Paper

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Design of the Torre de Collserola, Barcelona

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Introduction

In February 1988 the City of Barcelona held an invited competition for the design of a new telecommunications tower. They asked for the tower to be a 'monumental technological element… to enhance the image of Barcelona in the context of the 1992 Olympics'. The magnificent site overlooks the city from a height of 440 m above sea level. The new tower is called the Torre de Collserola, after the mountain on which it stands.

Ove Arup & Partners entered jointly with the architectural practice of Foster Associates (now Sir Norman Foster & Partners) and won the competition as the only non-Spanish team. In May 1988 this team was awarded a commission for the conceptual and scheme design of the tower complex. The tower, its supporting plant and equipment buildings and the associated infrastructure and landscaping were to be considered together as a coherent design solution within a site forming part of a designated national park. The commission was later extended to include detailed design and tender documentation for the tower itself, with a reviewing role during the construction phases of the project.

The client, formed specifically to build the tower, was a joint venture company of RTVE-Radio Televisión Española (the principal Spanish national television network), Telefonica (Spanish telecommunications), and CCRTV-Corporació Catalana de Ràdio i Televisió (Catalan television).

The project is part of the recent renewal of the City of Barcelona, which has seen much new infrastructure work, the construction of the 1992 Olympic facilities, and the renovation of much of the city's historic architecture, including the work of Antonio Gaudi.

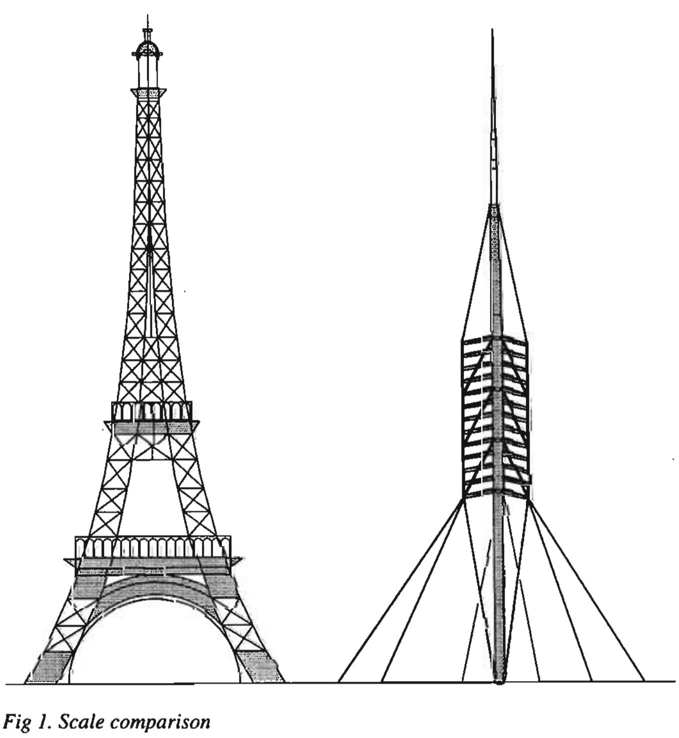


Fig 1. Scale comparison

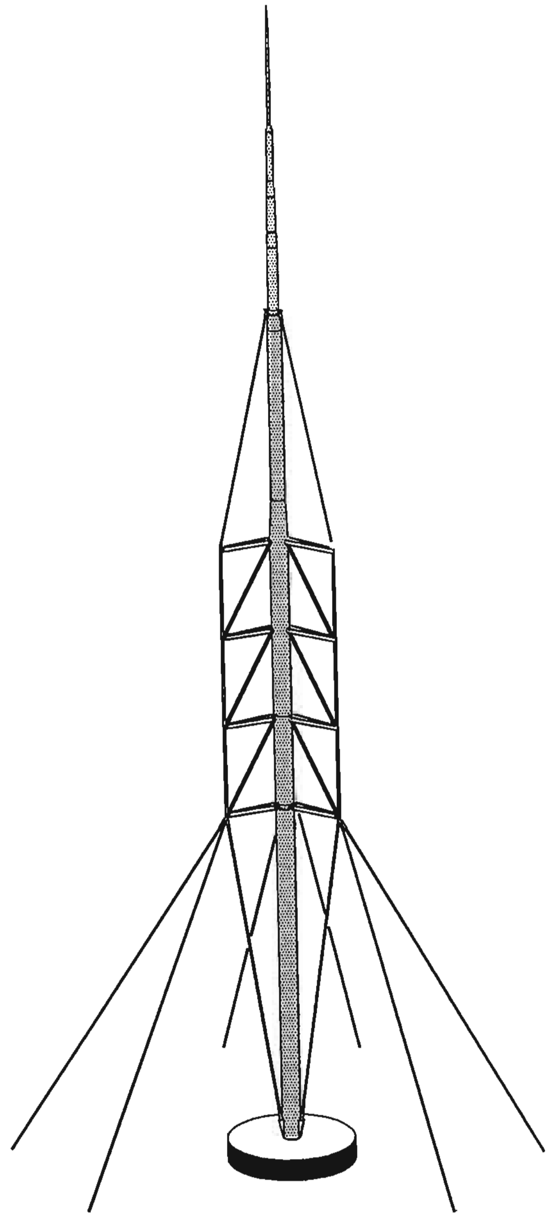


Fig 2. Primary structure

The brief

The brief called for a new telecommunications complex that would form part of the overall communications infrastructure of Spain. The project is also to be a symbol for the City of Barcelona as it enters the 21st century, and had to be designed to be sensitive to the natural beauty of the site, a national park planted with Mediterranean pines. The complex is divided into two parts:

- The tower, which houses the broadcast and relay antennae, signal processing equipment, and a public viewing gallery.
- The support building, which holds signal generation equipment and principal services.

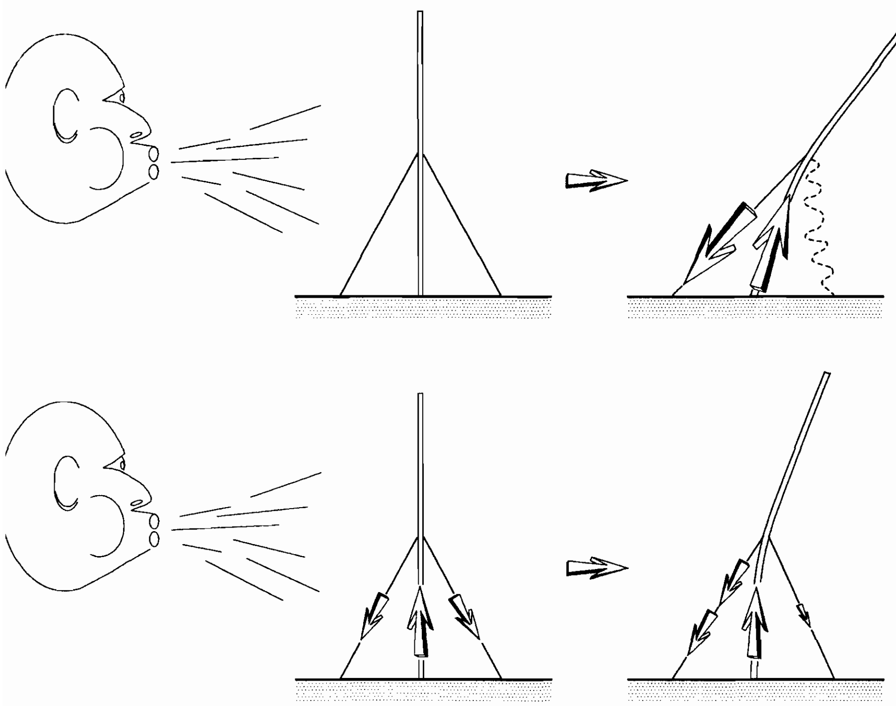


Fig 3. Benefits of prestress

The use of the tower can be summarised as follows:

— up to a level of +85 m:	none
— from +85 m to +152 m:	12 equipment platforms and antennae galleries for parabolic satellite dishes, and one viewing gallery, each of 550 m^2
— from +162 m to +206 m:	small antennae galleries for UHF/VHF transmissions
— from +210 m to +281 m:	even smaller galleries for mobile telecommunications and radio telephones
— at +281 m:	a crane to lift antennae onto the galleries

The brief required that broadcast signals should have unobstructed transmission paths as far as possible. This meant that no electrically conducting elements (including, critically, structural steelwork) could occupy a position in front of an antenna.

The operational efficiency of the tower is defined in terms of the amount of time each year in which the signals can be transmitted, the aim being to minimise broadcasting 'down-time'. This means that the tower has to be structurally stiff to limit twist and tilt of the satellite dishes under wind. In addition, the tower required power and datarisers to serve the antennae, a passenger lift, an equipment lift large enough to carry a 3.6 m diameter satellite dish, and an escape stair. Public and operational functions were to be kept separate for security reasons.

Design precedents

The amount of floor accommodation is much greater than for typical towers, and called for a fundamental rethink of traditional designs. This is perhaps more easily understood if it is imagined that the enclosed antenna galleries are converted to offices, in which case a firm with 700 staff could work there.

Conventional towers are built around a chimney-like shaft which encloses all the vertical services and access facilities, with cantilevered floor galleries. There are many examples of this, such as the towers in Toronto, Moscow, Berlin, and Emley Moor. For Barcelona, a 'control' design was carried out during the competition which showed that, for this type of tower, a slipformed concrete shaft at least 25 m in diameter at its base would be needed. This system separates the lateral load system from the vertical load system.

Towers such as the Eiffel Tower in Paris and many microwave towers worldwide use a tapering steel lattice. As an alternative, for small antenna systems, there are many examples of guyed masts. Some of these reach a

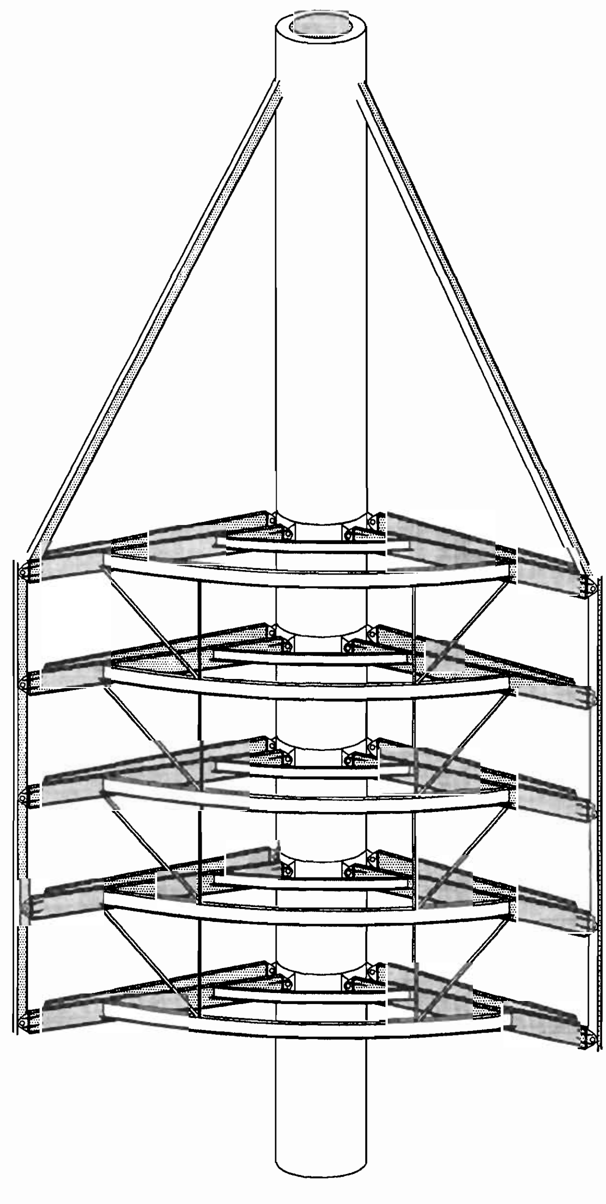


Fig 4. Floor systems

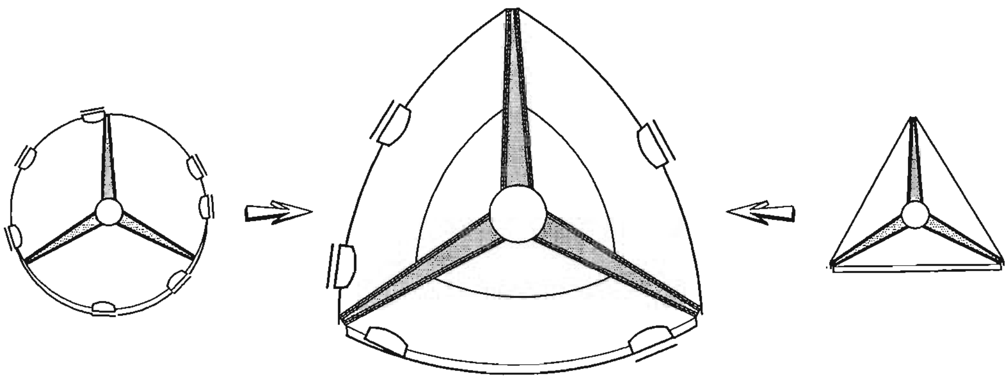


Fig 5. Evolution of floor shape

height of 600 m. There are also examples of cable-stayed towers, such as the Centrepoint Tower in Sydney, which use a network of steel cables to stabilise a comparatively slender concrete core.

A new concept

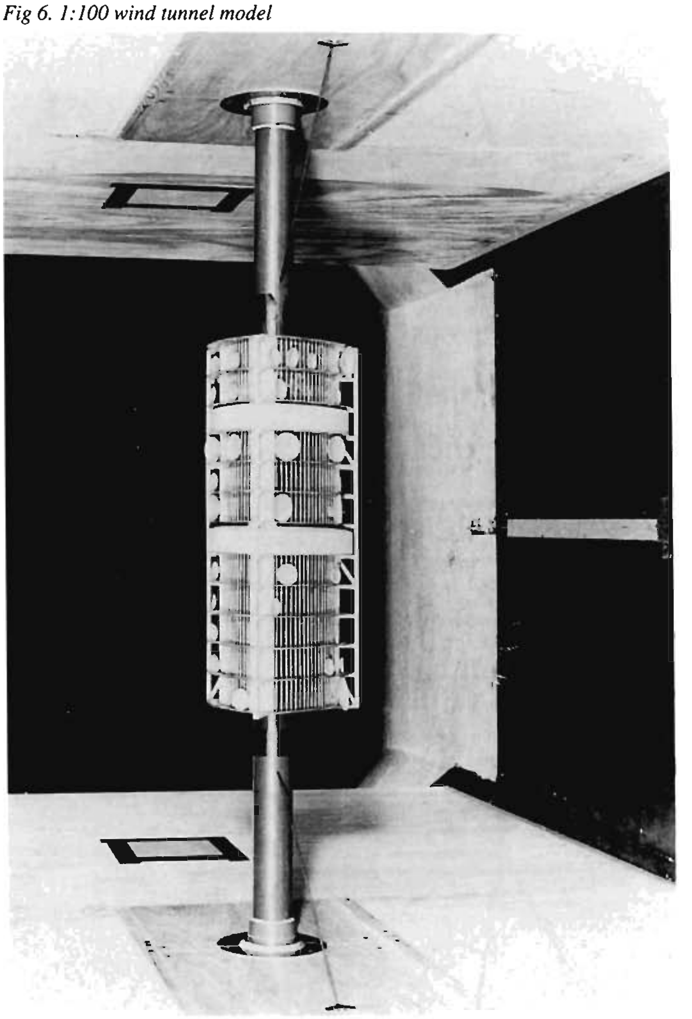
For the Torre de Collserola, OAP developed with the architect a structural system which integrates in a minimal way the requirements of the brief with a skeletal system that carries both lateral and vertical loads. In functional terms, the brief was satisfied with the following components:

- — the main floor system: a simple block beginning 85 m above the foundation
- — above this: a more or less conventional radio mast
- — on top of this: a crane
- — servicing this: power and datarisers, lifts and stairs

Primary structure

The primary structure is essentially a simple concept. Because the architectural expression of the project derives almost entirely from the engineered structure, the design was developed from the earliest competition solution into its final form by carefully evolving the quality, arrangement and material of each element. For example, the main floors are made with beams that taper to the tips of their cantilevers in response to their bending moments and shear forces. The geometry of the floors is radial, to standardise the components but, equally importantly, because it is beautiful to look at.

Fig 6. 1:100 wind tunnel model



The structure has three scales of expression. It is designed to read as a single unit in long views of the project from the City of Barcelona and its surroundings. Visitors to the tower complex see the structure at a larger scale, which brings some of the constituent assemblies, such as a complete floor, or a primary guy anchorage, into focus. It can be argued that these medium and long-range external views are the most important. Users of the tower view it from inside, seeing individual structural and functional components such as the cladding, lifts and stairs, but of course they are expected to spend their time running the sophisticated telecommunications systems rather than admiring the architecture.

The primary structure is made up of a slipformed circular hollow reinforced concrete shaft, braced by three vertical steel trusses spaced equally 120° apart on plan. Within the body of the primary structure, the trusses are made from high strength A510d steel. The lower truss diagonals are parallel strand steel guys, and the upper diagonals made from Kevlar fibre rope. The principal horizontal 'stability arms' of the trusses transfer wind-induced torsional forces to the shaft by acting as cantilevers on plan. To do this they are tapered in width from a maximum at the shaft to a minimum at the tip. The entire shaft and truss system is stabilised to the mountainside by six principal guys arranged in three pairs.

The geometry of the principal structural elements is standardised so that the external dimensions are the same anywhere in the tower. For example, the main vertical truss chords are made from pairs of plates which fit within a constant envelope of 600 mm × 500 mm, but whose thickness reduces from 50 mm at level 1 to 20 mm at level 13. This means that the connection geometry is also standardised, and the same fabrication jig can be used throughout.

To maintain stiffness during strong winds, the primary structure is prestressed by tensioning the principal guys. This means that the downwind guys do not go slack even under extreme winds. The primary structure is as stiff as a conventional highrise building, although its low inherent damping gives comparatively high accelerations, in comparison, for example, with guidelines for tall office buildings. The first translational mode of the structure has a period of 3.31 s. The first torsional mode has a period of 1.71 s. Taking this into account, the dynamic amplification of the response of the tower to wind loads is about 1.1 times its equivalent static performance. It is predicted that the tower will deflect 85 mm and tilt less than 1/12th° at a height of 152 m, under the broadcasting 'down-time' wind speed of 28 m/s (100 km/h).

Principal connections were tested analytically, and aesthetically with model and computer graphic prototypes. They are fabricated from steel plate and designed so that truss forces, up to 3100 t in the lower diagonals, are coincident at each node.

The primary structure has been checked dynamically against shock loading caused by sudden failure of the guy components. Instantaneous deformations increase by about 50 % compared with the long-term equilibrium condition after failure, which is reached after about 60 s.

The structure has also been designed to resist 50 % of the design wind load for a limited period until temporary guys can be installed and the permanent structure reinstated.

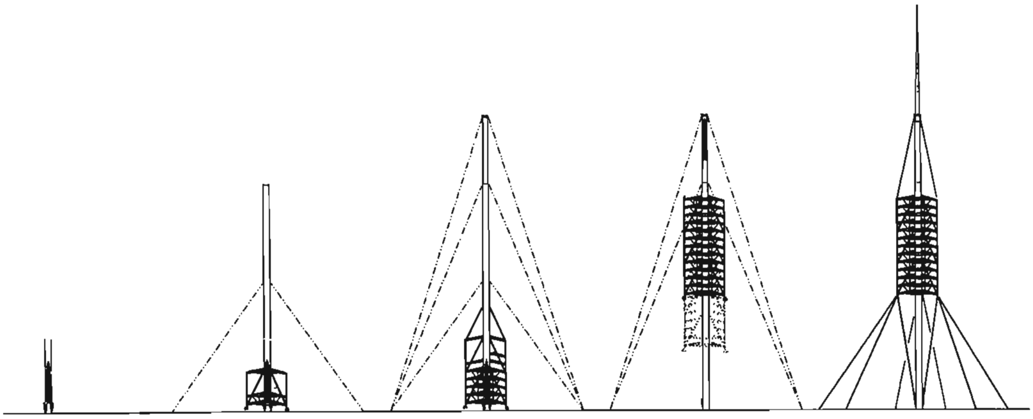


Fig 7. Construction method

Shaft

The central shaft has three uses. As a fundamental part of the structure, it carries the entire weight of the tower, plus the precompression resulting from the guy prestress, and resists torsional wind effects, with help from the principal guys. From 4.5 m diameter at the base, it tapers to a point. For the first 205 m the shaft is slipformed in reinforced concrete with a continuous 3 m diameter hollow core and a wall thickness that reduces from 750 mm to 300 mm. Above this level, to 288 m, is a steel mast which telescopes from 2.7 m to 2.2 m to 1.5 m to 0.7 m, and is topped by a small pointed crane.

In normal service the hollow shaft also behaves as a giant service riser, with continuous access to wave guides and power cables.

Lastly, in the event of a fire in the tower, the shaft acts as a 4 h fire barrier so that escape is possible by two routes. Outside the shaft is a conventional escape staircase, while, inside, the rather more cosy means of escape is by a ladder and platform system which runs 288 mm from the top of the tower to the foundation.

Floors

The floor design has to have a shape which balances the demands of broadcasting and structure. For broadcasting, the maximum perimeter would be obtained with circular floors but, for structural simplicity, a triangular grid spanning onto the primary truss system is the optimum choice. The interaction of the broadcasting circle with the structural triangle gives the characteristic shape of the floors.

The floors are supported by the same structure that resists wind loads. This helps to minimise the risk of load reversal under wind load, and also means that the strain changes in the structure under wind load are reduced.

There are 13 main floors with a storey height of 5.625 m. They are of composite construction with an open stainless steel grillage around the perimeter. The floors hang from the shaft by the three primary trusses and are linked together by a secondary interfloor hanger system. This reduces the effective span of the floor beams, cuts down torsional effects in the floor beams, and distributes heavy point loads between floors. The floors act as stiff rings to transfer wind loads to the primary structure. Each floor is connected to the shaft at only the three points where the stability arms touch the concrete shaft. The lifts and stairs fit into the 'virtual shaft' formed between these points.

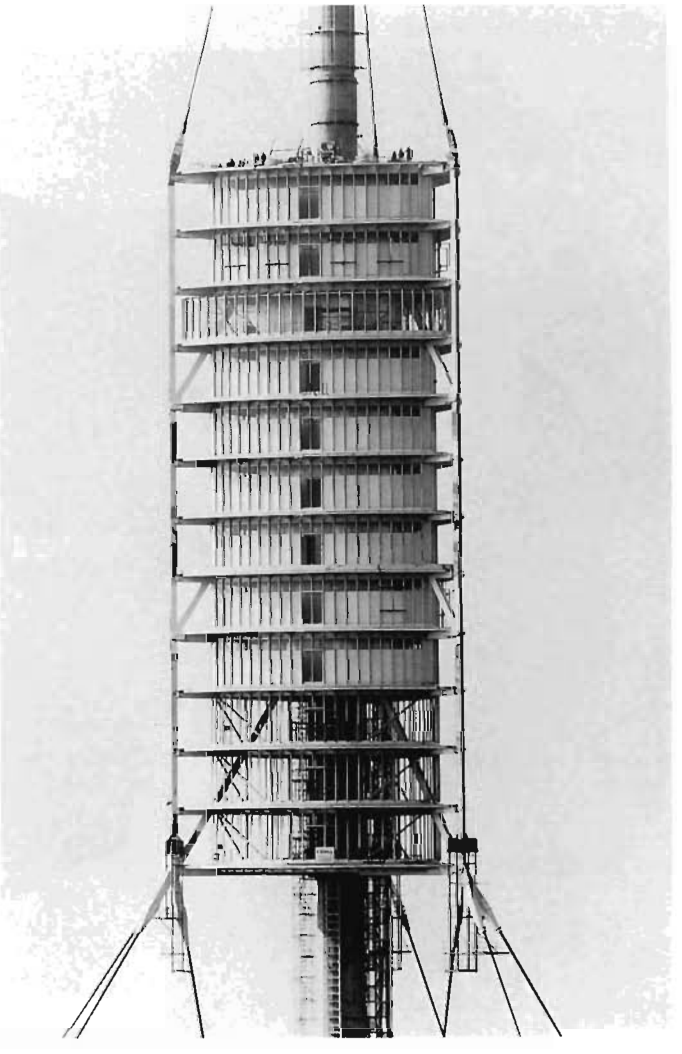
Fig 8. Lifting of floor assembly

Guys

The main guys are made from bundled parallel steel strands. For the ultimate load of 1700 t this requires a hexagonal guy of 320 mm diameter. Because the tower is on top of an undulating mountain ridge, the main guys have different lengths, so the design compensates for this by adjusting the number of strands in each guy to maintain equal stiffness.

The upper guys are constructed of Kevlar 49, which does not conduct electricity and so, being 'invisible' to broadcast signals, allows unrestricted transmission and reception. The material has excellent fatigue performance, about half the stiffness of steel and roughly the same strength. The Kevlar guys used in the project are each made from seven 50 mm cables, bundled and strapped together to remove the possibility of wind-induced oscillation of an individual component guy under wind effects. The guys are provided with an abrasion-resistant polymeric sheath and are protected against ultraviolet light, lightning and corona discharge. Cone-within-cone end

Fig 9. Completed floor assembly (Ben Johnson)



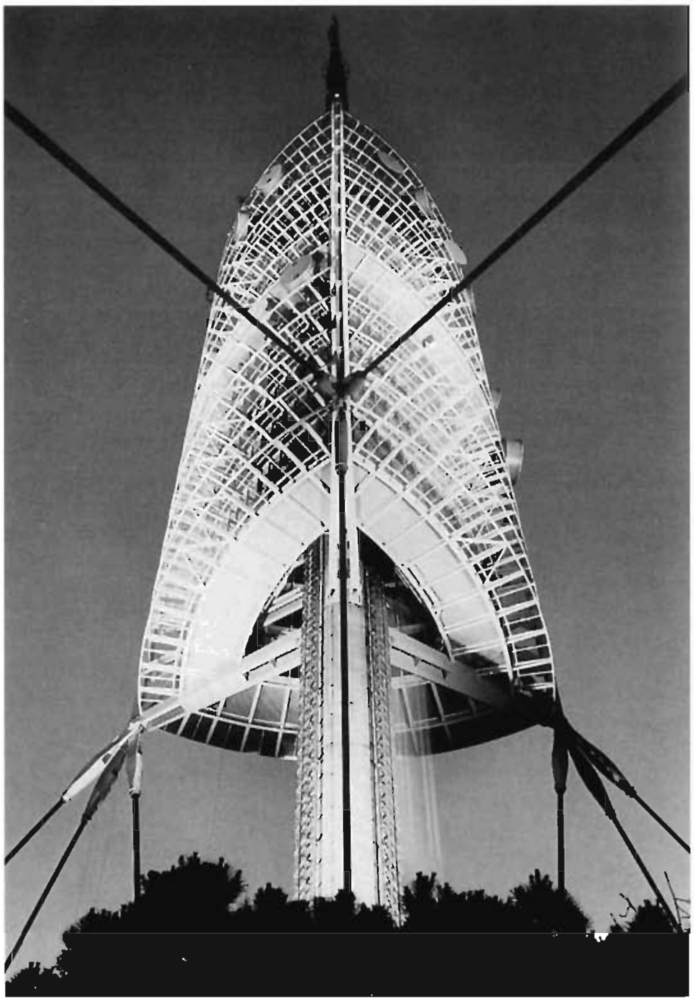
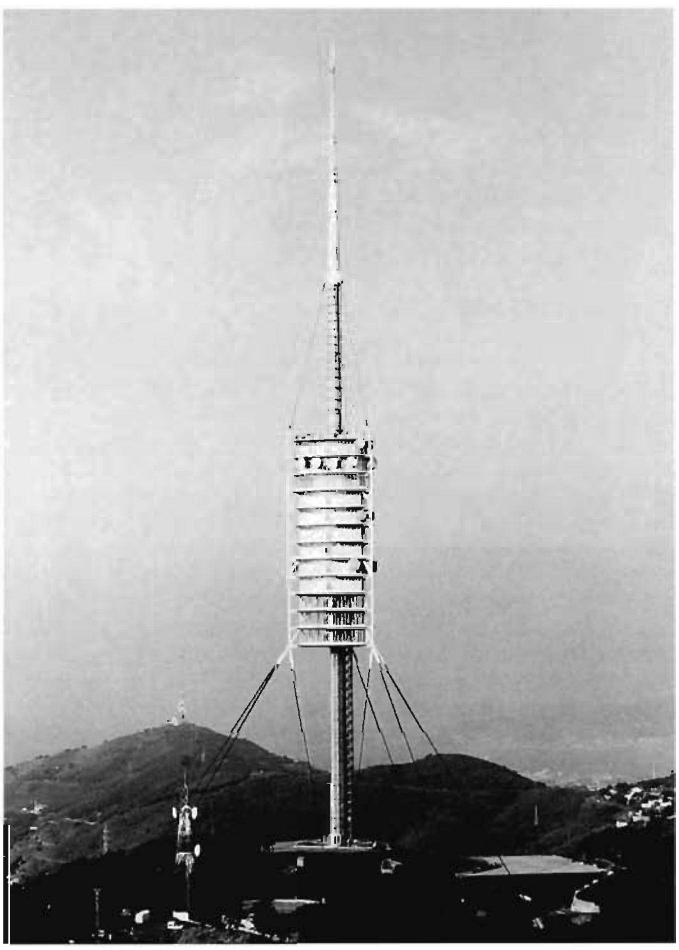


Fig 10. Finished tower (Ben Johnson)



connections have been used, with long radius internal curves to minimise local stress concentrations. Full-scale prototype testing of the guys and their connections was carried out by the manufacturer, Verto Phyllystran of Holland.

Analysis methods

The tower is structurally indeterminate, supported by guys that have significant weight. The tower is also subject to a magnification of wind forces due to dynamic wind effects. Within the range of design loads, and taking into account the benefits of prestress in the primary structure and its inherently low deformations under load, the tower exhibits moderately non-linear behaviour.

Among the external load effects considered were wind loads obtained from the wind tunnel test programme, full and partial live load, seismic loads, foundation movements, accidental damage, and temperature changes. Among the internal load effects are self-weight, shrinkage and creep of the concrete, and variations in the accuracy of application of the guy prestress.

Because of the large number of simultaneous load effects and combinations, the main analyses were carried out using OAP's principal linear elastic program GSA. Although the program is linear, it is possible to compensate for certain non-linear effects, such as guys going slack, by the use of temperature-induced prestress. The loads applied to the model were modified static loads, taking into account the results of non-linear and dynamic parametric studies carried out using other programs. Results from these modified linear elastic analyses were calibrated and checked against the relevant non-linear and dynamic analyses for key load combinations.

Studies into non-linear behaviour were carried out using the OAP programs FABLON and DYMAST, taking account of guy catenary effects and P–Δ effects by iterating the analysis through many cycles.

The response of the structure to dynamic loads was tested with analyses carried out using PAFEC. These included studies into the dynamic behaviour of the tower under wind and also dynamic studies into the effects of the sudden failure of guy components.

Wind engineering

Wind forces on the tower are unusually high. This is because of the exposed position of the tower in its mountainous surroundings, and also because the local topography produces an acceleration of the prevailing 'Tramontana' wind as it funnels down the valleys from the northwest and rises over the ridge on which the tower has been built. The result is similar to the airflow over an aeroplane wing. The effect was studied by wind tunnel testing carried out in two stages:

(1) A topographic wind study explored the speed-up of the wind because of the terrain, using a 1:4000 model of the surroundings 3.5 m in diameter.
(2) An aerodynamic and aeroelastic study tested the response of the tower, using a 1:100 carbon fibre model of the main floors.

These showed that the wind accelerates across the mountain ridge in gusts up to 59.9 m/s at the height of the main floors and 64.9 m/s at the tip of the tower. The clad and equipped floors experience maximum drag factors (C_d) of about 1.35. Measured twisting moments were comparatively small, and the tests showed that, for all credible wind speeds, the tower would not experience galloping instability or vortex shedding problems.

Geotechnical engineering

The site is underlain with mottled fractured schist, the top part of which is weathered. The rock is dry.

The tower foundation is a simple reinforced concrete disc 20 m in diameter and 5 m thick. It is founded 25 m below natural ground level, at the base of the pit which forms the public entrance to the tower and links the base of the tower to the signal-generation equipment in the support building. It carries the shaft working load of 10 700 t into the mountainside. Under this load, a small instantaneous settlement of about 10 to 20 mm is expected. As this shortens the main guys with a loss of pretension in each guy, a small adjustment of prestress is needed during the construction process.

The rock is jointed in blocks and it proved possible to rip it out without explosives, using a large excavator. The dip direction of the rock joints was an important consideration in the design of the anchorages for the main guys, which will carry a maximum working tension of 1260 t each. The anchor blocks were each made of 800 m^3 of mass concrete, shaped to maintain local rock stability during excavation. The technically attractive alternative of rock anchors was considered and eventually rejected on cost grounds.

Seismic hazard study

The tower site lies in an area designated seismic zone VII in the Spanish standards. This is equivalent to zone II of UBC 1982. Because of the importance of the project, as a fundamental part of Spain's communications infrastructure, a seismic hazard assessment was carried out. This showed, for a return period of 500 years, peak ground accelerations of 4.5 % g – 8 % g, compared to 10 % – 15 % defined in the standard. The structural analyses showed that, comparing these figures to the high wind loads on the tower, seismic loading is not a critical design condition.

Awarding the contract

For projects of this type, the main element of cost is in the techniques used to erect the structure rather than in the cost of the materials themselves. After an invited international tender, the Spanish contractor Cubiertas y MZOV won the contract with a construction programme of 18 months.

They submitted two offers. One was a conforming tender, using a method of construction which relied on assembly of the structure, unit by unit, in its final position. They also, however, submitted an alternative which was intended to minimise crane time by building as much as possible as close as possible to the ground. Their proposal was based around the concept of constructing at ground level the 13 floor platforms as a rigid body, and then jacking up the entire assembly weighing 2700 t through 85 m, into its final position. Their slightly reduced offer for the tower and the support building for this alternative was accepted by the client, who acknowledged that it contained a degree of risk.

Developing the erection method

Cubiertas commissioned the Madrid-based engineer Julio Martinez Calzon to assist them in the design of the erection sequence for the project. While keeping largely to the tender design, Cubiertas recognised the advantage of minimising the number of connections from the primary steel structure to the concrete shaft. To do this, they introduced an additional line of vertical structure at the inner end of the radial arms to collect together the forces from the 13 floors into four discrete levels, which coincided with the intersection of the primary structure diagonals with the shaft. During the development of the tender design, the client had specified that the hollow core of the shaft should be completely free of structure, to allow maximum freedom for the installation of power- and datarisers. After tender, this requirement was relaxed. This allowed the circular steel rings of the tender design, which transferred the forces from the three primary stability trusses into the shaft, to be replaced with a simpler, more direct, Y-shaped yoke passing through the shaft at levels 1, 5, 9 and 13. Wind-induced torsional effects were transmitted to these 'stability' levels by the action of the interfloor diagonals and mullions which were modified to act over four floors as vertical trusses.

The tender documentation developed by OAP proposed the early installation of the steel telescopic mast inside the partly complete concrete shaft, to be jacked up the hollow core and then telescoped up into its final position from the top of the concrete. To allow this, the four sections of the mast were designed so that they fitted snugly one inside the other, like a giant car aerial. The advantage of this was to remove the need for a very tall crane 290 m tall, replacing it by a more modest 210 m model. This was adopted as the final construction method.

Building the tower

After excavating and concreting the foundations, Cubiertas slipformed the central shaft to its full height of 205 m, with temporary stability guys at three levels. On top of this was a steel transition ring, which doubled as a platform for the jacking operations carried out by the Swiss company, VSL, and which also formed the connections for the upper Kevlar guys.

Around the base of the tower, surrounding the central shaft, the 13 floor platforms were constructed. The floors were cast on steel permanent formwork like a normal building, albeit with 5.625 m storey heights. By early June 1991, the 74m-tall, 13-floor assembly was complete, giving a stable braced structure with a total floor area of 7150 m^2, weighing 2700 t. After a champagne launching ceremony, the jacks were charged and the floors jacked up by a nominal 25 mm, and left hanging for a weekend to check that nothing was amiss. Then, guided by wheels, the floors were lifted from the ground to their final resting place, 85 m higher up the shaft, at a maximum speed of about 4 m/h. For security during this operation, the lifting guide system was provided with clamps which were designed to grasp the shaft in the event of high winds – although, fortunately, these were not needed. The final connection to the shaft was made by site welding to the Y-splices at four levels. Quality control on site welding was rigorous, with one of the principal elements welded 10 times before acceptance.

During the early stages of construction, the shaft was stabilised by tiers of prestressed temporary steel guys, 90 m, 160 m, and 200 m above the foundation. The top two tiers remained in position during the lifting of the floor assembly, to provide a degree of rotational and translational restraint at the top of the shaft, which, in turn, reduced the slenderness of the shaft and prevented it from buckling over its nominal length of 210 m in the temporary state. After installation of the main steel and Kevlar guys, the telescopic upper mast was jacked upward, sliding inside the concrete shaft through phosphor bronze bearings, from the same platform used to lift the main floor assembly.

Completion

The full height of 288 m was reached in November 1991, and completion of the tower, support building, and infrastructure work, took place in March 1992. Installation of the broadcasting antennae and signal processing equipment was complete in time for transmission of the Olympic Games in July 1992. Visitors to the tower are now able to ride in one of the glass lifts to enjoy a magnificent view of the city of Barcelona and the Mediterranean sea from the newly-opened public viewing gallery, 135 m above the base. I highly recommend it.