Fallingwater Is No Longer Falling

By Gerard C. Feldmann, P.E.

Fallingwater, Frank Lloyd Wright's masterpiece design, was a radical departure from the typical residence of the time. The reinforced concrete construction with long multiple cantilevered elements was ahead of its time. The problem was a design that left the main cantilevered beams under-reinforced. This under-reinforcement combined with the high compressive stresses and an ongoing creep condition in the concrete had caused deflections as high as 7 inches on a 15-foot cantilever. The structure could collapse suddenly under such conditions. Because of this condition, the structure needed to be shored as the permanent design was started. An external post-tensioning system was selected to strengthen the cantilevers and mitigate the deflections.

A STRUCTURE article, from July/August 2001 by Robert Silman and John Matteo, described the history of the structure, the monitoring of deflections and the analysis of the tiered cantilevered system. Fallingwater is currently owned and maintained by the Western Pennsylvania Conversvancy (WPC). The repair work was done during the winter of 2001/2002. The structural consultant for the WPC was Robert Silman Associates, P.C. (RSA). The Di Salvo Ericson Group (TDEG) was the post-tensioning consultant for RSA. TDEG assisted with the evaluation of the existing conditions and guided the post-tensioning design. The post-tensioning contractor was VSL and the repair contractor was Structural Preservation Systems, Inc (SPS).

Figure 1: Probe hole to allow inspection of cantilever beams

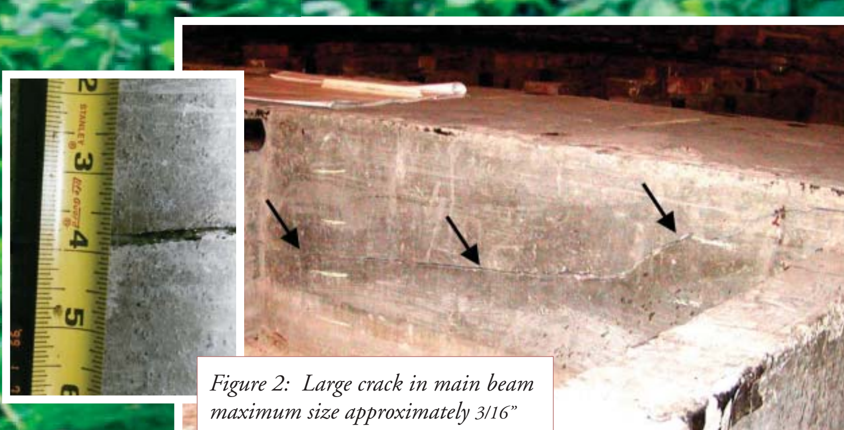

Figure 2: Large crack in main beam maximum size approximately 3/16"

Testing and Analysis

The pre-design non-destructive testing and probing established some of the parameters for the preliminary design. The initial probing was limited to non-destructive radar testing with a small hole and limited access under the floor. The floor was opened up further to allow more intensive observation and testing. (See *Figure 1*) Cores were taken from the beam to establish a compressive strength for the concrete. This work also established the amount of reinforcement in the beams, which had been a matter of contention during the original construction. (See *Robert Silman's sidebar*) It was during this probing that the first surprise occurred. A crack was discovered that effectively created a plastic hinge on the center cantilever beam at the face of the pier below. (See *Figure 2*)

The structure had been previously modeled to evaluate the original design. The final design of the external post-tensioning required a continuing refinement of the model to determine the overall effect of the large forces that were to be applied to the existing structure and the newly discovered "hinge". The culmination of this process resulted in the final design.

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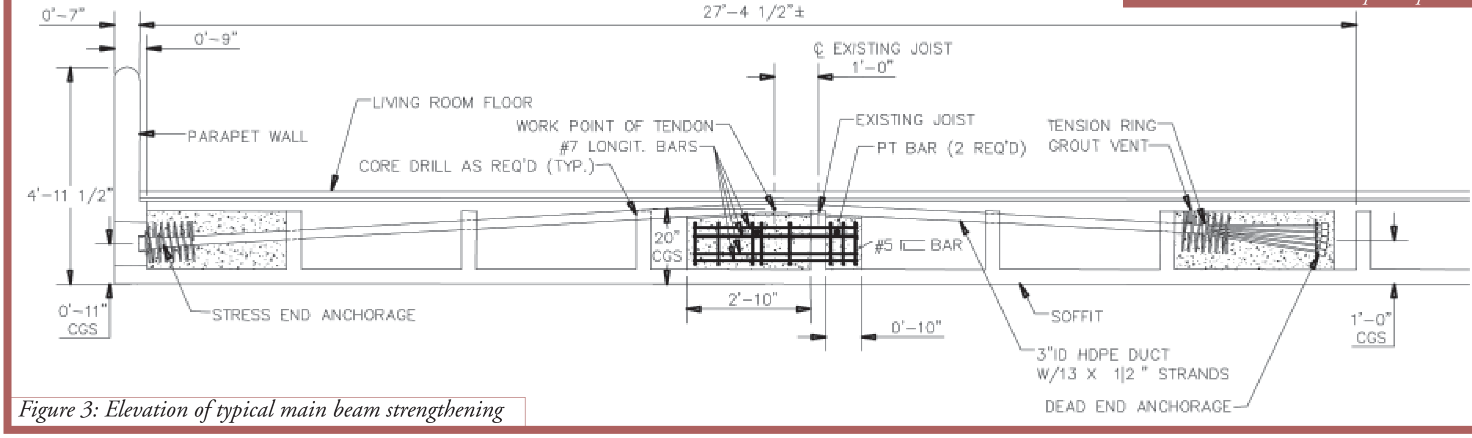

Figure 3: Elevation of typical main beam strengthening

Final Design

The discovery of the hinged beam added to the already complicated design. The exterior post-tensioning system was chosen, but the amount of post-tensioning force to be added needed to match the different demands of the individual cantilever beams. It was decided early in the design process that the existing deflections of the cantilevers could not be removed. The 60+ years of concrete creep and the suspected yielding of the reinforcing steel prevented this. The task at hand was to prevent any further deflection. This was to be done by reducing the dead load stresses in the reinforcing steel approximately 10 ksi. This level of force would create enough vertical forces to lift the cantilevers slightly.

The exterior post-tensioning needed to lift both levels of cantilevers, as the upper floor rested on the lower cantilevers through structural window mullions. The design forces chosen to reduce the stresses of the main cantilever created a problem with these mullions. The vertical component of the post-tensioning force at the end of the cantilevers would increase the mullions axial forces to near buckling. The forces involved in the strengthening system could not be reduced, so, after discussion with the WPC on the aesthetic impact, the structural mullions were beefed up by welding additional plates to increase the axial capacity.

A bonded, multi-span system was used for the main beams and a monostrand system was utilized on the joists. The final strengthening design resulted in the following tendons and forces on the three main cantilever beams: West and Center: 26 strands, 780 kips; East: 10 strands, 300 kips. See *Figure 3* for a typical elevation of the strengthening system. The post-tensioning stressing, diverter and dead-end blocks for the main beams were

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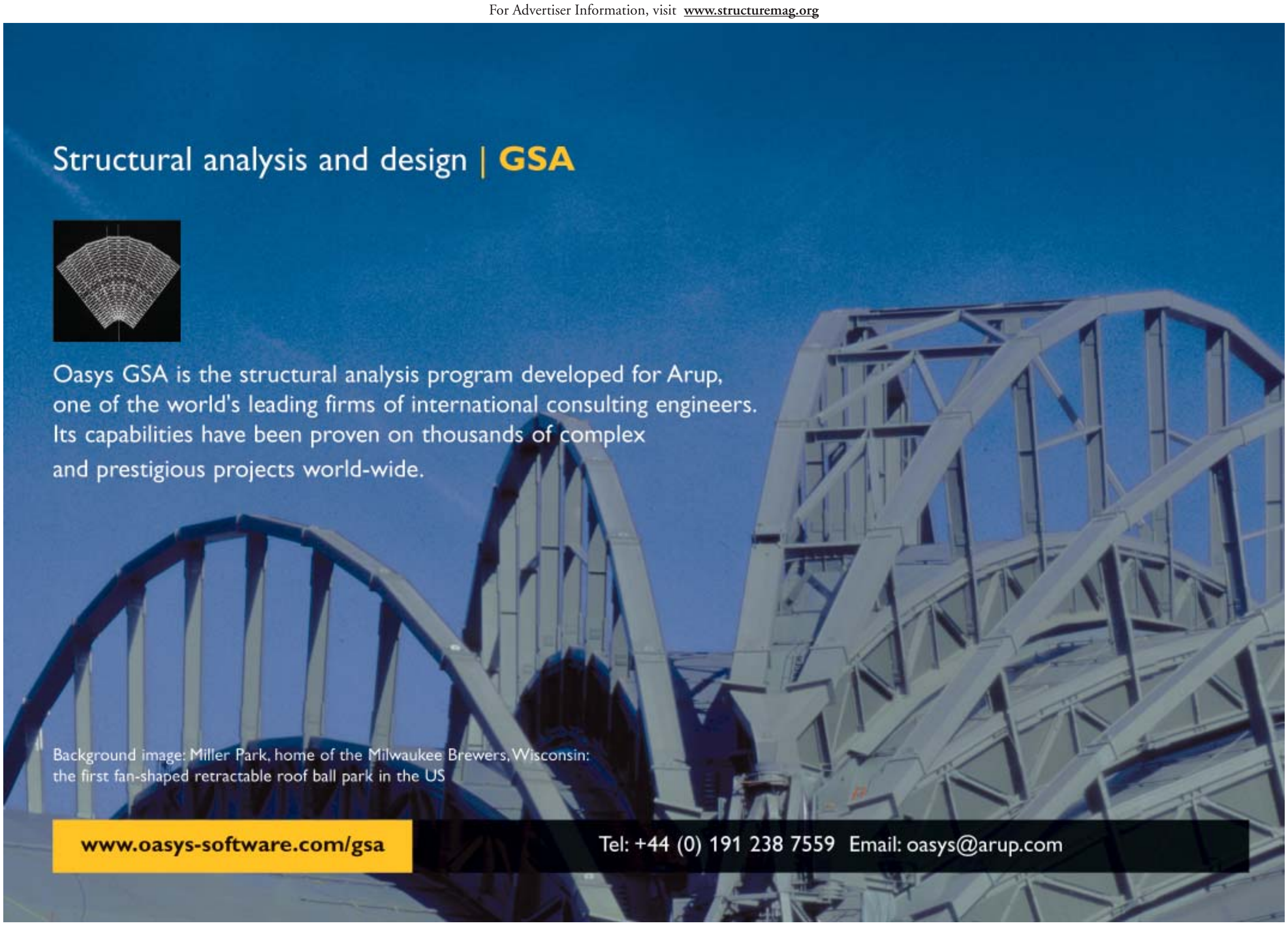

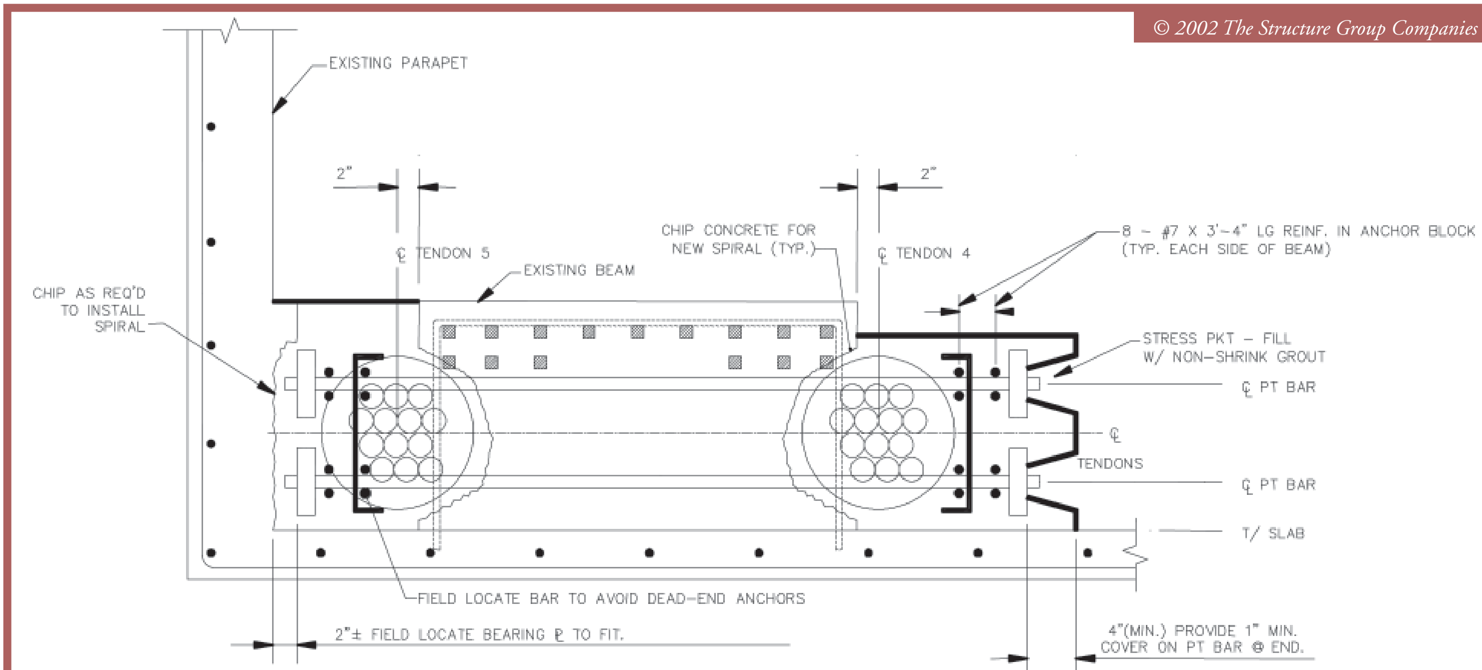

Figure 4: Typical stressing end detail showing transverse post-tensioning

also transversely post-tensioned to prevent splitting from the large horizontal forces. See *Figure 4* for the anchor block detail. The monostrand force was 43 kips per strand. The bearing ends of the cantilevers were more securely anchored to the piers below by coring and grouting rock anchors since the existing reinforcement in that location was found, from historic photos and radar investigation, to be minimal.

Construction

As with all rehabilitation projects, there are surprises when the structure is opened up. The investigation had observed and non-destructively tested the main cantilever beams on the interior of the structure. The full removal of the floor system, which was a time consuming project in itself as each piece of stone had to be cataloged to find its place again during replacement, uncovered other structural damage. The east side joist cantilevers were found to have effectively failed with large cracks. (See *Figure 5*) These joists were beyond repair, and were removed and replaced. Additional external post-tensioning was added in this location as well.

The installation of the main external post-tensioning proceeded. Cores were made through the parapets and perpendicularly through the main cantilever beams at the stressing and dead ends and at the high-point tendon deviations. *Figure 6* shows the typical reinforcement of the stressing end anchorage prior to casting. *Figure 7* shows the main post-tensioning in the process of being stressed.

Figure 6: Beam strengthening end block prior to casting.

The contractor suggested an alternate monostrand system for the east-west joists that utilized a crossover anchorage. This negated the need to core through the parapets and, thus, preserved the original structure. The main tendons were stressed incrementally over three days to slowly apply the large forces to the structure. *Figure 8* shows the crossover anchorage being stressed. The parapet penetrations were patched by SPS to within a ¼ inch of the surface to allow restoration specialists to patch the remaining material to match the surrounding finish of stucco and paint. The completed repairs are invisible to the eye. *Figures 9* and *10* show the restored interior and exterior, respectively. (See page 50 for *Figure 10*).

Figure 5: Failed cantilever joist discovered after floor removal

Figure 7: Stressing of main post-tensioning tendon

Figure 8: Monostrand being stressed at cross-over anchor

The structure was monitored for movement during and after the post-tensioning to field adjust the forces and to measure the uplift of the cantilevers. The maximum uplift, at the end of the west beam immediately after the post-tensioning, was 0.30 inches. The deflections of the other beams ranged from 0.08 inches to 0.18 inches. Long term monitoring has not recorded any further deflections.

Conclusions

The completed repairs did not change the appearance of this landmark structure. The use of the exterior post-tensioning was the most practical and least costly method to accomplish this task, but its impact on the whole building, both structurally and aesthetically, needed to be checked thoroughly.

Frank Lloyd Wright's architectural masterpiece is no longer "falling" and will continue to stand guard for many years over the waterfall from which it takes its name.▪

See Mr. Silman's sidebar on next page...

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Figure 9: Restored interior of main living room

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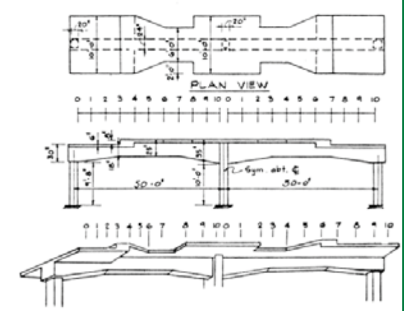

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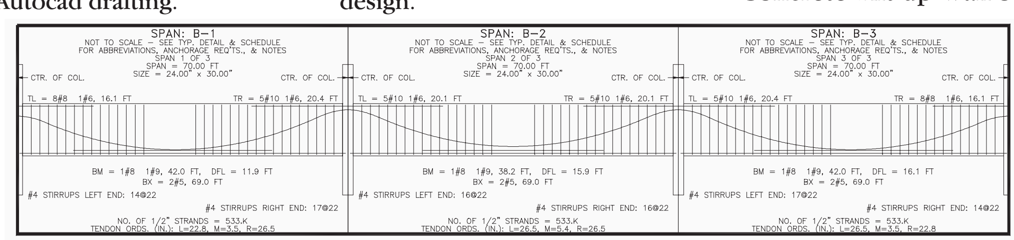

What Went Wrong

After much testing and non-destructive evaluation (NDE) work, our firm found that the major cantilever beams at the first floor were grossly under-reinforced — the reinforcing steel had exceeded its yield strength and the compressive stress in the concrete (approximately 4400 psi) was approaching the ultimate failure strength (approximately 5000 psi). How could this have happened? There were two structural engineers collaborating on the design, Mendel Glickman and Wes Peters, both highly competent. The theoretical problem was not difficult, even for 1936 — a simple concrete cantilever beam. Here are some of our theories:

1 The house was designed very quickly. Mr. Wright had visited the site just once and then had done nothing more. One morning he received a phone call from the owner, Edgar Kaufmann, who was visiting in Milwaukee, a three-hour drive from Taliesin, the architectural studio operated by Frank Lloyd Wright and staffed by his "apprentices". When he asked to come and see plans of the house, Mr. Wright invited him for lunch. He then gathered the apprentices around him and, in the space of one morning, proceeded to draw all of the floor plans. While they were at lunch, the apprentices finished the elevations. Virtually nothing changed from that original set and construction began soon after.

2 We suspect that the engineers were being pushed by the contractor to produce the structural drawings so that they could build the next piece. It is very possible that the engineers did not realize that the load from the second floor was being transmitted to the tip of the first floor cantilevers by means of the four reinforced mullions.

3 Even if these facts were so, there is still evidence that both the second and first floor beams were inadequately reinforced. It appears as though there were mistakes made. On the day that the formwork was decentered, the cantilevers deflected 1 3/8 inches and a significant crack opened in the second floor spandrel/parapet beam over the masonry supporting pier below. They called Mendel Glickman who went to check his calculations and when he returned to the phone he is reported to have exclaimed, "Oh my God, I forgot the negative reinforcing!" The originally designed reinforcing in the first floor cantilevers was also far too skimpy, not enough to even hold the weight of the first floor by itself.

Figure 10: Fallingwater as it looks today

4 The reinforced concrete contractor, also an engineer, determined by means of his own independent calculations that there was insufficient reinforcing in the first floor cantilever beams. The contractors wrote of their findings to Mr. Kaufmann, who passed the letter along to Mr. Wright. The latter, famously egocentric and unwilling to admit that he could ever be wrong, responded with a classic letter to his client, forcing him to choose between the veracity of the contractor or the architect. Mr. Wright's words went something like this: *"I have put so much more into this house than you or any other client has a right to expect that if I haven't your confidence — to hell with the whole thing."* Mr. Kaufmann backed down and deferred to Mr. Wright's power of persuasion. After a contentious exchange with Mr. Wright, the contractor "sneaked in" double the amount of reinforcing called for on the original design drawings. This was still inadequate for the dead loads of the concrete — they had apparently not accounted for the load of the second floor on the tip of the first floor cantilever either, even though it was clearly shown on the original drawings.

In summary, we never were able to conclude what went wrong. But wrong it did go and it remained in that dangerous state for 65 years until this elegant repair was completed.▪

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