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INTRODUCTION AND HISTORY

History of 10 Trinity Square

When the Port of London Authority was formed in 1900, Britain was the foremost maritime nation in the world. A prestigious building was needed to house the new authority and a site was selected overlooking the River Thames at Tower Hill, in the southeastern corner of the City of London. The architectural competition was won by Sir Edwin Cooper; construction was started by John Mowlen & Co in 1910 and completed in 1922. 10 Trinity Square was opened by David Lloyd George, the then British Prime Minister, in 1922. The building was damaged by German bombing during the Blitz in World War II with the original central rotunda being destroyed. The building is currently listed grade II* (particularly important buildings of more than special interest). One of the first meetings of the UN was held in the board room of the tower building and a more recent claim to fame was the buildings use as a film location for the Bond movie ‘Skyfall’.

Background and scope of the project

In 2008, the vacant building was purchased by the partnership of KOP Group and Reignwood Properties, both large multi-national property developers. In 2010, Reignwood acquired sole ownership of the property and appointed a design team to develop the scheme. The plan was to create a 5 star hotel with a number of private luxury apartments in the heart of London. The client’s scheme was taken to an advanced scheme stage before it was decided to open negotiations with a contractor for the construction stage.

The project team

Donban Contracting UK were appointed on a ‘design and build’ contract in the latter part of 2012. Barrett Mahony were engaged as structural and civil engineers by Donban. There was a short lead in time before the contractor started on site and this period was used to review the original scheme drawings and develop the detailed design to reflect the clients brief and to fit with Donban’s programme and budget. This involved a complete re-appraisal of the original scheme to reflect the proposed programme and internal space requirements. The design team and ourselves worked closely with Donban from the outset in order to progress the design as quickly as possible while preparatory works on site were at an early stage. TP Bennett were appointed as Donban’s architect to develop the original scheme and Malachy Walsh were appointed as M+E consultants.

The client’s vision was to convert and extend the original building from its previous office use into a five star 120 bed hotel with 38 private apartments on the upper levels. This would need over 19,500 m² of new floor area which would be located as new build within the existing central courtyard and as a vertical extension over the footprint of the original building. In order to achieve the internal column lay-outs required, the scheme involved the removal of the existing 5th floor and roof (which were a later addition in steel and precast concrete and not part of the original 1920’s structure) and the additional of 5 new stories, taking the original building from 4th floor to 8th floor level.
The central courtyard was to be in-filled with a new structure up to first floor. This new structure infill’s the courtyard to 1st floor and then continues up to 3rd floor level as a block around the perimeter of the courtyard. There is a separate central rotunda structure built up to level 3.

A new basement constructed within the courtyard, extending down to match the level of the existing basement areas below the original building. Finally, a new curved roof was to be added over the existing building and the new bays constructed around the perimeter of the courtyard. The new structure above 4th floor is constructed with a mixture of off-site elements and steel framed construction.

Separately, but under the same contract a new basement was to be formed below the original public garden located between Seething Lane and the existing building. This facility would provide both an automatic car staking system and a delivery vehicle loading bay.

Initial works and Foundations

Before the main construction works could start preparatory work including a detailed topographical survey of the existing building, a UXB survey and a ground investigation were completed. In addition, the first stage of an extensive archaeological dig that would progress in parallel with the phases of courtyard excavation was also undertaken.
Site investigation and ground conditions

The site investigation was carried out by Concept SI Ltd and comprised of a number of trial pits internally and externally around the original building to establish the dimension and depths of the existing footings and three 40m deep boreholes with piezometers. Pressure meter tests were carried out in each of the bore holes to establish in-situ soil strengths.

The investigation determined the presence of made ground up to 3m thick in the central courtyard area, overlying medium to dense river terrace deposits, 1-3m thick with the firm to stiff London clay located below. It is worth noting that London Clay is substantially less stiff than Dublin Boulder Clays, with the Dublin Clays being 6-8 times stiffer than the London Clays. Monitoring of the boreholes determined a ground water level of approximately 6.2m OD which would put the water level below our new basement level of 8.0m OD. Existing foundation depths were confirmed by rotary diamond coring to retrieve cores.

Excavation and piling within the courtyard

The first phase of works after the demolition the central courtyard block (that had replaced the original central rotunda bombed during WW2) was to install a perimeter retaining wall of 300mm diameter mini piles at 600mm c/c within the courtyard. The purpose of the piles was to provide a temporary retaining wall to allow the basement excavation to proceed to the depth of the new basement. This required a dig at, and below, the level of the existing courtyard wall foundations. These piles were cored through the out-stand of the existing courtyard wall strip footings and taken down into the London clays at depth. The piles were reinforced with a central CHS section and were tied with an RC capping beam. (which was propped back to the basement slab). The piles were installed by PJ Edwards to a design by Byrne Looby.

In order to limit the lateral deflection of the retaining wall piles, and the possible settlement of the existing facade (both lateral and vertical deflections were to be limited to 5mm for the excavation works) it was necessary to provide 130mm micropiles, pali radice, to underpin the existing foundations in areas of deeper excavation or where the existing footings were found to be at a higher level than the typical case. The design of these piles assumed that 50% of the vertical load from the existing walls would be supported by the micropiles while the remainder would still be resisted by direct ground pressure below the wall foundation. It was a feature of this dig that the existing foundations varied in thickness, level and outstand around the courtyard elevations. This was specifically the case for the excavation of the three lift core bases in each corner of the courtyard. Along with underpinning of these areas with micropiles, it was also necessary to undertake permeation grouting between the retaining wall piles to eliminate ground water inflow. This was successful and inflow of ground water was not an issue in any of the lift core areas.
Retaining wall piles and underpinning micro piles. (Photo courtesy of Byrne Looby and Associates)

Because of the varied construction activity in the confined courtyard space careful phasing of the work was necessary to keep things moving smoothly, this included an extensive archaeological dig which progressed in parallel with the various stages of excavation.

Barrett Mahony designed the basement temporary works propping and produced a construction sequence plan for the basement foundations and slab, ensuring that the perimeter capping beam could be propped from the on-going basement structure during the construction of the pile caps and basement slab. The initial phase of excavations left a berm around the perimeter of the basement to provide support to the courtyard walls and piled retaining wall but allowed the central areas of the piling mat to be installed at the required level. A Casegrande 105NG piling rig was craned over the building and piling for the main works was commenced while the underpinning and retaining wall piling was still in progress. As various areas of piling were completed the pile caps were constructed and then the central areas of the basement slab were constructed. This slab was used to prop the perimeter capping beam via props laid in slit trenches through the perimeter soil berm. The berm could be then removed in sections and the perimeter infill sections of the slab completed.

Micro piles are visible above and behind the capping beam. (Photo courtesy of Donban Contracting)

Basement slab pour sequence to facilitate propping of the perimeter capping beam

Capping beam props. (Photo courtesy of Donban Contracting)
The enclosed courtyard presented an obstacle to getting materials into the construction site and removing soil generated by the excavation and piling works. Therefore the first critical milestone on the programme was to get the three luffing cranes erected. The three lift core bases, one in each corner of the courtyard, serve as crane bases and it was critical they be constructed as quickly as possible. Due to the width of the excavation the propping of the adjacent capping beam was achieved independently from rest of the perimeter capping beam via large diameter diagonal CHS.

Working in such close proximity to a listed building presented a challenge in ensuring that movements of the existing structure were kept within agreed tolerances and minimised to a level which would avoid unacceptable damage to the existing building.

An extensive monitoring regime was put in place for this project to allow real time observation of movements and to identify longer term movement trends which would allow us to intervene in the construction process and alter the speed or phasing of work if necessary.

The elevations within the courtyard were fitted with a grid of reflector prisms mounted on the stone façade. Nine prisms were fixed to each wall for a total of 45 prisms within the courtyard.

The movement of the prisms was monitored by two total station EDM fixed to the existing building in locations which allowed full coverage of the courtyard.

The external elevations were monitored using vertical and horizontal tilt beam sensors mounted on each corner of the building, this allowed differential movements to be recorded without the need to locate EDM stations on adjacent buildings (Often a difficult proposition in London).

In addition, cracks identified on internal walls and slabs were monitored using ‘tell-tale’ gauges for the duration of the works.

Movements were reviewed daily via a web-site during the excavation works in the courtyard as the crane bases were excavated and then as the remainder of the basement was taken to formation level. In general, movements were found to be small, and only the vertical movement of the courtyard walls showed any discernable trend as the excavation progressed. The basement works were completed without any of the alarm trigger points being reached.
The tower building

During the course of the initial dimensional surveys and inspections it was determined that the tower building that forms the front entrance to 10 Trinity was out of plumb by approximately 300mm at eaves level. The tower is over 70m high and the eaves level is 45m above ground level and this figure represents a rotation of just under 0.25 degrees (compared to the 5 degree rotation of the Tower of Pisa). This rotation the outward rotational movement of the tower had dragged the adjacent wings of the building and caused vertical cracks in the elevation and floors at the junction with the longer wings at right angles to the tower.

The cost vs benefit issues of undertaking any foundation strengthening works to the tower raft were discussed in detail with the client. However, additional loading in the form a swimming pool at lower ground floor level and the removal of the substantial area of concrete thickening at the rear of the raft foundation necessitated that a geotechnical analysis be carried out establish the magnitude of likely movements before a firm decision could be made. We commissioned Byrne Looby Partners to undertake detailed time phased analysis, using Flac3D software of the raft with the new loadings and to consider the beneficial effects, if any, of underpinning the raft with a grid of mini piles. Long term settlements of the tower under various loading combinations with and without any intervention were quantified, and it was established that the current movements were both historic and on-going. The factor of safety on bearing capacity failure of the London clays was as low as 1.4 and the further long term settlements projected were likely to cause problems with finishes and facades if they were not reduced.

This analysis allowed us to develop a solution involving the installation of 125 no. 300mm diameter minipiles which would be installed through the existing 2.5m deep raft slab and which would, over time, create a piled raft foundation to greatly reduce further settlements over the next 100 years.

A series of in-situ load tests were carried out to establish how the tower’s vertical loads would be transferred to the new piles. Three test piles were cast in three cores formed in the raft. The first of which had a smooth bore, the second a heavily scabbled surface and the third a series of steel dowels drilled diagonally from the raft through the new pile. The pull out test confirmed that the maximum pile load of 500kN determined in the analysis could be achieved from the smooth bore core (via chemical adhesion and micro mechanical interlock on the surface of the cores). The result was a reduction in the programme as the contractor did not have to scabble over 250m of cored holes.
The piles were installed from lower ground floor level via pre-cored holes in both this slab and the raft slab below, Hill Piling installed over 125 piles to a depth of 18m below raft level over a 6 month period.

**New courtyard superstructure**

The courtyard structure was re-designed by Barrett Mahony, a change from the client’s original proposal for a steel and composite metal deck frame to an in-situ RC frame. This resulted in a more cost effective structure for Donban and also provided a laterally stiffer structure which ensured that wind loads on the new elevations were transferred back to the new cores and not into the existing structure below.

The new basement level within the courtyard is located above the water table level and consists of a 400mm thick concrete slab suspended on piles and piles caps with 160mm of cell core void former to accommodate the effects of clay heave. The new structure is supported on 160 no. 600mm diameter piles bored into the London clays to a depth of 30m. A 170m³ attenuation tank is located below the basement slab which will serve the new roof and courtyard spaces and will supply water to a site-wide irrigation system. The new structure is generally structurally independent and isolated, horizontally and vertically, from the existing building.

Waterproofing to the basement slab was achieved with a layer of bentonite membrane lapped with a vertical face of bentonite sheeting at the junction with the existing courtyard walls.

A 300mm thick reinforced concrete flat slab structure was adopted for the lower ground and ground floor levels with a concrete beam and slab structure at 1st floor level. The perimeter columns from basement to ground floor level were raked diagonally by 10 degrees from their positions at basement level, inboard of the existing courtyard wall footing to a position much closer to the existing wall above ground floor level.

The first floor slab has a 14m wide central opening which provides a double storey space. A two storey circular steel rotunda structure sits above this opening supported on a fan arrangement of concrete beams cantilevering 2.1m at 1st floor level. The area between the central rotunda and the perimeter structure blocks at first floor level will incorporate an ‘infinity’ water feature setting the rotunda within its own lake. The new rotunda is to be clad in a series of vertical aluminium blades and will have a roof garden at 2nd floor level.
The floor plate at 2nd and 3rd floor level are limited to a 6m wide bay around the perimeter of the courtyard. The perimeter columns of this structure provide support to the new 4th floor steel work which spans over the existing building footprint.

The programme for construction was phased to follow the production programme for the off-site construction system to be adopted above 4th floor level. It was important that these volumetric units be placed as soon as they were completed and to avoid the need for off-site storage. Thus the construction of the perimeter elements of the structure were given priority.

Lateral stability of the new courtyard structure is provided by concrete stair cores in each corner of the courtyard. These cores were also designed to resist the lateral loads to the new structure above 4th floor level located over the existing building. Thus the existing building only supports vertical loads from the new floors at 4th and above. To ensure that this is the case the floor plates of the volumetric units at 4th floor and above are isolated from the existing stair cores but are tied to the new core structures located in the courtyard.

**Seething Lane**

Seething Lane Gardens is a public park, managed by the City of London. Historically, diarist Samuel Pepys lived and worked at the Navy Office, which was originally situated on the present day Seething Lane Gardens. The site was subsequently occupied by warehousing, before becoming a public park.

A development of this size requires significant floor area for back-of-house areas and car parking. In order to maximise usable space within the existing structure, it was decided to locate these outside the footprint of the existing building. It was therefore proposed to construct a new double storey basement under Seething Lane Gardens, adjacent to the existing structure. The basement is approximately rectangular in shape, of maximum internal width 21 m and maximum internal length 74 m. The original gardens were removed for the duration of the excavation works and will be reinstated as an improved public space on top of the new basement.

The client’s original basement scheme featured a secant pile wall. The pile line ran underneath the public road and footpath, outside the boundary of the gardens. Asset search information and site surveys indicated a heavy concentration of services under the road. It was estimated that it would take up to a year to arrange diversions for these services, at a potential cost of £1M. It was therefore decided to move the outside edge of the structure within the footprint of the existing gardens. The implication of this was a reduction of floor space in the proposed basement. In order to mitigate this, BMCE proposed to redesign the structure as a steel-intensive basement, featuring a sheet pile wall. This would provide a gain of 500 mm around the perimeter of the basement, and would compensate for the loss in floor area. In coordination with the design team, BMCE reconfigured the internal structure to ensure that the original net floor area could be accommodated within the reduced footprint.
BMCE undertook a feasibility study of the sheet pile wall option, with particular consideration to vibrations and ground movements. The excavation is in a sensitive location, located just 1 metre away from the western façade of the existing 10 Trinity Square structure. Other nearby structures include St Olave’s Hart Street, a Grade I listed 15th century church. An outline design of the sheet pile wall was carried out, in order to evaluate sheet pile wall deflections. Ground movements outside the excavation were estimated using published guidance, including CIRIA C580. BMCE reviewed piling installation techniques with piling tenderers to ensure that sheet piles could be installed without causing excessive vibrations. BMCE specified vibration monitors, façade movement sensors, and pile inclinometers to ensure real-time data for building movements was available during the excavation.

On the basis of the pile preliminary design and the internal reconfiguration of the structure, the revised basement proposal received client approval. Since the retaining structure retains the public highway on three sides, it was also necessary obtain City of London approval for the proposals. An Approval in Principle document was submitted to the City Surveyor, to provide an overview of the design principles in the temporary and permanent cases.

The internal basement structure comprises a reinforced concrete frame. The roof of the basement is a transfer slab, which will support 2.5 m of reinstated soil, as well as landscaping features. There is a partial intermediate slab at Basement 1 level. This level is to be used for access and back-of-house activities for the main development. Two large voids exist at this level to allow additional headroom for truck parking and an automatic three-level car-stacker. The basement is to be linked to the existing 10 Trinity Square structure via a service tunnel at Basement 1 level. Reinforced concrete pavilion structures, extending above ground level will be placed at either end of the basement, and these will house vehicle lifts for the
Welded sheet pile clutches and water bar plate car-stacker and truck lift. The lowest basement level is a raft slab comprising suspended slabs on void-former spanning between ground bearing strips. The internal walls and columns within the basement are supported on the ground-bearing slab strips and are subjected to permanent downward loads from the structure and gardens above. Void former is used to reduce the heave pressures acting under the slab in the areas outside of the ground-bearing strips.

Waterproofing to the sheet piling is achieved by welding the clutch joints between pile sections over their full internal height. A steel flange plate is welded along the profile of the sheet pile wall at the junction between the slab and the sheet pile wall. At the interface with the lowest level slab, a bentonite seal is provided over the welded clutch as an additional protective measure. Waterproofing to the Basement 2 level slab is achieved with a proprietary tanking membrane, placed at the underside of the basement slab and ground beams. Particular consideration was given to the roof of the proposed basement, as this will be inaccessible following replanting of the Seething Lane Gardens. The roof slab will therefore be constructed with waterproof concrete, and a proprietary tanking membrane will be applied to the top of the slab.

The excavation was generally 10.5 m deep and involved the removal of 13500 m³ of material. Given the historical importance of the site, the excavation initially progressed to a depth of 5 m to allow for a detailed archaeological excavation. BMCE developed the initial temporary works scheme, and the final design was carried out by the groundworks contractor. In the temporary case, the sheet pile wall spans between steel walers at two levels which are propped across the full width of the excavation. A third level of propping is provided at the north end of the site, where a 13.5 m excavation is required for the truck lift.

The original tender scheme had indicated a thick basement slab on void-former. Tension piles were provided to transfer vertical load and to resist both hydrostatic and heave pressures. BMCE opted to investigate alternatives on behalf of their client and proposed a revised foundation scheme comprising a 600 mm deep suspended slab on void-former spanning between 1200 mm deep ground-bearing strips. No ground anchors were used.

A key design issue was the ability of the structure to resist uplift forces, and the consequent movement to the existing building. As the formation level for the Basement 2 slab is below the design water table level, the structure is designed to resist hydrostatic pressures. The ultimate stability of the structure for the uplift case was assessed by comparing the permanent downward forces against the hydrostatic force. The structure is also designed to resist heave pressure due to the swelling of the unloaded clays. The long-term heave pressures under the Basement 2 slab were advised by Byrne Looby.

The basement structure was analysed using RAM Concept software. The spring stiffness values for this model were derived from a multi-stage pDisp analysis carried out by Byrne Looby.
The design procedure consisted of an iterative process until convergence between successive runs was observed. The final spring stiffness values were then used in the RAM model to obtain slab settlements and bearing pressures under the ground-bearing portions. The RAM model was used to design the steel reinforcement in the Basement 2 slab, considering the most onerous load combinations for the upward and downward cases.

An assessment of the movements to the existing 10 Trinity Square structure was carried out by Byrne Looby using FLAC3D analysis. This analysis was based on the design loads and construction sequence advised by BMCE. The structure was modelled by considering idealised ‘slices’ through the structure. This was carried out at two typical locations in the basement, to represent the most critical conditions in the north and south of the basement.

The piling tender specification prescribed a limit of 10 mm settlement under the existing foundations during the piling works. It was also necessary to consider the possibility of long-term movements to the existing foundations. A total 10 mm settlement was considered as a sensible target for the long-term case. The FLAC3D analysis indicated that the maximum calculated settlement at the completion of the excavation, including an allowance for initial consolidation, was 9 mm. This indicated that the contract requirements were satisfied and gave the design team confidence to progress with the detailed design of this foundation option. The FLAC3D models indicated settlements of 17 – 18 mm in the long-term. However, it should be noted that the settlements obtained from the FLAC3D analysis are upper-bound values. The FLAC3D analysis is based on conservative soil parameters and does not consider the stiffness of the full existing building, which will act to mitigate the settlements to the western façade foundations. It was expected that the actual settlements would be approximately 60% of the FLAC3D settlement, based on initial readings observed on site. If this pattern continued, the calculated long-term settlements would reduce to 10 mm.

It was proposed to proceed with the intended foundation option and to use the Observational Method to assess existing foundation settlements during the excavation. Alternative options were considered in the event that the observed settlements were closer to the upper-bound values obtained from the FLAC3D analysis. These included reducing the areas of cell-core under the slab or introducing ground anchors. Such options had pros and cons associated with them and the potential settlement reduction was of the order of 2-3 mm.

Settlements at the conclusion of the excavation were below 5 mm. This provided the design team with confidence that the anticipated long-term settlements would be within 10 mm and that the existing structure was safeguarded, both during the works and in the long-term condition.

**Works to existing building**

There is a general principle that listed buildings are put to ‘appropriate and viable use’ with the recognition that this may involve the re-use and modification of such buildings. However, listed buildings cannot be modified without first obtaining Listed Building Consent through the relevant local planning authority.

10 Trinity Square, is a grade II* listed building, and it was important to consider both the conservation issues and the lead-in time for approvals when undertaking works that involved modifications to the existing building fabric.

![Existing Internal finishes within tower](image)

In many cases a more costly and complex solutions were adopted for such work to avoid interference with the areas of special interest, as the timeframe for securing approval from the relevant heritage bodies could be protracted. Most of the existing second floor level internal finishes were protected and could not be disturbed. However work in other parts of the existing building that effected existing/finishes structure also need permission from the heritage officer and the lead-in time...
required to secure permission from the relevant heritage body needed to be factored into the programme for both the specific area and its effect on the programme of the overall project.

**Existing structure and new loadings**

10 Trinity Square is one of the earliest reinforced concrete buildings in the UK. The floor plates are reinforced concrete slab, 140mm thick, spanning between concrete down-stand beams. These beams span perpendicular to the external walls and are supported on pairs of reinforced concrete columns (either side of the main corridor) and also on the external masonry/stonewalls. The external courtyard elevations are Portland stone interlocked with brick on the internal face.

The mansard elevations above 4th floor were a later addition and are a steel framed structure supporting solid precast concrete floor units.

The lower ground floor slab has an adjacent perimeter light well and there are two isolated areas of basement, one below the tower structure.

The foundations to the existing building are reinforced concrete; a mixture of strip, pad and raft footings typically founded in the terrace gravel just above the London clays. The level of foundation varies somewhat around the perimeter of the courtyard.

The lateral stability of the existing building is provided by the diaphragm action of the concrete floor plates coupled with a combination of cantilever action of these towers and possibly some vierendeel action of the masonry/stone facades.

Investigative opening up work confirmed the reinforcement to the existing slabs and the concrete columns. All reinforcement was taken as having a yield strength of 200kN/mm².

The lay-outs of the proposed apartments at 4th floor and above did not suit the original column lay-outs and it was decided that these two upper level would
be removed and replaced with a new structure better suited to accommodate the proposed layouts. The client’s original scheme adopted a conventional steel framed structure for these additional floors. The support of this structure involved partial demolition of the existing fourth floor plate and strengthening of the existing concrete beams at this level and below. Donban, in conjunction with Barrett Mahony proposed the use of volumetric construction, a state-of-the-art off-site construction system, rather than a steel frame, for these additional stories. This had a number of advantages including a reduction in the construction programme but the chief advantage lay in the fact that the walls of the units would act as the new vertical load bearing structure and remove the need for individual columns that would compromise the internal lay-out of these high end apartments. The use of these off-site elements is described later in this paper.

The demolition of the existing roof, 5th and 4th floor slabs was carried out in tandem with piling works within the courtyard. Non percussive techniques were used as far as possible to minimise the risk of damage to protected finishes down at level 2. Barrett Mahony were responsible for designing the temporary works for propping the existing floors and providing temporary stability to the existing structure while the demolition was carried out. It was necessary to create a temporary roof at 3rd floor level to weather the building below until the new floor plates where added at 4th floor level.

The addition of the new above 4th floor resulted in increases in vertical load in both the external masonry and the internal concrete columns and a detailed assessment was required to determine how the existing structure would handle these loads.

Material testing

As with any historic building an intensive desktop study was carried to source existing drawings. Given that the building was constructed in the 1920’s this proved difficult and very little information was available. Masonry and concrete cores were taken and tested to determine average and lower bound strengths and the condition of the in-situ materials.

The masonry test samples yielded compressive strengths of 20.9-44.9N/mm$^2$ with an average strength of 33N/mm$^2$. It was found that the lower bound strength was adequate for capacity checks on new load to existing piers in all cases.

The original investigation had taken 28 core samples from columns around the original building at different levels and the compressive strengths of the in-situ concrete ranged from 14.6-34.3 N/mm2 with an average strength of 22.1N/mm2. The initial loading calculations and the analysis of the existing columns with the new loadings applied from the additional storeys above showed that typically the lowest tested strength of 14.6N/mm$^2$ was sufficient for the new loadings.

However, there were local areas where increased floor spans meant that the new loads would be too high for existing column capacities based on this lower bound strength. A back analysis also determined that if his lowest strength was assumed to apply in these area the existing columns could not be justified for the original loads as suggested by the 1920’s London Building Acts. (an imposed load of 4.8kN/m$^2$ was suggested for office
buildings). We decided that it was necessary to carry out further core tests on columns in each wing and at each level to get a better representation of the available compressive strength in these specific areas. A further 22 no. core tests were taken with an average strength of 30.2N/mm² and based on a statistical analysis of the data both globally and on a quadrant by quadrant basis using BS EN 1379 1nad BS 6089 it was determined that no column strengthening was required to resist the new loadings. This was a similar situation for the floor slab and beams where the new loads did not present a problem for the existing structure.

The effect of differential settlements between existing foundations also needed to be considered. Within the existing structure the new transfer steel grillage at 4th floor did not load all of the existing columns by exactly the same amount. An analysis was carried out on a number of adjacent internal columns and external masonry piers where there was greater than 10% difference in total service load and it was determined that the differential settlement/heave movements were acceptable, being in the order of L/1000 or better between adjacent footings. The expected long term differential movements between the tower raft slab (now plied raft slab) and the adjacent corridor strip flooring was also checked and determined not to pose a problem.

A similar exercise was carried out between the tower structure and the adjacent courtyard structure that was partially supported on the outstand at the rear of the tower raft. The new courtyard structure was designed to resist the forces generated by the differential settlements between the columns on the raft and those on the courtyard piles.

**Robustness and stability**

The addition of additional storeys and a material change of use for the building required us to consider the issue of disproportionate collapse in the existing building.

The capacity of the existing slabs were checked to resist notional horizontal tie forces in both directions. In the direction of the span there was adequate existing reinforcement to resist the required tie forces and also to provide the necessary peripheral tie.

In the opposite direction, perpendicular to the span, the beams have been used as ties and it was shown that they were adequately reinforced to resist the prescribed tie force with respect to the floor area they support.

The existing columns and external masonry walls were checked as protected elements. The masonry piers with the lowest vertical loads were selected and checked at a number of levels for an explosive pressure of 34kN/m². The external masonry walls on the external elevations satisfied this requirement by three-pin arching action between the existing concrete floor plates. The masonry stair core walls were checked by the same method and deemed to be adequate.

**Remedial work to existing building**

An initial visual survey of the structure identified a number of cracks in the existing concrete floor slabs and masonry walls. The origin of these cracks was attributed to a mixture of reasons, early thermal shrinkage in historic concrete of high cement content, creep, foundation settlement/movements and bomb damage sustained during WW2.

It was determined that the larger vertical cracks in the facade and across the floors at the junction of the longer and shorter corridor wings were the result of tensile forces generated by the on-going and long term rotational settlement of the tower building.

![Typical cracking to internal and external facades](image)

The underpinning of the tower was carried out to reduce further movements to a minimum and in fact the predicted settlement after the installation was a rotation in the opposite direction to the original rotation. However we decided that the larger cracks to both the floors and facades needed to be repaired. Given the protected heritage aspect of the original building it was decided to adopt a...
minimalist approach to these repairs, limiting intervention in the original building fabric where possible. It was envisaged instead that the design and installation of new internal finishes within the existing building would have to take cognizance of the fact that this is an historic structure and that allowable movement tolerances would need to be greater that those associated with a new structure, particularly at the junction between the corridor wings and the existing stair cores.

The remedial works to the cracks in these areas would provide restraints only to on on-going thermal movements as the main action on the existing cracks. The other movement drivers having either already stopped eg shrinkage, or have been dealt with by alternative means eg underpinning of the tower raft.

The proposals was to repair the larger cracks in floor slabs were with cementious mortar and small diameter reinforcement bars laid at close centers in saw cut slots on the top of the slab across each crack.

The larger cracks in the external walls where tied with stainless steel rods either drilled longitudinally thru the wall at window reveals (Helifix ChemTies) or placed in saw-cut slots in the inner face of the wall (Helifix Helibars). We decided that any saw cutting of the external stone to install ties would likely cause more damage in the long term, thus cracks in the Portland stone were to be repaired with a colour matched grout. The programme for this work was pushed as close to the completion date of internal finishes as possible to allow any construction induced settlements to have occurred.

Fourth floor structure and the upper levels.

The client’s original proposal for the additional upper four floors was a traditional design of structural steel with composite metal deck slabs. The scheme involved partial demolition of the original 4th floor structure along with strengthening of the existing 4th concrete floor beams to provide support to the new structure above. In addition, some internal diagonal columns were utilised to achieve continuity within the structure and transfer additional loads away from the existing structure and back to the new build elements. This scheme had a number of columns located in positions that necessitated a compromise in the desired architectural lay-outs.

In conjunction with the contractor we proposed an alternative solution that would adopt a state-of-the art off-site construction system for these upper floors with the advantage of a reduced programme and a better defined load path both vertically and horizontally. And most importantly, the walls of the units would be load bearing and would remove the need for a traditional grid of columns structural columns that might compromise apartment lay-outs.

A steel beam transfer level was developed for 4th floor level. This grillage would support the stud lines of all the offsite elements at 4th floor and above and would cover the footprint of both new and existing buildings at this level. The grillage would be lighter than a concrete transfer slab and would also serve to move some of the new loads away from the existing structure and back onto the adjacent, new courtyard structure via a series of cantilever beams. This would also allow a more uniform distribution of new loads to the existing internal columns.
The grillage consisted of steel beams laid out on a grid to reflect the lines of vertical structure above; which formed the walls of the off-site elements. In some cases double beams were adopted to support the load above rather than the use of deeper single beam. This primary grid was then supported on secondary beams, within the same depth, to transfer the loads back to both the existing and new build vertical elements below. In some areas it was considered possible to simplify the grillage by replacing some of the existing concrete columns at 3rd floor level with a steel column supported on a concrete transfer beam supported on the cut down existing columns just above 3rd floor level.

Concrete transfer beams at 3rd floor

In order to maximise the head room within the existing building below the transfer deck it was necessary to integrate the mechanical, electrical and water services at the underside of 4th floor, to within the depth of these beams. This approach allowed the required clear height to be achieved but involved extensive co-ordination with the services engineer and in-house with our team designing the off-site elements. The complex interaction between the off-site construction elements and the supporting transfer steelwork necessitated a very detailed analysis that considered both the final structural configuration and the phased loading during erection. Etabs, GSA and RAM Concept were used in the modelling of the individual slabs and the overall structure above 4th floor.

Integration of service runs

Due to the tight tolerances of this type of off-site construction system the interaction of the steel grillage and the supported units above formed a key element of both the structural analysis and the erection sequence. In a number of areas of the grillage it was necessary to pre-deflect the transfer steel beams to ensure that the required tolerances between units could be maintained during the erection phase and in the fully loaded case.

Pre-deflection frame

This pre-deflection was carried out to accurately calculated values on a number of the transfer beams. Barrett Mahony designed a self-contained pre-loading frame that applied no load to the existing structure and that required only simple hand tools to achieve the required
deflections on site. The pre-deflection was carried out on a phased basis prior to erection of the off-site elements and was maintained in-place until the required height of structure had been reached, after which the frames were released and moved to the next area under construction.

Modern methods of construction- The Volumetric unit

The volumetric unit is not be confused with cold formed panel wall systems or with pod bathrooms or kitchens. For this project, with bespoke and varied architectural lay-outs there was very little repetition of the structure that could be achieved. The volumetric system is the off-site fabrication of traditional hot rolled steel and concrete elements into load bearing wall and floor units. These units are transported to site, craned into place and then locked together by welding to create rigid vertical and horizontal load bearing structure. In some cases, depending on the project, these units can have internal finished applied off-site, but this was not adopted for this project due the weight restrictions on the main cranes.

This off-site structural solution offers a significant advantage by including the structural columns in the walls that form the room layouts. The perimeter of each element is formed with a structural channel at floor level. Structural box section steel columns are utilised to form the walls at approximately 1200 to 1800mm centres where possible. The hollow sections do not always align from floor to floor and transfer beams are required in some areas within the thickness of walls to ensure vertical load transfer.
The ceiling is constructed with 150mm deep trusses which facilitate services, ceiling support and site access. These trusses also tie the opposing walls together and offer robustness to the construction.

The floor of each unit is constructed with lightweight concrete to reduce the weight on the transfer structures below. The off-site units extend to include corridors and so provide full coverage of each floor outside of the traditional build areas. Each off-site unit that is erected has an independent floor slab. These slabs are connected together on site in various locations, some of which provide for diaphragm action to take horizontal loads, and some to ensure that adjacent units act together for vertical displacements both on short and long term. The lateral stability of such a structural system is different from a traditional concrete structure where the floor diaphragm is deemed to be homogenous and able to transfer the in-plane load almost without further analysis. In this case a detailed analysis is needed to examine the discrete connection forces between the individual floor units and how these loads are transferred to the vertical braced elements, in this case the stair/lift cores. The connections between the units provide the necessary strut and tie forces to minimise differential displacements between the various parts of the diaphragm and ensure predictable interaction between the connection plates and the individual floor slabs. The connections are also checked for the accidental damage load cases, robustness and disproportionate collapse.

The floor slabs are connected to the new concrete stair/lift cores which are designed to provide the building stability. There is no physical connection between the new floor plates and the masonry stair cores of the existing building. Thus all additional lateral loads and are resisted exclusively by the new concrete stair/lift cores in the courtyard.

A detailed Etabs model was created to determine the distribution of load to the existing and traditional elements of the structure from the transfer steel grillage. Each individual element of the structure from 4th floor upwards is included in the Etabs model with individual floor slabs and their connections modelled to ensure accurate continuity effects. Individual structural studs have also been included in the model to ensure an accurate distribution of loads.

The model is then used to determine the loads on the existing columns, and masonry walls and the forces and deflection of the supporting steelwork in the transfer level.

The Etabs model allows for both vertical and horizontal analyses to be undertaken. The vertical analysis requires a complex model to analyse the distribution of loads based on the interaction of the off-site elements and the transfer steel deck. All the structural studs are modelled with the floor channel and concrete slabs included to ensure the interaction of each element is understood. Given that the deflections of the individual elements of the transfer steel can result in an uneven distribution of loading within the structure of the volumetric units it is critical to ensure the analysis addresses these interactions. In this case the effect of the support stiffness of both the transfer steel works and the settlement of existing structure must be considered when assessing loads on individual columns within in the stud walls.
Roof

The new roof of 10 Trinity Square was intended to provide a contemporary and visually striking solution to integrate the historic and new structure under a dynamic new surface.

In response, the clients architect, Woods Bagot, developed a hyperbolic parabaloid roof surface that would cover both buildings and leave the central courtyard open above the new rotunda. The roof is curved in two directions with the highest point located over the North West stair core diagonally opposite the tower building. The roof has a central ridge line of variable height that falls into the gutters, located around the courtyard perimeter and around the external building perimeter. These gutters have syphonic outlets and the rainwater is drained to a series of internal downpipes.

A 3-D Auto Cad roof topography drawing was produced by Harley, the roofing sub-contractor. This model showed the locations of each zinc panel and the supporting rail system. We then created a Revit model using the roof topography generated from the 3D drawing. Our model included the roof structure to support the panel rails and the included the volumetric units below onto which the whole roof structure would sit. The continuous support structure for the panel rails was provided by 200x90 PFC purlins and 1.2m c/c, laid flat, but stepped up and down as necessary to follow the curvature of the roof.

These purlins were supported on a grid of steel columns located so that the span of the purlin was limited to approximately 2.8m and that the span of the panel rail system was limited to about 1.2m. This gave a grid of approximately 2.8x1.2m of 80x80 SHS columns to support pfc purlins. The height of the columns vary from 0.5m up to almost 4m.

(Photo courtesy of TP Bennett/Woods Bagot)

There were a number of particular requirements and conditions that lead to the final design of the roof structure.

The contractor proposed the use of a panelised zinc rain screen cladding system as the outer skin of the roof with a separate weathered roof below this outer skin. This rain screen system was supported on a series of rails which in turn were supported on adjustable brackets at 600mm centres. A requirement of this system was that each of the bracket support locations needed to be flat and level; the roof curved profile would be generated by the variable adjustment of the height of these brackets. It was a requirement that the support structure for the brackets should be formed with continuous rails rather than discrete support points and that the level of the support points should reflect the minimum and maximum adjustment heights of the support brackets and the level of the gutter around the perimeter. Finally we needed to consider both the variable geometry of the roof and the how to support the roof the volumetric structure below.
The purlins are set-out to run parallel to the roof contours as far as possible to help minimise the number of locations where the purlins would need to step in level around the roof plan.

Even with this approach there where many changes in level required. The SHS columns will be site fixed to the roof of modules below.

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Roof support posts wind bracing is highlighted in RED

The internal weathered roof skin is an insulated/weathered timber deck supported on timber rafters and laid to falls. The rafters are supported on pairs of timber purlins bolted directly to the posts which are typically at 5.6m c/c. This deck is located approximately 750mm to 1150mm below the rain screen panel level and the deck will serve as a working platform during the erection of the rails and panels above.

The weathered roof is laid to a similar falls as the rain screen roof and drains directly into the gutter system at eaves level either side of the roof.

Initially we wanted to utilise the timber roof as a structural diaphragm, tying the columns back to the vertical wind braces. However, due to the changes in level of this roof and the discontinuities in the diaphragm, where the main plant room areas are located, it was not feasible to achieve the continuity and stiffness this system would require.

We decided to adopt traditional wind braces in the horizontal plane of the roof to transfer lateral loads back to the vertically braced bays. This bracing is located below the level of the weathered roof and is formed with SHS chords and tensioned 16mm diameter mild steel rods to minimise any take up movements when loads are applied.

Conclusion:

10 Trinity Square was a complex and fast tracked project that demanded a lot from both the contractor and the design team during the last two years. The project was one of the most challenging we had undertaken with volumetric units, principally due to the complex interaction between units, transfer grillage and the new and existing buildings below. The project was not a simple linear progression from foundation to roof; works were often on-going within the courtyard, the existing building, Seething Lane and on off-site elements at the same time. This needed a number of discrete groups within our office to design each of these areas independently; co-ordination of this design work and communication between the various teams both in Dublin and London required a determined effort from all involved. This ultimately led to a closer understanding between our engineers than would normally have happened on a less complex project.

Design Team

Contractor: Donban Contracting Uk Ltd

Architect: TP Bennett

Civil and structural Engineers: Barrett Mahony

Mechanical and electrical Engineer: Malachy Walsh