Retrofit of R.C. Beams & Columns by Finite Element Analysis

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Introduction

During the last week of March a few years ago, author was invited by a certain Company in an Indian city to review and advise upon measures to strengthen the ground floor and basement beams and columns of their six-storey building, consequent upon their raising two additional floors on top of their existing building, originally built eight years earlier.

Considerable work had already been carried out on strengthening the lower floor columns by their structural consulting engineer (SCE). The particular problem that initiated the author's involvement was the proposal by local experts to use Fibre Reinforced Polymer (FRP) bands to increase the shear capacity of the transfer beam B12 which carried the upper columns C9A and C15A down to its support columns C9 and C15. (Fig. 1.)

During his first visit to the site in March/April, author went round the entire building from basement to roof, examined the concrete and brick masonry, sometimes having the plaster removed for interior checking, did some light hammer tap tests, and took many measurements. He took numerous photographs for later examination, and made many freehand sketches and notes of critical details.

He held many discussions with SCE, and other building design and construction engineers, during which he obtained many clarifications on the history of the construction, and other technical matters.

Considerable planning and implementation of strengthening measures for the critical zone had been undertaken by SCE, experienced in reinforced concrete design and construction. Under his instructions, interior columns on certain lower floors had been strengthened by reinforced concrete jacketing, and further, interior finishing and full loading of the new floors were suspended till the problem was resolved.

Author not only endorsed his moves, but also proposed more stringent safety and enhancement measures to retain and improve the integrity of the building over the long term.

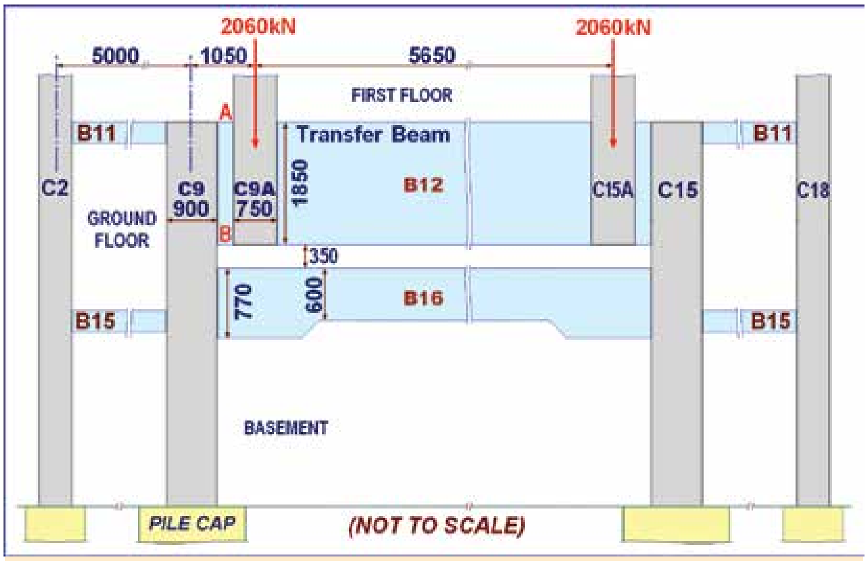

Figure 1: Transfer Beam B12 and Critical Shear Zone A

Definition of the Problem

From the information supplied to author, the problem could be defined as follows:

The original building was six-storeyed with ground floor plus five floors above it. (It is the practice in India that 'First Floor' is the floor immediately above the ground floor.) The structure had been presumably originally designed for the addition of one extra floor. The piles under the columns were designed with a higher factor of safety (F.S.), and hence would not pose any problem.

The source of the concern was at the ground floor level: Because of the regulations governing the entrance to underground car parking, internal columns C9A and C15A which had been spaced 5.65m (18.5ft) apart c. to c. from first floor upwards, had to be shifted laterally to C9 and C15 spaced 7.75m (25.4ft) in the ground floor and basement.

To enable this column offset, a deep beam (B12) was designed as a transfer beam, as shown in Fig. 1. The region between the two pairs of columns C9/C9A and C15/C15A was the "shear critical zone" marked AB in Fig. 1.

The total load received by columns C9A/C15A was stated to be 210 tonnes, equivalent to 2060kN (463Kips) each. This included the extra load of the purported casting of additional concrete approximately 150mm (5.9in.) thick on two floors during the original construction.

The average shear stress at the beam B12 support was 3.4MPa (0.49Ksi). Maximum permissible shear stresses for M-30, M-35, and M-40 concrete were 3.75, 4.10, and 4.40MPa (0.54, 0.60, 0.64Ksi), leading to factors of safety 1.1, 1.2, and 1.3 respectively, all less than 1.5 required.

Thus, already the transfer beam was in near-distress status.

SCE had proposed the solution of wrapping the shear critical zone AB with FRP to increase the shear strength.

As FRP was (in the local experience at that time) a novel solution, the owners wanted a second opinion and sought the author's guidance. This was the reason that various alternatives were being considered to strengthen the existing beam and provide for the safe transfer of the extra load down to columns C9 and C15 and thence to the pile foundation.

Constraints on Solutions

There was a gap of about 350mm (13.8in.) between the transfer beam and the beam at the ground floor level, somewhat less in places. SCE explained that the gap was intended to separate the lower beam (B16) action from the transfer beam (B12) action, so that the two beams could act independently to take the respective loads intended for them during the design stage.

While considering the different options, author was apprised of the following constraints on the proposed strengthening of the beam B12 and of the ground floor and basement columns:

1. No encroachment would be allowed into the 7m (23ft.) width of the driveway into the basement car park. Authorities would have to grant special permission for any intrusion. However, about 300mm (1foot) space beyond the stipulated limit had been left unused between the curved kerbs and columns C9/C15.
2. Any extension at the foundation level would have to be accommodated within the existing pile locations.
3. The ground floor had been let out to a corporate tenant until November. Transfer beam B12 formed part of one wall of this space, and hence only one face, namely the one on the driveway entrance side, could be accessed for any strengthening till then.

The non-availability of both sides of beam B12 logically led to the consideration of a temporary solution until November, with full loadings on the two new floors delayed until then, and a permanent solution after November.

For the temporary measure, SCE had suggested deepening of the transfer beam by providing additional concrete 350mm (13.8in.) wide and about 300mm (11.8in.) deep with suitable reinforcement in the pocket between the bottom of beam B12 and the top of beam B16, the gap being accessible from the raised floor level of the switch room, welding the new steel to the already existing rebars after chipping off the concrete around the existing rebars. The soffit of the added section was not to touch the top of beam B16, to avoid (according to SCE) transferring the high deflections from the superimposed load to the original beam B16.

Author's Approach to the Investigation

Material made available to the author

Available drawings of foundations, floor slabs and walls, columns and beams, kerb and ramp in basement, and staircases were supplied to the author. Author could not find any drawings of elevations of beam-column connections or of entire floors, and no cube strength reports for the concrete.

Additionally, the copy of a note from SCE outlining the shear stress situation in beam B12, and various measures he had taken and recommended to protect the structure, was also supplied. Author was able to tap SCE's experience from personal involvement in and site visits to the previous construction.

Author accepted the plan dimensions and beam and column cross-sections as his principal database. Not having any vertical elevations of complete floors, he made a few measurements on the spot, and tried to make educated guesses about the other vertical dimensions as and when needed.

For the material strength, author went by the statement on a drawing supplied to the author, with the prior stipulation by SCE that the concrete shall be of M-25 grade and conform to IS:456 revised, except B12, C9 and C15, concrete for which shall be of M-40 grade as per IS:456.

Author's assumptions and considerations

In order to offer a detached third-party view and an independent solution to the problem, the author decided not to base his recommendations on any undocumented or unverifiable properties of the materials or construction details. On the other hand, he took serious note of verbal comments based on personal experience (verifiable as necessary by site investigation) that might be to the detriment of the structural capacity or integrity.

Author decided to consider all options to carry the full maximum load for the entire building for the attributed area of the critical beams down to the foundations, rather than restrict himself to the shear strengthening of the critical zone. This decision was aimed to provide redundancy against unexpected contingencies in future.

In addition to the facts already stated, author accepted the following statements at face value as the basis for his further investigation:

1. The load on column C9A/C15A was 2060kN (463Kips).
2. The piles had a very high factor of safety (reportedly as high as 9).
3. The weight of extra thickness of concrete, about 150mm (5.9in.) placed on two floors to even up the stair landing and floor level was presumably included in the column load.
4. The original building had been designed for one additional floor, so that the current expansion by two floors would impose only one additional floor load.
5. The original roof, now to serve as the sixth floor, had all weather proofing removed and only a thin floor finish had been laid to offer normal floor live load capacity.

Author's conservative approach

Further, author also decided to be conservative in his approach, considering the dire lack of drawings, cube test results, and other documentation of the prior construction, and the overloading with extra thickness of two floors. Accordingly, he stipulated the following:

1. Not to take any cores from existing concrete, especially from the critical beam B12;
2. Not to resort to NDT of concrete, because of its inadequate reliability;
3. Not to take the strength of B12 and columns C9/C15 as M-40, but to restrict it to M-30;
4. Not to drill or hack into concrete in the critical zone, except as completely unavoidable, in which case, to keep it to the absolute minimum;
5. Not to expose main steel or links in the critical zone except as completely unavoidable, in which case, to keep it to the absolute minimum; and,
6. Not to weld to main steel or links in the critical zone, especially on the underside and in highly confined spaces, where the welding would not be precise or continuous, except as completely unavoidable, in which case, to keep it to the absolute minimum.

Author's reasons for the preceding conservative strictures were as follows:

Author was concerned that core drilling from the existing structure would be quite invasive. He opined that evaluations of concrete grade and strength of the original building would not lead to sufficient benefits to offset the risks involved in any invasive process, or to yield reliable results sufficient to justify liberal assumptions for future strengthening.

Even if core and non-destructive tests (NDT) were to be made on the existing concrete, cores taken outside the critical region would not assure the strength at the critical region.

As for NDT of existing R.C. buildings, current thinking was that the standard Ultrasonic Pulse Velocity (UPV) for testing concrete could estimate the concrete strength has only ±20% accuracy in lab specimens [Ref. 1], and ±25% accuracy in existing structures [Ref. 2].

Accordingly, even if a test estimated a core strength as M-40, it may be as low as M-30.

It was on this basis that the author recommended that no move be made to assess the grade or strength of B12, and instead, simply take it as M-30 in regard to permissible stresses, on the simple principle that any under-estimate thereby would be on the safer side.

While opinions might differ on the conservatism of above-mentioned recommendations, author opined that any intrusive procedures into long-standing heterogeneous materials with unsubstantiated properties would not be appropriate for a structure with the long life span desired.

Author's estimate of additional load

Obviously it would be sufficient to provide additional shear and/or alternative capacity to meet the excess loads caused by the extra loading from the two additional floors. This would also be consistent with the difference in age between the two concretes. At the same time, author wanted to check the final structure for the total loads also, to allow for time-dependent creep adjustments.

Author also decided to analyse the capacity of the existing structural elements to take loads to eliminate any likelihood of over-loading, especially in view of the two floors filled with extra concrete for levelling purposes.

Author calculated current shear capacity of the beam B12, as follows:

Permissible shear stress for M-30 with a factor of safety of 1.5 = 3.75/1.5 = 2.5MPa (0.36Ksi)

Capacity of B12 = 350(1850–100)×2.5 = 1530kN (344Kips)

Excess force to be taken by the additional elements = 2060 – 1530 = 530kN (119Kips)

As another criterion, assuming that the earlier load of five floors had already been transferred via B12 to C9/C15, the excess force to be taken by the prop would be the additional load from the two new floors, approximately pro-rated as (2060×2/7) that is, 588kN (132Kips).

The closeness of the two numbers reinforced the validity of the author's estimates.

Author decided to use a design load of 600kN (135Kips) for his retrofit solutions.

Single or two-step solution?

Although acceptable and even attractive in theory, author considered any temporary solution from May until November and a permanent solution thereafter as potentially raising fresh questions and creating additional problems such as:

- The seven-month dependability of the temporary solution,
- The postponement of full use of the additional floors till after November,
- The compromises to the permanent solution due to the temporary solution, and,
- The repeated disturbance to work (apart from the extra cost) after November.

Author was not in favour of a two-step solution, unless all options for an immediate permanent solution turned out to be impractical.

These considerations were author's philosophy while evaluating the situation and offering recommendations.

Pros and Cons of FRP for this Case

The initial solution proposed for shear strengthening by the owner's consultants was to apply FRP bands around the transfer beam at the supports to augment the shear capacity.

At the first meeting of the author with the owners consultants in March, recommendations were made for FRP shear strengthening of the region of load transfer of load from Columns C9A/C15A to the lower columns C9/C15 via Beam B12.

Author admitted that FRP had made great strides in R.C. strengthening in the last 10-15 years, and had considerable potential in the future for quick, neat, and efficient retro-fitting and repair of R.C. structures.

However, he raised the following concerns regarding FRP with particular reference to the owners situation:

1. The proposed bonding would be by U-strapping around the stem of the 'T' beam and not all the way around as in a column or rectangular beam, and hence the stress transfer in this case would be by surface bond of FRP with the decades old existing concrete and not by confinement within a FRP jacket.
2. More importantly, existing data on the in-service performance of FRPs went back only as far as 15 years at the subject time. While accelerated performance estimation techniques were under development, they were not ready as yet guidelines to extrapolate findings for long-life (100-year) structures such as the company building. [Ref. 3, 4.]
3. Those organisations using FRP for long-life structures had put in place periodic review and checking procedures, and subsequent planned maintenance and upgrading as necessary.

In this last concern, the author was influenced by the then recent tragedy of the collapse of the ceiling of the Boston tunnel in USA. In that case, the ceiling slab fell and killed a car-passenger, with the epoxy bond holding the slab to the rock anchor above having failed due to improper curing, discovered years later. [Ref. 5.] With no prejudice to the material or installation procedures by the vendors in the current case, author was sceptical if the FRP would be accessible to periodic inspection, and about the inspection procedures themselves over the years.

Although the FRP-vendor's representative offered to revert with their organisation's response to author's comments, the FRP vendor and installer would not be able to guarantee its performance for the life time of the building. Hence author recommended that because of the lack of data on the previous construction, and the unreliability of future checking and upgrading of the FRP as and when necessary, it would not be advisable to consider FRP for the beam further.

Risk Assessment

Laying all constraints aside initially, the author considered and evaluated solution options under two broad categories consistent with modern risk management, in which field he had become quite involved. [Ref. 6.]

Risk is generally taken as a function of two main factors mainly probability of occurrence of a mishap and severity of consequences if and when the mishap should happen. As the existing forces and future increased loads would be present continuously with narrow variations in the live loads, the probability was almost continuous, and the risk would be governed mainly by severity rather than in combination with probability. The severity of shear failure would be catastrophic.

In the hierarchy of risk management, risk control would first consider risk elimination and then, if not possible, attempt risk mitigation. As conventional design would advocate, and much thought had already gone into shear strengthening, author analysed shear strengthening solutions for risk mitigation first.

Shear Strengthening Solutions

Having decided not to use FRP, the options to increase shear resistance were as follows:

A. Increasing the existing R.C. section of beam B12 with a R.C. extension

(S1) Beam addition at bottom of Beam B12:

Proposed by SCE, a R.C. beam extension to be attached to the bottom of existing beam B12, as in Fig. 2 appeared to be the simplest solution

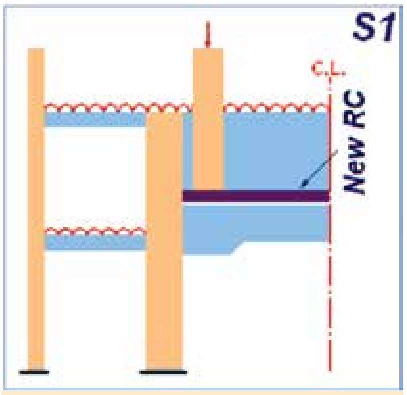

Figure 2: Increasing R.C. Section at Bottom

Both the above concerns (i) and (ii) would be aggravated by the fact that all access would be from only one side of the beam.

(S2) Beam addition at top of Beam B12:

The potential problems raised for solution (S1) could be avoided by simply moving the extra concrete from its suggested location below B12 to the top of the beam B12, as in Fig. 4.

In fact, the top of B12 was easily accessible from the owner company's

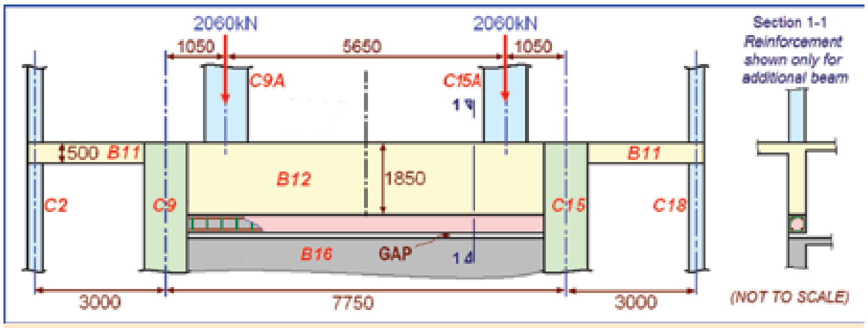

Figure 3: SCE's Solution for Additional RC Section

of all, if the only concern was to meet the nominal shear stress criteria of the IS Code.

His proposal was to cast a suitably reinforced R.C. beam 350×300mm (13.8×11.8in.) in the gap between B12 and B16, theoretically 350mm (13.8in.) but actually 320mm (12.6in.) in certain places. To integrate the new beam with the existing B12, he suggested exposing the links at the bottom of B12, and welding the new links to the old links. (Figure 3.)

Author raised the following concerns regarding the welding of thin links in the critical zone:

(i) Exposing the existing main steel and links by hacking the concrete would injure the present beam which was already in distress; and,

(ii) Welding rods in horizontal, vertical, and even up-nozzle positions within such a confined space may not be effective, may reduce their strength, and also cause embrittlement.

second floor office. The new R.C. beam, say 450×300mm (17.7×11.8in.) deep, may be laid on the exposed top. As it was needed only in the critical zone and for a short (development) length beyond it, the excess concrete may be tapered off to zero, after about 500mm (19.7mm) from the face of columns C9A/C15A as in Fig. 5, leaving a clear, level passage between them for people to walk by.

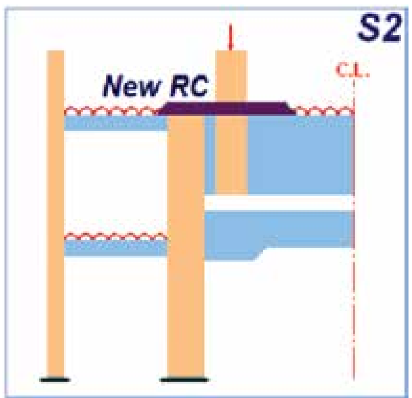

Figure 4: Increasing R.C. Section at Top

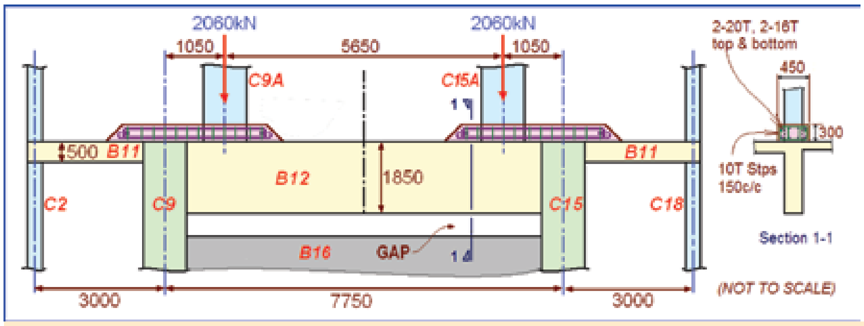

Figure 5: Author's Modified Proposal for Additional R.C. section. ('n'T = 'n' mm dia.)

In this top position, hacking of concrete, exposing of links, and even tack welding of old steel to the new steel would not be a problem. Welding in an awkward position in the extremely confined space – which was the main reason for the author's objection – would be completely eliminated.

While this alternative was more attractive than the first proposal, author still had the following reservations:

(i) As in the first proposal, exposing the existing main steel and links by hacking the concrete would injure the present beam which was already in distress; and,

(ii) The top additional longitudinal rebars would have to traverse the first floor columns outside the column profile, and then converge on both sides to the profile of the beam B12, which would reduce the effectiveness of the augmentation.

(iii) Possibility of embrittlement of steel during welding would remain.

Solutions S1 and S2 would not involve any need for access from the tenant's space.

B. Jacketing of the critical zone of beam B12 with R.C.:

(S3) Full jacketing of critical zone of Beam B12:

This would have been the best solution for shear strengthening, utilising conventional materials (concrete and rebars) and the long-time established practice [Ref. 7] of full jacketing around the rectangular portion of the 'T' beam B12 as in Fig. 6. This would be preferable to FRP jacketing for reasons already discussed.

Holes should be drilled into the flange slabs of the 'T' for the new shear links to go through from slab top. Hammer drills may be used for the drilling with a low hammer impact, but author would have preferred high-speed, non-impact rotary drilling.

The jacket would be of M-40 concrete with low shrinkage, high-strength concrete 12T or 16T rebars, around roughened concrete surfaces along top, bottom and both sides of the existing beam, as well as outside of column C9/C15, from outside to outside of the two pairs of columns plus extensions of about 150mm (5.9in.).

Tentative design for the R.C. jacket would be 75-100mm (3.0-3.9in.) thick concrete, with 2-legged 16T bar links spaced 100mm (3.9in.) apart in the critical zone, and longitudinal steel of 4 -16T bars at top and bottom. The new steel may be welded to each other to form a cage, with one or two horizontal bars along the vertical sides of the beam stem also.

(S4) Partial jacketing of critical zone of Beam B12:

This 'U' shaped partial jacketing of the beam stem (Fig. 7) was another proposal considered by SCE and the FRP vendor. Although the FRP solution had been discarded, a conventional R.C. partial jacket of similar shape would be feasible, with the vertical

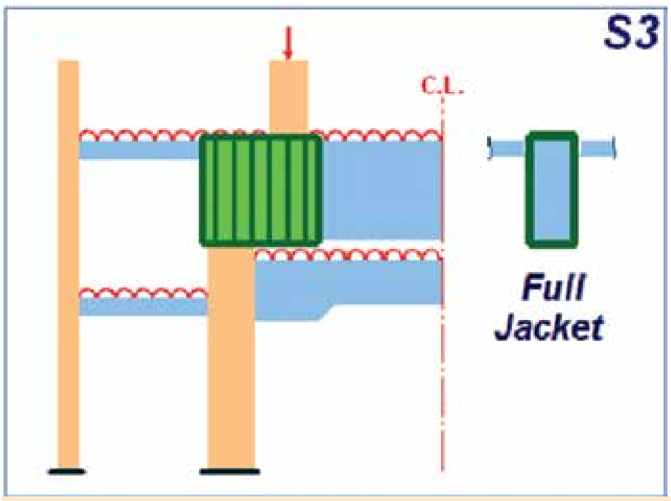

Figure 6: Full R.C. Jacketing of Critical Zone

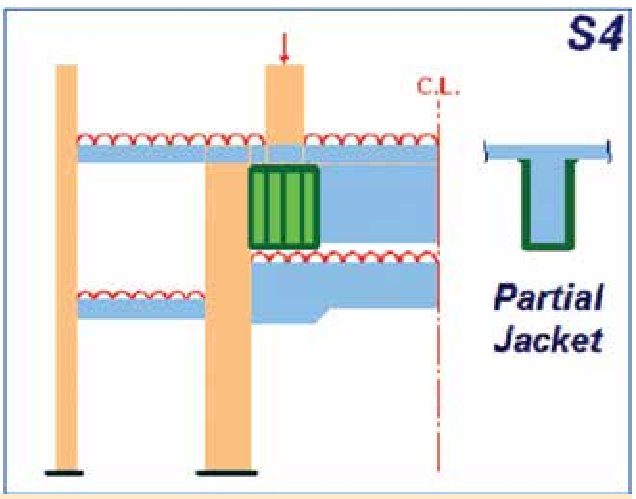

Figure 7: Partial R.C. Jacketing of Critical Zone

stirrups bent and inserted into drilled holes on both sides of the 'T' stem just under the slab, for about 80mm (3.2in.) penetration. This technique has also been in common use for a long time, although acknowledged to be much less effective than wrap-around jacketing.

Design would be similar to that of the full jacketing, with extra precautions for proper anchoring of the links at their top bends.

Both the S3 and S4 solutions would require access to all four surfaces of B12. The tenancy constraint would postpone this permanent solution to November, which in turn would require an interim solution. As access could not be obtained even for about two weeks in April/May, author relegated this to low priority.

To this, SCE proposed that half the jackets could be applied on the accessible switch room side of B12 immediately, and the rest of it applied on the tenant side in November.

Author recommended against any one-sided solution, because the lack of symmetry during such a temporary solution would result in the introduction of torque in the beam during the intervening seven months, which might adversely affect the load-carrying capacity of the beam. The subsequent integration of the old and new retro-fits might also pose problems.

Shear Elimination Solutions

It was while owner engineers were mulling over the shear mitigation options and their implementation problems that author explored the higher hierarchy risk control solution of risk elimination, in this case, elimination of the high shear in the critical zone.

Risk assessment often helps to reach out-of-the-box solutions. In this case, the principle that risk elimination was superior to risk mitigation in the hierarchy of risk controls prompted the author to re-examine the load transfer mechanisms and the involved structural elements.

It soon became obvious that the 320-350mm (12.6-13.8in.) gap between B12 and B16 could be a fortuitous opportunity to eliminate the shear problem altogether, certainly for the additional loads, and to a considerable extent – and in due course with creep, to relieve much of the shear stresses in the older structure also.

As the high shsear was due to the column offset, the author wondered, why not transfer the compression in columns C9A/C15A to the basement columns C9/C15 directly as compression rather than as shear to the ground floor columns?

As a prerequisite to this kind of solution, the column loads from C9A/C15A had first to be transferred via the ends of the transfer beam B12 to the top of the ends of the beam B16, by means of a R.C. filler piece, which the author referred to as a 'pad'.

Special considerations on the R.C. Pad

The pad between beams B12 and B16 would be a very critical element, as it must transfer all the fresh loads from B12 down to B16, and in due course, more and more of the former loads also.

Further, only two out of the six sides of the pad would be accessible – namely the front and one end – bringing in complications in the casting process. But these complications could be overcome with minimum trauma to the existing beams B12 and B16 by careful planning.

Hence only the best concrete and workmanship must be used, with:

- ♦ High strength, low shrinkage cement,
- ♦ Graded aggregates of maximum size limited to 12mm (0.47in.),
- ♦ Fairly dry mix, with least water-cement ratio,
- ♦ Well-vibrated concrete laid in small layers and poked into all corners,
- ♦ After initial packing and vibration in the gap, sealing all exposed sides and injecting rich grout, if possible and as necessary, to fill off all the voids; and,
- ♦ Proper curing for at least three weeks.

Reinforcement could be 4-20T bars at top and bottom, 1-20T bar on each side at mid-depth, and 10T links 150mm (5.9in.) c. to c.

A concern was raised by SCE whether the pad's insertion might transfer the full force of 2160kN (486Kips) and corresponding deflections to the lower beam.

The answer to this was of course that the original load had already caused all the needed deformation to the beams and columns, and the pad would convey only the additional deformations arising from the new loadings to the connected elements.

Further, being confined to the end segments, the pads would not increase the critical mid-span deflections of beam B16.

Check for the M-25 vertical compression capacity of beam B16 was made to ensure that the load transfer from the R.C. pad would not overload it.

Preliminary evaluation of shear elimination options with the pad yielded two approaches as follows:

(E1) By a corbel projecting from basement columns C9/C15 plus jacketing; and,

(E2) By inclined prop from pile caps.

Like the preceding solutions S1 and S2, this too would not involve any intrusion into existing structural elements or rented space.

(E1) Corbel under beam B16 and jacketing of columns C9/C15:

Corbel has been a classical solution for offset loadings from time immemorial, and could have been adopted during the original design itself. At this stage, it would still be possible, although somewhat complex to implement, as indicated in Fig. 8.

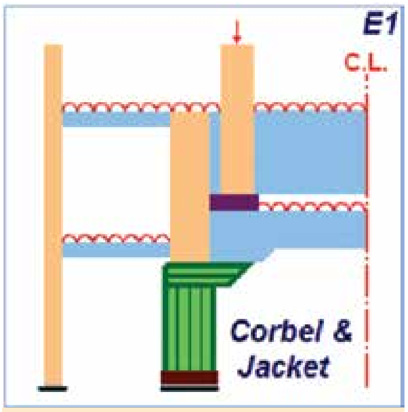

Figure 8: R.C. Corbel and Jacketing on C9/C15

The basic problem could be simply stated, with reference to Fig. 9, as a cantilever delivering a shear of 600kN (135Kips) and a moment of 300kN.m (443Kip.ft). There would be a downward force of 873kN (196Kip) at the inside face and an upward force of 273kN (61Kip) at the outside face of the basement column. The upward force would not be of any concern because it would be amply over-compensated by the massive compression from the existing storeys.

All that had to be done now was to transmit the effects to the bottom of the basement columns C9/C15, subjecting the pile cap only to compression but no tension or moment.

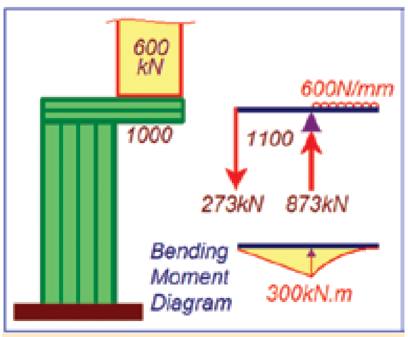

Figure 9: Corbel Solution

At the same time, in this solution, the shear problem had now migrated from the ground floor to the basement, and we would still have to resist the new 600kN (135Kips) shear at support of the corbel near the top of the basement columns. However, the difference would be that the shear design would be under better control.

A R.C. corbel solution with M-40 concrete with permissible shear stress of about 3MPa (0.44Ksi) would require a corbel cross-section of (600000/^{3}), i.e. 200,000mm^2 (310in^2), possibly by a 400mm×500mm (15.7×19.7in.) deep section.

On the other hand a steel corbel would require less than 5% of the concrete section, i.e. about 10,000 mm^2 (15.5in.2). Steel would be a simpler and faster solution from the bending moment consideration also.

However, steel fabrication would demand more skilled technology and would face more interfacing problems than R.C. with the basement columns.

(E2) Inclined support from under columns C9A/C15A to base of columns C9/C15:

The inclined prop was by far the most logical solution under the prevalent circumstances to take the vertical column loads directly down to the foundation, as shown in Fig. 10.

The prop may be designed for an axial load of about 700kN (157Kips), to resist a vertical component of 600kN (135Kips).

To resist the overturning component introduced by the inclined prop, two or three horizontal straps to take about 200kN (45Kips) must be introduced. Resulting shear in the columns would be very small.

The inclined prop might be effectively provided by steel. Standard

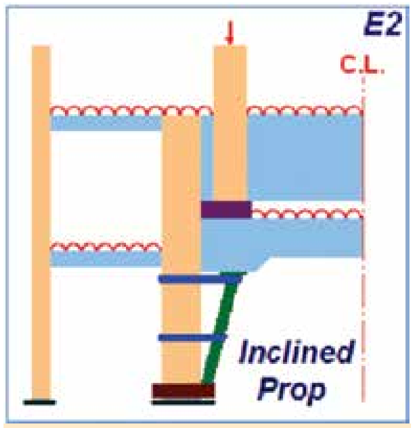

Figure 10: Inclined Support under Pad

rolled or built-up sections might be used for the prop, and rolled channels used for the straps, with the channel backs anchored to the sides of the column C9/C15 and/or their outside ends shackled to the far side of C9/C15.

Final Proposal

(E3) Full tapered jacket for basement columns C9/C15:

At that point, author decided to further develop the inclined prop solution into a tapered jacket for the basement columns C9/C15, with the appropriately increased width at the top and a minimum jacket thickness beyond the column width at the bottom. This would become the shear elimination solution E3, shown in Fig. 11.

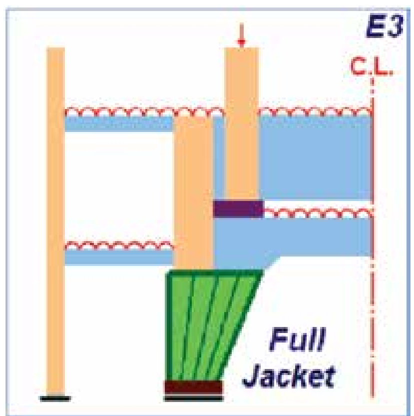

Figure 11: Full Jacket for Basement Columns

A critical evaluation of the clearances in the basement showed that there was available a clearance of about 300mm (11.8in.) beyond the required clearance of 7m at the basement. This would be sufficient for any jacketing of the basement columns at the base, over the pile caps.

This solution eliminated most of the deficiencies and difficulties of other solutions and met all required criteria. It would certainly take care of the new 600kN (135Kips), and in due course, may redistribute some of the earlier high shear stresses as compression into the lower reaches.

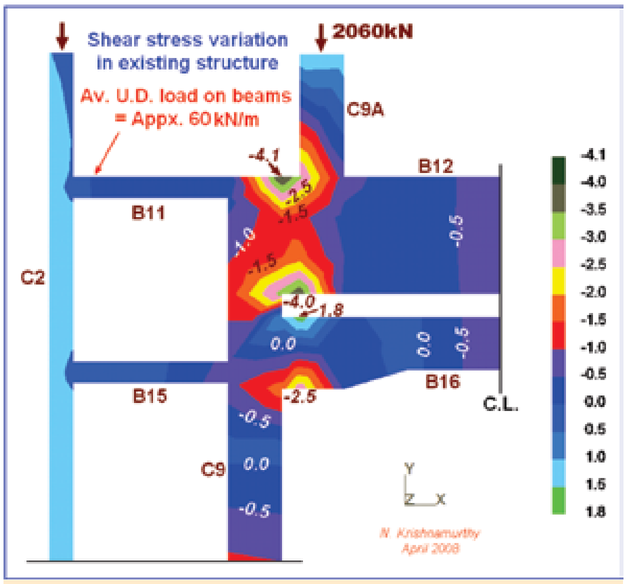

Figure 12: Shear Stress by Finite Element Analysis

Finite Element Analysis

Thus far, all solutions had been conceptual, and the time had come to analyse and confirm the assumptions and predictions for the various cases proposed.

Author modelled the critical zone and main columns and beams for the floor strip enclosing the critical zone with a simple 2D finite element model using GTSTRUDL, through which he could analyse various scenarios, focussing mainly on the critical shear stress distributions, and converging gradually on how the compression in C9A/C15A could be diverted directly to columns C9/C15 offset from the former by 1050mm (41.3in.).

To justify (to himself as much as to others) the feasibility and accuracy of his many hypotheses and retrofit solutions, and to separately identify the effects of the various proposed modifications, the author modelled the critical zone and surrounding regions by finite elements. Author analysed by FEA all the proposals mentioned herein, and many more besides.

The advantage of finite element analysis (FEA) is that it could give the variations of deformation and stress within the actual structure, for irregular shapes, loadings, and support conditions, as the author had learnt and utilised in his numerous previous applications. While quantitative interpretations of FEA demand expertise, qualitative distributions could highlight hidden problems, could be quite useful in understanding the actual structural behaviour, and in an evolutionary manner, point to potential improved solutions.

Figure 12 shows the region and configuration analysed by the author with FEA.

Apart from the given loading of 2060kN (463Kips) on C9A/C15A, author took the distributed load for each beam pro-rated from the total load for the attributable floor width of 5.65m (18.5ft.) for the beam line B11-B12-B11, computed as about 60kN/m (4.1Kip/ft.).

Figure 12 also shows the shear stress distribution of the structure as was. The conventional view that the R.C. beam shear stress follows a parabolic distribution from compression edge to the Neutral Axis (NA), and then remains constant beyond NA obviously was not quite correct in the special case of the deep beams in our critical zone. Maximum stress hotspots were seen to exist at both top and bottom junctions of the beam with the columns.

Before analysing various cases, he calibrated his final computer model by comparing critical (average) shear stress values between the finite element analysis (3.1MPa, i.e. 0.45Ksi) and direct computation (3.4MPa, i.e. 0.49Ksi).

Further refinement could narrow the gap, but would have taken considerably more time and resouces. Author considered the present accuracy as adequate for the present purpose, and suggested from prior experience an extra 15% allowance to compensate for under-estimation and stress concentrations. This accuracy would be adequate under the uncertain history and complex behaviour.

Results of Finite Element Analyses

From scores of finite element analyses the author carried out, Table 1 shows results for a few typical cases including the beam strengthening solutions, and the pad-column jacket case of Fig. 11.

The results are arranged in the order of decreasing maximum shear stress, that is, in the order of increasing factors of safety. The quantitative results may be taken to be approximate to about 15%.

Table 1 of FEA results confirms the anticipated positive effects of various retrofits proposed by the author, each enhancement progressively decreasing deflections and stresses and increasing factors of safety.

Table 1: Results of Finite Element Analyses

	Item→	Deflection	Beam Bending		Column Compression		Shear Stress						
No.	Location→	B12 C.L.	B12	B16	C9A	C9	Top	Middle	Bottom	Average	Factors of Safety		
	Model ↓	mm	MPa	MPa	MPa	MPa	MPa	MPa	MPa	MPa	M40	M30	M25
1.	As Is	3.0	-5.0	3.9	-12.8	-10.0	3.5	1.5	4.4	3.13	1.40	1.20	1.07
2.	Top Beam(S2)	2.9	-5.0	3.6	-13.7	-9.8	3.0	1.2	4.2	2.80	1.57	1.34	1.20
3.	Bottom Beam(S1)	2.6	-4.3	3.5	-12.1	-10.0	2.8	1.2	4.1	2.70	1.63	1.39	1.24
4.	Pad Only	2.1	-3.0	3.9	-12.0	-12.0	2.6	1.3	3.7	2.53	1.74	1.48	1.32
5.	Pad & InclProp(E2)	2.1	-2.9	3.6	-11.5	-11.5	2.4	1.3	3.5	2.40	1.83	1.56	1.40
6.	Pad & Corbel(E1)	1.7	-2.3	0.4	-11.0	-9.5	2.4	0.8	2.2	1.80	2.44	2.08	1.86
7.	Pad & ColJack(E3)	1.2	-1.9	1.2	-11.0	-5.5	2.3	0.8	1.7	1.60	2.75	2.34	2.09

1mm= 0.039in., 1MPa = 0.145Ksi

Case 1 highlights the fact that the clients building had already been pushing the envelope in regard to strength and safety. Case 4 demonstrates the dramatic effect of improving M30 factor of safety (almost) to the desired 1.5.

The permissible shear stresses for M-40, M-30, and M-25 concrete, with a factor of safety 1.5, are 4.40/1.5, 3.75/1.5, and 3.4/1.5, i.e. 2.93, 2.50, and 2.27MPa (0.425, 0.363, and 0.329Ksi).

For the "As Is" Case 1, the computed average shear stress is already 3.1MPa (0.45Ksi), not meeting the 1.5 safety factor required. Author found that the maximum shear stresses (from Table 1) may be much higher, almost equal to the maximum permissible value for M-40 concrete, resulting leaving no margin for safety.

Calculated average compressive stress in original C9A/C15A was about 10MPa (1.45Ksi). But FEA, which reflects the effects of not only the moments in the columns but also the stress raisers in the section changes, shows a maximum of 12.8MPa (1.86Ksi), which is more than the maximum permissible value of 11.1MPa (1.61Ksi) for M-25 concrete!

Again, while the calculated average compressive stress in C9/C15 was about 7MPa (1.02Ksi), the maximum could be as high as 10.0MPa (1.45Ksi) from FEA.

Luckily, maximum bending stresses are well within permissible values.

From Table 1, it is clear that the shear strengthening solutions (S1 and S2) would not be as effective in improving the shear factor of safety as the shear elimination solutions (E1, E2, and E3). Actually the inclined prop (Case 5) and the corbel (Case 6) solutions would have provided the desired factor of safety, except for the implementation problems. Case 7 of full column tapered jacket would not only be quite practical but also give the highest factor of safety.

Again, in regard to maximum compressive stress, Table 1 shows that the inclined prop solution Case 5 marginally exceeds the allowable 11.1MPa (1.60Ksi) for M25 concrete, but the corbel solution Case 6 and the full jacket solution Case 7 would effectively bring it down.

Figure 13 of shear stresses of a slightly different FEA model dramatises the improvement from (a) the existing structure, (b) by the addition of the pad, and (c) by the full-height jacketing of the basement column C9. In this model, the shear stresses are higher than in the extended model shown earlier, and in fact this was the reason the author went over to the extended model.

Comparison of Fig. 13 (a) and (b) shows how the pad would eliminate the 'hot spots' at the top and bottom corners of beam B12. However, the shear problem would be transferred to the lower beam B16, at the junction with the basement column C9, as was to be expected.

With the jacketing of the basement column C9, this shear hot spot has also been eliminated. The small dark spot at the top left corner of the junction of column C9A and beam B12 is a minor and local effect, not a problem in real situation.

After much deliberation, the pad-full jacket solution of Case 7 (S3) was voted as the optimum solution. Author then proceeded to develop the detailed design for the retro-fit.

Implementation Recommendations

Author next considered the practical problems of implementing the pad and jacketing solution. He reiterated his concerns as follows:

(a) The pad, while theoretically essential for all load transfer between B12 and B16, would depend entirely on its perfect installation for its effectiveness. In practice, it might be only partially effective, beginning

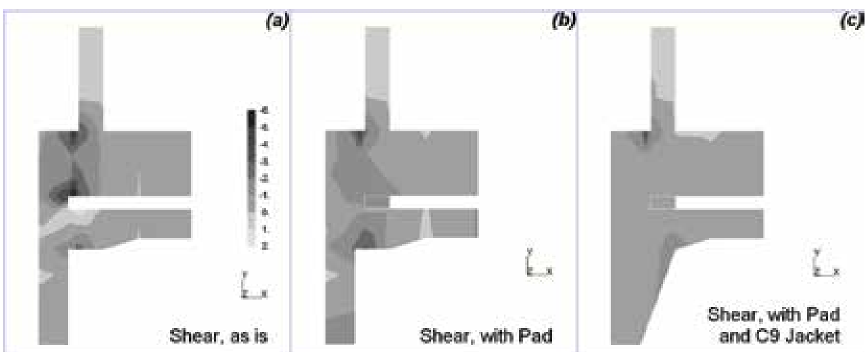

Figure 13: Improvement with Different Modifications to Existing Structure

to take load only after some deformation, however small.

(b) Both pad and jacket were add-ons to existing columns to handle the shear critical zone between C9/C15 and C9A/C15A. Their bonding with the existing concrete would be uncertain in quality and difficult in execution, as already described.

(c) As deep gouging and exposure of main steel and stirrups was not a safe procedure in the present structure, bonding between new and existing concretes would have to be achieved by means of jackets going round the pair of columns C9/C15 to their full heights. Scarification of the existing concrete surfaces, with possibly a few rebar dowels from the jacket reinforcement set into the existing concrete should suffice to provide reasonable bond.

The links and the rebars may be tack-welded (without burn-through) where they cross. For links across wide expanses of existing concrete, occasional stud-anchors may be provided, penetrating not more than 50mm (2in.) near estimated neutral axes positions or centre of members.

All this could be achieved right away without interfering with the tenant in the Ground Floor, and extending into the driveway by only minimal, tolerable and permissible amounts.

Precautions in surface preparation

For any casting of fresh concrete on existing concrete, the old surface must be roughened ("scarified"), not gouged.

Because of the already high stresses in the critical section, any surface chipping must be done very gently. Most engineers would consider chipping away the cover of R.C. members as standard practice, but author wished to emphasise the following precautions as recommended in sensitive cases.

- ♦ Use of hammers less than 9kg (19.8lb.) is recommended.
- ♦ Impact hammers must be set to the lowest energy level.
- ♦ Only the superficial surface mortar must be removed, exposing the first sand-cement layer of aggregate for good bonding, without dislodging any large aggregates.
- ♦ After wetting the scarified and wire-brushed surface enough for full absorption (but not dripping wet) a thin coat of rich cement paste should be brushed on just before the fresh concrete was laid.

Sequence of casting-related work

- ♦ Scarify all the contact surfaces, completing all the chipping and wire brushing.
- ♦ Erect the rebar cages, tack welding at crossings if possible.
- ♦ Cast the pads on both ends of the space between B12 and B16, and cure for three weeks.
- ♦ The two basement column jackets could be cast simultaneously by two crews. Local practice may dictate the details.
- ♦ Use M-40 concrete by way of abundant caution.

Impact on tenants

The tenants may have to be informed and reassured about safety and subsequent cleaning up and retrofit, regarding the following:

- ♦ Noise of concrete hacking and vibration for a few days (which they were already experiencing with the present construction);
- ♦ Some staining and wetting of the wall behind B12 during the casting of the pad;
- ♦ Some plaster cracks or light debris drop during the scarifying of beam and column sides;

Suggestions for future archiving and management

For future building management, author recommended the following additional steps:

1. Document all available and recoverable data on soil profile, foundation conditions, material properties, load estimates, design assumptions and procedures, and on-site ad-hoc decisions, events, and implementations from inception to completion of the original construction.
2. Debrief engineers, supervisors, suppliers and foremen connected with the past construction. Recover as many facts and figures from SCE. Archive any and all reports, sketches, photographs etc. that can be gathered from everyone connected with the project.
3. Gather and archive any and all documents, calculations, drawings, photographs etc. relating to the two-floor expansion, and enhancement of the columns and beams to take the extra loads of the two additional floors, including all the material received from the author and minutes of all relevant discussions and decisions.
4. Conduct a three-dimensional frame analysis of the entire building with the final dimensions and properties of all members, and archive, in both hard copy and soft copy, all details of the computer model, assumptions, and all results. Author predicted that in the next twenty five years, the design scene in India would change to what it already was in the advanced countries, namely to fully computerised Building Information Management. This analysis might be expensive at present, but at least, if the other recommendations were implemented, a future team could conduct the needed analysis.

Author believed that with all the preceding recommendations implemented, the company would be satisfying the regulations, as well as providing for direct transfer

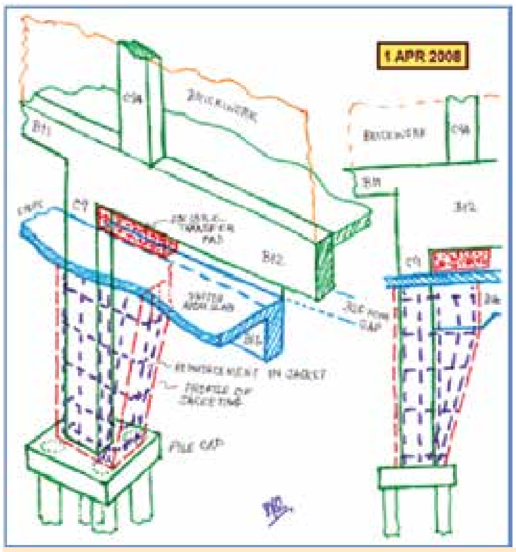

Figure 14: Schematic for the Pad and Jacket

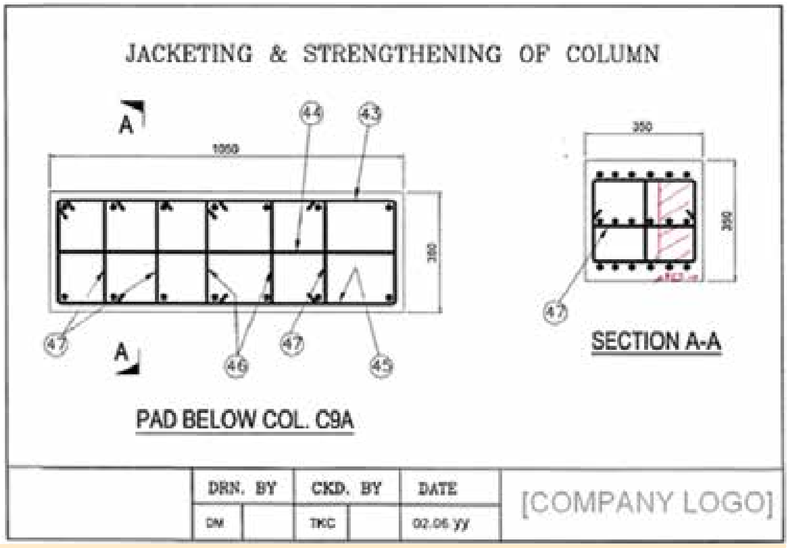

Figure 15: Pad Drawing

of the excess shear to the pile cap, with sufficient redundancy to meet reasonable contingencies in the long term.

Construction Process

Glimpses of subsequent actions to implement the author's design are given through figures and photographs. The two pads were cast first, and allowed to cure completely, and then the basement column jackets were cast. Pile cap top reinforcement was exposed minimally around the basement columns and the new rebars of the jacket were bonded to them.

Figure 14 shows one of the many schematics that author drew for the company's design department to prepare the drawings. Author also sent schematics for the formwork for the basement jackets.

Figure 15 shows the design drawing for the pad, and Fig. 16 shows one of the final drawings of the basement column retrofit design, with the author's notes for construction guidance.

Figure 17 depicts a photo of the jacket reinforcement for C9 before the formwork was put up for casting.

As is the convention in the professional consultancy environment in which the author is currently practising, author also sent formwork details and safe work procedures (SWP) for the concrete casting. However, being familiar with prevalent conditions in India, he fully expected that only the concepts would be adopted and details and methodologies could be different, dictated by local conditions, culture and resources. He was also aware and properly appreciative how good quality work can be achieved by local craftsmen under proper supervision and incentives.

Fortunately, the engineer supervising the rebar fabrication and concrete casting was a civil engineer

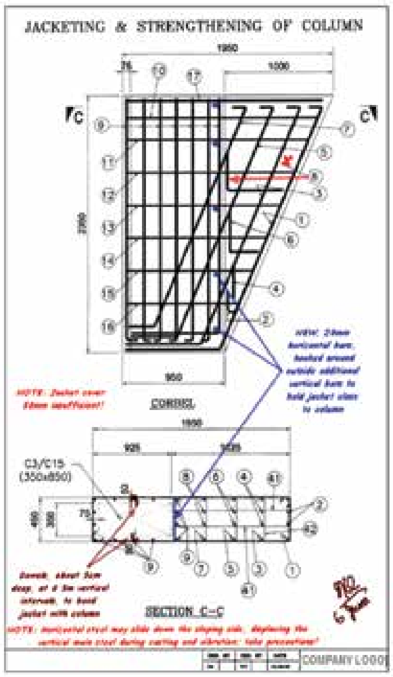

Figure 16: Jacket Reinforcement Drawing

Figure 17: Jacket Reinforcement for C9

Figure 18: Column C9, (Left) Before Retrofit, with Retrofit Jacket Outline Superposed, and (Right) After Jacketing

and veteran in concrete construction. Equally important, he was fully familiar with what the author was trying to communicate through his risk assessment, SWP, and other relatively modern approaches to construction management. Author knew that his design was in safe hands and his instructions would be followed fully in spirit if not to the letter.

Figure 18 shows photographs of column C9 before and after the retrofit.

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Acknowledgment

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