Stabilizing the Falling of Fallingwater: A Structural Rehabilitation Proposal for The Master Terrace

THEODORE M. CERALDI, AIA
Syracuse University

INTRODUCTION

The following introduction and summary of existing conditions is based on field observations, nondestructive testing, and computer modeling performed by Robert Silman Associates, Consulting Engineers. The existing master terrace at Fallingwater was never designed to carry itself as a true cantilever and must rely on the steel "T" sections built into the living room window (South Elevation) to transfer the load to the four main cantilever beams which are a part of the makeup of the living room floor structure. These sections were a working part of load transfer as indicated by the original design drawings produced by Wright.[1] The loads transferred by these sections are greater than originally intended due to the failure of the east & west continuous beams of the master terrace. In essence a plastic hinge has formed at the master terrace in each parapet beam. Recent measurement across cracks at these locations show an advancing downward deflection of the master terrace that must be arrested. "The rate of deflection may decrease but there is no indication that the ongoing deflections will stop."[2]

The concrete cantilevers of the living room have thus been over stressed in bending due to the failure of the master terrace to support a higher proportion of its load.

> "Based on the total deflections and apparently continuing deflections of the master terrace and the living room level and on the overstressed condition of many of the concrete beams, we conclude that remedial structural intervention will be required in the near future."
>
> - Analysis of The Master Terrace at Fallingwater, by Robert Silman Associates, dated May 17, 1996.

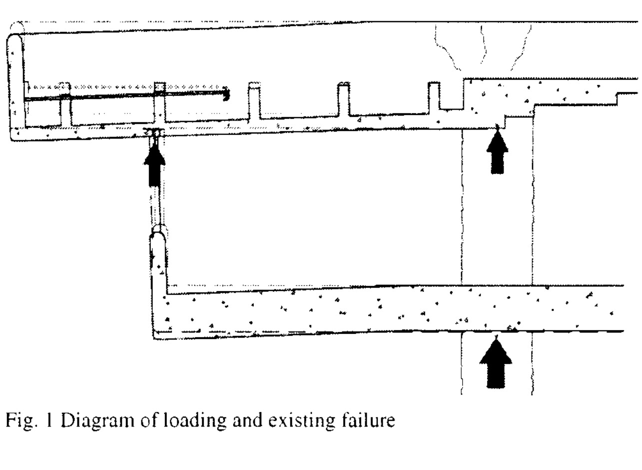

Fig. 1 Diagram of loading and existing failure

In the spring of 1997, the Western Pennsylvania Conservancy moved on this recommendation by installing temporary steel shoring under the living room cantilever to the stream bed below. The Kaufmann House is a National Historic Landmark and as such any work performed on the structure must meet applicable federal requirements. The cantilever deflections, therefore, can only be stabilized and arrested in their present attitudes. This must be accomplished without changing the appearance of the architecture or compromising the historic value of the house.[3] Presented are two possible solutions to resolve the stabilization of the master terrace.

A HISTORY OF CIRCUMSTANCE

Edgar Kaufmann Jr., son of the wealthy Pittsburgh department store magnate, was instrumental in getting Wright the commission to design Fallingwater. Kaufmann Jr. after joining the newly formed fellowship at Taliesin, in Spring Green, Wisconsin, introduced his parents to Wright. The Kaufmann Residence, at Bear Run Pennsylvania, would be one of Wright's first new commissions during this period at Taliesin, a second would be the work for Hib Johnson in Racine. Wright met Lilian and Edgar Kaufmann in the Fall of 1934. His first visit to Bear Run was in December of 1934, where a permanent weekend house was discussed for the extraordinary site. Wright spent the winter and spring of 1935 formulating his ideas for the Kaufmann house which he keeps to himself except for the following communications to Edgar Kaufmann:

> " The visit to the waterfall in the woods stays with me and a domicile has taken a vague shape in my mind to the music of the stream. When contours come you will see it," December 1934, F.L.W.[4]

> " Dear E.J.: We're working. You'll have some results soon. Frank Lloyd Wright." August 1935.[5]

In September of 1935 Edgar Kaufmann on a planned business trip to the Middle West decided to visit Wright and find out what was going on with his house. Kaufmann called from Milwaukee, according to a recollection of Edgar Tafel, one of Wright's apprentices, and upon hanging up the phone Wright ... "briskly emerged from his office, some twelve steps from the drafting room, sat down at the table set with the plot plan, and started to draw." The resulting house; the design of which was virtually unchanged at completion, was drawn and named Fallingwater by the time Kaufmann arrived some two and a half hours later.[6]

In October of 1935, Wright and Kaufmann agreed to a division of responsibilities for the construction of the house after a full set of

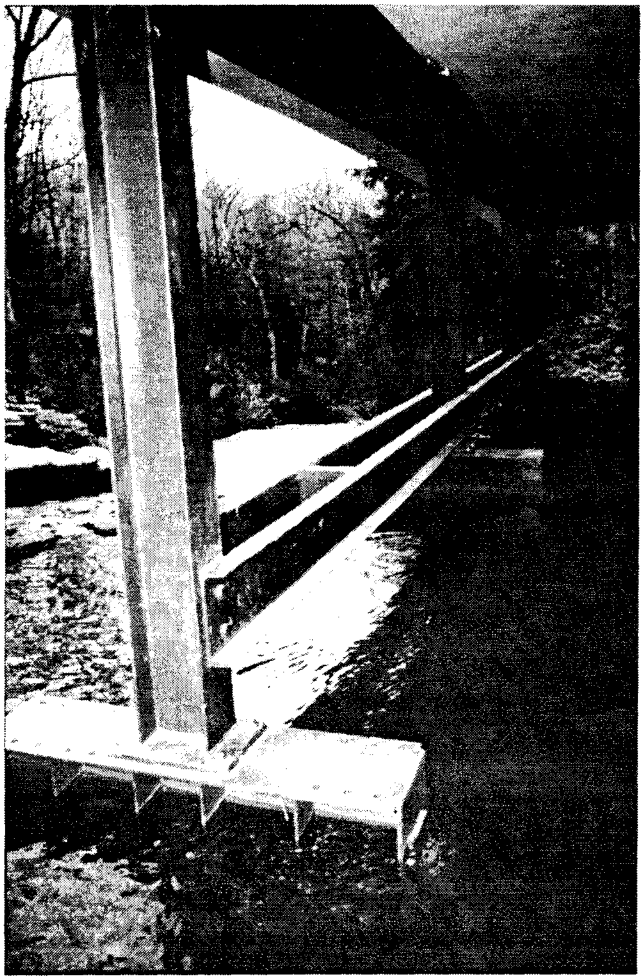

Fig. 2 Existing steel shoring placed under the living room in 1997.

drawings were produced and reviewed in Pittsburgh. Kaufmann puts Carl F. Thumm, his assistant manager at the department store, in charge of managing the construction of the house. Wright was asked to review a sample wall of stone, produced under Thumm's directive, in April of 1936. Wright rejected the wall and fires off a letter to Kaufmann in May 1936 trying to establish his tenure for the building of the house and put Kaufmann's man in his place. Shortly after, Bob Mosher, an apprentice at Taliesian, was sent off to the Kaufmann camp at Bear Run.[7] Later that month Wright selects a contractor based on a recommendation of one of his apprentices.[8] The contractor, Walter Hall, chiefly a stone masonry contractor, was signed on in June but did not show up on the job until July. Foundations and stone masonry work started in May, under the direction of Bob Mosher, was completed by the time Hall showed up. Hall was on the job in July as concrete bolsters under the living room slab were completed.[9] Hall moved very fast once he started and by the first week in August formwork was up for the first floor and the living room cantilevers.

August 1936 was the infamous month in the building of Fallingwater. A series of events began which would lead to the eventual failure of the cantilevers and the need for structural intervention today. The recorded events, paraphrased here, occurred between August 1 and December 23, 1936:

August 1-18 Formwork is installed for the first floor and living room cantilever. This and all subsequent formwork is set dead level without any "crown" as Wright called it. (Wright is not made aware of this until the structure is largely complete in December 1936).

Mr. Thumm gets back into the thick of it when Metzger-Richardson Co., engineers supplying the concrete reinforcing bars, as directed by the Owner and installed by the Contractor, add twice the number of 1" square bars to the large cantilevered beams of the living room. Bob Mosher does not stop this additional steel from being installed but informs Wright of what is going on.

August 19 Concrete is placed in the formwork of the first floor cantilever.

August 22 Forms are stripped and shoring is in place under the living room cantilever. Form work is started immediately on the Master Terrace.

August 26 Wright fires off a letter to Kaufmann and threatens to quit:

> My dear E.J.:
>
> If you are paying to have the concrete engineering done down there, there is no use whatever in our doing it here. I am willing you should take over but I am not willing to be insulted ... I am calling Bob back until we can work out something or nothing. ... I don't know what kind of architect you are familiar with but it apparently isn't the kind I think I am. You seem not to know how to treat a decent one. I have put so much more into this house that you or any other client has a right to expect that if I haven't your confidence—to hell with the whole thing. Sincerely, Frank Lloyd Wright, Architect

August 27 Bob Mosher is called back to Taliesin.

August 28 Kaufmann replies to Wright's letter and tells him "to hell with the whole thing." The post script to Kaufmann's letter, however, sets the tone for reconciliation.

August 29 Wright, having debriefed Mosher, fires off a letter to Hall telling him:

"If you imagine your meddlesome attitude to be either sensible or honest (we will not say ethical) something was left out of either your character or your education."

Wright ends the letter by calling Hall's work "treacherous interference."

August 30 Wright sends another letter to Kaufmann and defends his engineering concepts and design. He also denounces Kaufmann's engineer and states his case:

> "I have learned from experience with the earthquake proof Imperial Hotel and other buildings that the fiber stress in steel is safe at 25 to 30,000 lbs. and that the compression on concrete of 1,500 is entirely safe...In short Mr. E.J. Kaufmann (client No. 199) these assumptions of your engineer, to wit: 750 lbs. for concrete - plus a 40 live load - 20,000 for steel would double the cost of your construction because not only is there double the cost of your structure but the increase to carry the increase in weight would be considerably more. Frank Lloyd Wright, Architect."

August 31 Wright sends off another letter to Kaufmann with apologies blaming Hall and the steel company for the interference in the work. Attacking Hall as an ..."officious Yankee with no sense of proportion where he is himself concerned"..he writes at the end,..."Meantime your letter shows me that I do owe you and myself to get on the job. I'll come soon."

September Wright sends Edgar Tafel, another apprentice, to take Mosher's place on site.

October 1 Concrete for second floor and master terrace is placed in forms.

October 5-28 Form sides are stripped; shoring maintained in place;

formwork for West bedroom terrace, roof over master bedroom, and guest room is started; and Hall builds a shanty for the storage of his materials at the extreme South end of the master terrace cantilever which has yet to reached its 28-day design strength, normal for concrete.

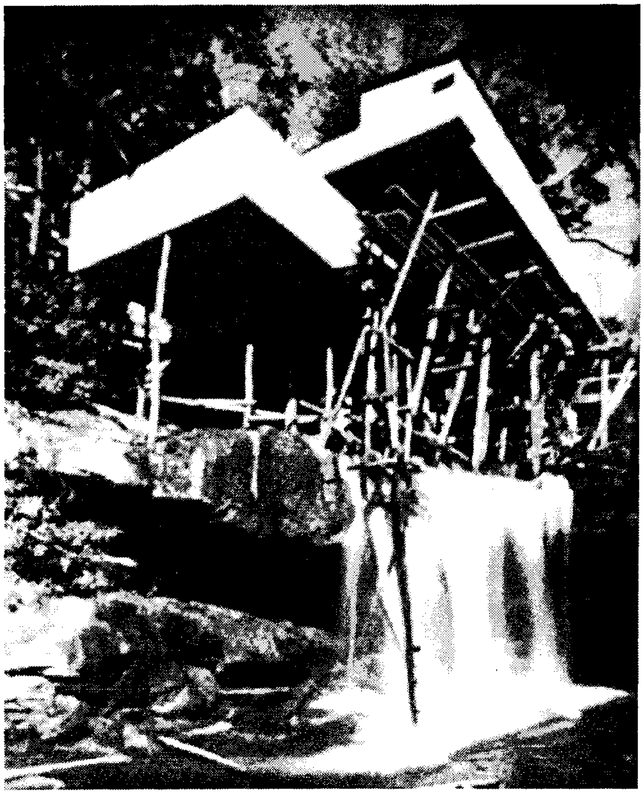

Fig. 3 Hall's storage shanty at the master terrace in place while form sides and bottoms are stripped and re-shored prior to 28 day strength of concrete.

October 29 Concrete placed in forms for master bed room roof, guest bed room, and west terrace. Hall's shanty remains in place, form bottoms are removed, temporary shoring replaces major formwork on living room cantilever, and master terrace cantilever.
October 30 Edgar Tafel, Wright's new man in the field, reports to Wright that two cracks have opened up in the master terrace parapets over the columns below.
November Hall starts a flurry of work to finish the concrete of the main structure before the cold winter sets in. The main structure of the house is complete by early December and many cracks have appeared as well as deflections of the cantilevered terraces as the shoring under has been removed.
December 8 Hall's shanty is still in place as Metzger - Richardson Co. report:

> "Structural cracks developed about the 1st of December. These cracks appeared in the curved cantilever beam and cantilever joists on the east side of the first floor; cantilever beam on the west side; wall around stair well on first floor; and in the parapet walls around Mrs. Kaufmann's terrace (master terrace) on the second floor. The cracks extend clear through the members, proving them to be structural" [10]

December Wright comes down with pneumonia
December 23 Wright, through Bob Mosher, orders concrete samples to be taken from the existing structure for testing.

Metzger-Richardson Co. were retained by the Kaufmann's to monitor the deflections of the house and advise on its structural integrity until they were replaced by the firm of Hunting Larsen & Dunnells (surveyors and engineers) in 1950. In April 1955, Mr. Kaufmann Sr. dies and the house remains with Edgar Jr. until he gives it to the public under the care of the Western Pennsylvania Conservancy in October 1963.

INTRODUCTION TO STABILIZATION

The following two proposals for stabilization of the master terrace at Fallingwater are based on a familiarity with the structural problems identified by Robert Silman Associates, and on analysis of the original construction documents obtained from the Frank Lloyd Wright Archives, at the Frank Lloyd Wright Foundation, Taliesin West. The proposal incorporates the following as design parameters (codes listed are standard to architectural practice in the state of Pennsylvania). The existing concrete structural system as dead load; design loads and loading conditions prescribed by BOCA code; AISC manual of Steel Construction; ACI 318-95; and ANSI load factors, where applicable. Dimension parameters are dictated by the stone floor and setting bed; the existing top of the slab under the master terrace; the coffered living room ceiling; and the availability of masonry construction as both counter weight and conduit for tension anchors to the concrete abutments below. The resulting designs shall require disassembly of the existing building in affected areas. Every effort has been made in the stabilization design to minimize damage and alterations to the structure, while providing an ease of installation. The design allows for stabilization without changing the outward appearance of the building or affecting its historic landmark status.

LOAD ANALYSIS

Several findings from Wright's drawings and early engineering reports indicate that the reinforced concrete structure of Fallingwater was not designed nor placed with the rigor and performance standards of the normally accepted practices of the time. In addition, the slab of the master terrace cantilever was placed much thicker then Wright had designed which adds considerably to the dead loads. Regardless of the particulars, it is clear that Wright's engineers did not anticipate the deflections and stress levels induced by the actual dead and service loads. Evidence supporting this existed as early as 1937, in a series of load tests performed by Metzger-Richardson Co. These observation-based claims were later substantiated through computer modeling of the structure under load in the Silman report of 1996. [11]

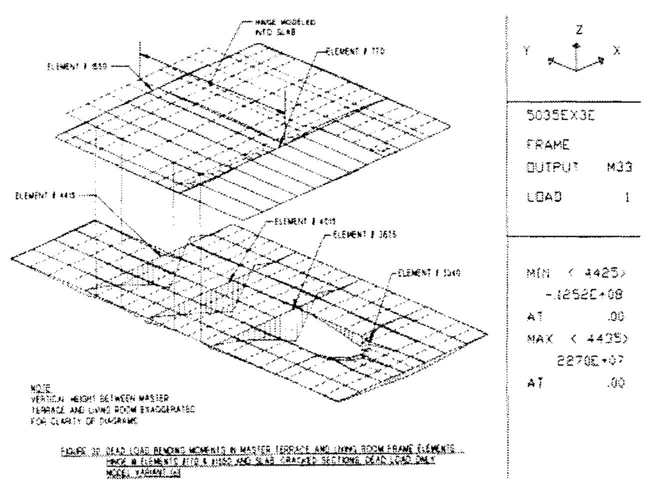

Fig.4 Dead load bending moments in master terrace and living room frame elements. Hinge @ elements #770 & #1550 and slab, cracked sections, dead load only, model variant (e) Silman

The concrete code, at the time Fallingwater was built, was limited. Indeed, the Johnson Wax Building test column, erected June1937, was required by code to be 30" in diameter throughout and not the 24" to 9" taper that was placed and field tested to everyone's amazement but Wright's. A lack of universally accepted model codes gave Wright and other architects of the time an opportunity to design forms, spans, and cantilevers that pushed the limits of structural design. Fallingwater was no exception and was to test the upset tray structural concept put forward by Wright and his engineers.

The proposed solution derives its load factors and design origin from the position of code compliance and architectural conservation. Accepted in Pennsylvania is the Building Official and Code Administrator (BOCA) Code. In order to realistically arrive at a solution, Fallingwater must be considered a Single Family Dwelling, allowing a minimum live load value of 37.6 psf to be used (BOCA 1606.1 with reduction allowance). The criteria for snow load (BOCA, 1610.4, table 1610.3) provides a design value of 22.5 psf. Deflections are restricted to 1/360 of the respective span. Steel design follows the AISC manual of Steel Construction, using an allowable stress based method. Prestressed and reinforced concrete design follow ACI 318-95 with ANSI derived load factors of 1.4 x dead load: 1.7 x live load; and a assumed 1.7 snow load by author.[12]

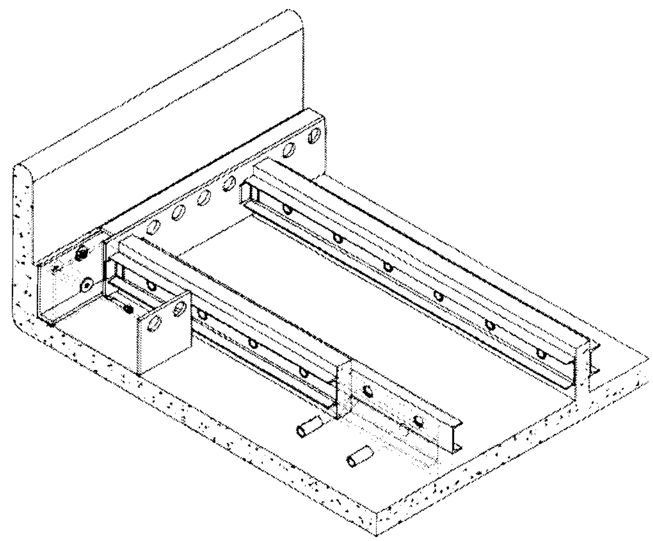

Fig.5 Partial isometric of main beam and sister joists in steel solution.

Load Transfer

The design and insertion of the proposed structural members are responsive to a transfer of load through the existing structure to the concrete bolsters and bolder at the stream elevation without eccentric loading of the existing structure. The structure will be relied on to transfer load in the following manner: Transfer the Y-Y axis torque from the eastern and western wings, through the respective continuous beams, to the proposed structural support systems. Transfer the load from the bottom slab of the master terrace, through the existing concrete ribs, to the proposed support system.

STRUCTURAL PROPOSALS

Steel

The piggyback nature of the existing structural system, intended or not, transmits a portion of the master terrace load through the "T" mullions of the living room windows, to the cantilevered living room beams below. The nonaligned nature of the existing structural hierarchy excluded it from consideration as an initial structural model to follow for the steel solution. Instead, the proposal embraced the more simplified propagation of loads from the master terrace through the stone masonry supports, then down to the bolsters and bolder. The requirements of this proposal demands a material that can resist enormous bending stresses (maximum over the stone piers), and maintain small depth and width dimensions. The resulting composite steel beam has a moment of inertia in the x-x axis of 1585 in4 at the location of critical stress. The profile of the beam changes in the zone of the free cantilever where it mechanically engages the existing structure. The beam sits within a saw-cut trough allowing it to sister the existing edge beam assembly.

The profile, strong in bending, has a relatively small capacity for resistance to buckling within its free cantilever profile and therefore requires lateral bracing. This sistering of the concrete joists will transform any tendency for buckling along the free cantilevers into a lateral shearing force. The existing sub-parapet beam and joists ribs maintain a functional role in transferring load, through mechanical connectors, to the new east and west secondary structural frame. Once the mechanical connections are made, the torsional stability of the existing beam/wing will be used to stabilize the steel cantilever. The more complex portion of the design for either structural model (steel or concrete) occurs in fastening the existing structure to the proposed. Through this mechanism, the existing concrete structure is taken on as a dead load to the secondary.

The mechanism, or fasteners, work through a function of the strength of the existing concrete and stress patterns the structure will assume from its interaction with the proposed secondary system. The method of attachment is directly related to the procedure of erection. The calculated design may be compromised by minor slippage between the existing concrete and new steel. Thus, anchor holes are to be drilled through the new cantilever steel after it is in place and secured to tension members. The existing cantilever must be jacked, and shored in place prior to this drilling. Jacking procedures are not a part of this proposal but they will necessitate critical calculations as to jack placement and elevating procedures. The steel cantilevered structural system shall have to resist a design load which produces a 820 kip-ft bending moment. The load will be slightly more for a concrete solution necessitating posttensioned in place and or the reintroduction of the "T" supports as a shared structural system.[13]

Concrete

The proposed prestressed concrete solution uses 9,000 psi concrete, and steel wire strands of grade 270 stress, draped in a profile expressive of its moment diagram. The maximum tendon eccentricity is 5" and occurs at the point of maximum moment which is approximately 8'-6" back from the southern terrace parapet. Two prestressed beams shall be formed and cast-in-place along the existing east and west parapet beams in the same attitude as the steel solution. Existing rib joists, cut in demolition, shall be reconnected by dowels and beam extensions cast integrally with the main beams. The beams shall also be doweled into the existing parapet assembly. After concrete has been cured to its 28 day strength, jacking shall occur to a final endblock reaction of 189,000 psi. This circumvents the construction procedure of jacking the entire cantilever into position as a prelude to fastening (as in the steel solution). The interstitial space around the wire strands is to be injected with high strength grout after jacking. The number, location, and embedment of dowels is identical to that required for the steel model solution. This move minimizes the density of steel reinforcement within the narrow profile of the prestressed continuous section. The east west ribs will be left to transfer load in the same manner as before intervention. Geometry of the existing east-west section of parapet and canopy will be utilized to provide torsional stiffening in concert with the prestressed beam assembly.

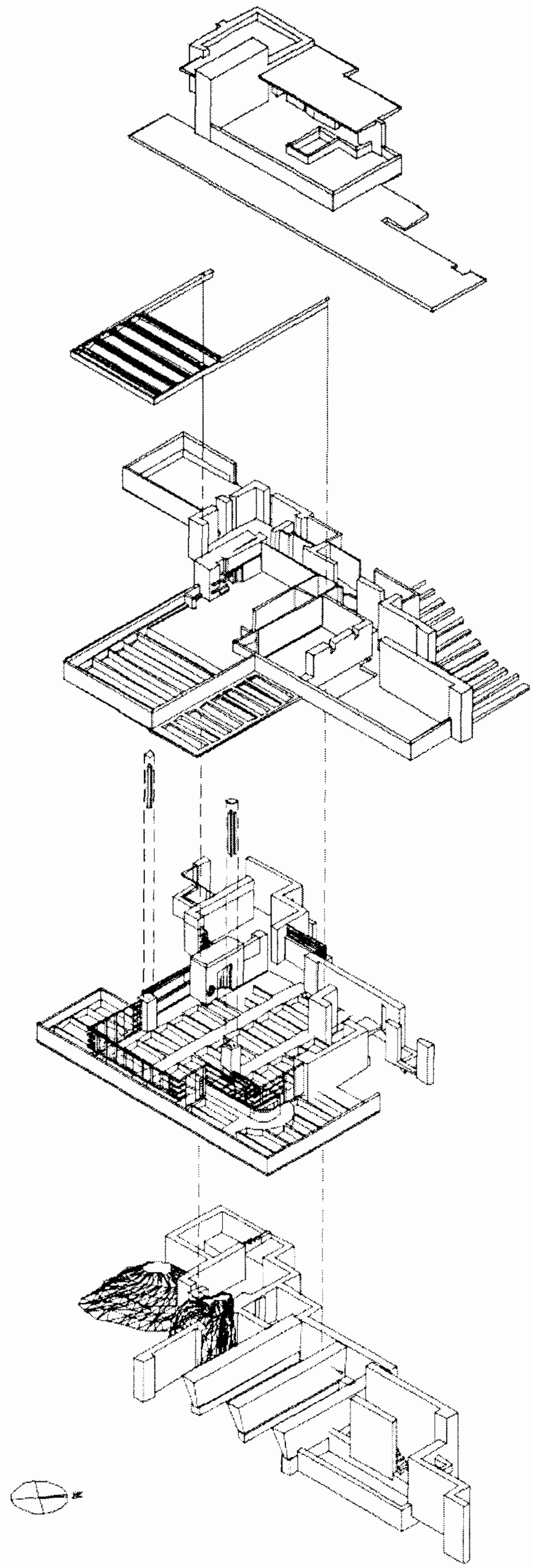

Fig. 6 Exploded isometric of the steel solution.

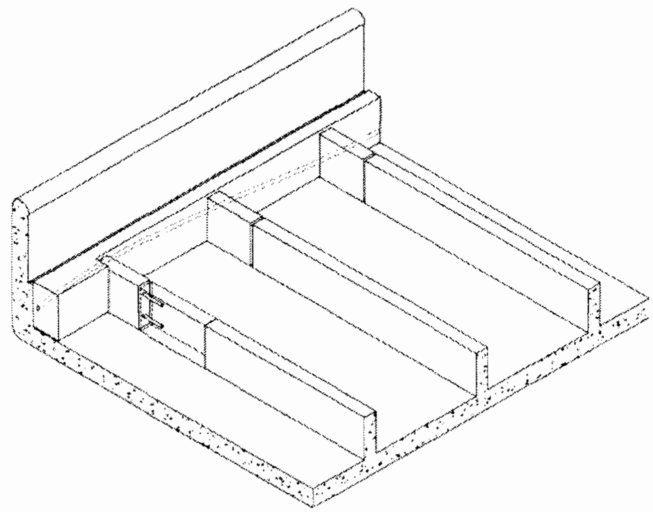

Fig. 7 Partial isometric of concrete solution.

The decision was made to reintroduce the steel "T" mullions as a support option and working part of load transfer as indicated on the original drawings by Wright. Assuming rigid support, the mullions reduce the service load design moment to 98.70K-ft. while allowing a margin of safety to be maintained in the steel mullions. The mullions are strong enough to resist failure under this reduced axial load condition. The east west posttensioned beams (no longer a true cantilever) transfer the load through the mullions to the living room below. The downside is that this design merely moves a portion of the load to the living room cantilever. The additional steel inserted here, by Metzger-Richardson Co. in August of 1936, will help to maintain a safe load transfer. The tension ties which occur in the steel solution will not be required in the concrete solution. The post-tensioning concrete solution shows reactions which do not require additional support other than the existing dead load of the building as was Wrights original intention. Considering factored and unfactored loads, 9,000 psi concrete, and a maximum tendon eccentricity of 5", the stresses induced by the design moments can be resisted within a cross section of 5" x 17".14

CONCLUSION

The steel and concrete solutions for stabilizing the master terrace at Fallingwater will both work in principal. The use of steel over concrete, or vice versa, requires careful analysis of structural and maintenance considerations not addressed in this paper. Stopping the present deflection is the intent of the proposed structural models. The solutions will not change the outward appearance of the architecture or the assembly parameters. The installation of either solution shall require disassembly of the stone floor and subfloor; stone masonry; window glass; existing concrete structural systems; unknown utility pipes and lines; and any built in furniture which might be damaged during construction. The installation of either system calls for extensive shoring and bracing along with a careful maintenance of same. The jacking of the cantilevers prior to installation of the steel solution, shall require procedural directives and monitoring by a structural engineer. Records on all removals for future reinstallation must be meticulously maintained, while removed materials will have to be properly handled and stored. The second half of the operation will be the careful restoration of the original assembly structures and associated finish patinas to the satisfaction of the Western Pennsylvania Conservancy.

The proposed concrete solution, although it diminishes the load transfer to the living room cantilever, would still burden that struc-

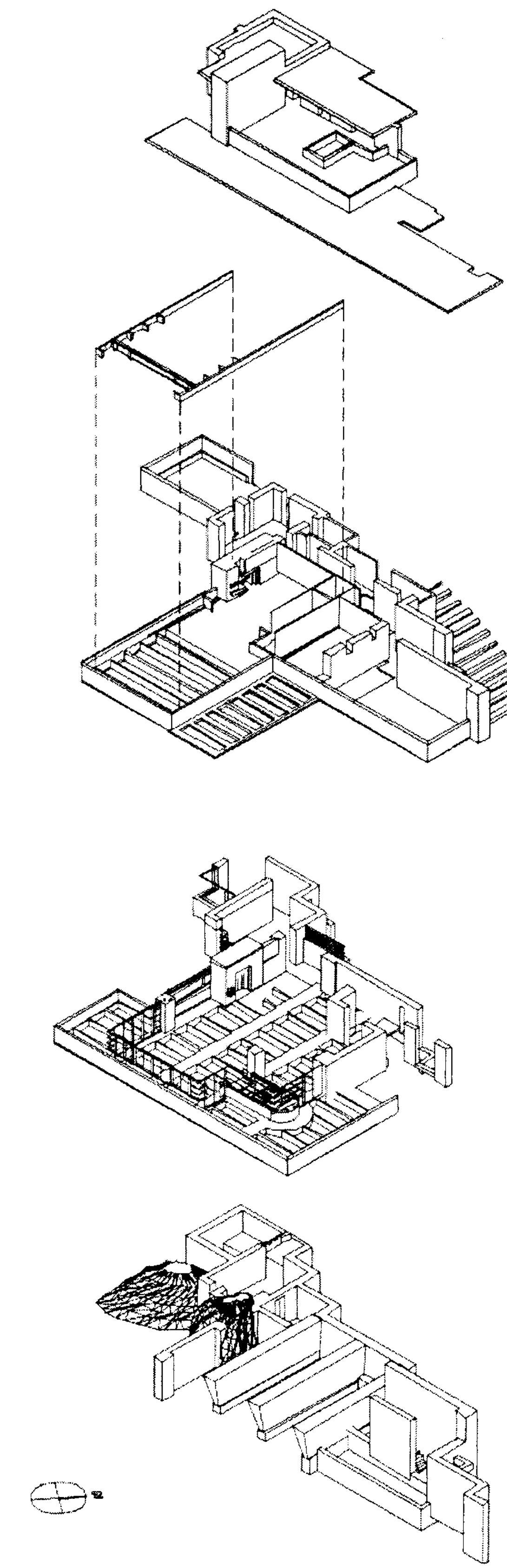

Fig. 8 Exploded isometric of concrete solution.

ture to carry a portion of the master terrace. It is anticipated that the living room cantilever beams must also be strengthened in order to guarantee a maintained stability to the entire coupled structure. Structural solutions for the living room cantilever could be implemented with similar cast in place posttensioned beams, and a further investigation of their present condition is required.

It is not the intent of this paper to draw any conclusions as to why or how the master terrace failed. Nor is it the intent to lay negligence on any one person or circumstance. A certain amount of ambiguity is evident between the intention and execution of the concrete work as designed by Wright's engineers, particularly in Wright's reliance on the contractor Hall to provide formwork crowned to counter initial deflections along the cantilevered extents without any such directive given. Additional concrete placed in slabs beyond the designed thickness increased the dead loads considerably. Form stripping procedure and re-shoring, with the addition of Hall's shanty (and its stored contents) as live load, was also highly questionable.

Architecture is an informed design process which is forever dynamic. Wright was a genius and similar to the architect of Hadrian's Pantheon (118-125 AD), pushed the envelope of concrete design and placement to it's limits. Fallingwater is a tribute to this process and a masterwork of twentieth century architecture that must be constantly maintained, conserved, and restored.

NOTES

[1] Robert Silman, Master Terrace Analysis. 1996 Executive Summary, p.1 ,the computer analysis firmly establishes this as was the original intent of the design drawings by Wright.

[2] Silman, Executive Summary, p.2

[3] This is indicative of the mission statement and some buildings that are National Historic Landmarks in accordance with the Secretary of The Interior Standards for The Treatment of Historic Properties. 1992

[4] Pfeiffer, Letters to Clients, Frank Lloyd Wright, The Press, California State University, from a letter to Edgar Kaufmann upon Wright's return to Taliesin after he first saw the site in December of 1934.

[5] Wright is placating his client here. He does not produce a drawing of the house until September of 1935.

[6] Edgar Tafel, Years With Frank Lloyd Wright, Dover, 1970, p.3 Edgar Tafel recounts the overheard telephone conversation between Wright and Kaufmann prior to his arrival at Taliesian. When Kaufmann arrived Wright immediately started his presentation and adds later that while E.J. and Wright were at lunch he and Bob Mosher drew up the remaining elevations and upon returning from lunch the master continued describing the house with the added elevations reinforcing his presentation. Wright never broke stride nor put pencil to paper prior to the phone

[7] Wright also asks Kaufmann to send the builder to Taliesin for a period of training, as Wright hints about his future relationship with the project stating..."..being an architect hundreds of miles away and a house for you in question I have to find my tools near you. I have explained all this to you many times. Now about money. You seem suspicious when I ask for it, and use the scissors to clip the sum. Don't be afraid. You aren't going to pay too much nor pay too soon. You won't be let down so don't you let me down." from a letter to Edgar Kaufmann, from F.L.W., May 4, 1936.

[8] Wright sends off a letter to Mr. Walter Hall at Fort Allegheny, Pennsylvania on May 13, 1936, asking him if he would be interested in building a "We have a house, chiefly masonry-stone work and concrete- which we are to build at Bear Run, Pennsylvania for Mr. Edgar J. Kaufmann of Kaufmann's Department Stores in Pittsburgh." Hall is chiefly a masonry contractor and the recommendation for his hire comes from Earl Friar, an apprentice at Taliesin, Wright does not know the anything about Hall but recommends his hire to Kaufmann on May 31, 1936, in a letter to Edgar J. Kaufmann,

he states "My object in getting Hall on the job is to save you that expense and insure cooperation with me. Probably some talk is necessary to clear up this point between us. But don't get headed in wrong on these matters. Seeing You soon-F.L.W."

[9] Apparently in a letter to Edgar Kaufmann, July 13, 1936, Wright notes that Hall is finally on the job and would have been sooner (back in June) if it were no for the intervention of Kaufmann's manager Carl F. Thumm. The briefing of Hall by Thumm is not recorded, but it certainty sets up a strained relationship from the beginning between the architect, client, and contractor.

[10] The initial report was given to the Kaufmann's after a site visit on December 8, 1936. The full report, with calculations was issued on June 1, 1937, Silman, pp. 5 & B-12.

[11] Wright defended his position to the end and conducted his own tests consulting with his engineers, Wes Peters, and Mendel Glickman.

[12] These load factors are higher than what was used originally by Wright and his engineers at Fallingwater. See appendix A for calculations.

[13] Malara, Anthony, Appendix A, Summary of Calculations for Proposed Steel Solution, 1998

[14] Malara, Anthony, Appendix A, Summary of Calculations for Proposed Concrete Solutions, 1998

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A. Malara, Appendix A, *Summary of Calculations for Steel and Concrete Solutions*, Syracuse, New York, 1998.

PHOTOGRAPHS

Figure 2. Photograph by Ted Ceraldi showing steel shoring in place from living room slab to the stream bed, installed in the spring of 1977.

Figure 3. Photograph from, Frank Lloyd Wright Archives, Avery Library, Columbia University, showing Hall's storage shanty at the master terrace.

DRAWINGS

Figure 1. Hofmann, Thomas Anthony, Syracuse University, 1998
Figure 4. Silman, Robert, Associates Consulting Engineers, N.Y., 1996
Figure 5, Hofmann, Thomas Anthony, Syracuse University 1998
Figure 6-8, Hofmann, Thomas Anthony, Syracuse University, 1998

This paper was prepared with the assistances of Stabilizing the Falling of Fallingwater: A Structural Rehabilitation Proposal for The Master Terrace

This paper was prepared with the assistance of Anthony Malara and Thomas Hofmann, graduate research assistants, at Syracuse University.

APPENDIX A

General Notes

All loading calculations and diagrams generated using RISA-3D software, version 3.0. Due to nearly identical design values for both the east and west side beams, only the diagrams for the west side are shown.

Summary of Calculations for Proposed Steel Solution

(See figure I.) Bending stress check
Structural steel used is A514 Grade 100. Quenched and Tempered Alloy.
Fy = 100 ksi.
Fb=assume Fb = .66Fy / .66(100ksi) = 66 ksi.
Sx (reqíd)= M / Fb = (803 k-ft (12î/ft)) / 66 ksi = 146 in3
Try built -up section d = 17î, b = 5î. top flange: tf = 1î , bf = 12î
Centroid y = 9.69î measured up from bottom
Ix-x = 1585.2 in4 ; St = 216.9 in3, Sb = 163.6 in3
216.9in3 >>146 in3 (OK.)

Shearing stress check

(See figure II.) Check web shearing stress.
Vmax = .40Fy = .40(100ksi)=40ksi
Design V = 71.63kips.
Total web surface area = 32 in2.
Total shear stress capacity =1280 ksi
Design shear stress = 71.63k / 32in2 = 2.24 ksi
1280 ksi >> 2.24 ksi (OK.)

Reinforcement for reaction points (3 and 4):
Impressive reinforcement for stone piers
In order to prevent localized failure of the stone masonry piers, compression reinforcement is provided by way of 4, 4" diameter standard steel pipe columns (A36 steel). These columns are to be grouped square, with pipe columns 5î on center. An A36 steel top plate,12î x 12"x1-1/2", is to be welded each column group. The finished assemblies, each inserted in a 12" x 12" clear space within the 2 respective piers, shall be leveled to receive the direct reaction of steel cantilevers. The design, transferring load from the bottom of the column group to the concrete bolster foundation has not been included in this article.
Column type: A36 steel, 4" diameter, wall thickness .237".
Use 9íeffective length.
@ 9' effective length, allowable concentric load = 52 kips.
4x 52 kips = 208 kips > 141.2 kips, (OK.)
(see figure III)

Cable reaction design for north ends of steel cantilevers

Upward load = 56.4 kips
Use 8, 7/16" Grade 270 stress-relieved seven wire strands.
1, 7/16" strand = .70 x 270ksi x.115 in2 = 21.7 kips.
21.7kips x 8 = 173.6 kips maximum allowable load.
173.6 kips > 56.4 kips, (OK.)

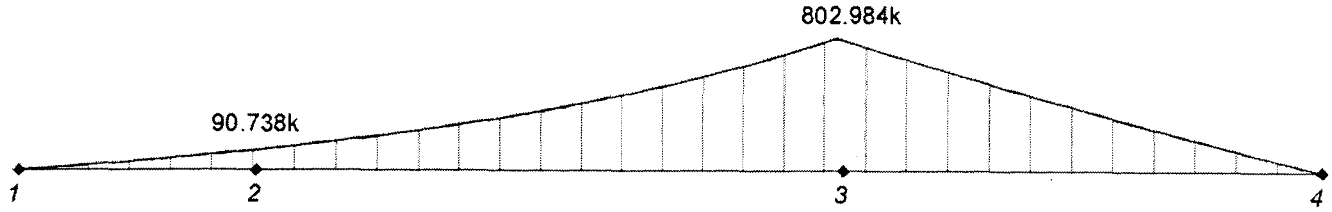

Fig. I Moment diagram for west side beam (steel solution, cantilevered element).

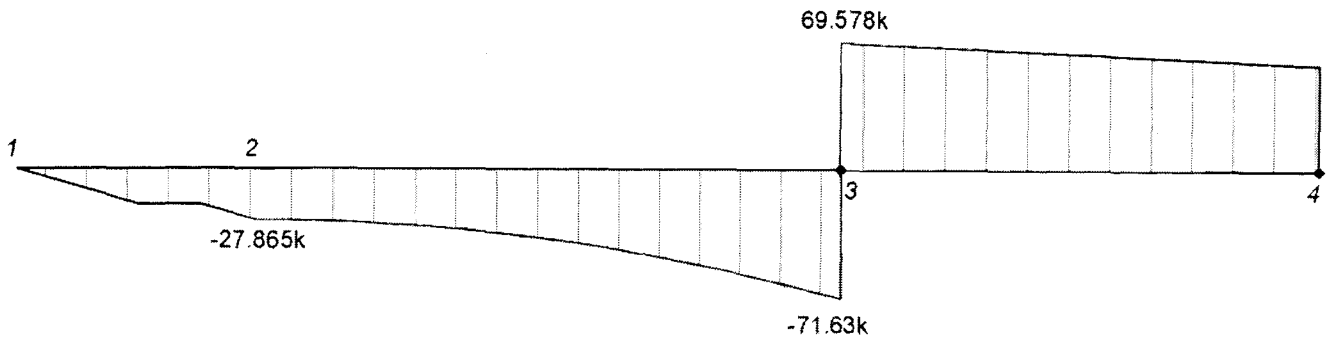

Fig. II Shear diagram for west side beam (steel solution, cantilevered element).

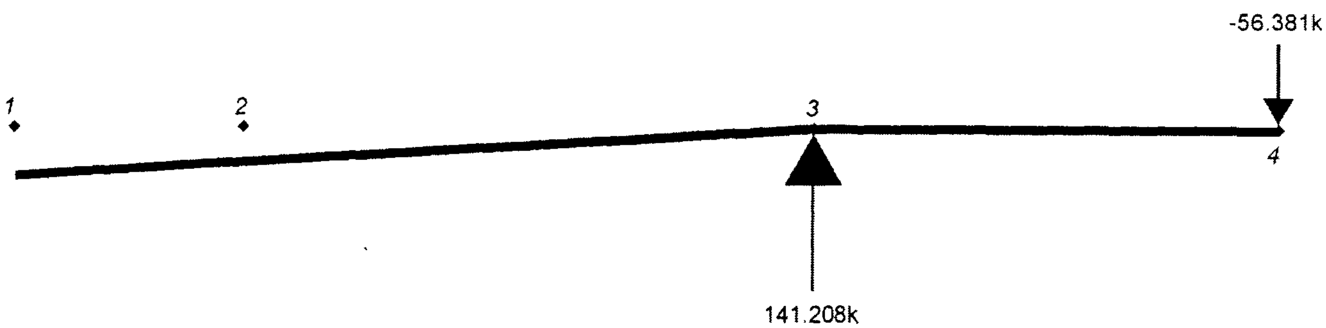

Fig. III Reactions for cantilever beam (steel solution, west cantilevered element).

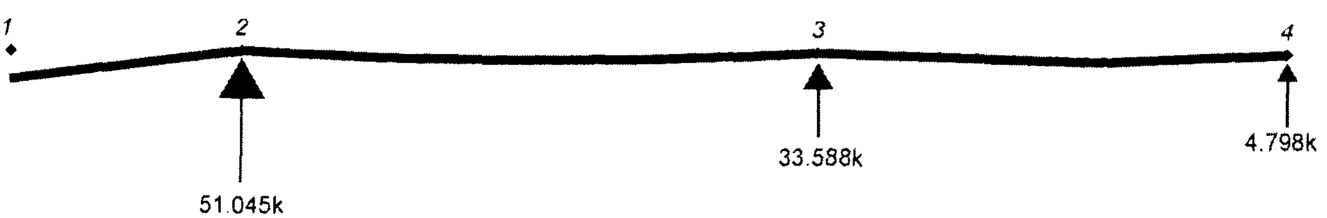

Fig. IV Reactions for continuous beam (prestressed concrete solution, west cantilevered element).

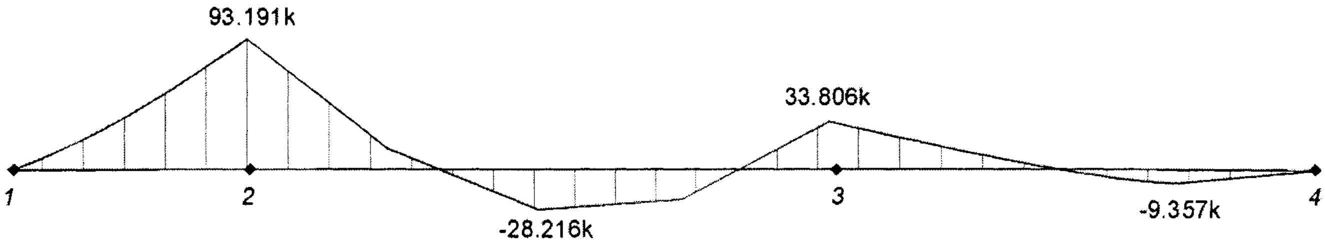

Fig. V Moment diagram for west side beam (PSC solution, continuous element).

Cables at uplift reaction at each beam are to be anchored within the stone and bolsters below with a 4" diameter, laterally inserted stainless steel pin (A316).
A316 stainless steel, Fy = 28 ksi.
Fallowable = .6(28ksi) = 16.8 ksi.
For pin, bearing surface = 4".
Allowable uplift load = [(8*7/16") * 4]*16.8ksi = 235kips, assuming full longitudinal cross section area of cable bears. (see figure IV)

Allowable stress check for steel T-mullions:
AT = 3.22 in2 (Silman, p.67)
Allowable stress of iT" = 16,250 psi (Silman, p.67)
16,250 psi x 4 x 3.22si = 209.3 kips
Point 2 compressive reaction (east and west sum) = 51 kips.
2' x 51 = 102 kips total <209.3 kips allowable. (OK.)

Assuming the additional compressive load from the proposed prestressed structural beams is tranmitted evenly over the 4 mullions (by way of the 2, C15 x 50 channels shown graphically), the maximum total compressive stress does not exceed allowble. Assuming a minimum angle size of 3 1/2" x 2-1/2" x 1/4" and A36 steel, the T-mullions are not in danger of failure from compression induced buckling. Masonry piers and walls (points 3 and 4, respectively), are assumed to have enough residual strength in their existing condition to preclude additional reinforcing using the prestressed option.

SUMMARY OF CALCULATIONS FOR PROPOSED PRESTRESSED CONCRETE SOLUTION

Design Values

("-"negative sign before stresses indicates compression).
Section Size: b = 5", h = 17".
ct = cb = 8.5"
Ac=85 in2
r2= 24.1 in2
Ix-x = 2047 in4
St = Sb = 240.8 in3
use emax = 5"
Pi = 189 ksi
Pe = 154.98 ksi
Aps = 1.053 in2 (9 x 7/16" seven-wire strands, Grade 270)
Assume fc' = (-)9,000 psi.
fti = 3x(9,0001/2) = 284.6 psi.
fci = (-)9,000 psi (assume transfer does not occur until 28-day strength is reached).
fc = .45 (9,000 psi) = (-)4050 psi.
ft = 6x (9,0001/2) = 569.2psi

Unfactored Loads:
1) Self-weight maximum moment MD:
i) @ point 2: 2.757 K-ft.
ii) @ point 3: 3.122 K-ft.
2) Service load maximum moment MS:
i) @ point 2: 93.2 K-ft.
Factored Loads:
1) Service load factored moment MU:
i) @point 2: 136.08 K-ft.
2) Service load factored shear VU:
ii) @point 2: 39.85 K. (see figure V0
Tension-side is on top, compression is on bottom.
Note that Msd is used rather than Md, since the service dead load transfer mechanism is cast integrally with the post-tensioned element.

Stress at transfer:
fb = [-Pi/Ac(1-(ecb /r2))] -Msd/Sb
fb =[-2223.5(-.77)] - 2466 = -754 psi
ft= [-Pi/Ac(1+(ect /r2))] +Msd/St
ft= [-2223.5(2.77) + 2466 = -3,693 psi
fci < fb, ft < fti, (OK.)

Stress after losses:
fb = [-Pe/Ac(1-(ecb /r2))] -Msd/Sb
fb =[-1823.3(-.77)] - 2466 = -1062 psi
ft= [-Pe/Ac(1+(ect /r2))] +Msd/St
ft= [-1823.3(2.77) + 2466 = -2585 psi
fc < fb, ft < ft, (OK.)

Service load final stresses:
fb = [-Pe/Ac(1-(ecb /r2))] -Mt/Sb
fb =[-1823.3(-.77)] - 1,184,400/240.8= -3515 psi
ft= [-Pi/Ac(1+(ect /r2))] +Msd/St
ft= [-1823.3(2.77) + 4919 = -131.5 psi
fc < fb, ft < ft, (OK.)

Through the use of 9,000 psi concrete and a maximum tendon eccentricity of 5", the stresses induced by the design moments considered for Fallingwater can potentially be resisted within a cross-section of 5" x 17". The overall compressive stress within the member precludes the need for any additional tension steel to be designed for the member. These figures do not account for endblock design or shear reinforcement design.