Title: Birmingham Gateway: Structural Assessment and Strengthening

Dr J.J. Orr, MEng (hons), PhD, FHEA

Lecturer in Civil Engineering, Department of Architecture and Civil Engineering, University of Bath, Bath,
BA2 7AY

j.j.orr@bath.ac.uk; 01225 383 412

Mr. D. Pask, BSc(hons), CEng, MStructE

Associate Director, Atkins Engineering and Major Projects, Atkins Ltd

Mr. K. Weise, MSc, Dipl Ing (FH), CEng MICE

Senior Structural Engineer, Atkins Engineering and Major Projects, Atkins Ltd

Professor M. Otlet, BSc(hons), CEng, FStructE, FSFE

Director, Atkins Engineering and Major Projects, Atkins Ltd

Abstract

Birmingham New Street is the busiest UK rail station outside of London. Growing demand following upgrade works to the West Coast Main Line has seen passenger numbers exceed the design capacity of the current station, which was constructed in 1967. To meet projected increases in passenger numbers, a redevelopment of the historic station is currently underway. Retaining all major structural features, the redevelopment is being undertaken over a live railway in the heart of Birmingham while maintaining existing passenger capacity.

This paper details the structural assessment and strengthening design work undertaken to facilitate the regeneration of Birmingham New Street. The assessment methodologies used in examining this historic concrete structure are discussed before the design of subsequent strengthening works is presented.

Keywords

Buildings, structures & design; Conservation; Risk & probability analysis

1 INTRODUCTION

Birmingham New Street Station is the principal rail hub for the City of Birmingham. In 2010-11 the station saw 24.7M entries and exits and 4.3M passenger interchanges [1], figures in excess of those envisaged during its design. With further increases in passenger numbers forecast in the rail sector [2], a major redevelopment of the station has begun to meet demand for the next 40 years.

Constructed between 1963 and 1967, the existing station replaced the original New Street Station which was completed 1851. The existing reinforced concrete building covers an area of 22,500m² in nine blocks separated by movement joints. Vertically the building has four main floor levels (Figure 1).

The construction of the main frame was principally in-situ concrete. Floor plate construction varies, being either all in-situ, in-situ with pre-cast concrete floor units and a structural topping, or precast bridge beams with pre-cast concrete floor units and a structural topping. Primary 'spine' beams run East-West following the line of the platforms, with secondary 'saddle' and 'rib' beams running perpendicular to these. All columns are aligned with an East-West minor axis (see Figure 2)

Movement joints allow each of the nine blocks to act as independent three-dimensional sway frames which provide global stability. Movement monitoring between each of the block was used to confirm this. The ability of the nine blocks to move independently was also measured through movement monitoring during the construction phase.

To improve passenger flow and capacity and revenue, a number of changes to the structure have been designed, as summarised below and illustrated in Figure 3:

- 1) New atrium across building zone 5 (see Figure 1);

- 2) Removal of internal intermediate floor slabs (Figure 3);
- 3) New cladding to external façade;
- 4) New retail building partly supported by building 7 (approximately 4 storeys above existing structure) (see Figure 1 for location);
- 5) Replacement of the multi-storey car park (see Figure 1 for locations);
- 6) Change of use of some areas from car parking to retail.

In combination, these works have changed the vertical and horizontal load envelope on the building, and for this reason a detailed assessment of the structure both before, during and after construction was undertaken.

The assessment stage was undertaken by implementing the historic permissible stress based British design code CP114 [3] along with relevant guidance documents as described later. Subsequent strengthening work was designed using the partial factor based BS 8110-1 [4]. The interplay between the two codes at various stages of the work are described in further detail below.

2 ASSESSMENT

The aim of the assessment was to consider the global effects the proposed modifications and new loading have on the capacity of the structure. The process of assessment set out to answer the question put forward by Happold *et al* [5] – “is the structure adequately safe now and will it remain so in the future? Can it be used for its intended purpose and can it continue to be in the future?”

To do this the current condition of the structure, and record drawing data, was reviewed and processed to inform decisions on the structural safety and reliability of the entire system. Levels of safety were considered with reference to both current and historical design codes of practice, as well as guidance produced by the International Standards Organisation [6].

The concrete design code used at the time of original design of Birmingham New

Street was the permissible-stress based CP114 [3]. Although this has now been withdrawn, guidance for the continued design and appraisal of concrete structures designed to permissible-stress based codes has been continually updated by the Institution of Structural Engineers (IStructE) [7]. The IStructE guide may be applied to “*all normal reinforced concrete building structures without limitation of size*” [7] and specifically covers sway frames such as that found at Birmingham New Street. It may also be possible to utilise guidance found in the Model Code [8].

2.1 Assessment process

An overview of the process of assessment undertaken at Birmingham New Street is described by the flowchart in Figure 4. The iterative assessment process was undertaken by collating the available reinforcement and geometric data to create three-dimensional structural models in STAAD.Pro [9] of both the existing and proposed structural schemes. The results of both analysis models were then used to carry out an assessment of each member affected by the building works. These results were then filtered using a series of more detailed criteria including more detailed intrusive surveys, resulting in a short list of members that may require strengthening in order to carry the proposed forces at a suitable factor of safety.

2.2 Data collection

An initial desk study collated several thousand archive and record drawings, site photographs and survey reports, from which a database of geometrical and reinforcement information was produced, covering some 750 columns, 3600 beams, floor slabs and walls. The majority of geometrical data was available and three-dimensional structural models of both the existing and proposed structures were then constructed.

In contrast to geometrical information, reinforcement information was less readily

available. With as much as 71% of the beam reinforcement information missing, it was not possible to carry out new surveys to collect the required data. Instead, the beam and column elements were divided into two categories that were subject to different methods of assessment. Category 1 elements were those for which both geometrical and reinforcement information was available; Category 2 elements were those for which reinforcement information was not available.

A column concrete strength of 41MPa (6ksi) and beam concrete strength of 28MPa (4ksi) was noted on the available drawings. To verify this, 133 concrete cores and 269 rebound tests were collected from which an estimate of the characteristic in-situ concrete strength was made using the method described in BS 13791 [10] and BS 6089 [11]. Test regions and core locations were identified and recorded following guidance in BS 13791 Annex D [10], and BS 6089 cl.5.5 [11] to minimise the influence of surface effects and concrete at the top of a pour on the compressive strength results. In accordance with BS 13791 cl.7 [10] the core strength was not modified to account for the orientation of coring.

The results of these investigations are summarised in Table 1, and these values were used in all of the assessment calculations. A high variability is seen in both sets of data, suggesting inconsistent concrete mixes. The result of the tests is to suggest that both columns and beams have similar characteristic compressive strengths.

It was found that the beam concrete strength was 18% higher and the column strength 19% lower than the recorded design strength. This contrasts to the CP114 [3] recommendation of a 25% increase in strength for concrete of this age.

Table 1: Concrete strength data.

	Median value (MPa)	Mean value (MPa)	Standard Deviation	Characteristic value, (MPa)
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Columns	60.6	61.1	16.1	34.6
Beams	50.0	48.9	9.39	33.4

Intrusive investigations found that square twist chamfered (STC) bars had been used as main reinforcement in all columns and beams. The yield strength (f_y) of the square twisted chamfered steel bars was shown on available reinforcement drawings as 414MPa (60ksi), this value was verified by testing. The yield strength of the mild steel circular secondary bars was also shown on reinforcement drawings as 250MPa (36ksi).

2.3 Loading data

The self-weight dead loads for the existing and proposed structures were calculated automatically based on the building model. Dead load moments in the columns were minimised by incorporating the construction sequence into the analysis, determined by studying available construction photographs (Figure 5). At lower retail floor level the in-situ rib beams were cast, with their formwork being struck soon after. Precast floor planks were then installed and tied together with a reinforced structural topping forming a 'T' sections with the beam and ensuring diaphragm action, before construction of the next level columns was commenced (Stage 1, Figure 5). All the dead-weight moments of the first floor level are therefore released into the lower column before the next level of the frame is constructed (Stage 2, Figure 5). This reduced the moments in the upper columns that were found in initial studies to have the highest utilisations. When the next column is installed it carries no self-weight moments until the upper floor is built (Stage 3-4).

By including this sequence, the self-weight moments calculated for each frame were found to reduce by between 30% and 40%. Such inclusions are particularly necessary when assessing existing structures, where the primary concern is the actual state of the building. Without the staged analysis, unrealistic outputs from the analysis model may have resulted in far more elements requiring remedial works.

The construction sequence analysis was achieved by resetting the analysis stiffness matrix and adding the active members, joint releases and self weight loading.

Imposed loadings used in the original building design were collated from the record drawings; new proposed loads were obtained from the design team. In older designs, the issues of lateral support and restrain were not considered in such detail [5] and so applying current notional horizontal loading (NHL) requirements would have been an onerous requirement. A letter was obtained from building control stating that NHL did not need to be retrospectively applied.

However as one of the key requirements was to assess the stability of the sway-frame building structure, horizontal loading representing the increased area of cladding exposed to wind load was applied as an equivalent horizontal load, taken as a percentage of the total permanent loads. The percentage was found by matching the NHL to the wind load. A value of 0.75% of total permanent loads was used as the NHL, and this was applied to each block in two orthogonal directions.

A 10% area load reduction was applied to live loads at lower retail, mezzanine and concourse levels for the assessment of spine beams and columns, based on BS 6399-1 (1996). No reduction was applied to the imposed dead loading, plant loading, nor to storage areas, roof areas, or car parks.

2.4 Analysis

A linear elastic analysis in STAAD.Pro using the models described above was then undertaken to determine member actions in each of the seven load cases given in Table 2.

Table 2: Loading combinations

Combination	Description	Combination	Description

1	Dead and live load	2-5	Dead, live and notional loads applied in each of the four orthogonal directions (where appropriate)
6	Dead and live loads with positive temperature change.	7	Dead and live loads with negative temperature change.

Moment redistribution in CP114 [3] is permitted up to 15% [7]. Given the age of the building there is a further potential for additional moment redistribution due to a combination of creep and plastic deformation of the concrete and the analysis therefore allowed 30% redistribution for dead load and 15% for live load. This level of moment redistribution is consistent with the values used in both CP 110 [12] and BS 8110 [4] (the concrete codes published after CP114 [3]). Creep and the time dependent behaviour of concrete leading to moment redistribution is further discussed by Scott and Whittle [13] where a redistribution of 30% is considered to be a conservative assumption.

2.5 Assessment

With the analysis output complete, the actions on each element in the frame were known and the assessment began with the primary aim of determining which of the 4000 elements would require strengthening works as a result of the building's rehabilitation. The criteria for determining which element would require strengthening are provided in Table 3.

Table 3: Criteria for strengthening

Category	Description	Notes	Criteria for strengthening
Category 1	Full reinforcement details available	100% Columns 29% Beams	Utilisation Factor > 0.71
Category 2	Reinforcement data	0% Columns	≥ 10% change in actions on pro-

	not available	71% Beams	posed model
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2.5.1 Category 1 Elements

Assessment of Category 1 Elements was undertaken by calculating for each a utilisation factor (UF) against probable failure in flexure, shear and torsion. The utilisation factor is calculated by dividing the effects of each action by the element's resistance against the action.

In a design to the permissible stress code CP114 [3], a stress reduction of 1.80 is used to determine steel and concrete permissible stresses. The member resistance is then calculated using these reduced permissible stresses. A stress reduction factor of 1.80 implies both a maximum utilisation factor (see above for definition) of 0.55, and that the member can carry a increase in its combination of actions of as much as 80% before it actually fails (i.e. when it reaches a utilisation factor of 1.00).

This condition is onerous for assessment purposes and since the risk factors associated with design and construction, as well as the geometry and concrete strength can be quantified, the allowable utilisation factor for this assessment was increased to 0.71 (which may alternately be thought of as a factor of safety of 1.40). This increase in utilisation factor was agreed through extensive discussions with an expert panel of the client's engineers. This level of utilisation still gives scope for overloading, hidden construction defects, and variations in concrete strength.

To calculate the utilisation factor for each element, its capacity was calculated using the methods given for design in CP114 [3] but without stress reduction factors, giving the envelope for $UF = 1.00$. The existing and proposed actions were then compared to this capacity to determine the element's actual UF in both the existing and proposed conditions.

For each column, IStructE recommendations [7] along with interaction diagrams were used to check combinations of both single and bi-axial moments and axial load while shear capacity was checked on both axes using the provisions of CP114 [3]. Category 1 beams were checked in flexure using a plane sections analysis, with shear and torsion capacities checked using CP114 [3], as summarised in Figure 6.

With columns and beams having up to seven layers of main reinforcement of various sizes and multiple shear reinforcement layouts the entire analysis of seven load cases was automated to output the maximum UF for each element. Hand calculations and external checking was utilised to verify the accuracy of the automated methods. For each element where the UF was initially calculated to be greater than 0.71, it was added to a 'long-list' for strengthening.

2.5.2 Category 2

Category 2 elements, for which no reinforcement information was available (see Table 3), were assessed by comparing the change in the effects of shear and flexural actions on the element between the existing and proposed computer models. Where a change in the effects in flexure or shear on the element was found to be greater than 10%, it was added to a long-list for potential strengthening.

For beams, two checks (denoted Local and Global) were undertaken for shear and flexure. The overall check compared the envelopes of moment or shear between the proposed and existing conditions, while the local check assessed how the forces at the end of an element were affected. This approach (shown for shear in Figure 7) ensured that beams with large percentage changes in a small value of force or moment could be filtered from the list.

2.5.3 Foundations

An assessment of the foundation capacity was undertaken. A maximum axial load increase of 45% was found in the proposed structure. Piles were subject to detailed design checks, and all were found to have adequate factors of safety.

2.6 Output

Loading drawings of the assessed structure were a key output. These plans show the loading for which the building has been assessed. Future changes in use or amendments may require new assessments, if their effect is to increase the total loading on the structure.

Of the thousands of elements assessed in this project, none of the 'Category 2' elements and only 43 of the 'Category 1' columns were subsequently found to require strengthening as a result of changes in loads on the structure due to the upgrade works. This reflects the iterative process undertaken by the assessment team. If a Category 2 element was found to require strengthening, investigative works could be undertaken to determine details such as reinforcement locations to allow a more detailed assessment. In addition, the majority of Category 2 elements were beams and many of these did not see significant increases in their design actions.

All of the 43 columns identified required strengthening in flexure about their minor axis. For security reasons, the columns identified as requiring strengthening will not be identified in this paper. In the following section, the process of the design, analysis and construction of the strengthening works is presented.

3 COLUMN STRENGTHENING

Achieving axial and moment strengthening of columns in sway frames poses unique challenges. It was not possible to brace the existing sway-frame due to the proposed

modifications to the structure and the difficulties of transferring the forces to the foundations at platform level. To increase live-load flexural capacity, two options were considered: 1) the addition of steel or fibre-reinforced polymer (FRP) reinforcement, continuous across the beam-column interface; or 2) to increase the lever arm of the existing column section through additional concrete compression zone depth. To increase axial capacity, two options were considered: 1) additional concrete cross sectional area; and 2) confinement of the existing column section with steel or fibre reinforced polymer wrapping.

A summary of the strengthening options considered, and reasons for their rejection, is provided in Table 4. The various methods are illustrated in Figure 8.

Table 4: Rejected strengthening options.

#	Strengthening method	Description	Reason for rejection
1	CFRP plate continuous between floors (diagonal through beam column joint)	CFRP plates epoxy resined to column face and continuous through joint in diagonal cored holes provides strengthening for an irreversible moment.	<ul style="list-style-type: none"> • No increase in axial capacity. • Difficult to core hole accurately, likely to damage beam tension reinforcement. • Alignment of CFRP within the joint and associated change in fibre direction reduces capacity.
2	Full height steel casing discontinuous between floors	Steel casing injected with non-shrink grout to confine existing column. No continuity between floors.	<ul style="list-style-type: none"> • Limited increase in flexural capacity; • Confinement to provide axial capacity is limited by column aspect ratio; • Heavy to handle on site; • Material intensive.
3	Full height steel casing	Moment continuity added to (2)	<ul style="list-style-type: none"> • Continuity difficult to achieve in

#	Strengthening method	Description	Reason for rejection
	ing with plates con- tinuous between floors	by steel plates across beam face to connect column casings.	practice, involving site welding and floor slab demolition; <ul style="list-style-type: none"> • Heavy to handle; • Material intensive.
4	Full height concrete encasement with NSM bars continuous between floors	Concrete encasement around existing column (for axial capaci- ty). Near surface mounted (NSM) bars added to provide continuity between floors and in- crease flexural capacity.	<ul style="list-style-type: none"> • Encasement thickness to achieve alignment of NSM bars is governed by beam position below each column. Some areas would require very thick en- casement (inefficient); • Chasing on column and beam for NSM bars will be difficult in practice, • Labour intensive

As the existing beam and column connections are all heavily reinforced, cutting through or drilling into this interface to add continuous reinforcement would be extremely difficult to achieve. The option of near surface reinforcement was considered carefully as it is potentially labour efficient to achieve in practice, with only small slots being cut into the existing structure. However, the requirement for 1) additional axial capacity and 2) moment capacity across the joint in this sway frame meant that this option would require both a concrete encasement and additional bars. It was therefore decided that to ensure an easily constructable solution that adds both flexural and axial capacity, a simple concrete encasement with discontinuous reinforcement was chosen for all columns requiring strengthening. This option increases the lever arm of the existing column reinforcement to add flexural capacity, while also increasing the column cross-section area for axial capacity.

Two encasement thicknesses were considered (100mm and 200mm), the thickness of the encasement being dependent on the increase in capacity required for the element.

3.1 Strengthening design

As noted in §2, the processes of assessment are quite different to those of design. Once a member was found to have insufficient capacity in the assessment, the process of designing the strengthening works was undertaken.

The following conditions were considered in the design of the encasement:

- 1) The encasement provides an increase in live-load capacity;
- 2) Longitudinal reinforcement in the column encasement should not enter the beam;
- 3) Confinement effects from the four-sided concrete encasement are negligible due to the high aspect ratio of the columns.
- 4) Due to the relatively high axial load in the columns, moment redistribution in the strengthening design was not undertaken (following §3.2.2.1 of BS8110 [4]);
- 5) Longitudinal and transverse reinforcement is used to control cracking due to temperature and moisture; transverse reinforcement also restrains the longitudinal bars.
- 6) Second order effects are applied to a limited number of slender columns.

3.1.1 Design process

In order to change from the permissible stress assessment to the partial factor strengthening design, the load combinations in accordance with BS 8110-1 [4] Table

2.1 were adopted. The intention of the partial safety factor method is to introduce statistical analysis and probability theory to calculate rational values for design loads and the partial safety factors [14].

Neal [14] describes how the permissible stress global safety factor of 1.8 for reinforced concrete was arrived at for use in CP114 [3]. To change from the permissible stress based CP114 [3] to the partial factor based CP110 [12], separate safety factors for loads (γ_f) and for materials (γ_m) were introduced. However, without the data required to undertake the required probabilistic calculations, in CP110 [12] the partial safety factors were simply set at 1.6 for live load and 1.4 for dead load, while the material factor γ_m for steel was set at 1.15 [14]. Overall, this provided a very comparable level of safety when compared to CP114 [14]. These choices were intended to be amended in later codes as '*new knowledge becomes available*' [12], allowing the intended probability based factors to be introduced. However BS 8110-1 [4] retains the original factors introduced in 1972.

The potential for errors arising from moving between calculations made in the two different codes was recognised at an early stage. The assessment and strengthening teams worked closely together to ensure a workflow was established which would not confuse the 'permissible stress' and 'partial factor' calculations and the computer models were used to provide checks on the calculation outputs. Oversight of the two teams by a senior engineer and standard quality assurance processes were used to further control the flow of information.

A Eurocode approach was not used as the project started before March 2010 and BS8110-1 [4] was the default code for all new concrete works on the project. Each column was analysed again using BS8110-1 [4] prior to designing strengthening works.

The staged construction approach described in Figure 5 was maintained in the new

analysis models and an iterative process of strengthening works design was adopted to account for stiffening effects of the concrete encasements, as shown in Figure 9(a).

For elements requiring strengthening, a 100mm four-sided encasement was initially assumed. The interaction diagram for minor and major axis bending was calculated in the same manner as in the assessment process, but with a factored load approach. The encasement was thus sized to bring the envelope of design load cases inside the interaction diagram, Figure 9(b). Biaxial bending checks of BS 8110 [4] §3.8.4.5 were also applied, and second order effects were considered for a small number of double-height slender columns.

In these calculations, monolithic factors for strength ($K_R = 0.8$) and stiffness ($K_K = 0.7$) were assumed in order to assess the behaviour of the strengthened section. These values were taken from guidance provided in BS EN 1998-3 [15], Fib Bulletin 24 [16], and the work of Lampropoulos and Dritsos [17], Breen et al. [18] (1994), and Eischen et al. [19]. They take into account the limit of in-plane shear transfer at the interface between the encasement and the existing column and effects due to preloading of the existing column. It is also required that the columns to be strengthened are undamaged and any loose concrete is removed.

Reinforcement stresses in the existing reinforcement were also checked in the design by analysing the existing structure under factored dead and superimposed dead loads. During the strengthening site works, sections of the station were to be closed to the public, which assisted the design team in estimating the reinforcement stresses. It was also recognised that effects over the life of the structure (such as foundation movements) can have a significant effect on the actual load distributions. However, as the lower bound theory was relied upon throughout, the chosen approach was deemed by the engineering team to be acceptable.

The resulting frame moments were used to provide reinforcement stresses at these loads. The proposed strengthened section was then analysed with factored live loading; the resulting stresses in any bar were checked to be less than the allowable design stress of 360N/mm^2 (based on a 414N/mm^2 characteristic stress measured in tests, with a reduction factor of $\gamma = 1.15$).

The utilisation factor (UF) for each member shortlisted for strengthening was calculated by comparing the factored analysis results with the ultimate member capacities as defined in BS 8110-1 [4]. The encasement dimensions were amended until the capacity of the section provided a $\text{UF} \leq 1.0$ and the stress in the reinforcement was within the limits set out above.

3.1.2 Interface shear

BS 8110-1 [4] was used to assess the contribution of aggregate interlock and friction between encasement and column to calculate the horizontal shear stress at the interface. Mechanisms that resist sliding and will therefore resist interface shear include the following:

- 1) Aggregate interlock between contact surfaces;
- 2) Friction due to clamping action normal to the interface;
- 3) Dowel action of reinforcement crossing the sliding plane.

The unstrengthened section supports the entire dead load, and so interface shear was checked against the increase in live load and imposed dead load applied to the strengthened section (Figure 10). This conservative check was made using the limits set out in BS 8110-1 [4]. This highlights the complexity of strengthening work design, as the extent of redistribution over time of the initial load from the original section to the new encasement is not known. This redistribution may arise from time dependent

effects and creep. However, the lower bound theory of plasticity ensures that the increased load can be carried by the structure. In reality, the spine beams will transfer most of the axial and moment forces into the encasement at the top and bottom of the column section. Confinement effects of a four sided encasement (when used) were not considered.

Shear keys (150x150x50mm deep), were added to the face of each column. In addition to improving the capacity of the interface, the presence of shear keys helped expedite and simplify the construction process by avoiding a requirement to pre-soak each column for two hours before casting [4].

3.1.3 Axial load capacity ratio

To assess the axial capacity of the strengthened section, a monolithic coefficient was chosen as described in §3.1.1. Lampropoulos and Dritsos [17] found that the most important parameters influencing the monolithic coefficient are the axial load and shrinkage of the concrete encasement.

In order to minimise shrinkage effects which would negatively influence the monolithic factor, a shrinkage compensating concrete (SCC) meeting the specifications of BD 27/86 [20] was chosen with a cement content of 600kg/m³ and compressive strength after one day of 27N/mm² [21].

The SCC encasement was installed up to 80mm below the soffit level. The remaining 80mm was filled after curing of the encasement with a high strength expanding grout [22] to ensure full contact is made with the main beams, accounting for shrinkage of the concrete encasement.

4 CONSTRUCTION

4.1 Introduction

Key to the strengthening works was a robust methodology for the construction of the encasement. In this section, a brief overview of the construction requirements and processes undertaken in the strengthening works is provided.

4.2 Methodology

A work area around each column was provided to limit access and loading by heaving machinery. Removal of floor screed and non-structural column nibs was achieved using floor and track saws with water suppression.

Preparation of the column surfaces was carried out with reference to BS8110-1 [4], and a three-headed hand scabbler and/or needle gun was used. Shear keys, described in §3.1.2 were formed using a diamond grinding wheel and pneumatic breaker.

Starter bars were installed a minimum of 100mm from the column corners and resin anchored (see [23]) to at least 110mm depth. The column reinforcement cage was then installed around the column. After construction and curing of a 100mm kicker, formwork designed to be low-weight for manoeuvrability and including valves to facilitate pumping of self-compacting concrete was installed around each column.

Using a water spray inserted into this formwork, each column face was wetted prior to the casting of self-compacting concrete in maximum 2.4m lifts. Each pour was completed when 80mm below the existing beam soffit. Concrete cubes were taken during casting for testing after 7 and 28 days. After formwork striking, each column was wrapped in light gauge polyethylene to aid the curing process. A minimum of five days after pouring the encasement, the 80mm gap between column and beam soffit was filled using expanding grout (§3.1.3). Prior to application of the grout all concrete

surfaces were dampened again. Cubes were taken for strength verification of the grout by testing at 7 and 28 days.

4.3 Results

The column strengthening procedure has been successfully applied to 37 columns in Phase 1 of the Gateway redevelopment. The remaining six columns will be strengthened between April 2013 and 2015. An example of the entire process is shown in Figure 11.

Monitoring of the structure continued throughout construction works, and any significant movement would have triggered an automatic response and review of the works being undertaken. All movements recorded to date have been small and within expected boundaries.

5 LESSONS LEARNED

The assessment process described in §2 was a lengthy and often iterative process. As new information was obtained or corrected the analysis models were updated. A list of elements requiring strengthening generated near the start of the project reflected the information available, and was reduced through the early assessment stages as more survey information became available.

- Although there were a large number of original drawings and site photographs available to the assessment team, sorting and interpreting these was a challenge. Once collated the resulting data set has proved to be invaluable to the assessment team;
- None of the original project files, which may have included some commentary on the design and construction processes, were available. Additional difficulties were found due to a change in contractor part way through the

original construction;

- The experience has highlighted the importance of retaining project documents to aid future works. All information gathered in the process of this assessment has been stored with the intent that it can be easily accessed and used for any future assessments of the structure that may be required;
- In an ideal situation, the collation and interpretation of drawings, surveys of missing information, intrusive investigations and testing would all take place prior to any assessment activity;
- The frame geometry, with beams wider than columns, informed the strengthening possibilities;
- Surprises uncovered on-site added to the project complexity. Modifications to the structure that were not recorded on the available drawings had implications for the assessment process;
- In the assessment of the original concrete strengths testing showed that age enhancement of the concrete cannot be assumed without verification;
- The addition of a realistic construction sequence to the analysis models reduced the number of columns requiring strengthening, helping to reduce the impact of the strengthening works on the overall project. Using this method in concrete frame design could also add economy to new designs;
- The use of 3D Laser Scanning to produce an as-built computer model could feasibly be employed for a project of this scale and complexity.
- Further research is required to address the long-term behaviour of concrete encasements, including effects of creep and redistribution. In future projects it would be desirable to undertake long-term monitoring of the stresses in the

concrete encasement to establish load take-up into the new parts of the structural elements and compare this with the theoretical calculations.

- Upon completion of the strengthening works, further work will be undertaken to analyse in more detail the movements of the structures both before and after strengthening and to compare these movements with theoretical predictions.

6 CONCLUSIONS

This paper has described the process of assessment and subsequent strengthening for axial and flexural loads of a 1960s reinforced concrete sway frame structure. Understanding how a structure may have been designed is one stage in predicting the capacity of its members. For analysis, detailed information of the actual structure being assessed is the most important criteria and the material tests of concrete and steel (taken from demolished areas) proved invaluable in the calculation of member properties.

The strengthening of sway frame structures is complicated by geometrical difficulties of adding reinforcement across beam-column joints, which tend to be highly congested. The technique used was able to add both axial and flexural capacity without interference with the sensitive joint location.

The assessment process undertaken illustrates the importance of record keeping in construction. Much of the information used in the assessment had to be taken from on-site measurements, in a costly and time-consuming process.

Monitoring of the structure continued throughout construction works, with relative movement between each of the nine blocks being continuously recorded. Future site condition surveys may also be undertaken to monitor the strengthening works in more detail. This may include checking the top of column grout condition and noting any

cracking in the new encasements.

Options for strengthening at the joints of a sway frame are limited and complex. The chosen solution is unusual and effective. Although sections of the station were subject to planned closures during the strengthening work (which assisted in the design team's estimations of the initial dead load condition for each column) Birmingham New Street station has remained operational to rail passengers throughout the upgrading works.

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Figure Captions

Figure 1: Plan arrangement (left); Typical vertical section (right)

Figure 2: Beam and column arrangement

Figure 3: Proposed works at Birmingham New Street

Figure 4: Process of assessment at Birmingham New Street

Figure 5: Original construction sequence

Figure 6: Column (top) and Beam (bottom) assessment

Figure 7: Category 2 Element assessment

Figure 8: Summary of rejected strengthening options (Table 4)

Figure 9: (a) Summary of assessment process; (b) Illustration of the strengthening process

Figure 10: Shear transfer (a) Typical section (b) and section analysis (c)

Figure 11: The strengthening process

Typical Section A-A

Internal floor slab removed at Upper Retail level. New atrium and roof over Building 5

Stage 1

Stage 2

Stage 3

Stage 4

Typical column

Where $\gamma = 1.80$
for $UF = 1.00$

$$A_v = \frac{bs(v - \gamma p_v)}{\gamma p_{st}}$$

2. Shear Capacity CP114 (1957)

Interaction diagram

Beam $UF =$
Action / Capacity

Typical 'T' beam

1. Flexural Capacity

Where $\gamma = 1.80$
for $UF = 1.00$

$$A_v = \frac{bs(v - \gamma p_v)}{\gamma p_{st}}$$

2. Shear Capacity CP114 (1957)

$$\frac{A_v}{s} \geq \frac{T}{0.8x_1y_1\gamma p_{st}}$$

$$A_s \geq \frac{A_v(x_1 + y_1)}{s}$$

3. Torsion CP114 (1957)

$$(v + v_t) \geq 0.5\sqrt{f_{cu}}$$

4. Shear & Torsion CP114 (1957); BS8110-1 (1997)

Global Shear Typical 'T' Beam:

Overall increase
 $= (V_{o,P} - V_{o,E}) / V_{o,E}$

Local Shear Typical 'T' Beam:

Local increase
 $= (V_{L,P} - V_{L,E}) / V_o$

(check both ends)

Steel Casing

Discontinuous between floors

CFRP Plates

Continuous Between Floors

Encasement with Near

Surface mounted bars
continuous between floors

Steel Casing

Continuous between floors

(a) Shear Transfer

(b) Typical Section

(c) Section Analysis

1
Existing structure

2
Surface preparation
Shear keys added

3
Link reinforcement
added top and bottom

4
Vertical
reinforcement
added

5
Remaining links
added

6
Kicker installed
Water saturation of
column face

7
Formwork installed
Concrete poured
from bottom up

8
Formwork removed
High strength grout
to top for load
transfer