15th World Congress on Computational Mechanics (WCCM-XV)
8th Asian Pacific Congress on Computational Mechanics (APCOM-VIII)
Virtual Congress: 31 July – 5 August 2022
S. Koshizuka (Ed.)

FORENSIC EVALUATION OF HISTORIC SHELL STRUCTURE: DEVELOPMENT OF IN-SITU GEOMETRY

JOSHUA A. SCHULTZ[1], KATRINA L. SPRINGER[2], JOSEPH A. ROBERSON[3], JACKSON M. CARROLL[4], MARK D. HEGBLOOM[5]

[1] Gonzaga University, 502 E Boone Ave Spokane WA 99258 USA, schultzj@gonzaga.edu, www.gonzaga.edu

[2] Gonzaga University, 502 E Boone Ave Spokane WA 99258 USA, kspringer@zagmail.gonzaga.edu, www.gonzaga.edu

[3] Gonzaga University, 502 E Boone Ave Spokane WA 99258 USA jroberson@zagmail.gonzaga.edu, www.gonzaga.edu

[4] Gonzaga University, 502 E Boone Ave Spokane WA 99258 USA, mhegbloom@zagmail.gonzaga.edu , www.gonzaga.edu

[5] Gonzaga University, 502 E Boone Ave Spokane WA 99258 USA, jcarroll4@zagmail.gonzaga.edu, www.gonzaga.edu

Key words: Concrete Shell, Historic Structure, Hyperbolic Paraboloid

Abstract. *When completed in 1961, the roof of St. Charles Church became the largest unbalanced hyperbolic paraboloid structure in the world and the only shell structure in Spokane, WA. Situated on an 8-acre site on the north side of the city, St. Charles is a modernist structure designed through partnership of Funk, Molander & Johnson and architect William C. James. This asymmetric structure is over 45.72m (150ft) and utilizes folded edge beams that taper from 1067mm (42in) at the base to a 76.2mm (3in) thickness at the topmost edge using regular strength reinforcing steel and concrete. The novelty of the shell structure serves both architectural and structural design criteria by delivering a large, uninterrupted interior sanctuary space in materially and economically efficient manner.*

Having previously completed an initial analysis of the structure, now, 60 years later, a complete structural forensic evaluation of the shell has been conducted using full point cloud laser scanning to generate a complete in-situ model. The in-situ geometry and historic loads are described and deflections as first steps in a full structural forensic study. Results of the current in-situ geometry are compared to the design geometry of original construction documents.

1 INTRODUCTION

The asymmetric pre-stressed thin-shell concrete roof of St. Charles church in Spokane, WA is a notable hypar (i.e., hyperbolic paraboloid) haecceity. The structure is a result of professional collaboration among the Spokane architect, William C. James, engineering firm Funk, Murray, and Johnson and professor T.Y. Lin of the University of Berkeley. The shell spans 39m (128ft) to either abutment and 45.72m (150ft) from either elevated end. The anticlastic shell utilizes folded edge beams that taper from 1067mm (42in) at the base to a 76.2mm (3in) thickness at the topmost edge in conjunction with standard and pre-stressing reinforcing steel to resolve the shear and moment in the edge beams [1].

Upon completion in 1961, the structure became the largest hyperbolic paraboloid in the United States and remains the only pre-stressed hypar shell structure in Spokane, Washington. The reinforced, thin shell roof allows the structure to economically span large distances without the use of interior columns or supports as shown in Figure 1.

Figure 1: Hypar Roof at St Chales, Spokane, WA in 2022.

Past research has investigated the construction documents and analyzed the structure using design geometry. This report focuses on developing the in-situ geometry and historical loading for the forensic evaluation of the structure for the last 60 years. The 1955 and 1958 building codes are used to calculate wind, snow, and seismic loads for the design in pursuit of a modern comparison. A laser scan of the roof was then carried out and, geometry was compared to the design documents. By comparing the modern geometry of the laser scan and the historical designs of the roof, a deeper understanding of the hyperbolic paraboloid roof structure's performance over the last roughly 60 years is attained.

2 BACKGROUND: HYPERBOLIC PARABOLOID

Early experimentation with hyperbolic paraboloid as a structure has been largely attributed to work by Antoni Gaudí and others in the during the end of the 19th century and into the early 20th. Gaudí's hanging chain models of the Sagrada Família reinvented nonlinear architectural design and inspired architects throughout Europe to invest further into unorthodox applications of catenary shapes and hyperbolic paraboloids [2]. Among them were two French engineers

Bernard Lafaille and Fernand Aimond, whose 1933 work with double-cantilever concrete canopies and structural analysis of hyperbolic paraboloids, respectively, laid the scaffolding for hyperbolic paraboloids in modern structures [3]. However, further work with these unorthodox architectural approaches was halted by the Second World War and interest did not return until 1945 at the war's completion. With the War having ravaged Europe's center, a migration of engineers and architects to the United States began and along with them a desire for innovation [4, 5].

The rise in popularity of hyperbolic paraboloids among architects, structural engineers and master builders began in the United States midway through the 20th century and had become a full-fledged icon for innovation by the construction of St. Charles Church in 1961. Massive projects like the saddle dome Dorton Arena in Raleigh, North Carolina inspired communities throughout the United States to invest in this economical form of construction. By the 1970s, however, the fad had passed with decreasing costs of structural steel and an increase in skilled labor across the country. The previous economic innovation had lost its sheen to post-modern architectural style and growing favor of structural performance and economic efficiency [4, 5].

Prestressed concrete plays a critical role in the structural integrity of the St Charles hyperbolic paraboloid and the structural approach would not function without its integration. Eugène Freyssinet, a French structural engineer, is considered to have pioneered prestressed concrete in the early 20th century due to his work with prestressing in bridges and thin-shell structures. Building on Freyssinet's early work with prestressed concrete, Chinese-American structural engineer Tung-Yen Lin developed standardized practices surrounding prestressed concrete in the mid-20th century leading to widespread use of the prestressing throughout the United States and around the world. The general principles of prestressed concrete developed by these two pioneers are to transform concrete into an elastic material and to develop a material/structure that is a combination of high-strength steel and concrete. These two principles allow prestressed concrete to be under tension when in a structure (which has been particularly useful for bridge design) and allows more extensive use of thin-shell concrete due to the additional structural support gained from high-strength steel [6].

3 HISTORIC LOAD DEVELOPMENT

St. Charles hypar roof was designed in 1959 and design manuals and reference materials of the period were consulted for development of historic loads. A few of the manuals and reference materials utilized include: 1955 Uniform Building Code (UBC), 1958 American Concrete (ACI 318-58), as well as the extant 1959 Spokane Building Code. These codes have been compared to those that are utilized today, such as ACI 318-19, American Society of Civil Engineers Minimum Design Loads (ASCE 7-16), International Building Code (IBC) 2018, as well as applicable service loads from the State of Washington's current building code reference, IBC 2015 [7-11].

3.1 Dead Load

Dead loads were developed using the geometry of the roof (both design documents and subsequent laser scan) and assumed reinforced concrete density as obtained from page 331 of the 1958 UBC of 2402 kg/m3 (150pcf). The edge beam varies in thickness from 76.2mm – 609.6mm (3 – 24in). The center cross- sections vary from 76.2mm – 203.2mm (3 – 8in).

Averaging the density by the varying thickness led to a dead load of 3.0kPa (62.5psf).

3.2 Lateral (Seismic) Load

The UBC 1955 was utilized to calculate the earthquake load on the roof using the seismic zone map showing that Spokane is in zone 2B. As a result, the horizontal force factor, C, will be doubled. Table 23-C gives an equation to calculate the value of C=0.067 for a roof where N is number of stories which equals zero. The resulting lateral force, F = 0.2kPa (4.2psf) is calculated using Section 3.3.1 and equation from page 313.

3.3 Roof Live Load (Snow Load)

Per Table 23-B the roof live load (i.e., minimum snow load) is taken as 0.96kPa (20psf).

3.4 Wind Load

Per section 2307 of 1955 UBC, the wind load was determined by a weighted average at mean roof height of (45ft), q = 0.85kPa (17.8psf).

3.5 Load Combinations

Governing load combination was found to be ASD Dead + Wind + Snow.

4 IN-SITU GEOMETRY

4.1 Laser Scan Procedure

To determine the in-situ geometry, the St. Charles Church underwent an exterior geometry scan utilizing a FARO Focus3D X laser scanner. After establishing the local control points, as shown in Figure 2, a full laser scan to point cloud model was completed.

Figure 2: Laser scanning set-up and control points

The resulting data is a collection of over 16 million data points with X, Y, Z coordinates as shown in Figure 3. Filtering the data, one million data points are utilized for three-dimensional analytical framework of the structure was able to be imported into numerous software to complete the in-situ portion of the Hypar roof study.

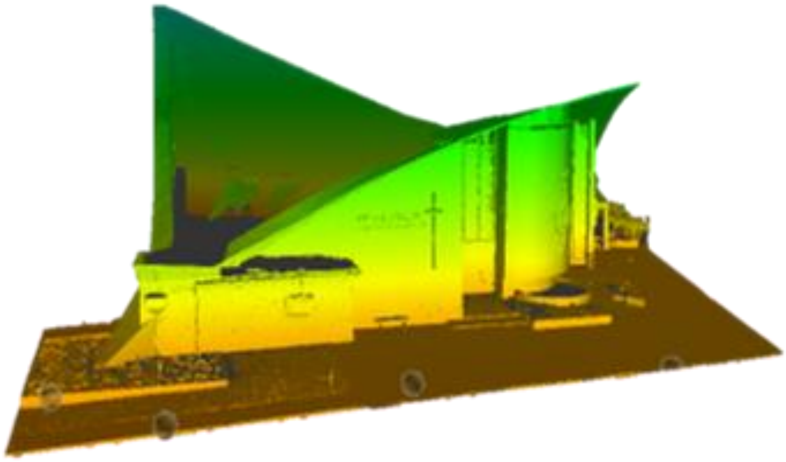

Figure 3: Laser Scan with elevation gradation in Autodesk ReCap

The point cloud was imported into Civil3D, and the roof and edge beam were traced. The coordinates of approximately 90 points per side were then imported into MATLAB. By using the Curve Fitting Toolbox, equations for each line and surface were found. These equations output 30 equally spaced nodes for each edge beam that were imported into RFEM. Since the laser scan validated that the roof is symmetrical, these points were mirrored, and an outline of the roof was created.

4.2 Roof Geometry

Figure 4 displays an overlay of the geometry from the plan sets and the geometry from the laser scan. From the comparison of the roof heights, there is a difference of 152.4 – 203.2mm (6 – 8in) between the tip-to-tip elevation as described in the construction documents and the in-situ geometry.

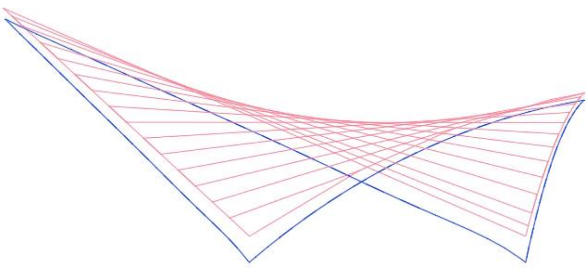

Figure 4: Original construction document geometry (red) vs in-situ scanned (blue) geometry

Based on field evaluations of the shell, there appear to be minimal tension cracks in the shell.

This observation, coupled with the reality of the large geometric stiffness, indicates that the initial formwork and structure were not placed exactly as provided for in the construction documents. From the lack of structural cracking, it is observed that minimal subsequent creep occurred.

This new geometry is shown in Table 1 and will serve as the basis for future studies. Due to the highly indeterminate nature of the structure, coupled with the large geometric stiffness inherent in the hypar shape, it is anticipated that the actual stresses will be significantly different from the calculated values that used the idealized mid-plane coordinates from the construction documents [1].

Table 1: In-situ geometry of the shell mid-plane from laser scan

Point	X (m)	Y (m)	Z (m)	**Point**	X (m)	Y (m)	Z (m)
1	11.2	64.6	545.9	**31**	47.5	60.4	529.1
2	12.2	64.6	545.6	**32**	47.8	60.4	528.8
3	13.4	64.3	545.0	**33**	47.9	61.0	529.4
4	14.6	64.3	544.4	**34**	48.0	61.9	530.0
5	15.8	64.0	543.8	**35**	48.2	62.8	530.7
6	17.1	64.0	543.2	**36**	48.3	63.7	531.3
7	18.3	63.7	542.8	**37**	48.5	64.6	531.9
8	19.5	63.7	542.2	**38**	48.6	65.5	532.5
9	20.7	63.4	541.6	**39**	48.8	66.4	533.1
10	21.9	63.4	541.0	**40**	48.9	67.4	533.7
11	23.2	63.1	540.4	**41**	49.1	68.3	534.3
12	24.4	63.1	540.1	**42**	49.2	69.2	534.6
13	25.6	62.8	539.5	**43**	49.4	70.1	535.2
14	26.8	62.8	538.9	**44**	49.5	71.3	535.8
15	28.0	62.5	538.3	**45**	49.7	72.2	536.1
16	29.3	62.5	537.7	**46**	49.8	73.2	536.8
17	30.5	62.2	537.4	**47**	50.0	74.1	537.1
18	31.7	62.2	536.8	**48**	50.1	75.0	537.4
19	32.9	61.9	536.1	**49**	50.3	76.2	537.7
20	34.1	61.9	535.5	**50**	50.4	77.1	538.0
21	35.4	61.6	534.9	**51**	50.6	78.0	538.3
22	36.6	61.6	534.3	**52**	50.7	78.9	538.6
23	37.8	61.3	534.0	**53**	50.9	79.9	538.9
24	39.0	61.3	533.4	**54**	51.1	80.8	539.2
25	40.2	61.0	532.8	**55**	51.2	82.0	539.5
26	41.5	61.0	532.2	**56**	51.4	82.9	539.8
27	42.7	60.7	531.6	**57**	51.5	83.8	539.8
28	43.9	60.7	531.3	**58**	51.7	84.7	540.1
29	45.1	60.4	530.7	**59**	51.9	86.0	540.4
30	46.3	60.4	530.0				

5 NUMERICAL ANALYSIS

For preliminary finite element analysis, consistent material properties of reinforced steel were assigned to all members of the roof. A modulus of elasticity of 48.2MPa (7ksi), shear modulus of 20.7MPa (3ksi) and specific weight of 3203kg/m^3 (200pcf) were assumed for the reinforced concrete. The edge beam varies in size from 76.2mm – 203.2mm (3 – 8in), so this was represented in RFEM by 8 different cross-sections. These members have a width of 304.8mm (12in) and a depth ranging from 149.86mm (5.9in) at the top to 609.6mm (24in) at the bottom. The concrete shell surface was given a uniform thickness of 152.4mm (6in) to represent an average of the shell's varying thickness. The abutments were represented by 10 pin connections at each end and the masonry walls were represented by 5 roller supports on two of the edge beams. The governing load combination from Section 3.5 was used. No pre-stress force was included in this model. The surface was divided into sixteen sections across each end which allowed for a variety in thickness in both the x and y direction as shown in Figure 5.

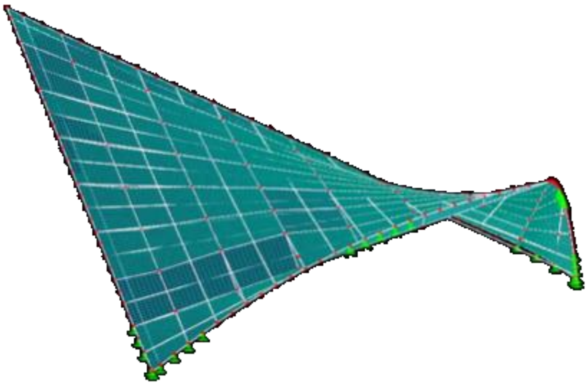

Figure 5: FEA shell elements for model (no pre-stress)

Maximum displacement was 36.1mm (1.42in) as shown in Figure 6. The maximum bending moment of the edge beam came out to be 457kN-m (31.3kip-ft). The maximum axial force in the edge beam was located near the abutments with a value of 1148kN (258kips). As with the last model, shear was relatively constant throughout the field with a maximum value of 99.3kN (22.34kips) in the edge beam. The maximum tension in the shell came out to 4.54kN (1.021kips) and the maximum compression came out to be -8.4kN (-1.89kips).

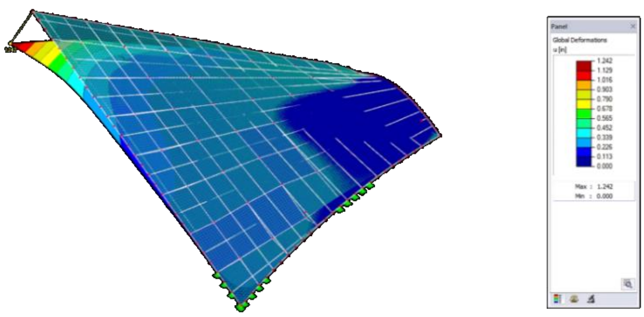

Figure 6: Deformation of FEA model using in-situ geometry

6 CONCLUSIONS

This research summarizes the historical loading and the in-situ geometry for the hypar roof at St Charles in Spokane, WA. The in-situ geometry was compared to the theoretical geometry that was provided in the construction documents and previously analyzed [1]. Actual geometry shows that the hypar has a difference of 152.4 – 203.2mm (6 – 8in) between the tip-to-tip elevation as described in the construction documents and the in-situ geometry. The resulting geometry and loading serve as baseline information for the full forensic structural evaluation which is ongoing.

REFERENCES

[1] Schultz, J.A. and Henriksson, V. Structural assessment of St. Charles hyperbolic paraboloid roof. *Curved and Layered Structures*, (2021) 8(1):157-166.
[2] Huerta S. Structural design in the work of Gaudi. *Architectural science review*. (2006) 49(4):324-39.
[3] Espion B. Pioneering hypar thin shell concrete roofs in the 1930s. *Beton-und Stahlbetonbau*. (2016) 111(3):159-65.
[4] Sprague, T. S. "'Beauty, Versatility, Practicality': the Rise of Hyperbolic Paraboloids in Post-War America (1950-1962)." *Construction History*, (2013) 28(1), 165-184.
[5] Sprague, T. *Sculpture on a Grand Scale: Jack Christiansen's Thin Shell Modernism*. University of Washington Press, (2019).
[6] Lin, T. Y., and Burns, N. H. *Design of prestressed concrete structures*. John Wiley & Sons, New York, NY. (1981).
[7] Uniform Building Code. International Conference of Building Officials, Los Angeles, CA. (1958).
[8] Uniform Building Code. Pacific Coast Building Officials Conference, Los Angeles, CA. (1955).
[9] International building code® 2018. International Code Council, Inc., Country Club Hills, IL. (2018).
[10] Minimum design loads and associated criteria for buildings and other structures: ASCE/SEI 7-16. American Society of Civil Engineers, Reston, VA. (2017).
[11] Municipal Code Section 17F.040.105 - Design Criteria. Spokane Municipal Code, City of Spokane, Spokane, WA. https://my.spokanecity.org/smc/?Section=17F.040.105 (Jan. 13, 2022). (2022).