DESIGN OF THE WORLD’S TALLEST BUILDINGS—PETRONAS TWIN TOWERS AT KUALA LUMPUR CITY CENTRE

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SUMMARY

Twin 451.9 m (1482 ft) tall towers just completed in Kuala Lumpur, Malaysia, presented a variety of design challenges related to tall buildings and slender members under wind load, and to construction methods in the Far East. Cast-in-place high-strength concrete for the core, perimeter columns and ring beams provides economical vertical load-carrying ability, stiff lateral load resistance, and inherent damping for occupant comfort. Steel beams on metal deck slabs provide efficient, economical and quickly-erected long-span floors which are easily adaptable to future changes in openings and loadings. The unusual tower plan has alternating cantilevered points and arcs, only 16 main tower columns, haunched wind frame ring beams 8.2 to 9.8 m (27 to 32 ft) long. Vierendeel outriggers at mid-height and sloped columns at setbacks. A unique arch-supported skybridge spans 58.4 (190 ft) between towers at levels 41 and 42, where the towers move more than 300 mm (1 ft) in any direction. A stainless steel pinnacle tops each tower. Extensive analytical, force balance and aeroelastic wind studies addressed individual tower behavior, influences between towers, pinnacle behavior, skybridge overall behavior and arch leg behavior. No supplementary damping was needed for the towers. Pinnacles have simple chain impact dampers. Each of the four arch legs has three tuned mass dampers for the three main modes of vortex excitation. © 1997 John Wiley & Sons, Ltd.

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1. INTRODUCTION

The PETRONAS Twin Towers just completed in Kuala Lumpur City Centre, Kuala Lumpur, Malaysia, each have approximately 218 000 m² (2.3 million sf) in 88 floors, as part of the 1.7 million m² (18 million sf) Kuala Lumpur City Centre mixed-use development (see Figure 1). Their structural frames use columns, core and ring beams of high strength concrete and floor beams and decking of steel to provide economy, fast construction and future adaptability. The slenderness of the towers, and of special elements within the project, required attention to wind behavior and damping. Each of these aspects is discussed in turn.

2. WHY CONCRETE?

Until recently, the world’s tallest buildings had steel columns since ‘typical strength’ concrete columns would be too large, taking too much rentable area. However, new materials such as silica

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fume and superplasticizers, automated batching equipment and strict quality control procedures now make high-strength concrete attainable on a production basis. For the best control, the owner contracted for an on-site batch plant capable of this production. High-strength concrete has numerous benefits for this site, such as the following.

(1) **Economy.** For columns, concrete is unbeatable on the basis of cost. High-strength concrete reduces member sizes, minimizing loss of usable space.

(2) **Efficient placement.** Using pumps, buckets, skip hoists or buggies, the contractor has flexibility in construction methods and maximizes use of the skills of the local labor pool.

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(3) **Simplified construction joints.** Complex beam/column joint geometry, stepped and sloping columns and intersections of beams and outriggers are handled by hooked and lapped rebar. Prefabricated cages give quick lay-up.

(4) **Functionality.** A concrete core provides fire-rated shafts as well as structural support.

(5) **Efficient lateral structural stiffness.** Member proportions inherent in the core and frame system sized for strength provide high lateral stiffness, needing no further ‘beefing up’.

(6) **Occupant comfort under lateral loads.** Concrete increases the average mass density and inherent damping of the towers, lengthens the building period, reduces motion perception and improves occupant comfort during wind without the cost and space penalty of special damping devices.

3. WHY STEEL?

Steel framing, rare for commercial construction in Kuala Lumpur, gave this project the following advantages.

(1) **Fast erection.** Flying forms were impractical for the unusual floor plan and surrounding ring beams. Slow forming and lathing of cast-in-place floors, compared to fast forming at concrete core columns and ring beams, would have delayed the overall project.

(2) **Flexibility in erection.** Steel beams and deck are not tied to the concrete form-and-strip cycle. Steel placement can be accelerated if necessary by working out of sequence, filling floors without scaffolding and placing deck on several floors at once.

(3) **Straightforward fabrication.** Most framing consists of simple-span infill beams between core and ring beams, and was fabricated by local firms. Use of local fabricators was important to avoid steep import duties imposed on off-shore fabricated steel.

(4) **Straightforward erection.** Most of the steel members are light in weight and drop in without involving other members, allowing non-crane placement where necessary and minimizing the demand for crane time.

(5) **Easy tenant renovations.** As a project intended for upscale tenants, the special openings, loading requirements, and other changes that arise during and after construction can be handled by steel more easily than concrete.

(6) **Reduced floor weight.** The extra weight of an all-concrete structural floor system was not needed and would have increased foundation requirements.

(7) **Minimized fireproofing.** Unlike most U.S. metal deck, a profile with dovetail ribs was used to provide rated floors with normal weight concrete of 110 mm (4.3 in.) for 1.5 hr and 125 mm (4.9 in.) for 2 hr rating without fire spray. Deck rolling was done in a new Malaysian plant.

(8) **Predictable deflections.** The perimeter consists of a series of cantilevered triangles and semicircles. The expected long-term creep deflection from concrete cantilevers would have required larger, more costly façade joint details than for steel cantilevers.

4. STRUCTURAL SYSTEM OVERVIEW

Cast-in-situ concrete is used in deep friction barrette foundations and the continuous cap/mat under each tower. Structural steel is used for long-span typical floor beams, supporting concrete-filled metal deck slabs. Structural concrete is used in foundations, in the central core, in sixteen tower perimeter columns and variable-depth perimeter ring beams, and in twelve smaller perimeter columns and ring beams around the ‘bustle’ (half-height mini-tower attached to the main tower). Outrigger beams link core and perimeter at levels 38 to 40 for additional efficiency.
5. FOUNDATIONS

The towers sit on stiff Kenny Hill residual soil, with erratically eroded limestone bedrock beneath. Since distance to bedrock varies dramatically across each tower—from 75 to over 180 m (250 to 600 ft) down—friction elements were used to distribute load gradually in the Kenny Hill. See Figure 2. Concrete barrettes (wall segments cast using slurry-wall techniques) up to 105 m (344 ft) long were used, with friction determined by full-scale load tests and improved by skin grouting (pumping cement grout at high pressure out of ports set along the barrette faces). Barrettes are longer where bedrock is deeper, to even out settlements. Barrette concrete was specified at 45 Mpa (6500 p.s.i. cube) strength.

The tower pile caps/mats are 4.5 m (14.75 ft) deep. The 13,200 m$^3$ (17,250 cu yd) of 60 Mpa (8700 p.s.i. cube) concrete in each mat was cast in one continuous 44 to 50 h operation. To minimize differential temperatures, chilled water was used and, ironically, the mat was insulated for a month.

6. CORE

Each tower has one central core for all lifts, tower exit stairs and mechanical services. Satellite bustle stairs have non-structural walls, since they would be less effective cores (see Figure 3).

Figure 2. Tower profile with foundations
For the PETRONAS Twin Towers, two virtually solid walls running North–South, and one running East–West, provide ‘Webs’, for the core ‘cantilever beam’, making the core quite stiff and efficient. As a result, it carries slightly more than half the wind overturning moment at the foundation. To resist wind the core has thick, heavily reinforced corners (see Figure 4).

The overall core varies from about 23 m (75 ft) square to $19 \times 22$ m (62 x 73 ft) in four steps, with outer walls varying from 750 mm (30 in.) to 350 mm (14 in.) and inner walls a constant 350 mm (14 in.) to avoid complications with lift shafts and the self-climbing forming systems. Above level 70, corners of the core drop back to avoid ‘pinching’ office circulation against the perimeter columns. Concrete grade varies in three steps from 80 to 40 Mpa, matching tower column grades (see Figure 5).

7. COLUMNS

Columns were cast in reusable steel forms and will be open to view at most floors after chipping fins, filling voids and bug holes, and priming the surface for finish painting.

The sixteen tower columns vary along their height from 2.4 m (7.9 ft) diameter to 1.2 m (3.9 ft). Five size increments minimized the time and cost associated with formwork changes. Concrete varied from 80 Mpa (11 600 p.s.i. cube, 10 000 p.s.i. cylinder) to 30 Mpa (5800 p.s.i. cube, 4900 p.s.i. cylinder) in three steps. The twelve bustle columns step in size three times, from 1.4 m (4.6 ft) to 1 m

Figure 3. Typical lower floor plan
Figure 4. Core wall layout (lower floor shown)

Figure 5. Column sizes and concrete grades

(3.3 ft) diameter. Bustle concrete grades differ from the tower since its casting occurs on a later schedule (see Figure 5).

Setbacks at floors 60, 73 and 82 are handled without transfer girders by using sloped/stepped columns over three-story heights (see Figure 6). A half-round outside nosing holds the critical face-of-column location for cantilever floor trusses. The inside half-round gradually shifts inward by adding infill panel forms at each floor. Sloping concrete columns use round forms with modified formwork ends. Above floor 84 slopes are greatest and steel columns and ring beams are used to avoid complicated, and therefore slow, concrete formwork construction.

8. BEAMS

Perimeter frames at tower and bustle use tapered or haunched ring beams all around (see Figures 3 and 7). Beam depth varies from 1.15 m (3.8 ft) at columns to a flat zone 725 mm (2.4 ft) deep at tower beam midspan or 775 mm (2.5 ft) at bustle midspan to pass ductwork. This works well for stiffness against sidesway, providing 34% more stiffness than a uniform beam of the same average depth. Span variations between floors due to column changes and setbacks (reduced building radius) are taken by
the midspan flats to maximize reuse of haunch forms. Beam concrete grades match column grades to simplify tracking and concrete pumping.

9. OUTRIGGERS

East–West outriggers link the core and perimeter columns at levels 38–40. Wind in this direction acts on a larger area due to the bustle. Three levels of beams linked by midspan posts were sized to help resist wind overturning while minimizing forces from differential shortening of core and columns.

10. SKYBRIDGE

A double-deck bridge spanning 58.4 m (190 ft) connects the two towers at the skylobby elevator transfer stations on floors 41 and 42, 170 m (558 ft) above grade, for easy circulation between upper tower floors with a minimum of lift transfer and possible crossover exiting through the other tower, reducing the required size of exit stairs below (see Figure 8).

Because of the great height and span, structural steel was used for lightweight and easier construction. Although single-span trusses could have been used, a two-hinged arch and continuous floor girders offered a shallow walkway structure, minimized expansion joint movement (joints move at both towers), self-centering action from restraint at the arch crown and a strong visual identity. The skybridge design considered the effect of complex tower movements on joints and members, including vertical midspan movements due to tower deflections, the aerodynamic response of the

Figure 7. Perimeter haunched beam details
1.1 m (3.6 ft) diameter pipe legs, fatigue, the response to sudden loss of support, creep and shrinkage movement and compensation, and bridge façade panel movements.

11. PINNACLE

Each tower is crowned with a 73 m (239 ft) tapering top which conceals a double-boom tower building maintenance machine, aviation lighting and lightning protection (see Figure 9). As at the upper tower floors, the sloping columns and stepped profile of this area render concrete framing impractical, so steel is used throughout. The lower pinnacle has eight internal, radial, painted structural steel frames stabilizing the upper pinnacle. The upper pinnacle is a single mast of tapering ‘squared circle’ cross-section assembled from four faces of curved, structurally functional stainless steel plate bolted to four stainless corner fins (see Figure 10).

12. DYNAMIC STUDIES—TOWERS

The dynamic properties of the main towers are important for cross-wind effects on the structure and for occupant comfort. Tower wind behavior was studied three ways.
12.1. Analytical modeling

Each tower was modeled in three dimensions using the SAP90 general analysis program, including perimeter beams and columns at all floors, a central column representing the core, and an outrigger beam/post system from columns to rigid offsets from the core.

Column gross section properties were used since working axial compression stresses exceeded any induced tension. Concrete elastic moduli were varied with design strength using the ACI 318 formula. Laboratory tests of field samples showed $E$ at or slightly above calculated values. Dynamic studies relate to short-term wind loading, so $E$ was not reduced for creep.

Haunched ring beams were modeled by uniform members of matching stiffness in ‘S-curve’ bending since frame wracking was of primary interest. Properties were taken as ‘cracked’. For the anticipated range of moments and beam designs, a typical value of $I/2$ was used as a good average. Enhanced stiffness at the beam-to-column joint was represented by a rigid beam offset of 0.5 to 0.8 of column radius to recognize flexibility within the joint itself. Lower column diameters, much larger than beam widths or depths, create greater rigidity. Beams would have little effect on column rigidity, so no rigid offset was taken in columns.

The ‘column’ representing concrete core properties in the full tower model was calibrated to match the behavior of a separate core model with each wall and beam represented. Outriggers were connected to proper core corner locations using rigid offsets.
12.2. Tower properties

Average mass density is important for the determination of dynamic behavior. For the PETRONAS Twin Towers, the mass density of the structure is 0.3 (18 p.c.f.), between steel-framed and all-concrete building values. Column and core sizes decrease with height, but floor area also reduces, giving relatively uniform density values. The lateral ‘first mode’ period in each direction is about 9 seconds. Torsional stiffness provided by the ring beam frame and the buslte reduces the ‘first torsional mode’ period to about 7.2 seconds.

12.3. Force balance wind model

A 1:400 high-frequency force balance model was used to determine along-wind and across-wind forcing functions for each overall tower. Based on the design wind speed of $35 \cdot 1 \text{ m s}^{-1}$ (3 s sample = 65 m.p.h. fastest mile) at 10 m elevation, wind forces for structural design were determined from the mean and standard deviation of wind forces measured and the buildings’ dynamic properties.

12.4. Aeroelastic wind model

The highly articulated nature of the façade may cause helpful aerodynamic damping as the buildings move. The twin tower configuration may cause interaction between towers through cyclic vortex shedding. Analytical procedures and force balance results are insufficient to study such phenomena. A 1:400 aeroelastic model with both towers hinged at their bases simulated along-wind and across-wind tower behavior with damping varied between 1 and 3% of critical.

12.5. Wind values from standards

Lateral loads inferred from wind tunnel models are related to building mode shapes and mass distributions. This is reasonable for dynamic across-wind excitation, but for a tower with varying mass/length may be inconsistent with quasi-static along-wind load. As a check on reasonableness, the recommended wind loads of ANSI/ASCE 7-88 were compared to the wind tunnel results in the longitudinal direction (ignoring the bustle area). For Exposure B (suburban, wooded) and octagonal plan shape (the plan is rounder than a square, but bluffer than a ‘rough’ circle) base shears and moments were very close to the wind tunnel values. At upper, low-mass floors ANSI-ASCE 7-88 gave somewhat higher shear and overturning values, which were used in design.

12.6. Results of tower studies

For the overall structural loading, the force-balance and ANSI/ASCE values were very close. Aeroelastic values using 2% of critical damping showed slightly reduced base forces at one tower and slightly increased base forces at the other. Because even a slight increase in damping for ‘50 year storm’ movements would reduce aeroelastic results, the design of both towers was based on the largest force-balance results for either tower.

For occupant response, an inherent internal damping of 2% and a ‘10 year storm’ gives peak accelerations at level 87 of 17 to 20 milli-g’s from force-balance tests and 14 to 18 milli-g’s from aeroelastic tests—well within the commonly used 21 milli-g guideline for comfortable office occupancy in long-period buildings. Therefore no supplementary damping, such as viscoelastic pads or tuned mass dampers (TMDMs) was needed or provided.

13. DYNAMIC STUDIES—SKYBRIDGE

The response of long, slender cylindrical bridge legs to vortex-shedding excitation at low wind speeds was of particular interest. Each leg is a 1.1 m (3.6 ft) diameter steel pipe spliced by butted bolted flanges. Pairs of legs diverge as they rise from a common lower support point on each tower (see Figure 11). All four legs connect rigidly to a pentagonal steel box girder supporting the bridge midspan. The four legs are effectively isolated from the rest of the bridge and the towers by joints of PTFE (‘Teflon’) on stainless steel: ‘ball’ joints at lower bearings, ‘rocker’ and ‘slide’ bearings at box-to-girder support pads and a ‘pin-in-globe’ bearing for lateral leg restraint at box midpoint.

13.1. Analytical modeling

The skybridge was modeled separately from the towers for ease of construction and data reduction. When necessary, tower movements were represented by enforced displacement of bridge supports. Member behavior was studied both through SAP 90 modeling and through classic mechanics formulas circulated in Mathcad, with good agreement of results. The dynamic properties of the legs were of special interest. The largest leg responses, called modes 1, 7 and 9, corresponded to 4 legs moving the same way, pairs of legs moving in opposition, and each leg in a pair moving in opposition (see Figure 12). As one would expect, differential movements between closer elements require more force, and thus yield shorter periods, than movements between more separated elements (see Table I).
13.2. Aeroelastic modeling

Since aerodynamic damping is critical for susceptibility to vortex shedding excitation, a 1:70 aeroelastic model with legs modeled in 7 segments was used to study this condition. Of particular interest was the effect of merging legs on vortex shedding response, since classical studies were based on the behavior of parallel smokestacks. The aeroelastic study showed vortex shedding excitation at low, common wind speeds for realistic, low levels of internal damping. Excitation to the level of destructive resonance was found to be unlikely, and was not a significant concern, but fatigue at connection flanges from the accumulation of millions of low-stress cycles was determined to be a potential concern.

13.3. Structural considerations

Wind speeds for the three likely modes of leg vibration are low and likely to be present a significant percentage of time. Therefore, millions of cycles of vibration can accumulate over the left of the skybridge. This can cause fatigue, a progressive growth of common, normally benign steel cracks. Fatigue failure could be avoided by lengthening the time needed for a crack to grow to damaging size through reduction of the number of stress cycles (impractical; requires fundamental bridge design changes), reduction of joint stress (helpful within limits), or reduction of the structural response in each cycle. Because structural response is inversely related to the amount of damping, adding even a little damping to a structure can be effective.

For the low, but possible, level of 0.15% of critical damping in the legs, vortex shedding could excite as much as 120 mm of crosswise leg movement. This is within leg strength limits, but as a
cyclic stress it would cause fatigue over time. The aeroelastic test showed dramatic decrease in
response as damping increased from 0.15 to 0.40% of critical, with excitation dropping to 10 mm
which lowers stress values to within limits for AISC fatigue stress category E, extrapolating the
allowable stress range for up to 35 million cycles. For design purposes, 0.5% of critical damping was
set as a desirable goal. Because different mode shapes have different locations of peak stress, their
cycle counts are not additive.

13.4. Damper solution

The need for supplementary damping was considered probable, but not certain. Because inherent
damping of the actual structure could not be determined until after construction was completed, but
access within the legs would be very limited at that time, the developer had dampers designed,
fabricated and installed in the legs before bridge erection.
Three tuned mass dampers (TMDs) within each leg are tuned for the three most important leg vibration modes. To treat all four legs, twelve dampers were required. A thirteenth was fabricated as a standard and is retained at the wind laboratory to permit easy study of TMD adjustments in the future if necessary.

Each damper is a cage of rectangular steel hollow sections, sealed for corrosion protection and sized to fit through the 860 mm diameter opening in each pipe leg segment end flange. The cage is tall and narrow; the TMD mounts to one side of the pipe interior, leaving half the pipe clear for access by ladder. Within the cage, a rigid steel arm ‘pendulum’ pivots from a bearing at the cage upper end. Stacked circular plates totaling 73 kg (160 lb) at the arm end provide the primary ‘mass’ and can be adjusted in location by nuts on a threaded rod. This permits ‘tuning’ the period of the device (see Figure 13).

The pendulum cannot hang straight down, is restricted to less than 1.7 mm overall length and is restricted in lateral travel. Therefore, gravity needs a ‘helper’ in the form of precompressed steel coil springs on both sides of the arm. Through appropriate selection of stiffness above and below the arm, it hangs at the proper slope to align with the bridge leg and has a short period. For the shortest periods, the very high stiffnesses required the use of nested coil springs. Any spring can be removed and replaced within the leg by forcing the pendulum to one side using provided turnbuckles and squeezing the spring with a clamp tool.

The ‘damper’ is provided by a long-life, maintenance-free, temperature-compensated custom dashpot between pendulum arm and outer cage. This ‘shock absorber’ provides 10 to 15% of critical damping for the TMD acting alone. Once mounted within the legs, it will lift leg damping to at least 0.5% of critical. The high TMD damping means that adequate leg damping will be provided even if tuning ‘mis-match’ develops over time.

Figure 13. Tuned mass damper in test fixture
With the completion of construction, actual leg behavior without supplementary damping (TMDs were ‘locked out’ by clamps) has been measured using an eccentric-weight ‘shaker’. As Table I shows, agreement is good for the lower two modes, but mode 9 is stiffer than anticipated. Legs merge before the bottom bearing and the leg pair resists twist due to high torsional stiffness of pipes. These two conditions create greater stiffness against mode 9 deformation than in the computer model. As-built damping for mode 1 is about 0.5%, apparently due to sliding action at top and bottom bearings, but mode 7 has 0.25% and mode 9 even less, since fewer bearings move with these mode shapes. The TMDs will be activated for all three modes.

14. DYNAMIC STUDY - PINNACLE

The stainless steel mast of the pinnacle atop each tower is a tapered ‘square circle’ in plan, with four continuous (field butt-welded) corner fin plates acting as ‘flanges’ and infill panels on the four faces acting as ‘webs’ (see Figure 10). The panels are curved to 3 m radius and bolted to fins using stainless steel bearing bolts. With 51 m (167 ft) of cantilever height, 14 m (46 ft) of ‘embedment’ length and width tapering from 2.6 to 0.6 m (8.5 to 2 ft), the mast is relatively slender and flexible. Mast behavior from vortex shedding was a concern. Halfway up the mast, 14 horizontal 300 mm (1 ft) diameter stainless steel pipes curved to 7 different radii from 1.8 to 2.9 m (6 to 9.5 ft) form a ‘ring ball’ representing the 14 States of Malaysia. Vortex shedding could affect these elements as well.

14.1. Analytical modeling

The structure from level 88 to mast top was modeled in SAP90, including supporting girder springiness and mast mass and stiffness variations. The mast first mode period was 1.3 s and the second mode was 0.5 s. The largest ring of the ‘ring ball’ was modeled separately, supported at quarter points; a period of 0.07 s was found.

14.2. Wind models

Wind tunnel models were not necessary for the pinnacle mast based on four factors: first the ‘squared circle’ shape disturbs smooth wind flow necessary for rhythmic vortex shedding; second, mast tape causes critical wind speeds for vortex shedding to vary with height—at any wind speed, only part of the mast will be excited, creating helpful aerodynamic damping; third, bolting of web panels creates frictional damping in the structure; and fourth, supplementary damping is being provided anyway—see below.

For the ball rings, no tunnel model was needed based on three factors: first, a very short period indicates high stiffness, minimizing possible vortex-induced deflections; second, the rings are curved—wind direction continuously varying along a ring segment creates a damping effect; and third, another connection was introduced to increase internal damping.

14.3. Damping methods

A simple impact damper is provided within each mast as inexpensive assurance against excessive wind-induced vibrations (see Figure 14. A galvanized anchor chain of 54 kg m$^{-1}$ (36 p.l.f.) mass density and 7.3 m (24 ft) length is suspended within the rope of the mast for maximum effectiveness against multiple modes of vibration. A neoprene rubber sleeve, similar to ship hawsepipe protection, wraps the chain to cushion the impact and provide a rebound effect. A 400 mm (16 in.) diameter stainless pipe surrounds the chain and sleeve and is fixed to the mast to provide uniform free travel
and rebounding. Effective free travel before striking the pipe is about 70 mm. Since the chain mass is
much less than the mast mass, the chain will travel this far during much smaller mast movements,
activating damping.

Ball rings are erected in quadrants and supported at their ends by extensions of the mast corner fin
plates. This leaves each ring quadrant free to vibrate on its own. To dampen such vibration and
maintain uniform parallel alignment between rings, an intermediate rib was added to each quadrant
which connects all the rings at midpoint. This creates inherent self-damping by linking the different
vibration characteristics of rings at different radii.

15. CONCLUSION

The PETRONAS Twin Towers at Kuala Lumpur City Centre in Kuala Lumpur, Malaysia, use mixed
construction for cost, schedule and usability benefits. The towers also provide several examples of
slender objects subject to wind excitation.

High-strength concrete in the core, perimeter columns, ring beams and outriggers gives economical
vertical core and column elements of reasonable size, saving rentable space. Concrete construction
uses relatively light, simple equipment and the skills of the local work force, simplifies connections in
joints of difficult geometry, and provides fire rated shaft walls in the core. Concrete benefits wind
behavior by inherent stiffness when sized for strength, greater mass leading to long, more comfortable building periods, and inherent internal damping reducing building response to wind gusts.

Steel beams and decking provide fast, flexible erection to meet an ambitious schedule, while permitting last-minute or post-construction changes for tenants’ special openings or loading requirements with minimal impact. The steel framing system used permits local fabrication and innovative non-crane erection methods, while the decking used provides fire ratings without firespray or thick or lightweight concrete fill.

Wind on the towers was studied by computer models, wind tunnel force-balance and aeroelastic models and comparison with codes. Inherent damping from the concrete is sufficient for the comfort of office occupants, and overall structural forces are consistent between the various models. Skybridge aeroelastic tests resulted in placing compact tuned mass dampers within each steel pipe leg to reduce vortex-shedding for long fatigue life. Pinnacle masts each have a Neoprene™-sheathed chain for additional damping. Pipe rings are connected to create inherent damping between them. These aspects of the PETRONAS Twin Towers show that dynamics under wind excitation can be significant for a wide range of element sizes, from 55 mm wide buildings to 0.3 m wide details.

The use of mixed construction materials and attention to dynamic effects brought the PETRONAS Twin Towers to a successful realization.

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