (2D) studies sought to demonstrate the comparative moments and shear forces on the cantilever walls both in their existing condition and as a result of the tower construction (Figure 9). As such, a staged analysis was completed where each step in the construction phase was sequentially added, with results reported at each step.

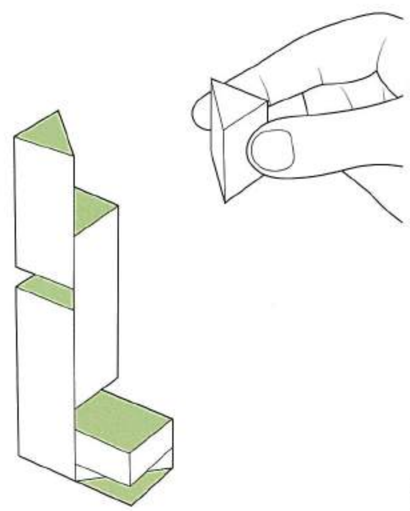
The presence of Thanet sands at a shallow depth, and the offset distance of the tower from the wall, indicated a maximum wall movement at the tip of approx. 15mm (height / 800), which was deemed acceptable. In practice, surveys showed movements of not greater than 10mm.

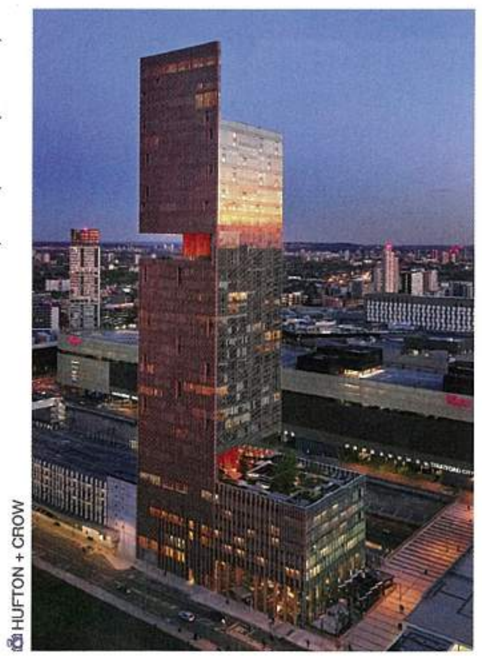
Wind tunnel testing

A wind tunnel test was carried out by BMT to obtain wind pressures on the building. Two scenarios where tested: one with a proposed new tower, which would have been slightly taller than Manhattan Loft Gardens, present quite close by to the north; and one without the proposed tower. In fact, during construction of Manhattan Loft Gardens, the proposals for the northern site changed and two slightly shorter towers were built, topping out at a similar time. No further studies have been done to reassess the effects of these towers, as the difference in the results with and without the tower was negligible.

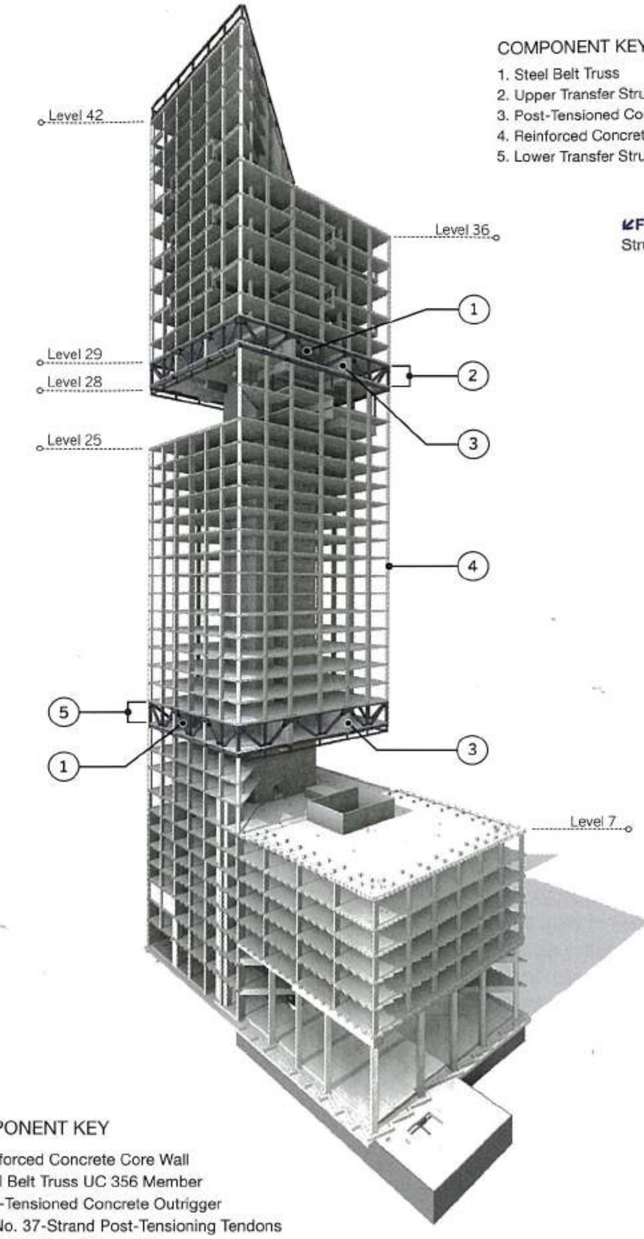
The wind force values for all relevant directions were extracted and the results were imported into a finite-element analysis program. Several load combinations were included for both scenarios and all wind forces were applied at each level of the structure. A non-linear p-delta analysis was conducted to assess the lateral stability of the building. Due to the structural thicknesses required at the outrigger zones, core thicknesses needed to be larger than normal, so the lateral stability results showed that the tower is relatively stiff compared to many towers of this height, in part due to the presence of the low-level buttress walls. The primary core walls are 900mm thick up to the first outrigger level, 750mm thick up to the second outrigger level, and 400mm thick up to the roof level. The analysis showed that wind actions do not control the design of the lateral stability system.

←FIGURE 2: Site constraints: plan view of Olympic park and high-speed rail line (looking northeast)



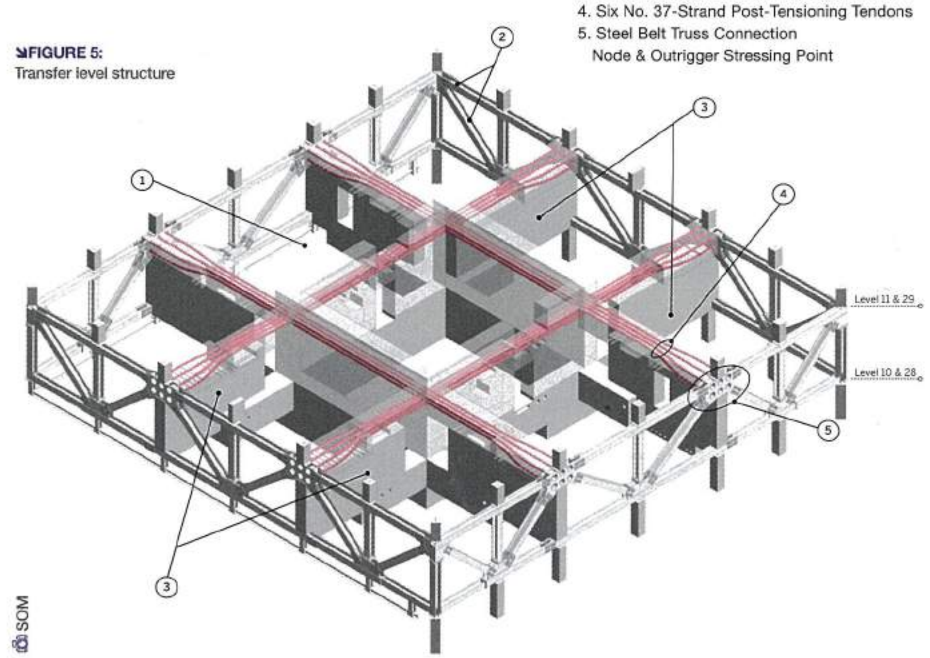


↑↗FIGURE 3: Architectural concept



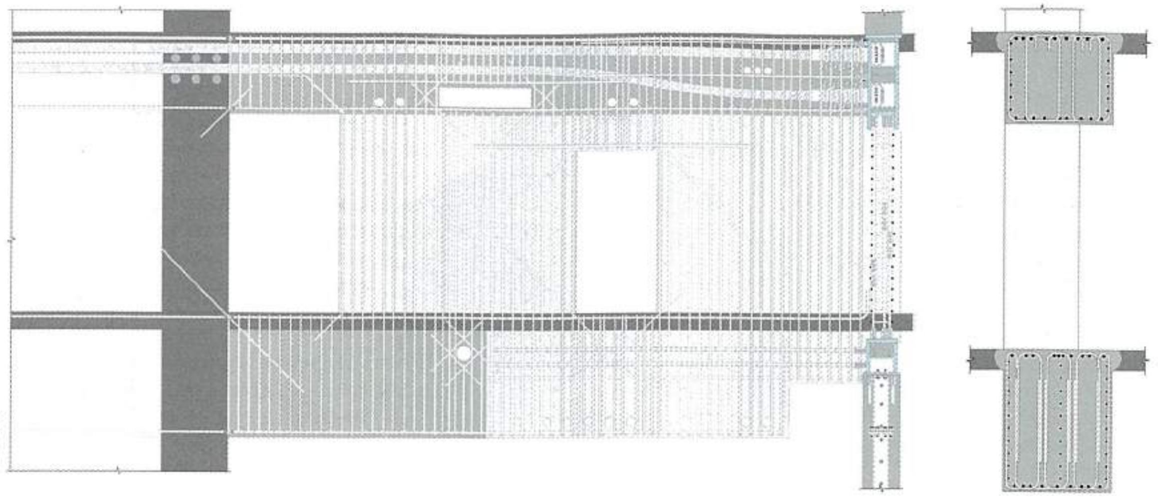
COMPONENT KEY
1. Steel Belt Truss
2. Upper Transfer Structure
3. Post-Tensioned Concrete Outrigger
4. Reinforced Concrete Column, Typical
5. Lower Transfer Structure

↙FIGURE 4: Structural system



COMPONENT KEY
1. Reinforced Concrete Core Wall
2. Steel Belt Truss UC 356 Member
3. Post-Tensioned Concrete Outrigger
4. Six No. 37-Strand Post-Tensioning Tendons
5. Steel Belt Truss Connection Node & Outrigger Stressing Point

↘FIGURE 5: Transfer level structure



↖FIGURE 6: Post-tensioned outrigger elevation and section

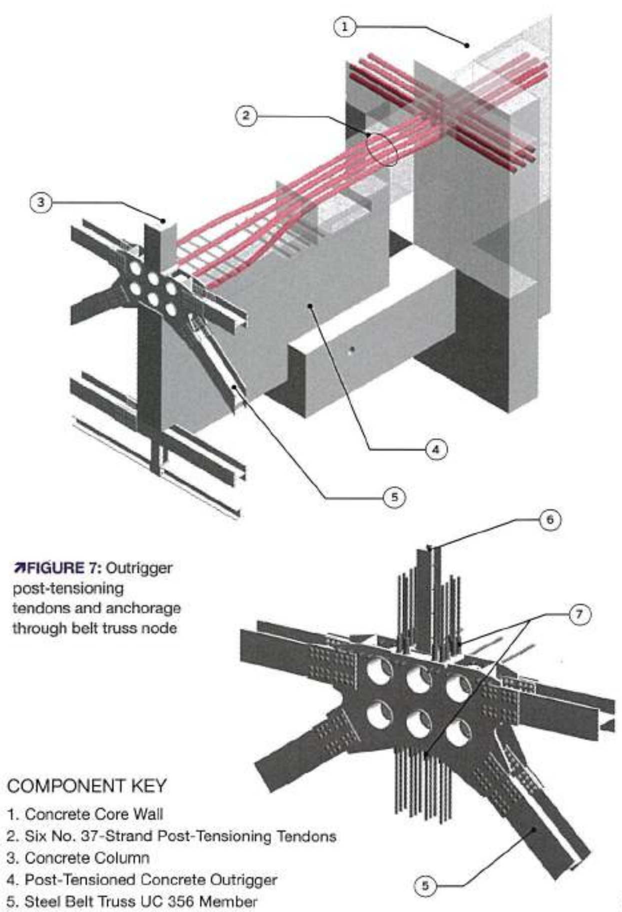
Post-tensioned outrigger transfer system

The tower's sky gardens are three storeys high, extending from level 7–10 on the southwest half of the tower and from level 25–28 on the northeast half of the tower. At each of these locations, 11 of the tower's 24 perimeter columns are interrupted and column loads are completely transferred back to the central core wall.

The transfer system consists of eight post-tensioned outrigger walls emanating from, and concurrent with, the central core (Fig. 5). These, in turn, support a perimeter steel belt truss. The post-tensioned outrigger walls are nominally one storey deep, but the lower portion of the walls extends an additional 1.7m below the level 10 and 28 slab levels, bringing the total outrigger depth to 5.5m.

Each outrigger cantilevers approx. 9m from the central core wall to the perimeter column line and carries the vertical load from up to 15 storeys above and the weight of three perimeter columns (per outrigger). The outrigger walls are typically 750mm thick, but increase in thickness to 1300mm near the connection to the main core walls, and to 1700mm at the connection to the steel belt truss node.

The typical concrete strength was C40/50 for the core walls; at both outrigger levels, the concrete strength was C60/75 for the walls and outriggers.



↗FIGURE 7: Outrigger post-tensioning tendons and anchorage through belt truss node

COMPONENT KEY

1. Concrete Core Wall
2. Six No. 37-Strand Post-Tensioning Tendons
3. Concrete Column
4. Post-Tensioned Concrete Outrigger
5. Steel Belt Truss UC 356 Member
6. Embedded Steel Column Stub
7. Mechanical Reinforcement Couplers

It is notable that the outrigger system is limited to a depth of one floor, and even more remarkable that all four pairs of outriggers at each transfer level are situated between and around residential apartment units.

In most tall buildings that utilise outriggers (typically, for lateral stability), an entire floor or a double-height floor is given up as saleable space, being used instead for mechanical equipment or storage because the bulk and complexity of the outriggers obstruct normal usage.

In the Manhattan Loft Gardens tower, however, all the floor space at the transfer floors, except for one small bay at level 28, was utilised for residential units. This allowed the developer to maximise the efficiency of the tower, but presented a major challenge to coordinating services and circulation at these levels.

Most notably, the residential use of the transfer required large, 2.7m tall, rectangular door openings in the outriggers to accommodate corridors, at the structurally disadvantageous location of maximum shear transfer between the outrigger and core. The resulting discontinuity in the shear transfer from the outriggers to the core required substantial thickening of the walls above and below the opening and careful consideration of the pour sequencing and shear transfer mechanism (Fig. 6).

Each of the eight outriggers at a transfer level carries an ultimate concentrated load at its tip of up to 13.5MN. The outrigger acts like a cantilever deep beam with the upper portions of the beam attracting tension and the bottom compression.

To counteract and balance the tension force, six, multistrand, high-strength, post-tensioning tendons were introduced. Each of the six tendons consists of 37 individual strands made up of wires with a 1860MP breaking strength, giving a total breaking load of about 10.3MN per tendon.

The tendons were arranged to pass through and directly against the steel belt truss connection node (Figs. 5–7). With this arrangement, the post-tensioning served a dual purpose: allowing the tendons to both stress the concrete outrigger and help secure the connection between the steel truss and outrigger.

The tendons for each outrigger are located in two horizontal layers and are offset vertically so they can interlace as they approach the core wall. One row of tendons for the outriggers spanning north–south passes over and through the tendons for the outriggers spanning east–west.

At the corners of the core, 12 large post-tensioning tendons pass in close proximity to mild reinforcement for the longitudinal and vertical outriggers, as well as significant reinforcement for the core wall. Avoiding congestion and collisions in this zone of the core wall required careful coordination and detailing of every reinforcing element (Figures 6 and 10).

The decision to introduce post-tensioned outrigger elements was advantageous in minimising and controlling deflections. Initial structural schemes for the tower transfer system considered trussed structural steel outriggers, embedded in the main core walls, to lighten the structure and introduce more open space that would facilitate routing of services.

While the steel truss outriggers were feasible from a strength perspective, the resulting vertical deflections were not negligible and would have required corrections to the slab construction elevations, as well as additional monitoring, survey and alignment procedures during construction.

In contrast, the use of post-tensioned concrete outriggers resulted in small deflections due to the much larger stiffness of the concrete elements. More importantly, deflections that occurred during construction were counteracted with each incremental tensioning of the outrigger tendons.

Just after completion of the outrigger levels, two of the six tendons in each outrigger were stressed and the tips of the outriggers moved up by approx. 4mm. After this initial stressing, five floor levels of concrete construction were built above, and the tips of the outriggers deflected downward.

At this stage, two more of the six tendons were stressed, five more floors were built, and the process was repeated until all six tendons were fully stressed and the 15 concrete floors above were complete.

Upon completion of the floors carried by the outriggers and final stressing of all six tendons, the elastic deflection of the tips of the outriggers was approx. 3–4mm upward. Subsequently applied loading and the resulting long-term deflections of the outriggers were minimal, and are expected to bring the tips of the outriggers to the neutral position.

Due to the negligible outrigger deflections, no special corrections or unusual curtain wall connections were required – the construction above the outriggers was, for all intents and purposes, the same as any flat-slab and reinforced concrete-frame residential construction; tolerances were no different than for a conventional building of similar height (these conclusions were arrived at through rigorous sequential analysis discussed in the next section).

Constructability considerations

The greatest challenge in designing the post-tensioned outriggers was not the analytical modelling and engineering design per se, but rather designing for practical considerations and in a manner that ensured the outriggers could be built easily without undue time and complexity.

Given that the outriggers are integral with the primary core walls at the transfer levels, a conventional design and detailing approach would have required pausing and disassembly of the tower core's jump-form construction to allow placement of the outrigger post-tensioning and reinforcement. This approach would have resulted in significant additional cost and time.

Working together with Bouygues UK, SOM engineers instead designed the outriggers to be constructed after completion of the main core walls, and thus allowed the jump form to advance above, and concurrently with, the outrigger construction. This approach was made possible by leaving out large portions of the core walls at each corner during the initial jump-formed pour (Figure 11). These large 'leave-outs' at the corners were the full thickness of the core and thus provided a full-depth shear and bearing interface between the outriggers and core. The leave-outs were formed in the normal jump-forming sequence by introducing 'stay-form' to isolate corners, while allowing the reinforcement, post-tensioning ducts and couplers to pass through the pour joint.

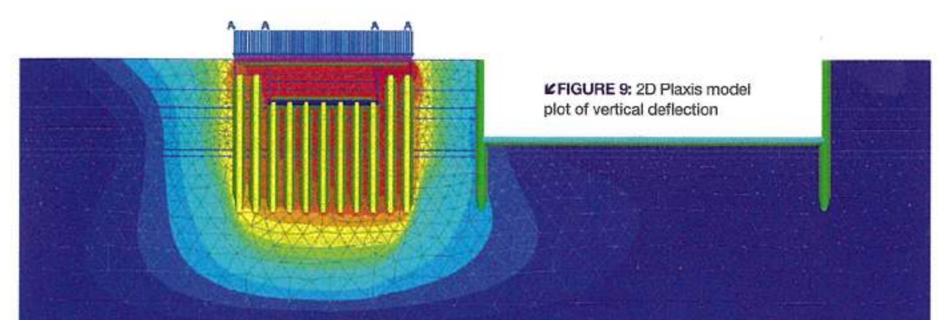
Placement of the outrigger reinforcement, post-tensioning ducts and steel belt trusses required cooperation between concrete and steel trades. Additional time was needed to place the complex outrigger reinforcement. This consisted of dense, conventional, vertical shear reinforcement, typically 25mm bars, extending the full 5.5m depth of the outriggers, and 32mm and 40mm longitudinal bars.

The large shear forces in the outriggers and anti-bursting reinforcement requirements for the post-tensioning strands suggested densely spaced, closed shear ties; however, such an arrangement would have been difficult to install in the field. Alternatively, using a series of overlapping U-bars for shear reinforcement would have resulted in a doubling-up of congestion at the face of the outriggers. The final design utilised a combination of closed ties and U-bars, longitudinal laps and mechanical couplers to reduce congestion.

The final reinforcement detailing was determined to allow simple, sequential placement of each bar. To ensure the constructability of the outriggers, each piece of reinforcement was modelled in 3D by SOM and the model was used to 'walk through' the construction sequence, identify conflicts and verify detailing choices (Figure 12). The 3D modelling of the reinforcement was also beneficial for locating holes and mechanical reinforcement couplers in the structural steel, all of which had to be precisely fabricated by the steel contractor.

At the outer ends of the outriggers, the post-tensioned elements connect to the perimeter steel belt trusses. The location of this interface occurs where the post-tensioning tendons would typically be stressed. Rather than attempting to avoid this collision (by moving the post-tensioning stressing points inwards, away from the truss, for example), SOM engineers embraced the concurrence of these two systems as an opportunity to simplify

↗FIGURE 8: Stratford site, 2005 (looking north)



↙FIGURE 9: 2D Plaxis model plot of vertical deflection

↘FIGURE 10: Outrigger post-tensioning ducts at corner of core

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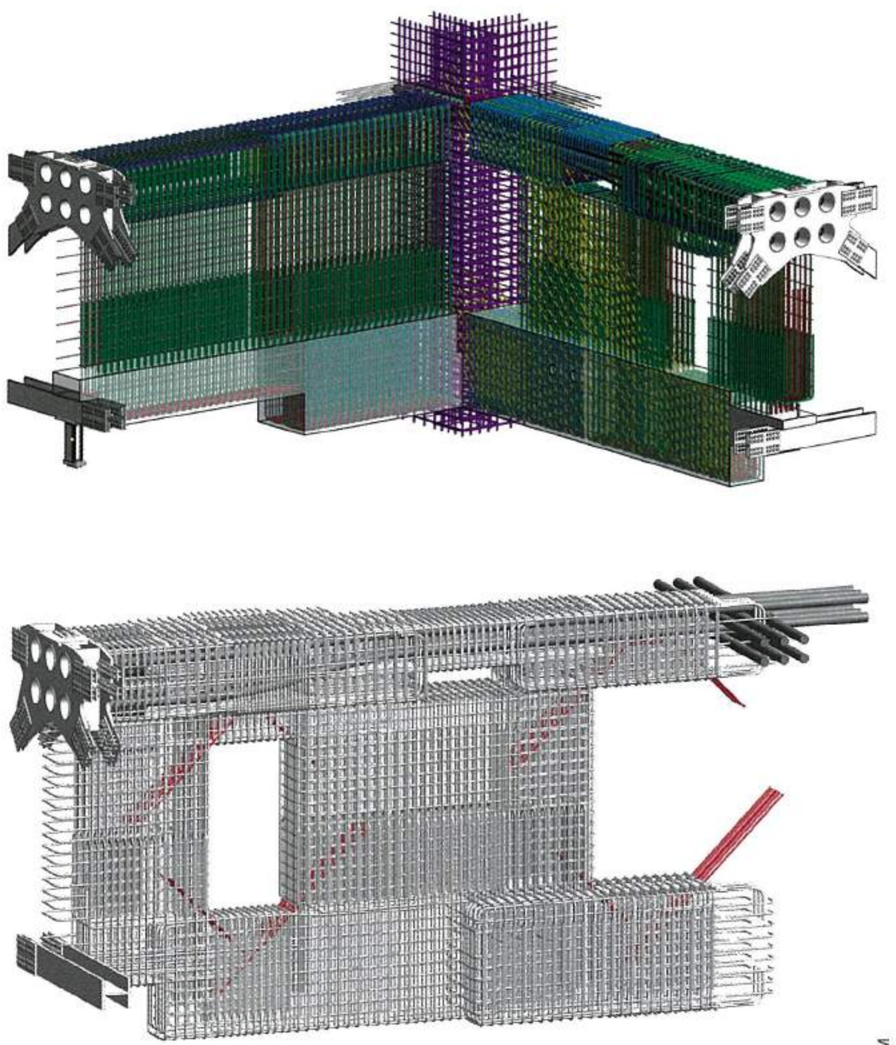
↑↗FIGURE 11: 'Leave-outs' at corners of core for outrigger construction



↑FIGURE 12: Model of outrigger reinforcement to demonstrate reinforcement placement sequence

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and enhance the connection between the steel truss and allow easy access to the post-tensioning tendons.

The layout of the post-tensioning holes in the steel truss node was dictated by minimum tendon-spacing requirements and the truss node geometry was configured to be as compact as possible given this constraint. The truss nodes consist of two, shaped gusset plates, each in the same plane as the belt truss flange members, connected by stiffening plates and hollow round tubes through which the post-tensioning tendons pass.

The post-tensioning strands first pass through larger 360mm holes in the outer plate and a round steel tube connecting the two plates; they bear directly against the annular space around smaller 215mm holes in the inner shaped plate.

The longitudinal outrigger reinforcement is also mechanically coupled (threaded) directly to the back side of the inner shaped truss connection plate. The space between the inner and outer truss node connection plates is filled with concrete, as is the space directly below the truss node, so that the truss node and outrigger form one contiguous composite element.

By post-tensioning *through* and *against* the steel truss node, the truss is securely connected directly to the concrete outrigger by (among other mechanisms) the tremendous friction force that results from the steel truss node bearing against the outrigger (Figure 13). Furthermore, the steel truss node serves as a large stiffening and anchorage plate for the tensioning strand termination and helps to distribute the anchoring forces evenly across a larger area of concrete than could otherwise have been achieved with conventional post-tensioning anchorage terminations.

Finally, the direct mechanical connection between the truss node plate and the longitudinal outrigger reinforcement provides additional confinement to resist bursting forces in the vicinity of the anchorage.The hybrid steel-truss-node and post-tensioning anchorage connection is therefore truly stronger than the sum of its parts.

Steel belt trusses

A continuous steel belt truss connects to and spans between the post-tensioned outriggers. It cantilevers approx. 9m from the tips of the concrete outriggers out to the corners of the tower. The steel belt trusses are nominally one storey deep (4.05m between the centres of the truss chords) and inhabit the space between levels 10 and 11 and levels 28 and 29. The trusses are located just inside the building envelope, and the diagonals pass directly in front of the glazing.

The steel belt trusses utilise standard, wide-flange, UC 356 rolled shapes up to a maximum size of 340kg/m. The strong axis of the members was oriented horizontally, as is typical in bridge construction, to simplify the connections by allowing flange forces to be transferred through shared gusset plates.

All field connections were made with 27mm, grade 10.9, high-strength friction bolts. The 27mm bolt size was the maximum-diameter bolt that could fit in four rows across the width of the UC 356 sections and was chosen to minimise the connection sizes. The steel truss was designed and

↙↓FIGURE 13: Concrete-to-steel connection

↘FIGURE 14: Interior view of steel belt trusses

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detailed to be functional, compact and elegant, but no special considerations were given to aesthetics or alignment with architectural finishes, as it was originally intended to be clad.

In the original architectural design for the transfer floors, the steel truss diagonals were to be boxed out with plasterboard and wood and enclosed to match the typical interior finishes.

However, on seeing the structural outrigger and steel belt truss elements taking shape, the owner, Harry Handelsman, took a liking to their unadorned industrial character and directed a last-minute design change so that the steel trusses would remain exposed to view, along with the outrigger truss concrete. This design change required intumescent fire proofing of the steel with a high-grade finish, but otherwise the original truss detailing choices remained unchanged.

The character of the outriggers and belt trusses inspired the design of the apartment interiors at the transfer floors and the final interior design celebrates the structural engineering design that supports the tower's improbable geometry (Figures 14 and 15).

Construction sequence and staged analysis

The presence of the sky-garden notches and the associated column transfer structures posed an engineering paradox. If the columns under one side of the outrigger and belt trusses were built in a traditional bottom-up sequence and the outrigger and belt truss transfer system initially sat atop of, and engaged half of, the columns below, the transfer structure would be stiffer. But subsequent gravity loading of the core would be asymmetric and would result in lateral drift of the tower (the tower would tend to lean, as more load would go into the core on the side with the notch).

In principle, it would be possible to predict these deflections and make corrections during construction; however, a corrective approach would add complexity to construction and it would be difficult to accurately predict such movements.

Compounding these challenges, the resulting structure would be highly indeterminate – the portion of vertical load transferred by the truss, versus the amount that would pass directly through the truss to the columns below, would be nearly impossible to predict accurately. It would depend on the relative stiffness of the truss system versus the axial stiffness of the columns, all of which would be effectively changing with time as the building changed during construction and the concrete elements crept and shrank.

To eliminate these challenges and uncertainties, SOM engineers chose to transfer not just half the columns, but all the columns at each outrigger level. By transferring all the columns, the outriggers and belt trusses were loaded in a balanced, uniform manner and the resulting deflections were symmetric and predictable. Loads on one half of the outriggers balanced the loads on the other half, the core was evenly loaded, and the tower did not deflect laterally during the initial construction.

Ensuring that all columns were transferred was accomplished by designing a temporary gap or 'leave-out' into the columns just below the belt trusses. The columns below the belt trusses were not initially poured (and the encased steel column connection and mechanical reinforcement couplers were initially left disconnected; Figure 16). During construction, a 50mm gap was visible and the width of this gap was visually monitored to ensure no load was transferred to the column stubs below the belt trusses (Figure 17). No special survey, monitoring or measurement of the outrigger deflections was required during construction, but the gaps at the embedded steel column connections served as a tell-tale: if unusual movements were to occur, it would be readily apparent looking at these connections.

After the level 10–11 transfer structure was completed, conventional concrete post-tensioned slab construction proceeded atop the outrigger platform for levels 12–25. At this point in the construction, 100% of the dead load from levels 10–25 was transferred into the core walls and no column connections were engaged below level 10.

The process was repeated above level 25: the level 28–29 transfer structure was first completed, and then levels 30–36 were again built as conventional concrete construction (Figure 18), with no column connections below level 29. Upon completion of level 36, two tiers of floors were fully transferred to two transfer systems and no columns were engaged the full height of the tower.

The sequence described above was fully documented and described in SOM's construction documentation (Figure 19). Thus, the contractor's means and methods were prescribed to some degree by the engineering design.

Above level 36, where the highest sky garden is located, the tower is cut back to a triangular extrusion and continues up to the roof level (42) only on the northeast side. The centre-of-mass of the structure above level 36 is thus offset to the northeast and construction of each level of floor above level 36 caused the building to lean slightly. To minimise this lean, the decision was made to 'lock up' and completely connect the 22 columns below each outrigger.

At this stage, the majority of the gravity load for the tower was already transferred through the outriggers. Upon initial 'locking up' of the columns, no additional load was carried by these columns, but the building's lateral system was suddenly stiffer, as each transfer floor now formed a partial outrigger by engaging 13 of 24 perimeter columns. This additional lateral stiffness reduced the tendency of the building to lean while levels 37–42 were constructed.

In this manner, only *balanced, symmetric* gravity loads were applied to the disconnected outriggers, resulting in initially symmetric deflections. *Asymmetric* gravity loads (from level 37–42 construction) were applied to the structure with connected outriggers so that their ability to resist lateral drift was mobilised.

As discussed previously, the loading of the structure from levels 37–42 – and due to all subsequent wind and imposed gravity loads – is highly indeterminate. It is not easy to predict how much load will be carried by the outriggers and how much will go directly through the trusses into the columns below.

To account for this uncertainty, several different construction and loading sequences were analysed to consider the effects of delayed column lock-up and variations in the timing of the cladding and superimposed dead load. The worst-case forces resulting from these scenarios were used for the final design.

In addition to different loading sequences, load combinations were considered with reduced superimposed dead loads to capture scenarios where gravity load would be beneficial (i.e. to find the worst-case tension in columns just under the outriggers).

Because the structure is indeterminate and the load path could vary greatly depending on the construction sequence, the final member design represented an envelope of different construction and loading scenarios. For example, the design of a lower perimeter column controlled by compression is governed by the scenario where the columns are locked up as intended, but the superimposed dead load is applied late, so that more superimposed dead gravity load goes directly through the columns.

The design of a steel belt truss member is typically governed by the sequence where the superimposed dead load is applied early, before column lock-up, so all the superimposed dead load must be transferred through the steel trusses.

↗FIGURE 15: Interior view of post-tensioned concrete outrigger and steel belt truss

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↘FIGURE 16: Embedded steel columns below belt truss disconnected during construction

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↖FIGURE 17: Steel belt truss

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↘FIGURE 18: Construction of concrete floors above upper transfer floor

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Column reinforcement for columns directly below an outrigger is typically governed by the temporary tension condition where there is little or no compression in the column (initially), but wind or asymmetric gravity loading causes tension in the columns due to the partial lateral outrigger action of the transfer floors.

At these locations, where wind-outrigger-action-induced column tension could exceed gravity load compression, all the columns were detailed with mechanically coupled splices that helped ensure a robust tension load path (Figure 20). Columns that might encounter tensile forces are typically just under the outriggers, where gravity floor loads are smallest.

For an indeterminate structure like the Manhattan Loft Gardens tower, where load paths are sensitive to construction sequence, linear static analysis (or even non-linear, non-staged analysis) often leads to erroneous results. The load in any member in the structure can only be determined by tracing a specific staged construction history. Each analysis run represents a very specific sequence of construction and loading events and the results from different load cases cannot be combined or superimposed independently of the build and load sequence. This presents challenges for applying load factors and calculating creep and shrinkage effects.

The approach taken was to run separate service-load (SLS) and ultimate-load (ULS) staged analyses (and usually several of each). In the SLS scenario, the member self-weights were added in the sequence the members were built, and imposed and wind loads were incrementally added to, and then removed from, the completed structure in the staged analysis run. The same approach was used for ULS loads, but with the appropriate factors applied at the time the loads were applied.

The staged analysis results provide a record of the load in a member as it changes with time

↘FIGURE 19: Prescribed construction sequence

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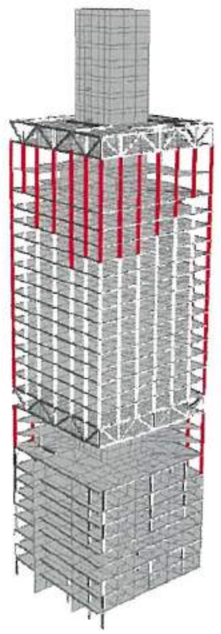

↖FIGURE 20: Columns subjected to tension forces and mechanically coupled column splices

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and changes in the building state. To calculate the relative deflections in the columns and floors supported by the outriggers, for example, an SLS staged construction sequence was run that followed the anticipated build sequence and the time at which cladding and superimposed loads were applied. For each floor and cladding attachment point, the deflections occurring *subsequent to slab construction*, but *up to cladding installation*, were computed and summarised in a movement and tolerance report provided to the frame contractor and cladding contactor.

These movements were required to ensure that the cladding bracket connections were designed to allow sufficient adjustment to compensate for any deviations from the theoretical that occurred before the cladding was installed.

The deflections that occur at each floor level *subsequent to cladding installation* are due to the additional loads and the weight of construction above that gets added to the structure after a level of cladding is installed, and also due to the long-term creep and shrinkage of concrete. These *subsequent-to-cladding* deflections were also traced and provided to the contractors – to ensure that the cladding movement joints were sized adequately to allow for the movements that occur after the cladding is installed.

It is typically not the absolute movements, but the *relative* racking movements between adjacent glass panels that can be cause for concern. The location of the greatest racking movement on Manhattan Loft Gardens is between the tips of the belt trusses and the tips of the outriggers. This was estimated to be about 10mm over 9m or L/900 (Figure 21).

Robustness and redundancy

The seemingly unstable form of Manhattan Loft Gardens might suggest a structure that is less robust than a standard building. But, in fact, the presence of the transfer structures provides a level of robustness and redundancy well beyond what one would find in a more conventional building.

Manhattan Loft Gardens is inherently insensitive to removal of, or damage to, critical parts of the structure because its design already includes systems that allow the removal of columns. The connection of the outrigger transfers to (half) the columns below brings further redundancy and allows multiple load paths. For example, any one column at any location in the tower can be removed: the load that was formerly carried in compression can be hung in tension from the column above and taken up to the outrigger level above.

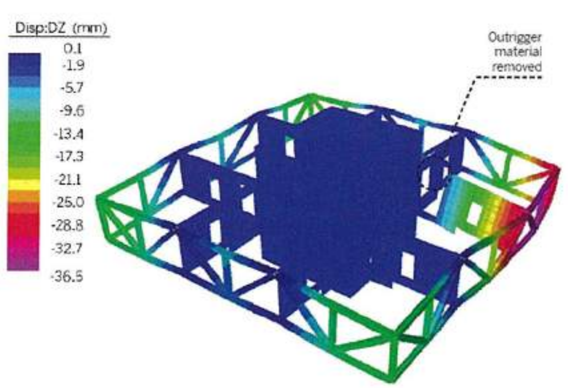
The belt trusses allow redistribution of loads between columns and make the structure more resilient. It was shown that even in the extraordinary circumstance (well beyond any code requirement) that an entire post-tensioned outrigger experienced a shear failure, the steel belt truss could redistribute load to the adjacent remaining outriggers and prevent collapse (Figure 22)

Conclusions

The successful completion of Manhattan Loft Gardens demonstrates that an ambitious architectural design (Figure 23) is not incompatible with cost constraints and constructability. The engineering analysis carefully accounted for the uncertainties inherent in a highly indeterminate structure and adopted an overall envelope approach to structural design.

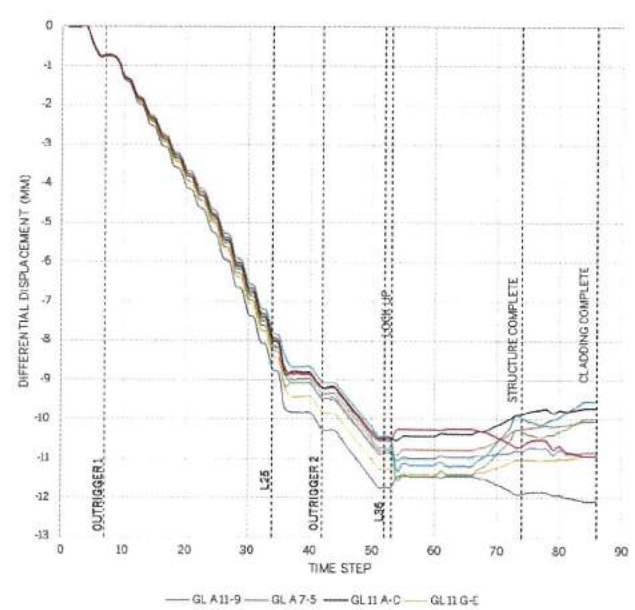
The hybrid post-tensioned outrigger and belt truss transfer design provided balanced platforms on which to stage conventional concrete residential frame construction. The transfers were integrated into and around residential floors and the structural design added to the value and appearance of the apartments.

Perhaps most important to the project's success was the engineers' focus on simplicity and constructability from the outset of the design. This approach helped ensure the project could be realised swiftly and without undue additional cost and complexity – designing for how a structure is to be built is as important as designing what is to be built.



Deflection in lower outrigger walls with material removed

↙↘FIGURE 21: Differential vertical deflection versus time between tips of outriggers and corner of belt trusses



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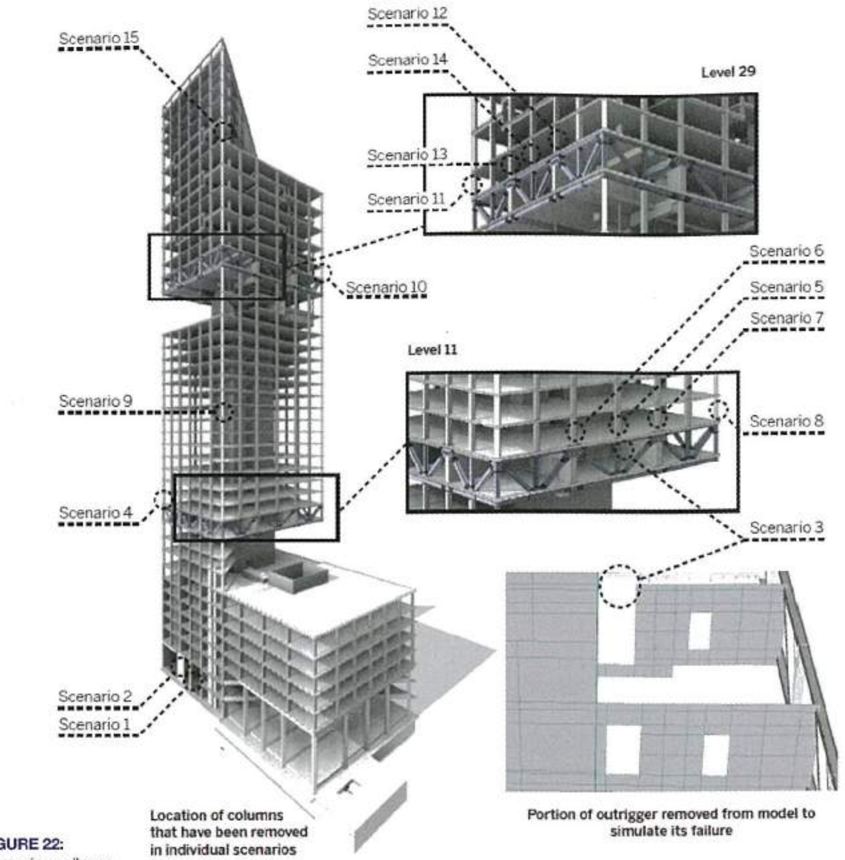

↗FIGURE 22: Progressive collapse scenarios considered

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"DESIGNING FOR HOW A STRUCTURE IS TO BE BUILT IS AS IMPORTANT AS DESIGNING WHAT IS TO BE BUILT

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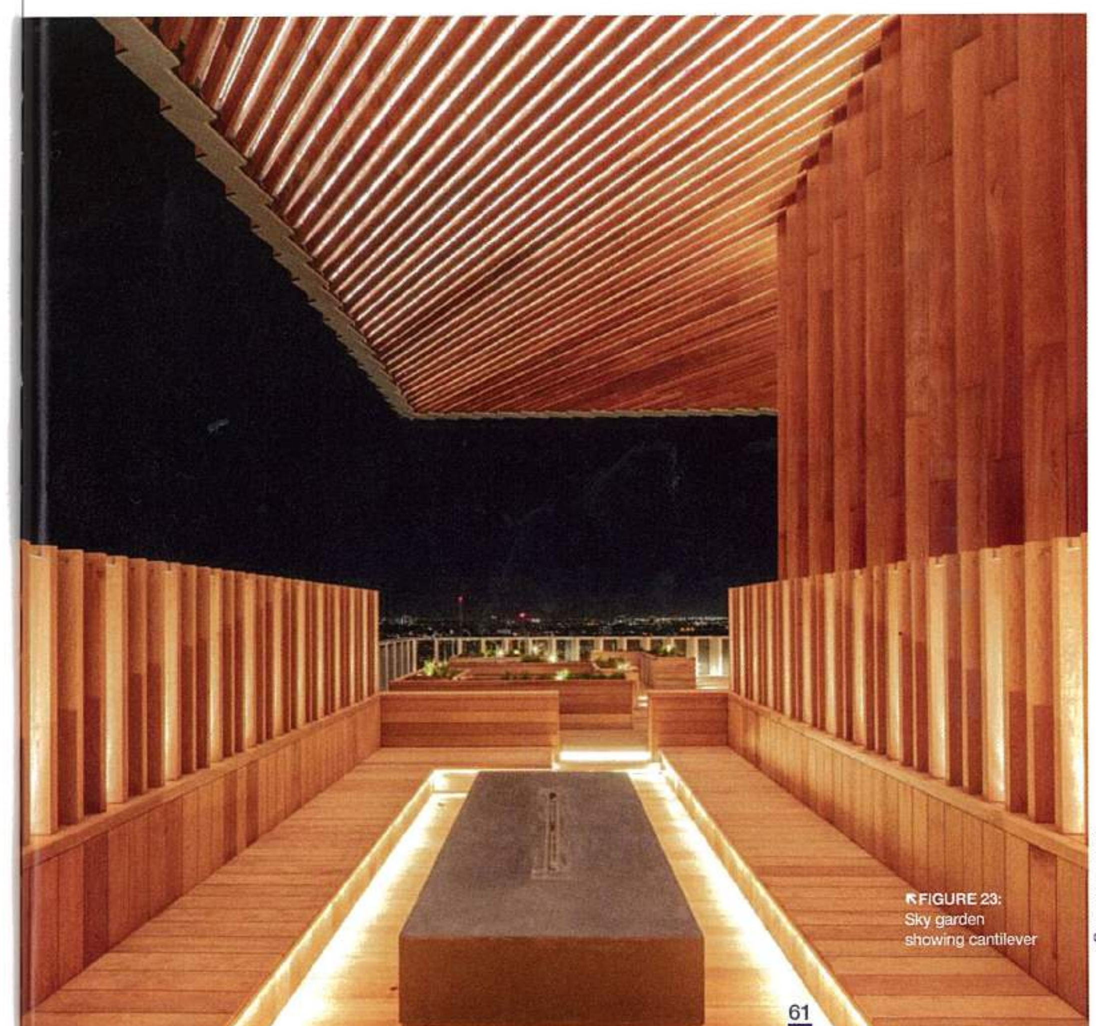

↖FIGURE 23: Sky garden showing cantilever

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