



CONSTRUCTION OF THE SHARD OF GLASS

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ABSTRACT

Described by some as the new icon for London, and by others as a “shard through the heart of historic London”, The Shard has had a high profile start to life. But regardless of your point of view on its place on the London skyline, it must be seen as an engineering triumph. The tallest building in Western Europe, constructed in the heart of a congested city area, constrained by sensitive, historic structures in the midst of a global financial crisis, is nothing short of outstanding. Robert Bird Group provided construction engineering services for the project covering primarily the jumpstart construction sequence and the crane strategy.

INTRODUCTION

London Bridge Station handles over 300,000 passengers a day, is less than 1km from the city of London and has direct links to the financial hub of Canary Wharf. The case for a premium development in this location was clear. The Shard forms the centrepiece of the London Bridge Quarter Redevelopment, which will also include an extensive revamp of London Bridge Station, new office buildings, and a new bus station. Sellar Property Group appointed Renzo Piano to develop the concept for this vertical city.

The Shard stands at 310m tall making it the tallest building in Western Europe. The tower is made up of offices from level 2 to 28, restaurants from 31-33, the Shangri-la hotel from level 34-52, residential apartments from 53-65 and viewing galleries from level 68-72. It is built over three levels of basement which house plant, car parking and facilities management services. An additional 18 storey structure called the Backpack is located on the eastern elevation providing additional office space.

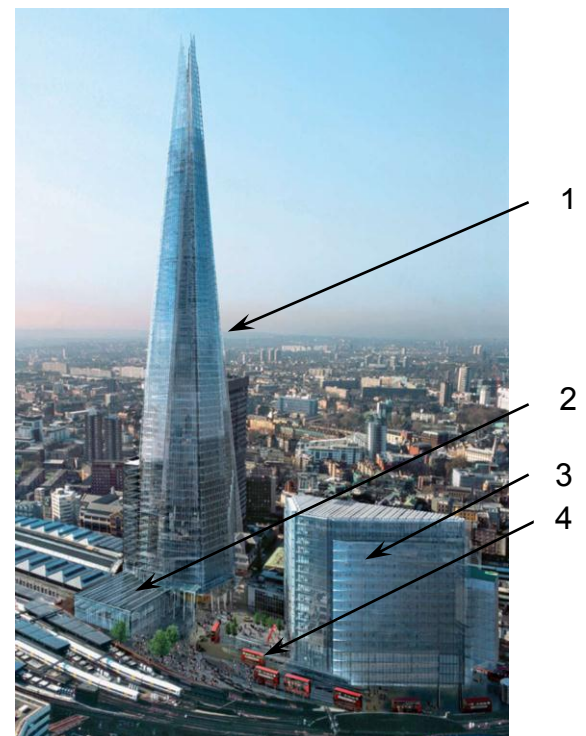


Figure 1 – Render of London Bridge Quarter
1. The Shard, 2. New London Bridge Station Roof,
3. The Place, 4. New Bus Station

Developer: Sellar
Architect : Renzo Piano Building Workshop
Associate Architect: Adamson Associates
Structural Engineer : WSP Group
Services Engineer: Arup
Project management: Turner Townsend

Main Contractor: Mace
Concrete Frame: Byrne Bros
Steelwork: Severfield Reeve Structures
Piling: Stent
Facades: Scheldebouw
Construction Engineer : Robert Bird Group

CONSTRAINTS

The site is heavily constrained by existing infrastructure which has had a profound effect on the form of the basement construction and the methodology that was adopted.

The site is located immediately adjacent to London Bridge Station, which was constructed in the 19th century on shallow founded masonry arches. Also surrounding the site are the Jubilee Line tube and escalator tunnels, disused tunnels and shafts, a heritage roof over London Bridge Station, and a cast iron water main. All of these assets were sensitive to the ground movements induced during the basement construction.

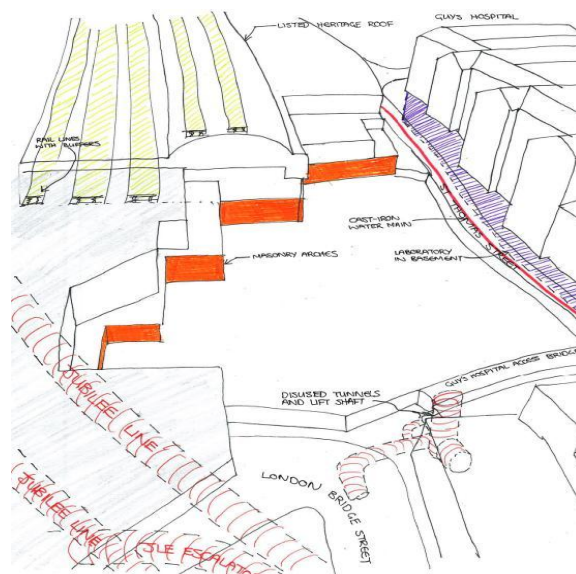


Figure 2 – Site Constraints

THE STRUCTURE

To achieve the tapering form of the structure and the architect's intent, a number of transfer structures occur throughout the building. Toward the base, perimeter column spacings are 6m reducing to 3m through the hotel and residential, and 1.5m through the spire. The larger column spacings and clear floor spans are within the office space up to level 40. Steel was used for these areas to provide larger column free space, and an integrated service zone of 700mm was used. Above level 40 the use changes from office to hotel and residential. Through these levels the structure changes to insitu reinforced concrete columns with post-tensioned flat slabs. The concrete construction provides better control of acoustics for this use and provides a smaller interstorey height. The change from steel to concrete also takes advantage of the additional weight in the superstructure and the better inherent damping qualities of concrete construction to improve the dynamic performance. This allowed the removal of viscous dampers that had originally been proposed. At level 72 the structure changes back to steel to achieve the lightweight, transparent spire.

As the structure changes use and the building tapers back, new perimeter column lines are introduced. This requires a series of transfer structures on the buildings perimeter. These are accommodated by multi-storey trusses (where these could be incorporated within plant stories) or three level vierendeel trusses where the views would be obstructed.

All lateral stability for the structure is provided by the central core. An outrigger system located in the plant room between floors 66-68, mobilises the perimeter columns to improve the dynamic performance of the structure. As the use of the structure changes and demand for lifts decreases, the core reduces in plan area. The primary core walls range from 450mm thickness to 800mm thickness.



Figure 3 – The Core

The shard is built over a three level basement, formed of external secant pile walls propped by the ground floor, basement 2 slab and basement 3 raft. The basement 1 level is a mezzanine that does not extend across the full basement. The raft is 1m deep though the majority of the basement and 3m deep below the core. The Shard uses a pile assisted raft, founded on large diameter bored piles that seat 50m below ground level in the Thanet sands.



Figure 4– The slipform rising through the steelwork.

CONSTRUCTION METHODOLOGY

A number of innovations were brought to the construction methodology to overcome the challenging site constraints, and to address the complexities of the superstructure construction. These innovations included a jumpstart methodology, construction of the raft in a single pour, and a recovery crane strategy.

JUMP START METHODOLOGY

The risk of ground movement related damage to existing structures, and the risk of delays due to in-ground archaeology and demolition, forced a rethink of the construction methodology. Byrne Brothers and Robert Bird Group developed the concept of the jump-start sequence for the construction. The jump start refers to constructing the basement in a top down fashion, but also commencing the core construction prior to completing the foundation raft.

The jump start sequence delivered a number of benefits including:

- Providing a rigid prop to the head of the secant wall, significantly reducing the risk of ground movements when compared to conventional flying shores.
- Providing a larger logistics platform at street level.
- Enabling an early start to erection of superstructure steelwork.
- Providing suitable elevated bases to allow erection of the tower cranes.

Top down construction refers to construction with embedded retaining walls, where the slabs are constructed from the top down supported on steel plunge columns and excavation is undertaken by moling below the slab. This is in contrast to conventional methods where the walls are propped by temporary works during excavation, following which the slabs are built from the bottom up.

Plunge columns are steel columns that are plunged into the wet concrete of the bored pile and used to provide vertical support to the suspended slabs. The Shard's plunge columns were installed in 1800mm diameter rotary bored piles. The piling platform was at ground floor level and the pile cutoff was about 15m below ground level. In order to install the plunge columns to tight tolerances (1:400 verticality, +/- 10mm position) a hydraulic guide frame with laser alignment was used. The guide frame was lowered into the pile casing after the reinforcement cage was installed. Following this the piles were concreted up to cut-off level. The plunge column was then installed in the guide frame, aligned, and plunged into the concrete. The strength gain and workability of the concrete has to be balanced with the time to install the plunge column, otherwise refusal can occur prior to achieving the required bond length. The bore hole between platform level and pile cut off is then backfilled to maintain the stability of the ground when the pile casing is removed. For most plunge columns they were backfilled with pea gravel, however for heavily loaded core plunge columns foam concrete fill was used to enable a greater degree of restraint to lateral buckling of the plunge columns.

The plunge columns had to be incorporated within the thickness of the core walls, whilst avoiding existing piles within the core footprint. The core walls internally were 450mm and externally were 800mm. This restricted the plunge columns to a maximum width of 350mm and 600mm respectively to leave space for confining reinforcement and tolerances.

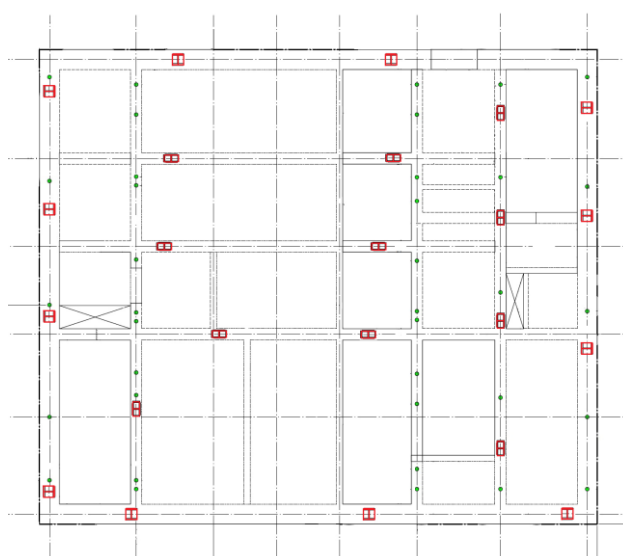
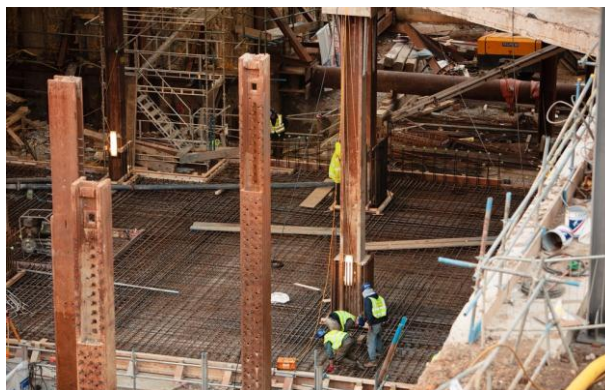


Figure 5 – Core Plunge Column Layout

Load transfer from the plunge column to the pile is achieved by friction between the steel and the concrete. Maximum bond lengths of 5metres were used in most cases and additional fin plates were required on many of the columns to increase the bond area. Shear studs, distributed over an 8m length of the plunge columns, were selected as the most appropriate method for transferring load from the core to the plunge columns. Shear studs provided a ductile failure mechanism that would give significant cracking and signs of distress prior to failure, and the load transfer is spread over a larger length of the structure. Openings in the core wall adjacent to the plunge columns were cast solid, and broken out once the lower core walls were completed.



*Figure 6 – Shear studs on core plunge columns.
B2 slab reo being fixed.*

THE SEQUENCE

The initial strategy for the jump start involved launching the core from the ground floor level. The preliminary design indicated that the core plunge columns would be capable of supporting the superstructure to a maximum height of about level 30. A detailed programming exercise showed that with commencing the core from ground level, superstructure construction would reach level 30 and have to stop well before the basement raft was complete. This was due to the relatively slow excavation progress from the moling operations. Therefore the jumpstart scheme's programme savings were unlikely to be fully realised.



Figure 7 – Excavation to B2. Ground floor ring beam is evident.

An alternative scheme was then developed whereby the ground floor was partially constructed in a ring beam, leaving a central hole for the core to rise through. This propped the retaining walls and enabled open-cut excavation to level B2, which was quicker than the moling process. Once at level B2, the slab was cast with a grillage of beams linking the core plunge columns together. From there, the slipform commenced and the moling operation was limited to a single level of basement excavation. This enabled a faster completion of the raft, thereby preventing the need for stopping the superstructure construction at the hold point.

Each stage of the superstructure construction is limited by the amount of load induced in the plunge columns and the slenderness of the column. Overturning moments from wind and out-of-balance superstructure loads are resisted by axial load in the plunge columns. Base shear is resisted by the B2 and GF slabs bearing against the retaining walls so the plunge columns are braced. The critical stage for the plunge column design was when the raft excavation was completed, leaving a large unrestrained length of plunge column between underside of B2 and the pile cut off level. The plunge column design also considered lateral loads induced in the plunge columns from ground movements of the basement box and accidental impact loads from the excavation equipment. Once the basement raft was cast, it provided additional rotational restraint to the plunge columns, reducing the slenderness and allowing construction to proceed until the limiting criteria of the load transfer between the core and plunge columns was reached.

The effective length of the plunge columns was analysed taking into account, the rotational fixity provided by the length of plunge column cast into the core wall, the rotational fixity provided by the connection to the pile, and the stiffness of the surrounding London clays. Options of bracing the plunge columns to limit the effective length were discarded due to the number of programme hold points this introduced, the obstructions to the excavation and the relative stiffness of the restraints compared to the plunge columns.

THE SLIPFORM GRILLAGE

In order for the slipform to commence from the level B2 slab it was necessary to provide a structure to support the slipform. A grillage of beams was cast on the core wall lines at the underside of B2 level. This supported the wet weight of the concrete and the jacking loads from the slipform until the walls were capable of arching direct to the plunge columns. This grillage also provided a suitable clearance to the underside of level B2, to allow the casting of the final section of core wall from B3-B2 with a letter-box pour.

The jumpstart method achieved a 4 month programme saving over the baseline construction method, and offered a significant reduction in risk of delays because the superstructure could proceed unhindered.

CRANES:

The site posed a number of challenges to the crane strategy. The top down construction sequence meant that the basement slab would not be in place for a significant period of the construction, hence conventional piled or gravity crane bases were not suitable. Steel grillages supported by the secant piles and plunge columns were adopted for three of the external climbing crane bases. As the core was supported on plunge columns during the jumpstart, it was decided not to tie the tower cranes back to the core. The cranes were erecting freestanding to a height of 66m which enabled the core construction to be completed prior to tying and climbing the cranes.

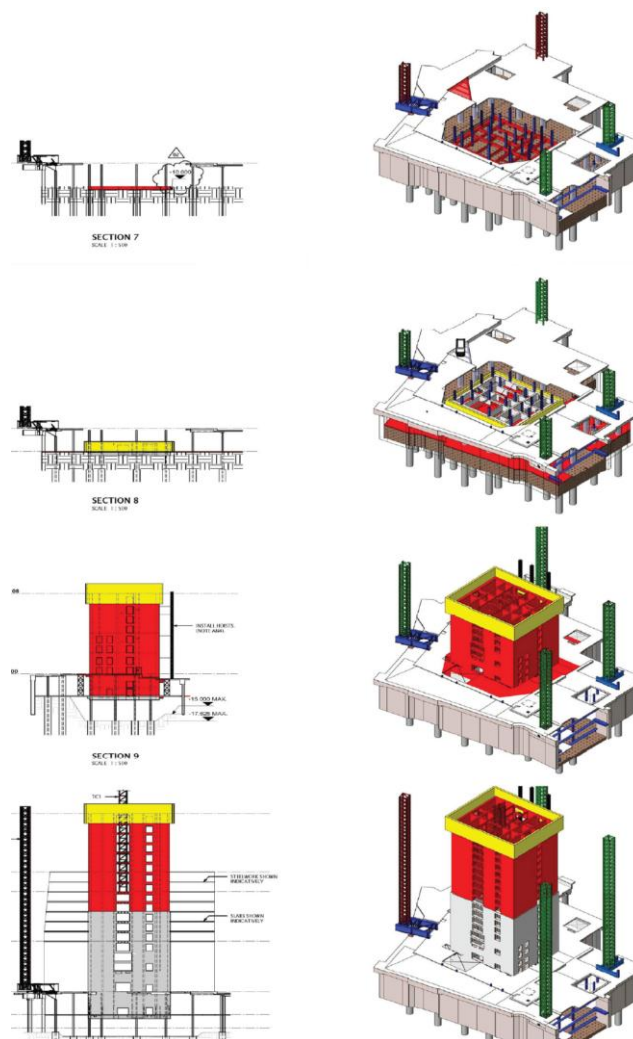


Figure 8 – Extracts from the jumpstart Sequence

1. Plunge columns installed, ground floor ring beam constructed, excavation to B2
2. Construct B2 and slipform launch grillage, construct slipform
3. Commence core construction, infill remaining ground floor slab, excavate to formation
4. Continue superstructure construction, prepare B3 raft
5. Construct Raft and infill the B3-B2 wall (not shown)

An increase to standard crane base deflection limits was negotiated with the crane manufacturer to ensure the stability of the cranes while still keeping the bases within economical limits.



Figure 9 – TC2 and TC5 Crane Bases

CRANE TIES

The steelwork erection worked by fast-track methods. The steel was erected in quadrants two floors at a time by riggers working from deck-riders. Following the steel erection were the decking and stud welding operations. Two floors below this (an exclusion zone for safety from dropped objects and welding), was the placing of reinforcement and concrete. This lag between erecting steel and placing concrete provided an opportunity for final alignment of the steel, full bolting and shimming of connections, and defect rectification. However it also posed a challenge for crane operations because the nearest full strength diaphragm was 20-30m below the leading steelwork. This lag prevented the cranes being tied to the slab edge. Bracing the steel floorplates in plan and tying to the steel frame was avoided due to the lateral deflections of the floorplates, and the difficulties this would cause during setout of the perimeter steel.

These issues lead to the development of ties that connect direct back to the core. These ties, were up to 30m long, and called the “Mega-Ties”. They were formed into a truss in the horizontal plane, and in the vertical plane they were supported from the steelwork. These Mega-Ties were replaced with a conventional tie to the slab edge when the concrete diaphragm had achieved its design strength. The Mega-ties could then be relocated to the next level.



Figure 10 – Mega-Ties on TC2 and TC3

INTERNAL CLIMBING CRANE TC1

Tower Crane One was one of the most challenging installations on the site. Internal climbing within a core is always a complex operation with a number of challenging constraints to overcome including:

- Finding a suitable zone within the core with sufficient clearance for the tower and climbing mechanisms,
- Manoeuvring climbing equipment weighing several tonnes through the core without a crane hook,
- Managing personnel crushing hazards due to the crane moving within the core.

However, with high-rise cranes the most challenging issue is managing the wind-risk. Very-low wind speeds are required for crane climbing operations. Waiting for an appropriate window to climb the crane can stop core construction for weeks. The project team decided not to climb TC1, but to suspend it from the slipform and allow the slipform to drag the crane up the core. This removed 15 crane climbing operations from the programme and the associated wind-risk.

MacAlloy bars were used to suspend the crane from the upper grid of the slipform. An additional 8 22 tonne jacks were incorporated in the slipform design, immediately adjacent to the crane. The crane tower was supported in a skid system with adjustable rails to allow the crane to be guided up the core. The skids would bear against the core walls when resisting overturning moments.

The scheme posed a number of additional risks because faults with the slipform could potentially endanger the stability of the crane. Control measures were put in place to mitigate these risks including: using some of the most experienced slipform personnel in the UK, careful planning and mix designs, monitoring of the crane skid adjustment every 2-4 hours during operation, and a monitoring device to alert the rig operator if the crane skids became jammed in the core.



Figure 11 - TC1 within the slipform

CRANE RECOVERY

Recovering the cranes posed one of the greatest challenges on the site. In order to climb the crane down after construction, the crane would have to be positioned outside the building footprint. Due to the tapering facade, this would have required crane tie lengths in the order of 30m long, and the crane was so eccentric to the building that the torsions created would have required significant strengthening. As such, Robert Bird Group developed a recovery crane strategy to allow the removal of the crane from the building without the need for long ties.

Once construction was above level 40, post-tensioned concrete floors were used. The demand for hook-time was significantly less than with the steel floors so only two cranes were required to service construction. This consisted of the core crane, TC1, and TC5. The tapering geometry provided an opportunity to transfer to a crane supported on a grillage cantilevered from the permanent structure. As the facade tapered away from the crane, the base could be close to the building edge, yet the machinery deck of the crane was far enough from the structure to weathervane in storm winds without clashing. A CTL180 (with some modifications to reduce component weights) was used, supported on a steel grillage. The grillage was supported from the perimeter HSC columns and resultant lateral forces were transferred to the core by the adjacent floor slabs.



Figure 12 – TC7 Jib Removal

The fractures between shards at level 72 provided an ideal position for a recovery crane. However, as no cranes on the market had the required duties, a cantilevered temporary platform had to be constructed to allow the derrick to be positioned close enough to the pick-up point. A CDK 100-16 hydraulic derrick was used for the recovery.

The cantilevered platform was constructed in a modular arrangement to allow easier dismantling at height. A Palfinger knuckle boom crane was used to recover the Derrick and the cantilever platform. All components were then returned to ground level using the inclined construction hoists.

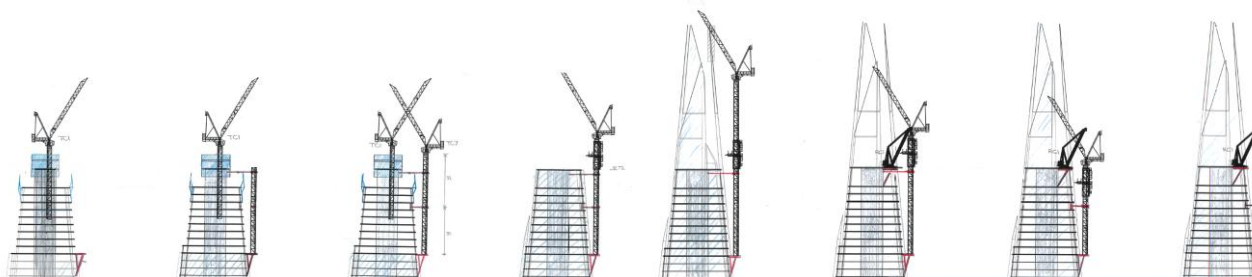


Figure 13 – Tower Crane Recovery Sequence. TC1 erects TC7 base, tower and tie to core, TC1 erects TC7, TC7 dismantles TC1, TC7 climbs to full height and completes construction, TC7 erects recovery derrick then climbs down to L75. L66 tie is connected, L72 tie is disconnected. TC7 climbs down to L72. Derrick removes TC7. Palfinger crane removes Derrick. Derrick and Palfinger components returned to ground level by the construction hoist.

CONCLUSION

The innovation in the architecture and structural engineering of the Shard was matched by the innovation brought to the construction methodology. Careful planning and site management allowed four months to be removed from the programme along with mitigating potential delay risks with the excavation. The jumpstart methodology for the Shard won the Institute of Civil Engineers (ICE) London awards for 2010.

Jump starting is a suitable method for many sites with deep basements. Several other projects using this methodology have since begun in London.

Recovery crane strategies can provide a cost-effective and safe means of removing cranes from high-rise structures. As more complex sites are developed and increasingly complex building profiles are constructed, recovery cranes will become more of a necessity in the UK than an option.



Figure 14 – Tower Crane 7

ACKNOWLEDGEMENTS

The author would like to thank all those who assisted in the design and construction of these works with special mention to those below.

Mace: Tony Palgrave, Adrian Thomson

Byrne Brothers: Donald Houston

Robert Bird Group: David Seel, Guy Wood

Figure 15 – The New London Skyline

