The Spire of Dublin

Cormac P. Deavy
BE, CEng, MIStructE, FIEI, MICE
Arup

Andrew Allsop
MA(Cantab), MESc, CEng, MICE, FWES
Arup

Keith Jones
BEng, CEng, MIStructE
Arup

Keywords: The Spire of Dublin, Ireland Monuments, Design, Stainless steel, Vortex shedding, Damping, Erecting, Maintaining

Synopsis
The Spire of Dublin, described as the largest monument in the world, is 120m tall, 3m diameter at the base tapering to a point at the pinnacle, and is fabricated from shot-peened stainless steel. The simplicity of the structural form belies the complexity of the engineering design. There are many interesting features in the design of this slender monument including the approach to material selection, wind engineering and vortex shedding, damping, fatigue and construction. This paper describes the design and construction of the Spire.

Project background
The project originated in 1998 when Ian Ritchie Architects (with Arup as engineering advisers) won the open international design competition for the O'Connell Street Monument. The competition was organised by Dublin City Council (formerly Dublin Corporation) and administered by the Royal Institute of Architects of Ireland. The brief asked for a monument with 'a vertical emphasis, an elegant structure of 21st Century contemporary design'.

Ian Ritchie Architects' design was announced as the competition winner in December 1998, from an international field of 205 entries. Arup collaborated with Ian Ritchie Architects during the competition stage and undertook the detailed structural, geotechnical, services (mechanical, electrical and public health), and wind engineering design and supervised the on-site construction activities. The Flint and Neill Partnership acted as independent checkers on behalf of Arup during the detailed design phase. The specialist architectural and aviation lighting design was undertaken by others.

An injunction against the construction of the Spire was sought in April 1999 resulting in a High Court case. An Environmental Impact Assessment, carried out by McHugh Associates, ensued. Mr Noel Dempsey, the Minister for the Environment eventually gave the go-ahead for the project in December 2000. The design team was remobilised by June 2001. The project was tendered in November 2001 and construction on site and fabrication off site commenced in May 2002, with erection complete in January 2003.

The site
The site, 10m × 7m in plan, is located on O'Connell Street at the junction with Henry Street and North Earl Street (Fig 1) in the position formerly occupied by Nelson's Pillar. The 40.8m high monument to the memory of Nelson was erected in 1808, the foundation stone having been laid by the then Duke of Richmond. William Wilkins of Norwich designed it, but the statue of Nelson was by an Irish sculptor, Thomas Kirk.

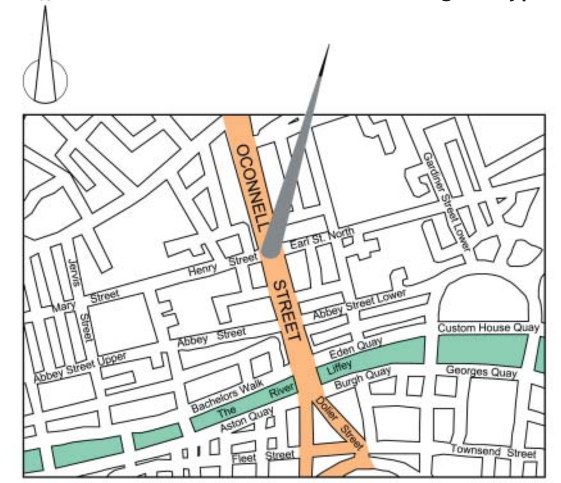


Fig 1. Keyplan

Nelson's Pillar was funded by public subscription and cost £6856. In 1966, it was blown up by the Irish Republican Army. The head of Nelson has been preserved by the Dublin Civic Museum.

Project brief
The project brief was to 'reinstate a monument which has a pivotal role in the composition of the street… The monument should be a new symbol and image of Dublin for the 21st century (such as, for example, the Eiffel Tower is for Paris and the Statue of Liberty is for New York)… The monument shall have a vertical emphasis, an elegant structure of 21st century contemporary design… The chosen materials shall be appropriate for a civic location, durable, adaptable to new technologies and require low maintenance in the future.'

Project description
Conceptually the structure is simple. The tip of the Spire is 120m above ground level. At the base it is 3.0m in diameter tapering to 150mm diameter at the tip (Fig 2). The Spire is fabricated from rolled stainless steel plate grade 1.4404 to BS EN 10088[1], (formerly known as AISI grade 316L). Generally the plate is 20mm thick although increased to 35mm for the bottom 4m and reduced to a minimum 10mm near the top. The plate is finished with a shot-peened finish for the height of the Spire as well as a mirror polish finish near the base.

The Spire was fabricated in eight sections (frusta), the longest being 20m in length. The lowest seven frusta are joined by internal bolted flange connections. The two uppermost frusta are connected by a threaded connection. Two tuned mass dampers (TMDs) are installed in the fifth frustum of the Spire.

The structure is founded on reinforced concrete piles and the basement chamber required a soil excavation of approximately 8m diameter by 5m depth. Piles were installed using specialist-drilling rigs to socket them into the underlying rock.

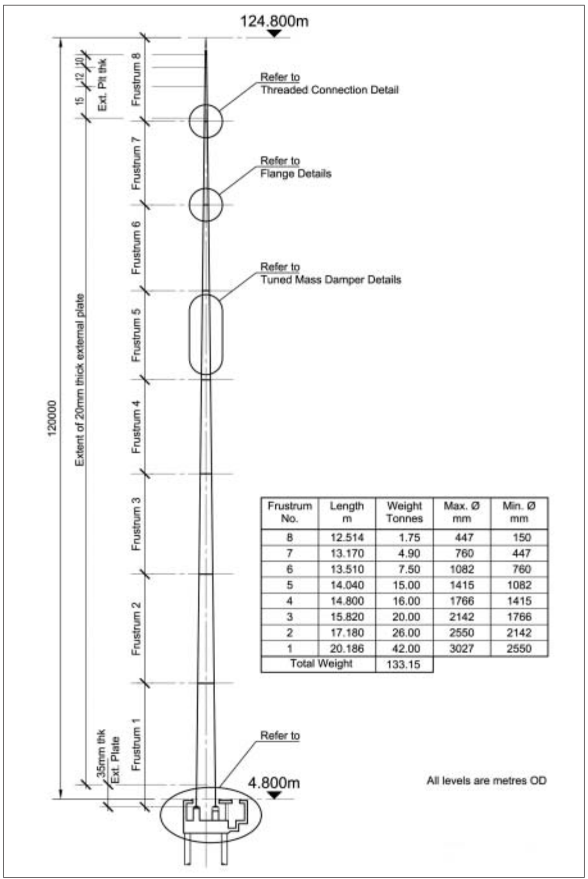
A 7m diameter circular base of bronze is laid at the base of the monument, this is flush with the surrounding paved area. There is an underground access and maintenance chamber to accommodate electrical and drainage equipment.

Material specification
At the competition stage many materials were considered for the construction of the Spire including aluminium, kevlar, timber, titanium, carbon steel and stainless steel. Material attributes such as durability, mechanical properties, ease of connectivity, surface finish, ease of maintenance and whether they were perceived to be reliable technology, were considered. Stainless steel was selected as the most appropriate material.

The term stainless steel is used to describe the family of corrosion resistant alloys that contain more than 10.5% chromium and a maximum of 1.2% carbon. Exposure to the air causes a thin, stable, chromium oxide film to form on the surface which provides the corrosion resistance. If the film is damaged by abrasion, it reforms and thereby maintains the corrosion resistance.

In practice there is a large number of stainless steels with differing degrees of strength and corrosion resistance, which are primarily dictated by chemical composition. They can be sub-divided into four principal groups, austenitic, ferritic, martensitic and duplex, designated by their microstructure. An austenitic stainless steel was chosen for the Spire. Austenitic stainless steels are based on 17-18% chromium and 8-11% nickel additions; the corrosion resistance can be further enhanced by the addition of up to 3% molybdenum. They are characterised by:

- excellent corrosion resistance to general or uniform corrosion;
- mechanical properties comparable to mild steel;



Frustrum No.	Length m	Weight Tonnes	Max. Ø mm	Min. Ø mm
8	12.514	1.75	447	150
7	13.170	4.90	760	447
6	13.510	7.50	1082	760
5	14.040	15.00	1415	1082
4	14.800	16.00	1766	1415
3	15.820	20.00	2142	1766
2	17.180	26.00	2550	2142
1	20.186	42.00	3027	2550
Total Weight		133.15		

Fig 2. Key elevation

- inherently tough, exhibiting good levels of toughness across a broad temperature range;
- high ductility making them amenable to cold forming;
- readily weldable when using conventional fabrication techniques.

The most commonly used austenitic grades in civil/structural applications are 1.4301 (304), 1.4307 (304L), 1.4401 (316) and 1.4404 (316L). Their mechanical properties are covered by current European specifications, such as BS EN 10088[1]. Grade 1.4404 (316L) was chosen for the Spire. The design was carried out in accordance with the principles of the SCI 123 guide[2] BS 5950[3] and BS 7608[4]. Arup consulted with TWI Cambridge to establish an appropriate fatigue (S–N) curve for the design of stainless steel.

A comprehensive structural stainless steel specification was created specifically for the project. This set the quality standards required at all stages, and covered all aspects of supply, fabrication and erection. This included the detailed specification of the base materials, welder qualifications, welding procedures, weld acceptance criteria, fatigue criteria, tolerances and erection.

Wind and vortex shedding

A '100 year return' wind event is likely to be exceeded only once in 100 years. The likelihood of this occurring is high over the lifetime of the structure. Therefore, an engineered structure is designed with safety factors intended to reduce the risk of failure to a several thousand-year return wind event. Hence it is not expected to fail in any windstorm short of one which would be classed as a major economic disaster.

Slender structures with a smooth surface are potentially subject to two principal wind responses: normal wind buffeting response and abnormal wind instability responses such as large amplitude vortex shedding movements, flutter and galloping. For the Spire, the circular cross-section negates flutter, which necessarily involves torsional movements, which change the aerodynamic angle of attack to the wind.

Lateral galloping responses are also not generally possible with a completely axially symmetrical section but over certain wind speed ranges galloping can be caused by small linear features e.g. a cable, fin or groove. The lack of such features and the taper of the cylinder were important to eliminate this risk. Smooth polished cylinders can also suffer from 'in-line' galloping at certain wind speeds. In this case also the taper was sufficient to eliminate the risk.

'Vortex shedding' is an instability of the flow pattern past a structure resulting in cyclic changes in the wind force. Large amplitude vortex shedding is a resonant response when the frequency of vortex shedding (characterised by the non-dimensional Strouhal number) coincides with one of the natural frequencies of the Spire. The first mode of vibration of the Spire has a theoretical frequency of 0.274Hz. The second, third and fourth modes of vibration have theoretical frequencies of 0.839Hz, 1.730Hz and 2.820Hz respectively (Fig 3).

The response to vortex shedding is a maximum at a critical wind speed, which depends on the aerodynamic behaviour of the cross section and the size and natural frequency of the structure. The aerodynamic behaviour of a cylinder also depends on windspeed and diameter through the Reynolds' Number. The Spire is thus subject to a range of different critical wind speeds due to the taper of the structure. The response is dependent on further aerodynamic factors such as surface roughness and wind turbulence. The response is potentially aeroelastic; that is to say, the motion of the structure affects the flow and is potentially very sensitive to the available amount of energy dissipation (damping) of the structure itself.

The resonant response to vortex shedding is greater in the cross-wind direction than the along wind direction and may be much greater. There are two current methods of calculating cross-wind response in accordance with the Eurocodes, the sinusoidal excitation method as formulated by Ruscheweyh, and the Vickery method[5, 6]. For both these methods, the response is dependent on the mass and damping through the non-dimensional Scruton number as shown in Fig 4.

Either method may be more conservative depending on the Scruton number. However, the sinusoidal excitation method does not predict the known sensitivity of the response to damping and aerodynamic properties. The Vickery method was therefore used to calculate the cross wind response to vortex shedding as it allowed a proper parametric investigation.

The Scruton Number is defined as:

Scruton Number = $4\pi m_z \beta / \rho D_z^2$

Where

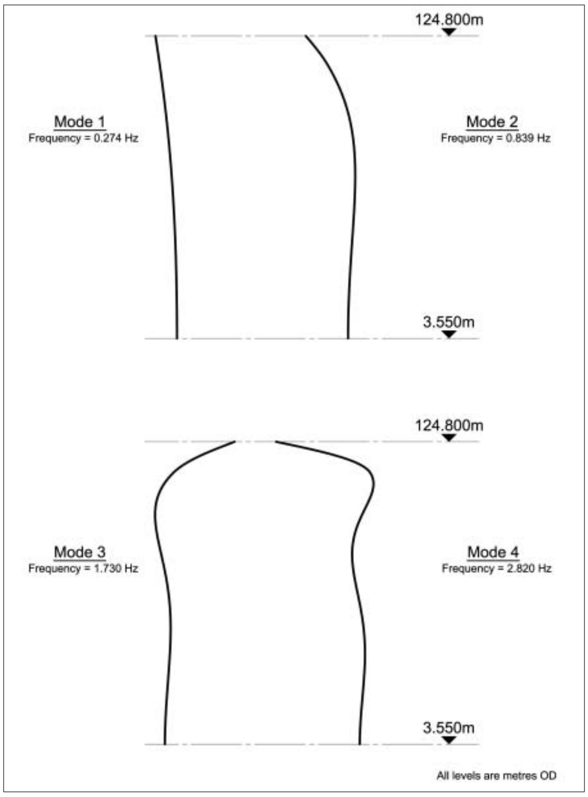


Fig 3. Natural frequency

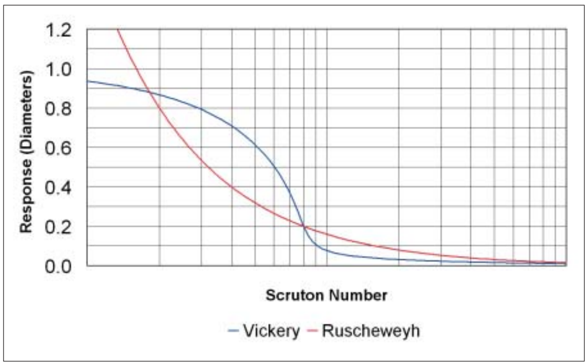


Fig 4.
Resonant response vs Scruton number

m_z = effective mass/length for vortex shedding referred to height z
β = damping in structure
ρ = density of air
D_z = diameter where vortex shedding frequency equals structure frequency.

For the Spire, only the mass and damping in the structure can be adjusted without changing the geometrical form.

The consequences of the vortex shedding and buffeting responses could manifest themselves in several ways. Overly visible motions could be upsetting to some people and large responses could be an immediate strength issue.

As a result of the wind engineering work, it was found that designing the structure to withstand extreme wind speeds alone would not be sufficient to prevent potentially severe vortex shedding induced fatigue damage. Vortex shedding occurs at lower wind speeds and hence more frequently than the extreme wind speed event. Four basic methods are available to improve a structure of this nature's performance under vortex shedding:

- Stiffen the structure such that vortex shedding occurs at a wind speed above those occurring at the site.
- Increase the thickness of the steel so that the peak stress range induced is below the fatigue threshold of the material.
- Increase the mass of the structure to reduce the dynamic response.
- Increase the damping in the structure to reduce the dynamic response.

The implications of implementing each of the four methods outlined above were evaluated so that the most appropriate solution could be chosen.

Damping

To properly consider the option of adding damping it was necessary to work up several possible schemes to a stage where they could be assessed. The following methods of adding damping were considered:

- constrained layer damping,
- tuned mass dampers (TMD),
- impact dampers,
- internal cables preloaded and anchored with springs and dashpots.

The possible solutions were assessed in terms of effectiveness, durability, initial and long-term cost, maintenance and risk. The chosen solution was to add two tuned mass dampers to suppress the dynamic response of the first two modes of the structure.

The introduction of tuned mass dampers meant that there would be no fatigue issues at the bottom of the Spire. It was much less easy to dismiss the risk of fatigue due to higher frequencies of vibration towards the tapering top.

Vortex shedding right at the top of the Spire could be reduced or eliminated by introducing some aerodynamic porosity (holes). This fitted in nicely with the scheme for architectural lighting but did not entirely eliminate contributions from modes 3 and 4, especially since the holes themselves increase stress concentrations thus reducing the fatigue life. It was necessary to assess whether the inherent damping of the stainless steel plate alone was sufficient to eliminate the vortex shedding from modes 3 and 4. A comprehensive literature search indicated that the minimum level of inherent damping for similar materials or structures could be as low as 0.05% of critical[6-20]. Arup therefore commissioned the University of Sheffield to test a sample of the stainless steel plate identical to that used for the Spire. On the basis of these test results a minimum level of inherent damping of 0.1% of critical was used in the design. This was sufficient to eliminate vortex shedding from modes 3 and 4.

Tuned mass dampers

To counter the effects of wind-induced vortex shedding for the lowest two modes of vibration, a tuned mass damping system was built in the Spire's fifth frustum (Fig 5). Designed by Arup and Motioneering, of Guelph, Ontario, the passive damping system consists of a mode 1 TMD and a mode 2 TMD. Each TMD consists of a stainless steel mass suspended on cables. Each mass is connected to the Spire internal surface, via four viscous damping units. Adjusting the length of the mode 1 TMD suspension cables tunes it to the frequency of the first mode of vibration. The mode 2 TMD is tuned similarly.

When tuned, the TMD oscillates 90° out of phase from the primary structure. The motion of the mass relative to the primary structure provides the opportunity to dissipate energy in the viscous damping units.

The location of the mode 1 and mode 2 TMDs was governed by the corresponding mode shapes of the Spire and the practicality of installing the required mass and components within the tapering Spire. The aim was to locate each TMD at the position of maximum displacement for that mode shape, thus maximising the ratio of effective mass to modal mass, and hence the amount of damping added to the structure. For mode 1, the maximum displacement is at the tip of the Spire, however it was not practical to locate the TMD there. The chosen position was, therefore, at a level of 77m above the base. For similar reasons, the mode 2 TMD was located at a level of 70m above the base.

TMDs are not static objects, therefore a zone of motion relative to the 'at rest' position of the TMDs was defined. At the centre of the Spire, it was necessary to continue the rainwater drainage pipe, cold water supply pipe and cable conduit past the TMDs, without encroaching on the zone of motion.

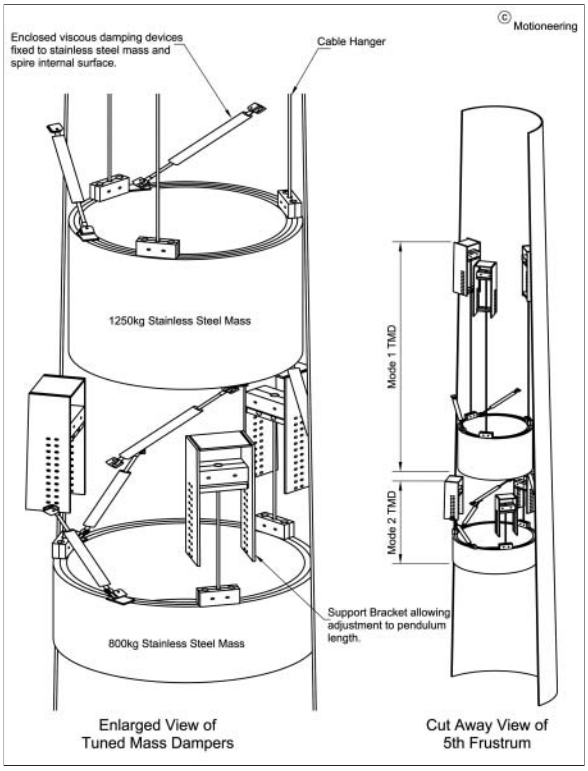


Fig 5.
Tuned mass dampers

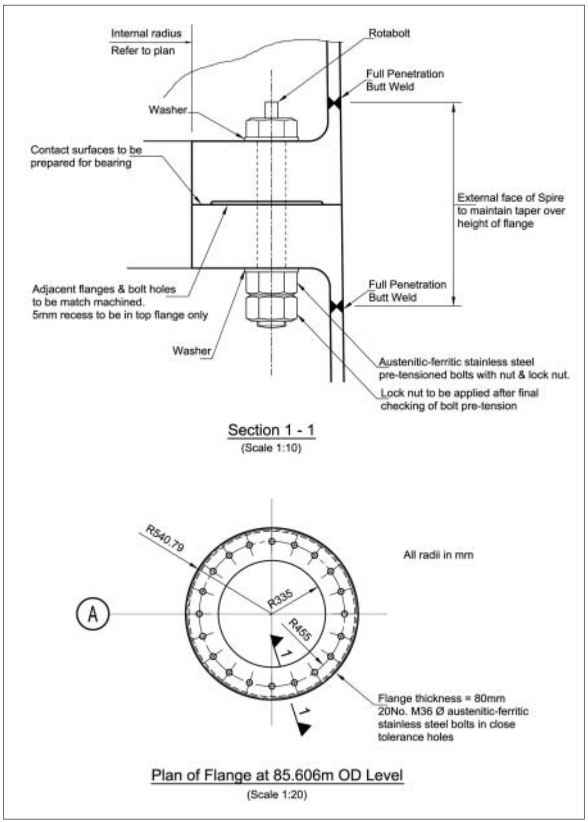


Fig 6.
Flange details

Similarly, the raise and lower system for the lighting rig had to be capable of passing through the TMDs. To enable maintenance access past the TMDs a lockout system was installed for each TMD.

The mode 1 and mode 2 TMDs have been designed to provide optimum damping for the range of critical wind speeds at which either mode 1 or mode 2 vortex induced oscillations will occur respectively. However, these critical wind speeds are relatively low, and at higher wind speeds the displacement of the Spire due to normal gusting wind response could cause the TMDs to displace and touch out on the internal surface of the Spire. Bumpers were therefore designed to prevent damage to either the TMDs or the Spire.

Connection of frus

Bolted flange connections

The architectural requirement for a smooth external surface to the Spire resulted in the design of internal bolted flanges, since on-site welding between adjoining frusta was not appropriate. The Spire structure is subject to stress reversal due to its oscillatory response under wind loading. The design and configuration of the bolted flange connections were governed by fatigue and the need to maximise the fatigue life due to wind-induced cyclic stresses. Guidance was taken from the CICIND *Model Code for Steel Chimneys*[21], and Bouwman[22]. A prestressed internal flange, with a recess in the upper flange, was developed (Fig 6). Prestressing the bolts in excess of the maximum design tensile stress in the shell means that the flange joint is always in compression (although varying levels of compression) and not subject to stress reversal. The fatigue life of the joint is further enhanced by the radiused internal angle and locating the circumferential weld away from the flange. The weld caps being ground flush reduced the local stress concentration factors associated with the weld toe.

Brück of Germany fabricated the flanges from stainless steel forgings. The forgings of maximum thickness 165mm and maximum diameter 2550mm have similar mechanical properties to the Spire shell austenitic stainless steel. The bolt holes in adjacent flanges were match reamed by Radley Engineering to ensure the accuracy of the close tolerance holes (–0.00mm/+0.15mm). The bolts vary from M36 to M60 over the height of the Spire. To avoid long term creep under pre-stress, austenitic-ferritic stainless steel was specified for the bolts.

Threaded connections

As the Spire tapers, the internal diameter reduces, thus restricting internal access. It was not therefore possible for the final connection (between frustum 7 and frustum 8) to be bolted similar to the lower flange connections. The combined length of frusta 7 and 8 is 26m. A threaded connection was therefore developed in order to facilitate handling in the workshop during fabrication and transport to site. The threaded connection consists of an internal flange section with upper and lower external threads which receives both frustum 7 and frustum 8, each having internal treaded ends (Fig 7). Brück, of Germany, manufactured the threaded connection. The threaded flange is subject to cyclic stress reversal and, therefore, a fatigue life calculation to BS 5400[23] and BS 7608[4] was carried out.

Perforations

The uppermost part of the Spire (frustum 8) contains architectural lighting. To allow the light to be seen, the Spire plate is perforated with a large number of small diameter holes.

The perforations have a dual purpose, as they also mitigate the effects of vortex induced oscillation. Guidance was taken from Walshe and Wooten[24] on the hole density specified. A total of 11 692, 15mm diameter holes were hand drilled in frustum 8. The perforations cause stress concentrations, which were found to be particularly significant in terms of fatigue design. Figure 6(a) in BS 7608 gives guidance on the stress concentration factors for a single hole in a plate. A finite element model was used to calculate the compound effect of multiple holes on the stress concentration factor. After consultation with Professor Burdekin of UMIST, a design methodology using an increased stress concentration factor and the fatigue (S-N) curve established with TWI Cambridge, was developed.

Holding-down bolt design

Cyclic loading, due to wind effects, dictated that the Spire holding down bolts needed to be preloaded. The Spire is prestressed to its concrete foundation with a 350kN tensile holding-down force in each of the 48 M60 bolts. Benefits of using preloaded holding down bolts include[25]:

- Consistent load paths at, and above, the grout interface result in (a) the transmission of shear by shear friction and (b) a reduction in the compressive bearing stress on the grout due to bending.
- Avoiding separation at the foundation interface, which can occur under tensile loads, such that when the cycle reverses the grout is subjected to an impact force as the crack closes.

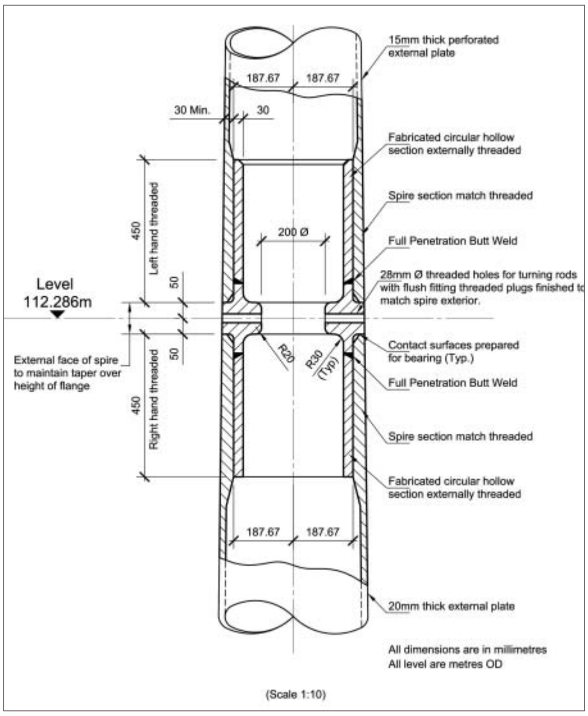


Fig 7.
Threaded connection

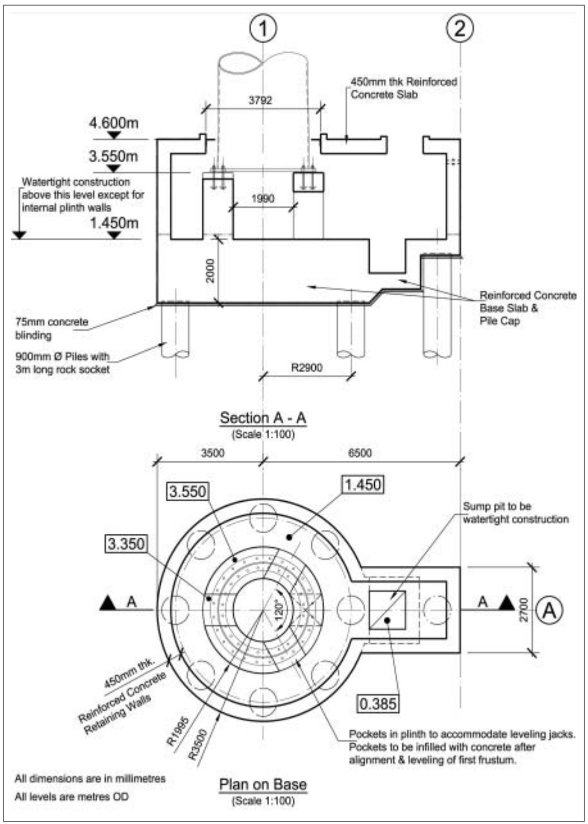


Fig 8. Substructure

This cyclic impact process can, over a period of time, cause 'grinding' of the grout.
- Reducing the range of the stress cycles experienced by the bolts due to wind loads on the Spire. This significantly improves the fatigue performance of the bolts.

Substructure design

The substructure consists of a reinforced concrete chamber 7m in diameter and 5m deep. Within the chamber is a reinforced concrete annular plinth, 1m thick, 4m diameter, to which the Spire is bolted (Fig 8).

The design life of the substructure is commensurate with the Spire superstructure. Retaining walls are designed to resist aggressiveness of ground, carbonation-induced corrosion of reinforcement and chloride-induced corrosion. To counter these effects the design included:

- high quality concrete grade C60X (low water/cement ratio; sulphate resistant portland cement;
- stainless steel reinforcement;
- 75mm cover;
- high standard of workmanship (development of procedures to ensure compaction, achievement of cover);
- Waterproof tanking.

Geotechnical engineering

The various layers of the geological build-up of the site is given in the following table:

Depth below ground level	Description
0 to 3.2m	made ground - slab, hardcore,old masonry walls
3.2 to 5.7m	silty sandy GRAVEL with cobbles (water bearing)
5.7 to 11.7m	very stiff to hard sandy gravelly CLAY (black boulder clay) with cobbles and occasional boulder
> 11.7m	Moderately weak to moderately strong LIMESTONE, MUDSTONE and SHALES

The imposed loads from the earth and groundwater pressures were based on the soil profile outlined above. Groundwater at approximately 4.5m below ground level (BGL).

The Spire foundation is a simple reinforced concrete disc 7m in diameter and 2m thick supported on a ring of reinforced concrete piles. It is founded 7m below ground level, at the base of the chamber previously described. The tension in the piles due to the overturning moment on the Spire is resisted through friction in a 3m rock socket.

Fabrication

A detailed fabrication methodology was developed early in the design to outline, step-by-step, how the structure could be fabricated. The methodology was discussed with fabrication specialists to understand the issues that one could expect to encounter during fabrication.

The basic method is that each stainless steel plate was first cut to the required developed conical shape then rolled to form a half 'can'. Two sections were then welded longitudinally to form a complete 'can'. Each 'can' was then welded circumferentially onto the adjacent section until a frustum was formed.

Care was taken during all stages of fabrication and erection to prevent contamination of the stainless steel surface by carbon steel or iron, which would corrode once exposed to the atmosphere thus giving the appearance that the stainless steel is staining. The stainless steel was protected during fabrication using appropriate adhesive protective films.

To ensure that the full corrosion resistance of the stainless steel was maximised careful attention to detailing and fabrication was required by:

- Avoidance of dirt entrapment by specifying smooth finishes.
- Avoidance of crevices by using pre-stressed bolted connections and using closing welds, dressing/profiling welds.
- Reduction in the likelihood of pitting by removing weld splatter, pickling stainless steel to remove unwanted welding products and avoiding pick-up of carbon steel particles by using workshop areas and tools dedicated to stainless steel.

Erection

The erection of the Spire was akin to the structure: conceptually simple, but technically challenging. Conceptually, each frustum was delivered to site on the horizontal, raised to the vertical using two cranes (topped and tailed) and then lifted by a single crane onto the previous section, at which point the internal flange bolts were tightened.

Among many technical challenges to be overcome were:

- The alignment of the 48 M60 holding down bolts to receive the 20t first section.

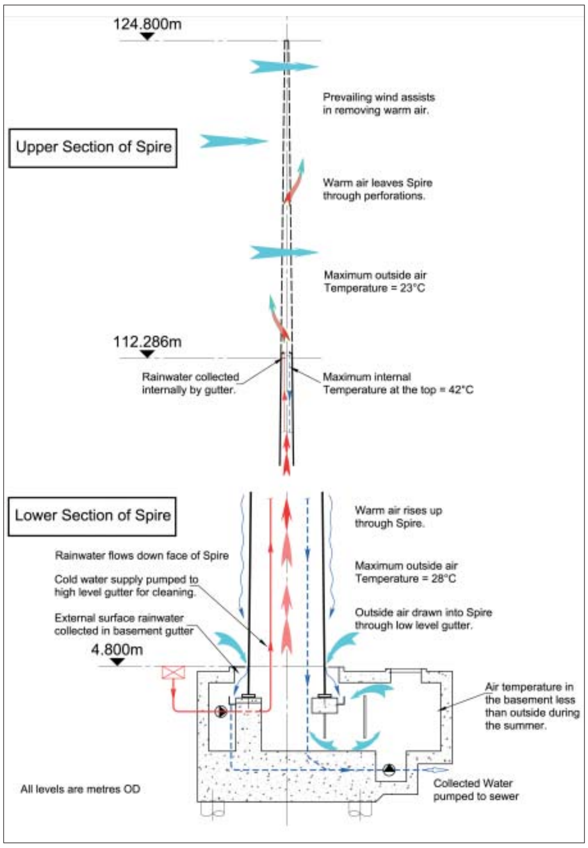


Fig 9. Services

- The levelling of the first section and subsequent grouting of the base plate to the concrete plinth.
- The safe erection of each frustum, particularly as the height increased, and internal diameter decreased.
- The temporary stability of the partially completed Spire prior to the final erection and releasing of the TMDs.

A stainless steel template was cast into the concrete to guarantee the alignment of the 48 holding down bolts. This was trial assembled with the first frustum of the Spire in the fabrication yard, prior to placing on site.

The levelling of the Spire was carried out with three jacks. Once level, the sections of baseplate between the jacks were grouted and allowed to cure before the jacks were released. The jacking recesses were then grouted.

The safe erection of each frustum was achieved by the careful planning, skill and experience of GDW, combined with McNally Crane Hire, under the supervision of the Siac/Radley Joint Venture.

As each subsequent frustum of the Spire was erected, the natural frequency and response to wind loading of the partially complete Spire changed. Arup calculated each temporary analysis case to determine the natural frequencies and predict the wind speed at which vortex induced oscillations may occur.

Maintenance, monitoring and inspection

Maintenance

The Spire has a 120-year design life and like any building, routine maintenance is required. Wherever possible, those items requiring the most frequent maintenance are placed either in the service chamber or near the base of The Spire, e.g. electrics, sump pump.

The location of the TMDs was governed by design considerations and therefore access via an internal fall arrest ladder system is provided to the TMDs. This allows visual inspections and maintenance of the TMDs to be carried out.

The perforations at the top of the Spire mean that water ingress is possible. A drainage gutter is located at the top of frustum 7 to collect the majority of the water. Access to this drainage gutter is restricted by the diameter of the Spire. Therefore, a dedicated cold water supply pipe is used to flush the internal gutter drainage pipe at regular intervals (Fig 9).

Internal access to the architectural and aviation lighting at the top of the Spire is also restricted by the tapering diameter. These items are, therefore, installed with a raise and lower system, so that they can be lowered to the base of the Spire for maintenance. Ian Ritchie Architects, in conjunction with the contractor, developed the raise and lower system.

The raise and lower system consists of a winch near the base of the Spire and a cable reeler at the top. In the event of the cable reeler at the top of the Spire failing to operate, external access is required. Provision has been made in the design and construction of the Spire for man access to the top, without the use of large capacity cranes.

External access is gained via a rope system using a line of external anchor holes at 750mm centres up the Spire surface, which are capped off with a removable (threaded cap). These caps are removable from inside initially but are then removed externally at the upper levels where the taper precludes internal removal. Access to the first 20m of the Spire is gained via cherry picker or mobile access platform. This provides access for general cleaning required for the stainless steel of the Spire. A collapsible access platform can be erected by the rope access team at the top of the Spire to provide a safe working platform.

Monitoring

Monitoring of two components of the Spire; the flange bolts and the TMDs is provided at the request of the client. As described earlier, the fatigue design of the bolted flange connections utilises an applied prestress to the flange bolts. The flange bolts were modified to incorporate the Rotabolt, a proprietary pre-load control system. Standard bolts are modified to incorporate a gauge pin, drilled and tapped into the shank of the bolt. The gauge pin is toleranced such that there is an air gap between pin and bolt. As the bolt is tightened the air gap closes. When the specified prestress is applied to a bolt, the Rotabolt cap does not turn. However, should any loss in prestress occur, the cap becomes loose. Frequent monitoring of the flange bolts and reapplication of the flange bolts if necessary is therefore possible via the internal access ladder.

The TMDs are monitored using accelerometers on the Spire and each of the TMDs, and an anemometer on a nearby building. The data from the accelerometers allows the displacement and frequency of the Spire and the TMDs to be calculated. The anemometer measures the wind speed and direction. From this data, the damping of the TMDs and the Spire can be calculated and monitored. Should any change in either the Spire or TMD properties occur the monitoring system notifies the client.

The TMDs consist of a number of small components. Internal access has been provided to each of the components for inspection, replacement and recommissioning if necessary. Replacement of each of the components can be carried out in a prescribed sequential method without detrimental effect to the performance of the structure.

Acknowledgments

Structural Engineer: Arup; *Services Engineer*: Arup; *Client*: Dublin City Council; *Architect*: Ian Ritchie Architects; *Quantity Surveyor*: Davis Langdon Everest; *Planning Supervisor (design)*: Arup Consulting Engineers; *Contractor*: Siac Radley Joint Venture; *Site Investigation Contractor*: Irish Drilling Limited

REFERENCES

1. BS EN 10088-1:1995 *Stainless Steels – Part 1 List of Stainless Steels*
2. SCI Publication 123 *Concise Guide to the Structural Design of Stainless Steel* (Second Edition)
3. BS 5950: *2000 – Structural Use of Steelwork in Building*
4. BS 7608: *1993 – Code of Practice for Fatigue Design and Assessment of Steel Structures*
5. EN1991-1-4: 2005 *Eurocode 1: Actions on structures, General actions, Part 1-4: Wind actions* (draft)
6. Pagnini, L. C. and Solari, G.: 'Damping measurements of steel poles and tubular towers', *Engineering Structures*, 23, 2001
7. Ruscheweyh, H., Langer, W., Verwiebe, C.: 'Long-term full-scale measurements of wind induced vibrations of steel stacks', *Wind Engineering and Industrial Aerodynamics*, 74-76, 1998
8. Ruschewyh, Hans, Galemann, Thomas: 'Full-scale measurements of wind-induced oscillations of chimneys, *Wind Engineering and Industrial Aerodynamics*, 65, 1996
9. Ricciardelli, Francesco: 'On the amount of tuned mass to be added for the reduction of the shedding-induced response of chimneys', *Wind Engineering and Industrial Aerodynamics,* 89, 2001
10. Verwiebe, Constantin., Berger, Georg W.: 'Gemessene Dampfungsdekremente von Stahschornsteinen und deren Bewertung im Hinblick auf die Bauart', *Stahlbau,* 68, 1999
11. Johnst, D. J., Britton, J. and Stoppard, G.: 'On increasing the structural damping of a steel chimney', *Earthquake Engineering and Structural Dynamics* , **1**, 93-100, 1972
12. Yam, L. H., Leung, T. P., Li, D. B. and Xue, K. Z.: 'Use of ambient response measurements to determine dynamic characteristics of slender structures', *Engineering Structures*, **19**/2 p 145-150, 1997
13. Robertson, A. P., Hoxey, R. P., Short, J. L., Burgess, L. R., Smith, B. W. and Ko, R. H. Y.: 'Wind-induced fatigue loading of tubular steel lighting columns', *Wind & Structures,* **4**/2, 163-176, 2001
14. Von Hirsch, G.: 'Aktive und passive Kontrolle dynamischer Verformungen von schlanken Strukturen unter Windbelastung', *Konstruktiver ingenieurbau – berichte,* 35/36
15. Tilly, G. P., Cullington, D. W., Eyre, R.: 'Dynamic behaviour of footbridges', *IABSE Periodica 2*, 1984
16. Bachmann, H. and Ammannm W.: *Variations in structures induced by man and machines*, IABSE, 1987
17. 'Vibration problems in structures', *Bulletin D'Information,* No. 209, 1991
18. Barltrop, N. D. P. and Adams, A.J.: *Dynamics of fixed marine structures,* 3rd edition, Butterworth – Heinemann, 1991
19. Grootenhuis, P.: *Damping mechanisms in structures and some applications of the latest techniques*, Imperial College London, 1972
20. Lazan, B. J.: *Damping of materials in structural mechanics*, Pergamon Press, 1968
21. *Model Code for Steel Chimneys*, May 1988, CICIND
22. Bouwman, E. P.: 'Bolted connections dynamically loaded in tension', *Proc. ASCE, J. Structural Division,* ST9, 1982
23. BS 5400 : *1982 – Steel, Concrete and Composite Bridges, Part 10 Code of Practice for Fatigue*
24. Walshe, D. E. and Wootton, L. R.: 'Preventing wind-induced oscillations of structures of circular section', *Proc. Inst. Civ. Eng.,* 47 (1970) 1-24
25. Bloomer, D. A.: 'Stainless steel ventilation stack: WEP Sellafield', *The Structural Engineer*, **71**/16, 17 August 1993