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12. The National Westminster Tower, City of London (England)

Owner: National Westminster Bank Limited
Architect: R. Seifert and Partners
Engineer: Pell Frischmann and Partners
Contractor: John Mowlem and Co. Ltd.
Construction Dates: 1971-1980

Planning and Layout

The unconventional plan shape of the National Westminster Tower stems firstly from the restrictions imposed by the site shape and boundary conditions, and secondly from aesthetic and engineering considerations.

The initial triangular plan area of the main tower was first modified to form an irregular hexagon, quite a good shape for providing stability against lateral forces, and this was further refined by the introduction of re-entrants and the rounding of the corners, to produce the final plan shape.

This gave a core which, despite its contortions, approximated to a cylinder of 25 m diameter, a cylinder of course being the ideal geometrical shape for providing stability against lateral wind forces from all directions.

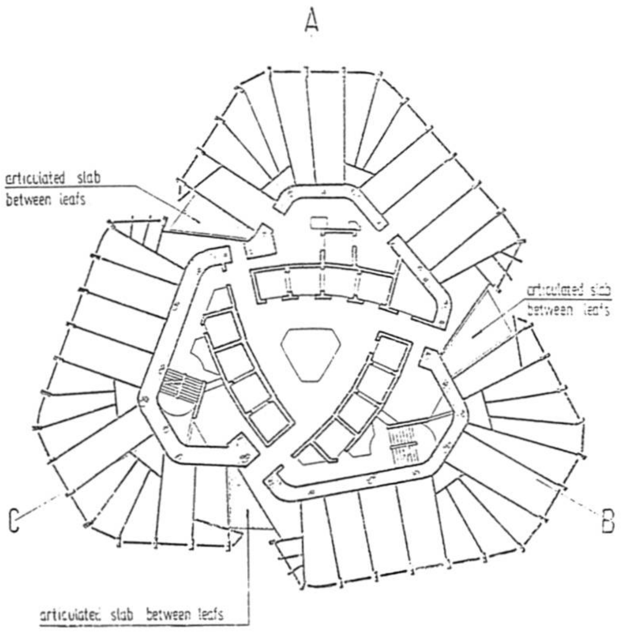
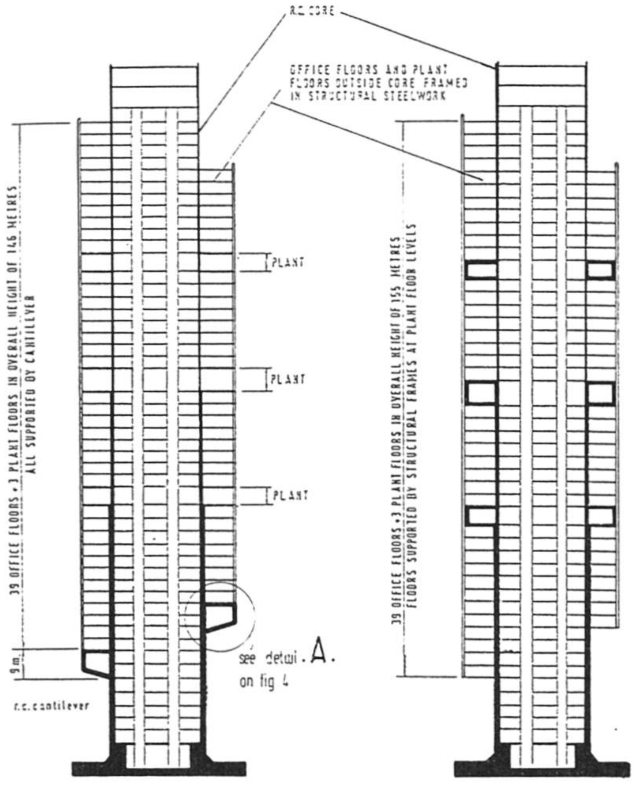


Fig. 1 Horizontal section through tower



FINAL SCHEME INITIAL SCHEME

Fig. 2 Vertical section through tower

The core was intended to house all lift shafts, staircases, toilets, ventilation shafts, services etc., and was therefore subject to numerous penetrations. Design attention was immediately focused on the determination of the stress distribution within this major three-dimensional structural element and concurrent with various wind studies being carried out at the National Maritime Institute, the construction and testing of a 1/50 scale perspex model was commissioned at City University.

Such studies revealed that the inner core walls contributed relatively little to the lateral stability of the core as a whole, and the major contribution was by the outer core walls acting alone, which despite the presence of major openings at five specific positions at every level, responded as a monolithic whole, and not as separate linked elements, the stress levels at and around the major openings being of acceptable values.

Structural Design

Whilst the core was always foreseen as being of concrete construction, the framework to the office floors surrounding the core was always envisaged as being in steelwork, for its strength to weight advantages over other materials, prefabrication and speed of erection. Initial structural considerations were devoted to taking advantage of the greater structural depth available at the levels of the proposed three intermediate plant floors, from which the office floors would be part supported and part suspended. Such an arrangement had the benefit that there would be no encroachment of the necessary structure into usable office space. However, subsequent development of both plant and structural requirements made it apparent that in these positions they were incompatible with one another, and it was resolved that the office floor superstructure should be structurally supported in a way completely unaffected by services considerations.

This led to the proposal to adopt a concrete cantilever at the base of each leaf of the office superstructure, this arrangement being preferable to any alternative scheme, such as suspending from the top since the construction of what was likely to be a difficult and highly critical structural element could be carried out at the lowest and most readily accessible level.

With a central concrete core resisting all lateral forces, the steel framework itself could be designed as a simple pinned framework, and full advantage was taken of this to design all joints as simple bearings, thus not only reducing steel fabrication costs, and erection difficulties to a minimum, but also permitting the occurrence of relative movements between the steelwork and the concrete core without causing distress in the steel framework.

The outer edges of the floor beams were supported on perimeter steel columns extending the full height of the superstructure and bearing directly onto the concrete cantilever. To keep within a previously agreed column width of 200 mm – in earlier schemes the columns had been either hangers or supporting a small number of floors – the lower lengths of the columns, where working loads were of the order of 650-700 t, were composed of 4 No. 50 mm thick H.Y.S. plates rivetted together to form a solid section 250 mm × 200 mm. At cantilever level, these were extended downwards 4.5 m into the concrete cantilever to transfer the load partly by bond and partly by end bearing.

To achieve as much repetition as possible and hence allow worthwhile production schedules for each steel element, only two sizes of floor beam were detailed and the columns were grouped according to their loads in multiples of 50-100 t.

Aids to Construction

The construction of the foundations, the lower parts of the central core, especially at the junction with the cantilevers, and the cantilevers themselves, presented a number of very complex problems, not the least of which was now to incorporate within the structure, intensities of reinforcement previously considered totally impracticable.

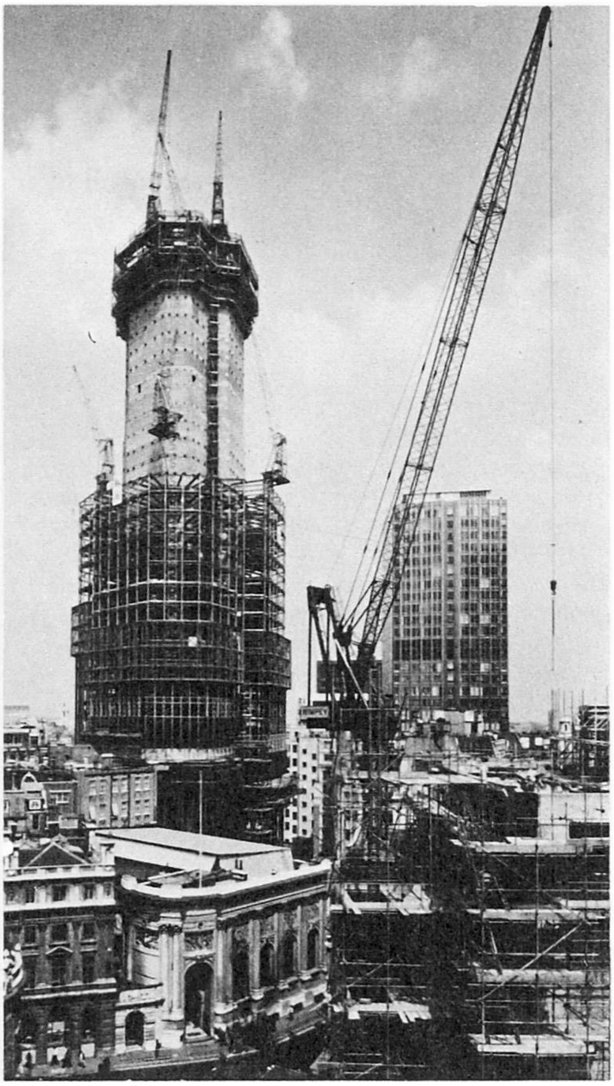


Fig. 3 Tower under construction

Detailing of such reinforcement in a three-dimensional form demanded the development of very specialised techniques, and the subsequent shaping of the material to the required standards, necessitated the design and manufacture of purpose-made machines.

Reinforcement congestion was reduced as far as possible by staggering splice positions where these were unavoidable, and the use of threaded couplers to join straight vertical bars, but there still remained the problem of placing and compacting high strength concrete.

Numerous trial mixes were prepared and tested, culminating in the use of a high slump, highly sanded, self compacting mix, which gave excellent results. All concrete material was mixed on site under strict control and supervision.

(Dennis C. Lippard)