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Design of the roof for the Grandview Heights Aquatic Centre, Surrey, Canada

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Synopsis

Constructed to meet the diverse needs of one of Canada's fastest-growing cities, Grandview Heights Aquatic Centre (Figure 1) features an undulating roof structure with hanging timber 'cables' suspended between large concrete buttresses. It is believed to be the world's largest-spanning pure catenary timber roof, highlighting wood's potential as a cost-effective, structurally efficient and aesthetically pleasing building material for aquatic facilities.

Introduction

Surrey, a suburb of Vancouver with a population of more than 500 000, has experienced unprecedented growth during the past several decades and is expected to surpass Vancouver as the province's most populous city between 2020 and 2030.

When the city began its search for an integrated design team for the Grandview Heights Aquatic Centre (GHAC) project, its aim was to build an iconic 'destination pool' that would be a catalyst for civic growth and a pivotal first piece in a larger recreational master plan for the area, to be built over the next decade.

The client had gained a reputation in recent years for its high expectations – Surrey procurement staff demanded functional and striking architecture that would change the face of the burgeoning metropolis and establish it as a city in its own right, rather than a 'bedroom community' of Vancouver commuters.

GHAC was to be designed to LEED (Leadership in Energy and Environmental Design) certification standards and to meet stringent International Swimming Federation (FINA) regulations for regional, provincial, national and international sporting events. To host competitions, the facility would need a 10-lane, 50m, Olympic-size competition pool and a dive platform, along with seating for up to 900 spectators.

It was, however, also crucial to balance the facility's function as a recreational hub where Surrey's diverse community would feel welcome – and so a 500m² leisure pool with lazy river, waterslide, hot pools, sauna, universal changing rooms and fitness centre were included in the programming (Figure 2).

Structural engineers at Fast + Epp pioneered a novel approach for GHAC in collaboration with HCMA Architecture + Design. The aquatic facility became one of the Vancouver-based firm's more ambitious projects in its 30-year history, although Fast + Epp engineers are no strangers to complexity – internationally-recognised projects include the 2010 Richmond Olympic Oval 'WoodWave' roof, VanDusen Botanical Gardens Visitor Centre (Vancouver) and the world's tallest contemporary wood building, a 17-storey mass-timber student residence called Brock Commons at the University of British Columbia.

Concept design

For an aquatic centre of such size, the design team recognised the roof structure as a crucial point of visual interest. An initial prompt from the architect to consider spanning in the counterintuitive long direction led to the idea of a catenary roof structure.

Figure 1 Grandview Heights Aquatic Centre

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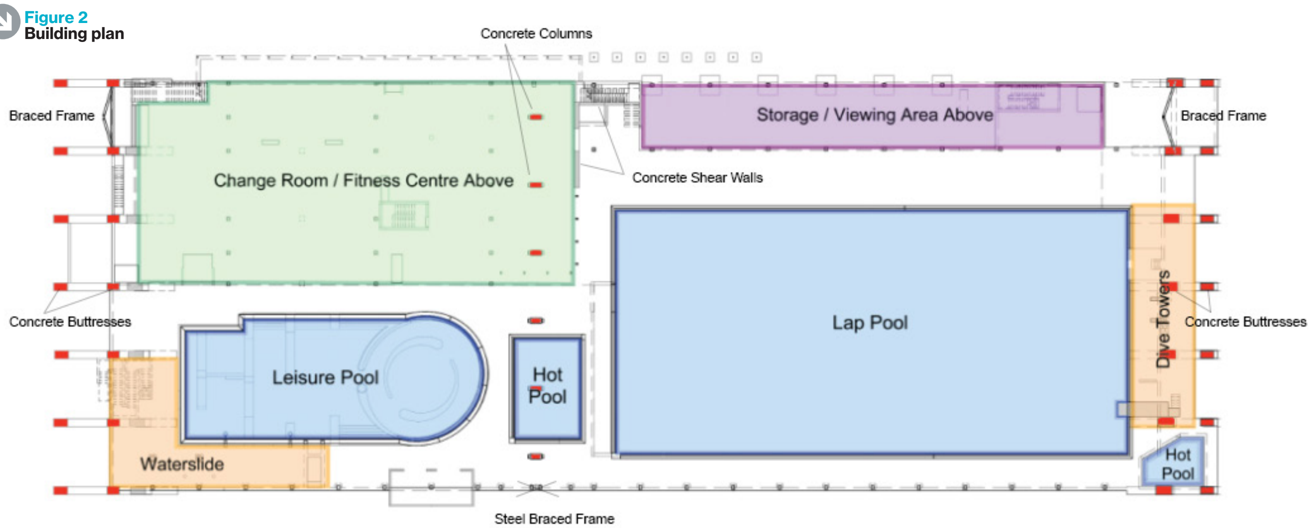
Figure 2
Building plan



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Economy of design is typically achieved by spanning primary structural elements in the shorter direction of a building. For GHAC, this would have meant spanning 35m across the swimming pools instead of 100m over both the leisure and lap pools. When the architect challenged the team to 'think outside the box' and explore spanning the primary structure in the longer span direction, it was readily apparent that this break with convention had the potential to substantially increase costs.

Yet the design team was confident: an earlier arena project by Fast + Epp had taken a similar approach, the building was bisected with a steel Vierendeel arch spanning in the long direction, which in turn had facilitated a wood structure with a more reasonable span between the arch and exterior walls in the shorter span direction. Not only did this result in a very dynamic form with timber components, but it also proved to be a cost-efficient solution because it reduced the area of the building envelope.

In the case of GHAC, there were two points in the building that would govern height – a 10m dive tower at one end and a water slide tower at the other. The roof structure could drop over all other areas including the second-level fitness room. Additionally, the architect sought to maximise security and accessibility with clear views from the central lobby into the natatorium, and to provide a linear orientation of the roof parallel to the pool lanes to benefit competitive swimmers as they progressed through the water.

In light of these constraints, the team suggested a slender and light timber catenary structure with multiple, shallow, glue-laminated timber (glulam) 'cables' (Figure 3). The architect initially thought this idea somewhat audacious – shouldn't steel cables support wood components, as seen in similar, slender, precedent-setting structures such as Eero Saarinen's design for the main terminal at Washington's Dulles Airport, which used steel cables with thin concrete infill, or the Portuguese National Pavilion for Expo '98 with a thin reinforced concrete catenary slab? These structures relied on sufficient self-weight to resist wind uplift, whereas a timber catenary structure would be significantly lighter.

But a typical steel-cable roof structure with infill timber components would be more connection intensive and would be more susceptible to long-term corrosion in an aquatic environment, the team reasoned. It would also require additional measures to resist wind uplift. So it was decided that wood should be used for GHAC's hanging suspension roof. The architect jumped on board with the unconventional approach and the entire team worked hard to overcome obstacles.

While timber is more commonly used in compression elements such as arches when designing long-span roofs (particularly when splices can be achieved through relatively economical bearing connections), exceptions confirm the rule. In the case of GHAC, the catenary structure effectively 'shrink-wraps' the building with a thin, warped roof structure, resulting in a structural depth of 300mm, versus an estimated 3000mm depth if conventional steel trusses were spanning

Figure 3
Catenary concept

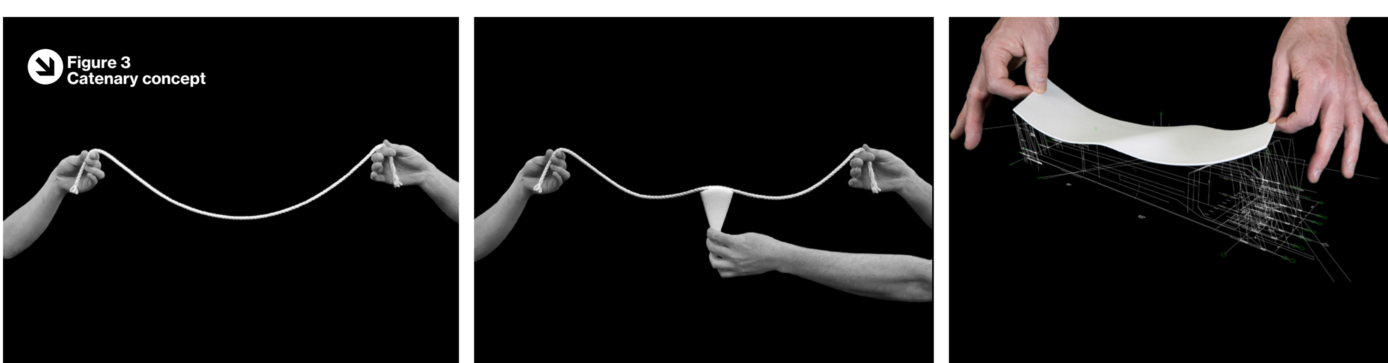

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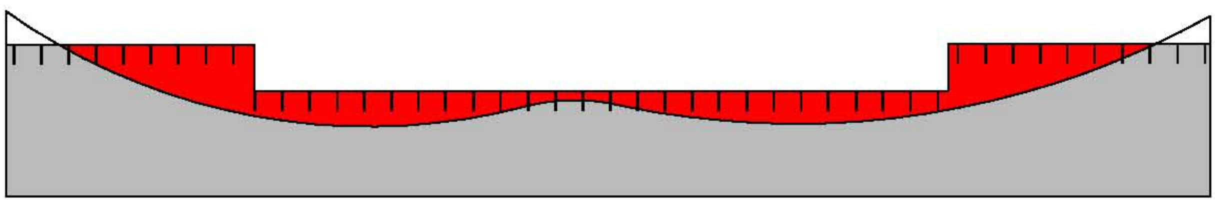
Figure 4
Volumetric efficiency. Red shading denotes simplified truss system option, while grey denotes final scheme



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in the short building direction. This design achieved an estimated 20% reduction in building volume and envelope costs, not to mention future energy cost savings by reducing the volume of air to be heated and dehumidified. These savings more than offset the cost of tension connections at splice locations and end conditions (Figure 4).

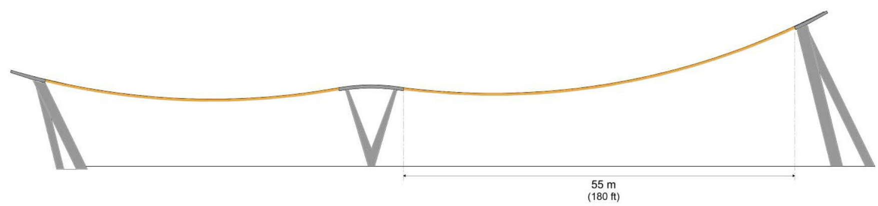
The design team performed some preliminary analysis, whereupon it was quickly decided that a mid-span V-column support between the two pools would be introduced to reduce the spans from 100m to 55/45m, in order to minimise complexity and cost. A catenary structure consisting of pairs of 130mm × 266mm glulam 'cables' spaced at 800mm centres became the primary tension element that would span between concrete buttresses at the ends of the building and the mid-span concrete V-columns (Figure 5). The west-side buttress was outward leaning to provide space for the waterslide platform, while the east-side buttress was inward leaning to provide clearance between the stacking 5m and 10m dive towers.

The glulam cables were covered with a double layer of 16/12mm plywood decking. Concrete slabs over all three support points served as connecting diaphragm elements, transferring loads into the buttresses and V-columns. The buttresses could resist overturning by resting on backfilled raft foundations, founded at the lower pool slab raft elevation (which already governed the excavation depth).

Overall stability of the structure in the north–south direction was provided by perpendicular brace frames within the buttresses, at each end of the building. The central roof section was stabilised by a double-height concrete shear wall in the lobby area. Meanwhile, from east to west, buttresses provided support to the ends of the building, while a concrete shear wall at the north wall and a steel brace frame at the south wall ensured stability to the centre of the building.

Refinement of the roof geometry was paramount to the success of the structural concept. The clear height requirements varied drastically from extremely high at dive towers and the water slide to low over swim areas. The roof shape was warped in order to minimise building volume as well as to create a slope for rainwater management. The buttresses were slightly tilted at each end to enhance the dynamic aesthetic of the resulting undulating, wave-like roof form.

Figure 5
Building section



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Initially this resulted in no less than 14 radii of glulam cable curvatures and prohibitive costs for each custom glulam jig manufacture – enough to sink the structural concept. However, the geometry was then refined so that only one radius of curvature and jig was used for every glulam cable. By simply lengthening and raising the ends of each adjacent glulam cable slightly, the warped roof geometry was achieved by much more economical means. The spaghetti-like glulam cables were erected on site in just 12 days (Figure 6).

The facade structure, meanwhile, reached up to 20m high and was constructed with steel tube columns that serve a double function – they not only resist wind loads, but were perforated and connected to the basement air supply ducts, acting as ventilator ducts to prevent condensation at exterior glazing. This eliminated costly and unsightly mechanical ducting (Figure 7).

Nevertheless, no project is all smooth sailing – pioneering a novel structural approach presented a range of technical challenges for the design team to overcome.

Design challenges

Roof deformations

When the decision was made to pursue a catenary-type roof structure, the immediate follow-up question was, 'How large are the anticipated deformations and how do we deal with them?' Catenary structures change shape based on the loading, to achieve equilibrium and stability.

Because of this change in shape, any structure or building component connected to the catenary roof must allow for these movements, both vertically and horizontally. Under a balanced and constant load, the catenary shape would remain relatively static, but under variable loads (such as snow) there was a possibility of snow drifting or sliding into different regions of the roof, reducing loading in some areas, and increasing loading in others. Canadian building codes provide snow distribution tables for curved roofs and require a minimum snow load variation of 100% of snow load in one area and 50% of snow load in an adjacent area to determine the most negative structural effect.

Initial calculations based on a hinged, central V-column support yielded up to 1200mm vertical deformation under unbalanced snow conditions (which nearly caused the architect to faint!). Considered unacceptable, it was decided that the central concrete slab spine supported by the V-columns would be locked in to prevent any lateral movement in the direction of the catenary span. This was achieved by placing a concrete shear wall at one end of the slab and vertical steel bracing at the opposite end.

Subsequent vertical deflections were calculated to be in the range of 300–400mm – numbers that still would have been impossible for the building envelope curtain wall detail to accommodate with standard, off-the-shelf hardware. However, the design team chose not to specify custom components for such a large facade in order to reduce the costs and liability. The largest movement allowed for by an off-the-shelf facade envelope connection was 200mm – the new deflection target. The design team concluded that the probability of realising such extreme unbalanced loading in southern British Columbia, where snowfall is very wet and not prone to drifting, was minimal. Furthermore, this was considered a serviceability issue rather than a life-safety issue; therefore, sliding facade connections were designed for maximum vertical movement of 200mm (Figure 8).

Sliding snow was also a concern, as the specified roof membrane was slippery and roof slopes approached 40° from horizontal in some locations. To reduce the risk of sliding snow accumulating in the centre of the span and causing unacceptable deformations, a snow consultant was retained to design a curb system for snow retention (Figure 9).

Wind uplift

A further concern related to wind forces, with respect to both static behaviour and dynamic excitation.

Static behaviour

One of the benefits of the catenary shape is the resulting small structural sizes. Designing a member primarily for tension loading yielded a slender section that could achieve the necessary capacity. However, wind uplift forces could work against the design of such a lightweight roof – the relatively light timber cable structure had insufficient self-weight to prevent wind forces from lifting the roof.

While extra weight could be added (e.g. concrete topping) to overcome wind uplift, the additional weight in turn would trigger larger loading and higher design forces. Adding steel hold-down cables inside the building would be unsightly and was therefore not considered a viable option.

GHAC's design found the sweet spot between self-weight and the wind uplift forces. A timber cable structure differentiates itself from traditional steel cable structures in that it has some inherent bending stiffness. Wood cables were sized to have sufficient strength to resist snow loads and self-weight in tension, and just enough strength and inherent stiffness to resist wind uplift as skinny compression arches – a close-to-perfect balance and an efficient use of material.


Figure 6 Glulam cables

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Dynamic excitation

Given the extremely slender profile of the roof, the question arose of whether the roof could be subjected to unacceptable dynamic excitation, effectively becoming a 'Galloping Gertie' of Tacoma Narrows fame.

In discussions with the wind consultant, it was estimated that the geographical location, building orientation and roof shape meant wind frequencies likely to affect the roof were less than 1Hz. It was anticipated that if the natural frequency of the roof was above 1.5Hz, there would be little risk of excitation.

Figure 7 Facade column air supply

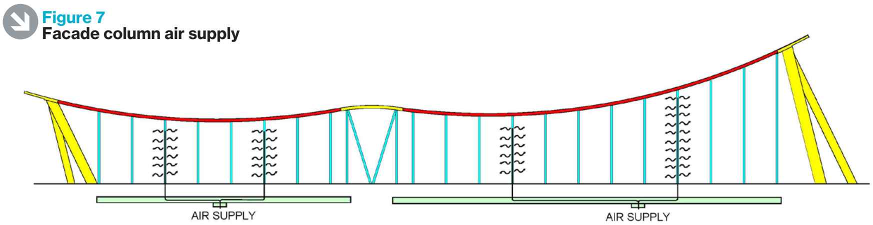

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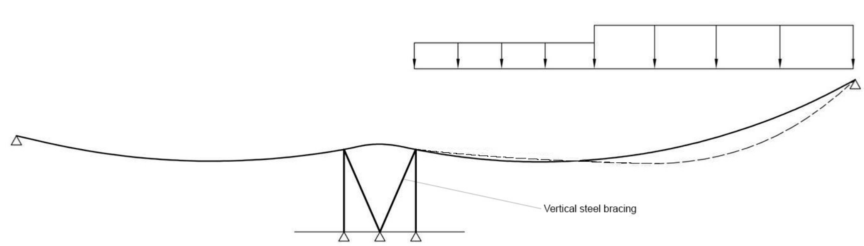
Figure 8 Deformed shape with unbalanced loading



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When analysed as a two-dimensional, simple-span timber cable (first mode), the natural frequency was in the range of 0.9–1.0Hz. When analysed as a three-dimensional warped roof structure, taking into consideration boundary restraint conditions as well as the stiffening effect of preload from self-weight, the frequency rose to 1.35Hz.

Engineers felt the proposed warped roof geometry, as well as the damping effect of glued roof insulation, would sufficiently mitigate the potential for resonance. A 150mm thick, glued-on insulation and backing panel with membrane roofing on top was attached to the primary plywood glulam cable structure. However, in order to confirm these assumptions and avoid costly wind tunnel modelling, *in situ* testing was conducted using accelerometers, a metronome and a 'jumping party' as soon as the roofing was installed.

Crews on site found it exciting to observe how the stiffness of the roof incrementally increased from the individual hanging glulam condition to the plywood sheathed condition, and finally to the fully-roofed condition (which included the snow sliding prevention curbs for further added stiffness).

Based on the *in situ* testing, the wind consultant estimated the natural frequency of the roof to be 1.7Hz. Coupled with favourable damping effects, the design team was confident the roof would perform well under wind load conditions and no further wind tunnel testing, roof stabilisation or boundary condition adjustment (e.g. raising the end parapet height) was required.

Fabrication challenges

The goal of the design team was to wrap the building as tightly as possible to reduce its volume. The dive tower had specific requirements from the diving regulatory body for clearance from the edge of the platform; thus, this element defined the highest point of the roof (Figure 10). At the opposite end, the waterslide governed the height requirement, as did a fitness area at the low point of the catenary. Furthermore, slope for roof drainage was required in the perpendicular-to-long span direction. The decision to set the central concrete spine slab at a constant elevation to reduce geometric complexity and provide consistency throughout the building rounded out the geometric setting points for the roof and resulted in a warped roof form.

The first iteration of the roof was performed with multiple radii forming the curve of the roof. This optimised the shape but added cost and complexity to the glulam fabrication. Knowing the glulam cables would be built using fixed jigs (and a different jig for each radius), the design team worked to develop shapes that would use the same radius or nesting radii.

As previously mentioned, all glulams were designed to have an identical radius (with only one jig required) and the warped geometry was achieved by slightly increasing the length and elevation of each adjacent glulam cable. This greatly reduced expected fabrication costs and allowed the team to stay within its budget.

Figure 9
Snow retention curbs on roof

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The radial geometry was chosen not because of optimal structural behaviour, but rather for ease of fabrication. In reality, the catenary shape is not a constant radius and bending moments develop when forming the catenary shape. However, due to the shallow depth of the glulam cables, the bending moments are small when compared to the tension stresses and overall capacity.

Glulams were installed with the larger dimension placed vertically, yet the roof slope (perpendicular to the glulams) varied from almost flat to approx. 10°. To allow for this slope, the glulam supplier produced shims to suit each location. The cross-slope varied along the length, so cutting each glulam would have required a cut that curved in two directions. Hence, a shim solution was developed. The plywood diaphragm had some flexibility to absorb small angles, but the shims allowed the large geometry changes necessary for the overall roof shape.

Connection design and erection

Early in the design process, consideration was given to how the roof structure could be erected – should it be installation with single-piece glulam erection or a prefabricated panel-type erection procedure? Speed of construction was a key issue on site due to southern British Columbia's heavy winter rains; crews needed a strategy to minimise exposure of the wood to moisture before it was covered. The contractor was encouraged to schedule roof construction for summer months to reduce the probability of extended rain exposure.

Glulam 'cable' lengths were constrained by 25m transportation limits; hence, the longer span required two splices while the shorter span only needed one. Typically, timber connections that involve fastening wood on site with bolts or screws require more care and attention during installation than simply connecting steel to steel with bolts. Thus, connections were developed that would not require wood to be connected on site (Figure 11).

GHAC connections consisted of five steel plates to link four glulams together. Each glulam end had a galvanised steel plate screwed to its inside face with 134 screws. This plate had three holes to accommodate 25mm diameter steel bolts. The twinned glulams were joined together with a single 22mm thick steel plate and connected with six steel bolts (three in each pair of glulams). These bolts were easily installed on site and did not connect to the wood directly, allowing

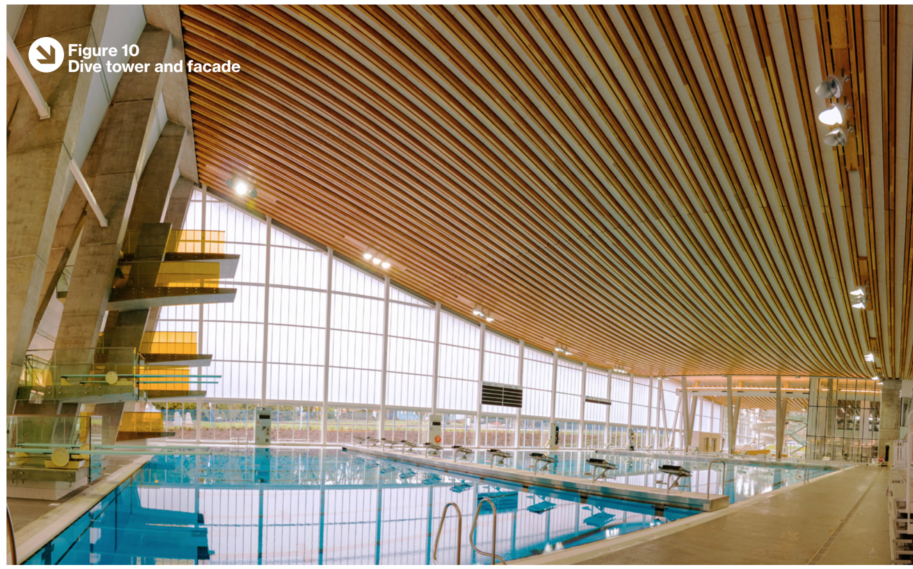

Figure 10
Dive tower and facade

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Figure 11
Glulam cable splice

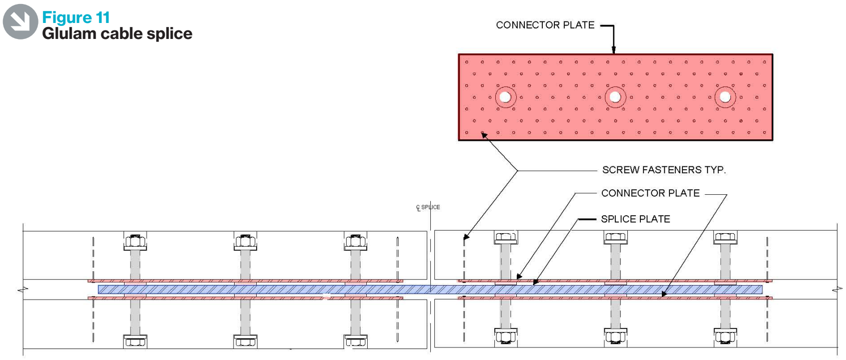

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Figure 12
Plywood decking over glulams

ANDREW GOODBRAND, ELLISDON

Figure 13
Erection sequence

ELLISDON

basic ratchets and wrenches to be used for the connections.

Several concerns were raised regarding a panelised solution. First, it was thought a panelised approach would likely require multiple costly jigs on site for prefabrication to achieve a short erection period. Laydown space on site was also limited. Shop prefabrication of such large panels was also considered impractical from a shipping perspective. Finally, connecting multiple glulam ends all at one time under tight tolerance conditions was also considered risky.

To avoid these potential problems, twinned glulam cables were installed one at a time. The plywood diaphragm was installed after the glulams were hanging in place, also to ensure there was no shear in the diaphragm from tension effects (Figure 12).

The slenderness and length of the glulams required a spreader beam to support them and prevent large deflections during erection. The spreader beam could handle the glulams (connected on the ground) on the shorter span but was not long enough for the longer span with three sections of glulam (two splices). Therefore, an overhead crane lifted two preassembled sections into place, while a smaller mobile crane lifted the remaining single section. After the shorter and longer span sections were connected to the concrete roof slabs, an erector in a boom lift installed three bolts at the splice location (Figure 13).

Even for this more complicated side, each lift only took 15–20 minutes. The entire roof was erected in 12 crane days, with the plywood diaphragm installed directly after glulam erection. Surface applied membrane was installed over the plywood to protect the timber from rain until the finished roof was completed.

Figure 14
Hollow perforated facade columns

NICK RUSSELL

Mechanical integration

Mechanical ducting and piping can become the bane of clean architectural expression. Aquatic environments typically require large-sized feeder ducts to distribute air throughout the building. Furthermore, sprinkler systems with associated piping are also typically required for larger timber structures.

In the case of GHAC, an analysis was performed by the code consultant which led to the rather intuitive conclusion that, given the height of the roof structure, the size of the timber components and the aquatic environment, no sprinklers would be required.

In order to avoid excessive exposed mechanical ducts, primary distribution ducts were located within a below-slab plenum space adjacent to the south-side facade, with feeder ducts transferring air into hollow, perforated, rectangular steel facade columns. Hence, the columns not only resist facade wind loading but also do double duty as mechanical distribution ducts. Utilising structure to perform more than just single-duty load support is an example of achieving structural sustainability (Figure 14).

Sustainability and social impacts

The GHAC project represented outstanding value for money and met the City of Surrey's budgetary expectations. The 8830m² facility was constructed for a hard cost of CAD \$44M; about CAD \$4983/m² or CAD \$463/sq.ft – and comparable to a steel solution. The final roof design minimised building volume and represents engineering efficiency and striking architecture, lending credence to the saying 'good structure is good architecture'.

Since its opening to the public in March 2016, GHAC has surpassed anticipated visitor numbers and exceeded the client's expectations. Initial reaction to the superstructure design of the building suggests that its striking aesthetic expression and ambiance will make it a favourite for years to come (Figures 15 and 16).

Project credits

Owner and client: City of Surrey
Structural engineer: Fast + Epp
Architect: HCMA Architecture + Design
General contractor: EllisDon Construction Services Ltd
Mechanical engineer: AME Consulting Group
Electrical engineer: Applied Engineering Solutions Ltd
Civil engineer: Binnie Consulting Ltd
Acoustics: Daniel Lyzun & Associates
Landscape architect: PFS Studio
Glulam fabricator: TBC

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Figure 15 Exterior west buttress

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Figure 16 Leisure pool and hot pool

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