

Design, Installation and Management of Remote Pedestrian Structures in Tasmania

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ABSTRACT

The Parks & Wildlife Service (PWS) in Tasmania manages a large network of walking tracks and associated pedestrian infrastructure including bridges, viewing platforms, elevated walkways and safety barriers. The design, installation and ongoing management of these structures presents challenges that are rarely encountered in a more urban setting. These can include difficult access, high rainfall, snow loading, freeze-thaw actions, coastal environments, bushfires and floods. The PWS has developed a range of practical solutions for managing assets in such environments. These include specialised structural designs, helicopter transport and lifting systems, visitor risk assessment tools and a risk-based approach to prioritising inspections and maintenance. This paper presents a summary of the designs, systems and management methods developed by PWS over recent years, including some examples of successes and failures.

INTRODUCTION

The Tasmania Parks & Wildlife Service (PWS) currently manages about 2.9 million hectares of land, or 43% of the state by area. This managed land includes about 2,715km of walking track, as summarised in the following table:

| Track Class (AS 2156) | Length (km) | Description (AS 2156 track condition) |
|-----------------------|--------------|---|
| 1 | 11 | Generally a broad, hard surfaced track suitable for wheelchair use. Width: 1200 mm or more. Well maintained with minimal intrusions. |
| 2 | 53 | Generally a modified or hardened surface. Width: 900 mm or more. Well maintained with minimal intrusions. |
| 3 | 375 | Generally a modified surface, sections may be hardened. Width: variable and generally less than 1200 mm. Kept mostly clear of intrusions and obstacles. |
| 4 | 1,060 | Generally distinct without major modification to the ground. Encounters with fallen debris and other obstacles are likely. |
| 5, 6 and routes | 1,216 | Limited or no modification to natural surface (except where required for environmental purposes). |
| Total | 2,715 | |

By length, the majority of the track network is AS 2156 Class 3 - 6. These lower standard tracks are mostly remote and can generally only be accessed on foot (sometimes several days walk) or by helicopter. Helicopter access is convenient but expensive, with current commercial rates in the vicinity of \$2,500 per hour. It is also dependant on weather conditions and can be problematic in forest and mountain areas.

On the walking track network, there are currently about 500 identified pedestrian bridges, 150 viewing platforms and numerous elevated walkways and safety barriers.

DEVELOPMENT OF ENGINEERING PROGRAM - 1990'S TO PRESENT

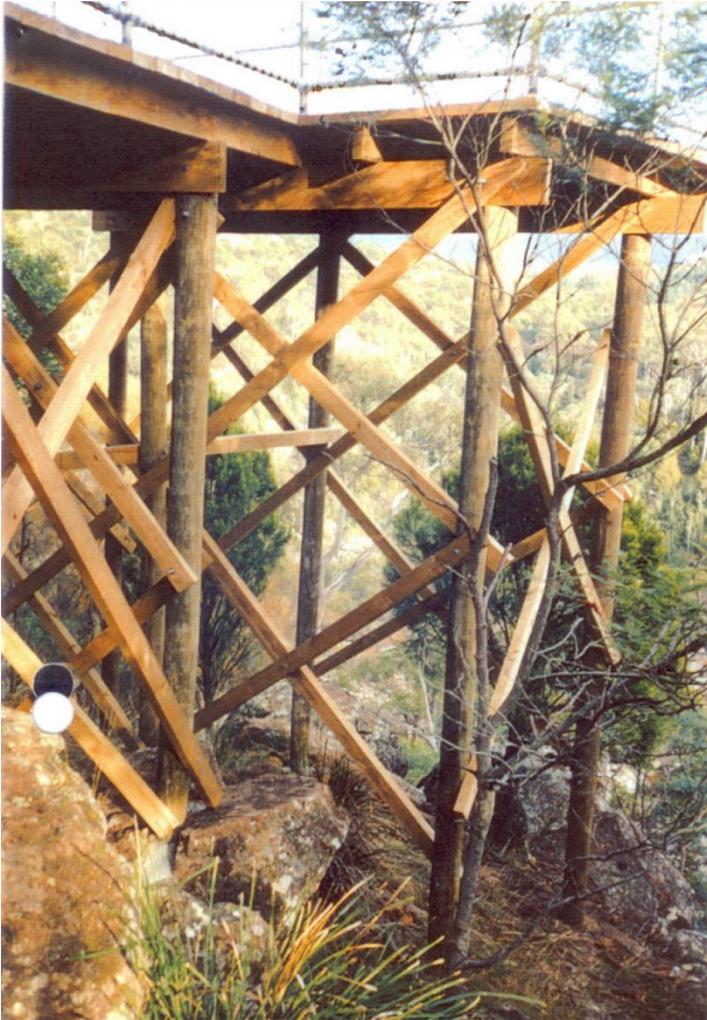
Prior to 1995, professional engineering services relating to design and inspection of structures at PWS were ad hoc, at best. In many cases structures were built by local staff with little or no engineering input. Most construction was from CCA treated radiata pine, often with nailed connections and minimal foundations. Construction was sometimes of a low standard, with minimal (if any) documentation and formal inspections were rarely undertaken.

In 1995 the collapse of a viewing platform at Cave Creek in New Zealand resulted in the tragic deaths of 13 students and one park ranger, with another four seriously injured (Noble 1995 and Bowling 2014). The shockwaves from this tragedy triggered dramatic changes in management of public infrastructure throughout New Zealand and Australia. At PWS in Tasmania, a critical “risk of closure” program was initiated, with all “elevated” pedestrian structures being deemed potentially unsafe and at risk of closure, unless certified fit-for-purpose by a professional engineer.

Between 1995 and 2005, PWS engaged engineering consultants to inspect and certify hundreds of walking track structures, throughout the state. During this period, many structures were replaced, strengthened or closed due to safety issues. Of particular concern were a number of identified structures that had the potential for catastrophic collapse, with possible consequences on a similar scale to those experienced at Cave Creek.



Typically low standard infrastructure dating from the 1980's, predominantly treated pine



Example of a structure that was closed during the first round of engineer inspections, post 1995



Footing detail

Following the first round of inspections and the resulting emergency works, closures and upgrades, funding for the initial safety program was largely exhausted. PWS moved into a new phase where the longer-term issue of the sustainability of routine inspections and maintenance became important considerations.

There was clearly a risk that if not managed strategically, the cost of engineering services would become unsustainable and available resources would not necessarily be directed where most needed. A prioritised approach was required that would provide acceptable inspection and maintenance regimes for different classes of asset, based on risk. The system would need to address the large number and type of assets, the visitor type/setting and also take into account the difficulties and cost involved in accessing remote sites. It was recognised that not all risk could be eliminated and that, particularly with limited resources, a more focussed and prioritised system for delivery of inspection and maintenance programs was required.

The publication of AS 2156 *Walking Tracks* in 2001, provided an improved framework for designers and inspectors to specify appropriate infrastructure for walking tracks of different standards. The concept of an “effective fall height” for elevated structures was particularly useful. At PWS many of the guidelines in AS 2156 have been adopted (and in some cases developed and adapted further in departmental policy).

In 2002 the Tasmanian Civil Liability Act was passed by parliament and officially proclaimed. This important legislation is still current and provides a framework for public authorities to manage their duty of care in maintaining public infrastructure and conversely to define what would constitute negligence. Under this act in Tasmania, it is now possible to manage public assets and visitor safety within a defensible framework, with a significantly reduced risk of litigation.

Within this context, PWS has developed its own asset management and visitor risk policies and recruited a small team of professional engineers and asset inspectors who can carry out inspection and maintenance work in a strategic and efficient manner. As part of this system, prioritisation of inspections is important, with high risk assets receiving the most scrutiny and low risk assets the least. To achieve this, an inspection schedule for each walking track structure is developed using a simple risk-based calculation or a “scorecard” that is simple enough to be used in the field. The scorecard sums points to give a risk score for each asset. Points are applied for:

- Fall height (measured to AS 2156);
- Structural complexity;
- Materials;
- Environment;
- Risk of damage;
- Age of the structure (as a percentage of design life);

A recent addition to the scorecard is a risk/consequence matrix which adjusts the output depending on the consequence of failure, for a given risk score. A summary of the two parts of the scorecard is presented on the following page.

| Effective Fall Height - Ref.AS2156.2-2002 | | Ref Score |
|--|--|-----------|
| <1.5m | | 0 Points |
| 1.5 to 3m | | 6 Points |
| 3 to 10m | | 10 Points |
| >10m | | 15 points |
| Structure Type | | |
| Stand alone handrail or fence, non-critical | | 0 Points |
| Stand alone safety barrier, critical | | 3 Points |
| Simple low-level post & deck structure e.g. platform/boardwalk or stairs < 1.5m high | | 4 Points |
| Simple short spanning structure e.g. simple beam pedestrian bridge < 6m long | | 5 Points |
| Simple long spanning structure e.g. simple beam pedestrian bridge >6m long | | 7 Points |
| Complex structure e.g. platforms >1.5m high, multi-span or truss pedestrian bridge | | 10 Points |
| Critical high-level structure with little or no redundancy e.g. cantilevered platform or suspension bridge | | 15 Points |
| Materials | | |
| Masonry, stone or concrete structure | | 0 Points |
| Stainless steel, aluminium or fibre composite structure with stainless steel fittings | | 2 Points |
| Treated timber with galvanized or stainless steel fittings | | 4 Points |
| Galvanised steel structure (may have treated timber decking) | | 7 Points |
| Painted or non-coated mild steel structure | | 10 Points |
| Untreated/non-painted timber | | 12 Points |
| Structure with critical rock/soil anchors or below-ground cable anchoring system, e.g. suspension | | 15 Points |
| Climate/Environment - susceptibility to deterioration | | |
| If subject to normally dry inland weather conditions (includes caves) | | 0 Points |
| If subject to normally wet forest or high rainfall | | 4 Points |
| If subject to high altitude (extreme cold) or high wind | | 6 Points |
| If subject to coastal or other corrosive forces | | 10 Points |
| Mechanical Damage | | |
| Low risk | | 0 points |
| High risk e.g. damage resulting from fallen trees, flood, vehicles or boats etc | | 3 Points |
| Age of Structure as a % of Design Life (Life Phase) | | |
| 0 to 25% (near new) | | 1 Point |
| 25 - 50% | | 3 Points |
| 50 - 75% | | 7 Points |
| 75 - 100% (near end of life) | | 10 Points |

| CONSEQUENCE - SEVERITY OF HARM / LOSS Pick highest applicable row | | | RISK OF INCIDENT SCORE (Part A Total) | | | |
|--|-------------------------------|---|---------------------------------------|--------------------------------|--------------------------------|--------------------------------|
| Injury | Financial Impact | Social Impact | 0 to 19 | 20 to 29 | 30 to 39 | >40 |
| None | Negligible | Negligible | <input type="checkbox"/> AIP 1 | <input type="checkbox"/> AIP 1 | <input type="checkbox"/> AIP 2 | <input type="checkbox"/> AIP 3 |
| First aid treatment | Economic loss <\$1k | Inconvenience to <10 persons p.a. | <input type="checkbox"/> AIP 1 | <input type="checkbox"/> AIP 2 | <input type="checkbox"/> AIP 2 | <input type="checkbox"/> AIP 3 |
| Casualty treatment | Economic loss \$1k - \$10k | Inconvenience to 10-100 person p.a. | <input type="checkbox"/> AIP 1 | <input type="checkbox"/> AIP 2 | <input type="checkbox"/> AIP 3 | <input type="checkbox"/> AIP 4 |
| Serious injury (e.g. permanent disability) | Economic loss \$10k - \$100k | Inconvenience to >100persons p.a. | <input type="checkbox"/> AIP 1 | <input type="checkbox"/> AIP 2 | <input type="checkbox"/> AIP 3 | <input type="checkbox"/> AIP 4 |
| Fatality | Economic loss \$100k - \$500k | Significant loss of amenity for >100 persons or community | | <input type="checkbox"/> AIP 3 | <input type="checkbox"/> AIP 3 | <input type="checkbox"/> AIP 4 |
| Multiple fatalities | Economic loss >\$500k | Major loss of amenity for >100 persons or community | | | <input type="checkbox"/> AIP 4 | <input type="checkbox"/> AIP 4 |

PWS Scorecard – for elevated pedestrian structures

The “asset inspection programs” (AIPs) generated from the scorecard for each structure are associated with a hierarchy of inspection programs, as follows:

| | Program Name | Inspection Schedule |
|-----------|--------------|--|
| Low Risk | AIP 1 | <ul style="list-style-type: none"> Asset Inspector: 6 monthly, 12 monthly or 36 monthly (depending on remoteness of site) Engineer: Not applicable |
| | AIP 2 | <ul style="list-style-type: none"> Asset Inspector: 6 monthly, 12 monthly or 24 monthly (depending on remoteness of site) Engineer: If requested by an Asset Inspector |
| High Risk | AIP 3 | <ul style="list-style-type: none"> Asset Inspector: 6 monthly Engineer: Every 5 years |
| | AIP 4 | <ul style="list-style-type: none"> Asset Inspector: 6 monthly Engineer: Every 3 years |

Using the scorecard and the (above) inspection programs has streamlined the inspection effort and maximises the use of available resources, with a focus on the assets presenting the highest risk to the public. This approach is logical and aligns with the intents of the Civil Liability Act.

PROBLEMS WITH OLDER STEEL STRUCTURES

Some early versions of steel bridges at PWS were not always successful considering the cost and effort that went into their construction, and in many cases the lifespan of these structures was disappointingly short. Problems encountered include:

- Corrosion of painted steel in coastal environments (often due to poor quality coatings or lack of maintenance);
- Corrosion of galvanized steel in coastal environments (particularly high exposure zone / marine zone);
- Corrosion due to inappropriate use of inline/electroplated galvanized products (typically these products only have a 100g/m² zinc coating, compared to 500 g/m² typical for hot-dip galvanized);
- Poor detailing, leading to structural or maintenance problems;
- Lack of maintenance in general;
- Splitting of RHS members due to water ingress and freeze/thaw action (more on this below);
- Flood, snow and ice damage (often inappropriate design).

Splitting of Hollow Sections, Freeze-Thaw

A number of earlier PWS steel structures were damaged by freeze-thaw splitting of hollow sections, a relatively little known but serious problem in colder climates. When it occurs in critical members, freeze/thaw splitting is a serious defect that, if undetected, could potentially lead to collapse of a structure.

The phenomenon has been discovered in a number of older painted (not hot-dip galvanized) steel truss bridges in Tasmania. These structures were typically fabricated with “hermetically sealed” members, using continuous welds. The intention was that oxygen would be excluded from the inside of the hollow sections (thereby limiting internal corrosion), and that they would remain dry internally as water would not penetrate the welds.

Despite this logic, in practice it has been found that moisture will almost always enter exposed “sealed” hollow sections. A plausible theory is that differential air pressures drive moisture through microscopic flaws and capillaries in the weld material and other defects. This moisture, once inside the member, never escapes and can accumulate over time into a significant volume. Pooling occurs at the lowest part of the member and this will gradually burst or split the section if subjected to repeated freeze-thaw cycles.

It appears that rectangular/square hollow sections with wall thicknesses less than 4mm are the most vulnerable to damage. An early warning sign is bulging of the sides of the section. This is followed by fine crack formation and then splitting, usually at the cold-formed corners.

Hot-dip galvanized structures are generally not susceptible to this problem because all hollow sections are fully vented for the galvanizing process. The vent holes (if left unplugged) generally allow free drainage of the sections, alleviating any risk of water accumulation.

The UK structural safety website CROSS/SCOSS has an interesting article on this issue: <http://www.structural-safety.org/publications/view-report/?report=3211>



RHS Splitting Example 1



RHS Splitting Example 2



**RHS Splitting Example 3a - Bridge at very cold location with significant fall height
Localised splitting was detected in the lower chord of this structure.**



RHS Splitting Example 3b – The bridge was heli lifted to the base, the split chord was replaced, drain holes were drilled in critical members, the structure was painted and returned to service

Snow Load

PWS bridges in alpine areas (>900m in Tasmania) are normally checked for snow loading in accordance with AS 1170.3. A shape coefficient, $\mu = 0.7$ is used (equivalent to a 0° roof pitch, not the ground snow load). The probability of exceedance is determined using AS 1170.0 for a 50 year design life and importance level of 2, giving:

- ULS: 1/150
- SLS: 1:25

At many locations in Tasmania the snow load case is not critical for a bridge. For example, at a typical lower alpine site (say altitude 950m, semi-sheltered, Terrain Class 1.0) the design snow loads are 2.1kPa and 1.5kPa, for ULS and SLS respectively. As is demonstrated below, if the design live load Q is 4kPa, then the factored live load alone is no greater than the snow load combined with the combination live load. Hence the snow load case is not critical for the design. This is demonstrated as follows:

- ULS: $S_u + \psi Q = 4.5\text{kPa}$ (where $\psi=0.6$) is no greater than $1.5Q = 4.5\text{kPa}$, and
- SLS: $S_s = 1.8\text{kPa} < Q = 3\text{kPa}$

This situation changes dramatically at altitudes above 1200m, particularly in areas of snow accumulation. At these locations, according to AS 1170.3, design snow load can be as high as ULS: 14kPa and SLS: 10kPa, which is many times higher than normal live loading and necessitates careful design of all structures.

Snow Load Design Issues - Plateau Creek Bridge Failure

The following case study is of a 9m modular truss bridge, installed at Plateau Creek on Cradle Mountain. The bridge was installed initially as a temporary structure and the intention was to remove it before winter. It was not designed for snow loading and little thought was given to this issue, partially because there was no significant risk to the public due to the limited fall height.

As it turned out, the bridge was not removed during the first winter and survived snowfalls unscathed. It was left insitu for the following winter, which was more severe, as shown in the following photographs.



Bridge under early winter snowfall at Plateau Creek (Cradle Mountain behind)



July snow cover



August snow cover



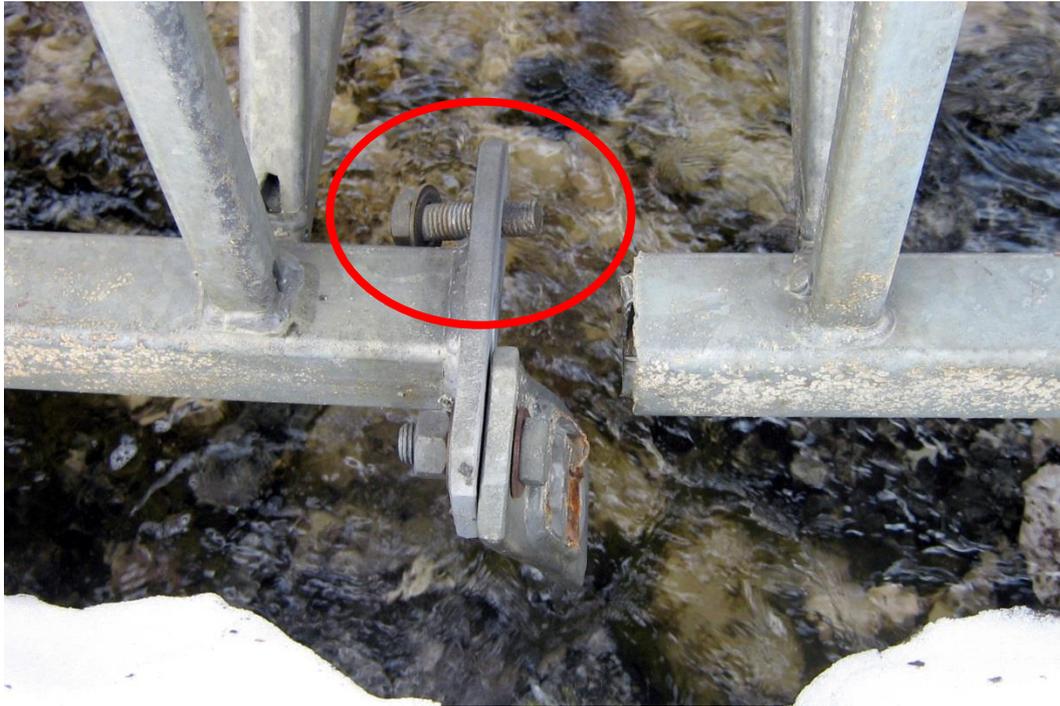
Spring thaw: a problem was revealed



Tension failure of the module connections (upstream side)



Interestingly, the downstream side failed in a different mode



Failure initiated by thread stripping of an M16 bolt

It is thought that the failure was initiated by thread stripping on the inner M16 tension bolt on the downstream side. This resulted in an eccentric load on the connection plate and a “peeling” failure of the corresponding weld. High loads were then transferred to the upstream side of the structure, where the tension connection failed in a more conventional manner, in tension at the connection plate welds.

It is estimated that the snow load on this structure was about 10kPa, well above the design live load of 4kPa. Irrespective of this, the failure mode was interesting and there were a number of contributing factors:

- Sustained loading, well above the design load;
- Bolt tension forces amplified by plate prying action (possible increase in bolt tension of up to 30%);
- Bolt Cat 4.6 installed, not 8.8;
- No gussets on the end plates.

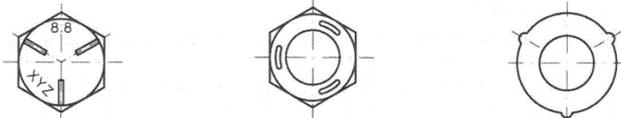
Lessons Learnt

There was a misjudgement made in installing a non-snow rated structure at this location, whether temporary or permanent. From a designer’s point of view there was some concern, as the original design calculations indicated that the structure *probably* should have survived this loading. The generic design drawings for future bridges were subsequently modified as follows:

- Emphasis on the correct bolt category (8.8);
- Connection plate thickness increased (to reduce prying forces) and gussets added to highly loaded structures;
- Connection plate weld increased.

Following this failure there was a safety focus on similar critical tension connectors during routine inspections. One particularly large and high risk bridge was found to have been installed with incorrect bolts at similar connection plates (again Category 4.6 bolts instead of 8.8). The contractor was notified and offered to rectify the defect at their own cost, despite the issue being discovered almost 10 years after construction. To reduce the risk of this occurring in the future, the following detail is now placed on all such drawings to emphasise the difference in the bolt categories:

M16 Category 8.8/S galv. bolts at all module connections. Bolts to conform to AS/NZS 1252, with identification markings as indicated below.



Example of drawing note regarding use of Category 8.8 bolts

IMPROVING THE STANDARD OF INFRASTRUCTURE

Over the last decade, much of the remaining lower standard infrastructure dating from the 1980's and 90's has reached the end of its serviceable life and/or no longer meets visitor requirements. When designing new structures there has been an effort to improve the overall amenity, satisfy a higher standard of visitor safety, increase the service life and reduced maintenance costs.

Much of the basic infrastructure that was built in the 1980's and 90's had a serviceable life of only 20 - 30 years and there were numerous examples of structures that local staff had built and replaced up to three times in their careers. It has been shown that in many (but not all) cases, there are considerable life cycle cost benefits to be gained by installing more highly engineered structures (e.g. service life of 50 - 100 years), which are custom designed for the site conditions to require minimal maintenance. The following are typical attributes of these more highly engineered structures:

- Partially or fully prefabricated;
- Typically long single spans for bridges;
- Timber, steel or FRP beams for spans <9m;
- Galvanized steel trusses for spans 9 -12m;
- Light weight walk-through trusses for spans 12 – 24m;
- Use of FRP materials, particularly for decking;

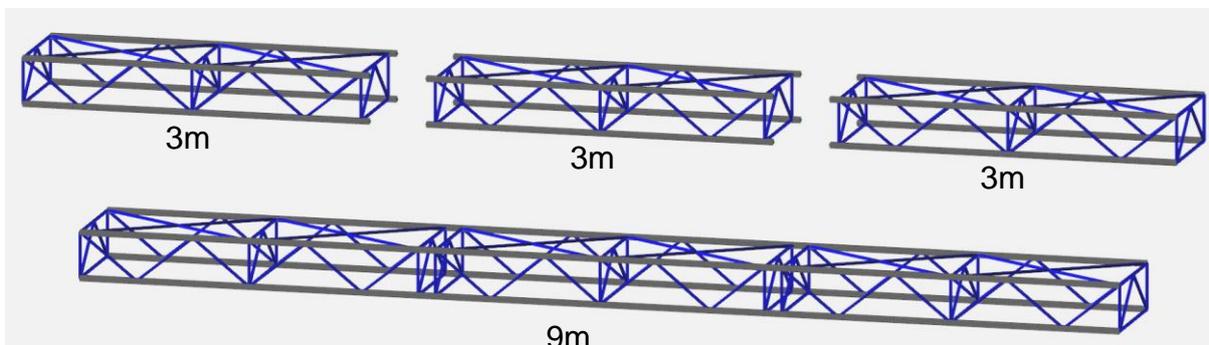
Prefabricated components can be manufactured to a high standard in workshops in a quality assured environment. The components are generally delivered to site by helicopter for fast and efficient installation, often using PWS work crews.

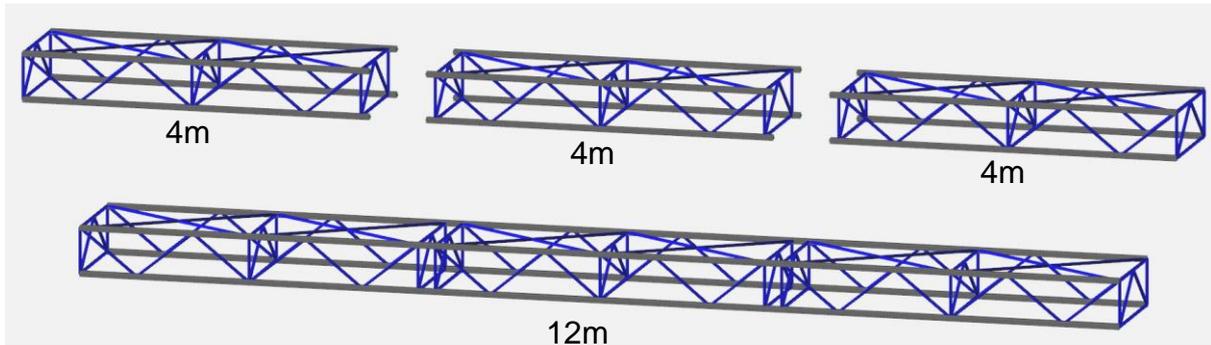
Modular Bridge (Under-Truss Type)

For bridges with spans in the range 9 – 12m, a hot-dip galvanized under truss from welded SHS section can provide an economical spanning element, with all structural components below deck level.

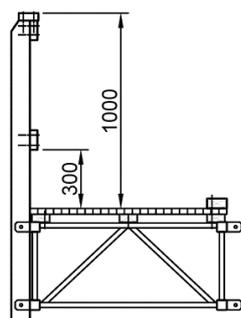
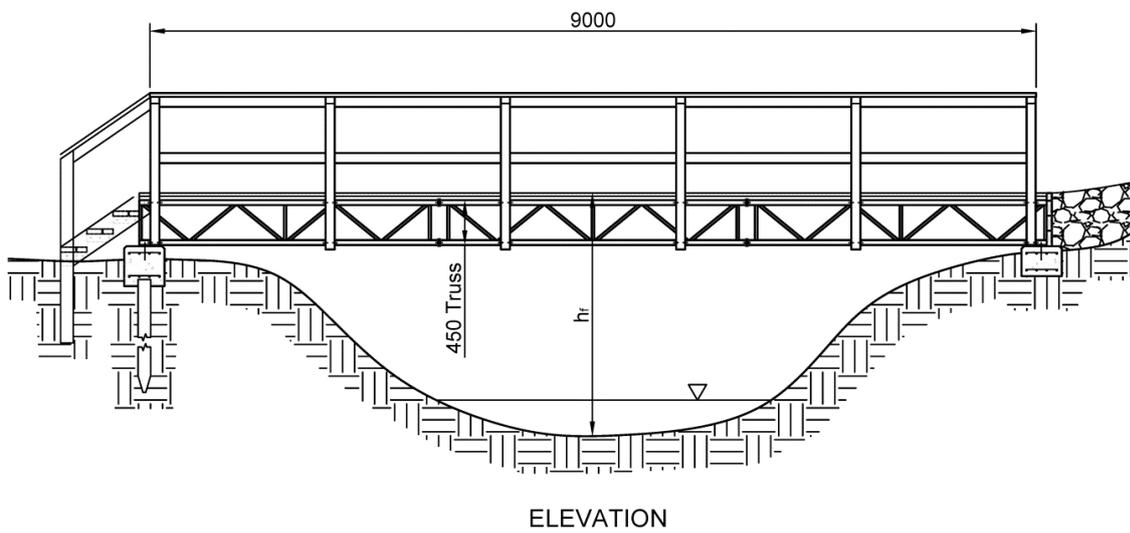
The modules are fabricated in either 3 x 3m lengths for a 9m bridge or 3 x 4m lengths for a 12m bridge. The deck width is generally 1m, which is suitable for bridges on AS 2156 Class 2, 3 and 4 tracks. The design features:

- Fully vented SHS truss for safe hot-dip galvanizing. The modules fit easily in the available bath (maximum bath size in Tasmania measures 6500 x 1050mm and 1400mm deep);
- Vents, as well as being mandatory for the galvanizing process, allow free drainage of water to prevent freeze-thaw splitting of the SHS members;
- Modules can be transported on a small truck/ute or even a power barrow and can be lifted manually;
- Single module mass 100kg (9m bridge) or 185kg (12m bridge);
- Heli lift mass (full structure with deck, but no barriers): 525kg for 9m length or 850kg for 12m length. This load can be carried by a B2/B3 “Squirrel” helicopter;
- Modules bolt together, either prior to heli lift, or on site;
- Safety barriers (if required) are fitted after installation.



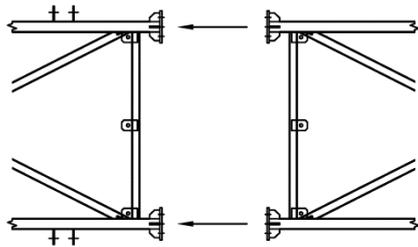


Modular truss bridge, 9m and 12m lengths

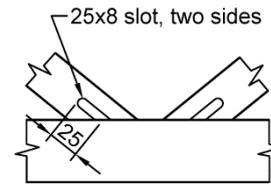


| Class 3 Track - Safety Barrier Requirements | |
|---|-------------------------------|
| $H_{eff} < 1m$ | None |
| $1.0m < H_{eff} < 3m$ | Type C barrier, one side only |
| $H_{eff} > 3m$ | Type C barrier, both sides |

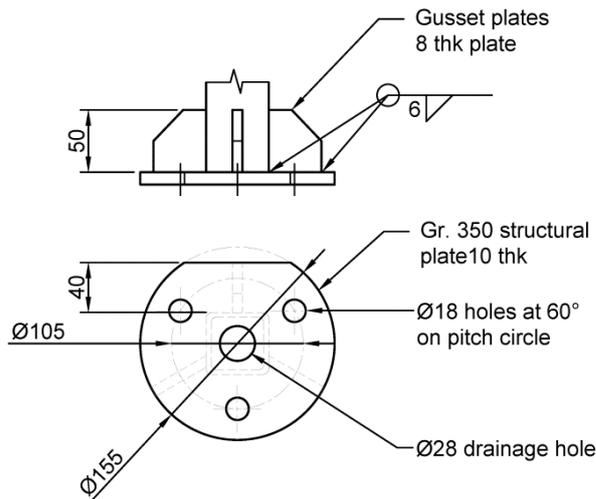
9m Modular Truss Bridge - General Arrangement



MODULE CONNECTION PLAN



VENTING



Material:

A..... 50x4 SHS Grade C350LO to AS 1163
 B..... 35x3 SHS Grade C350LO to AS 1163
 Bolts: M16-50 Grade 8.8/S galv. to AS/NZS 1252.
 Total no. 24.

Finish:

Hot-dip galvanize to AS 4680, 390 g/sq.m

9m Modular Truss Bridge – Structural Details

The modular bridges are relatively stiff and light compared to simple beams of the same length. Typical design deflections are as follows:

| | Deflection | |
|-----------------|-------------------------|------------------|
| | Self Weight + Dead Load | Live Load (4kPa) |
| 9m span | 6mm (span/1500) | 20mm (span/450) |
| 12m span | 10mm (span/1200) | 30mm (span/400) |

Since 2010, it is estimated that 20-30 bridges of this design have been installed by PWS, some at very remote locations. The design has proved robust and reliable with only one recorded failure, which was due to excessive snow loading (as discussed previously).

Installation is generally completed within 3-4 days:

- Personnel mobilised to site (walk or fly), measure and mark out;
- Construct abutments (concrete or timber);
- Truss delivered directly onto abutments in a single heli lift (with deck fitted);

- Fit safety barriers, as determined by site requirements and the appropriate standard for the track (e.g. AS 2156);
- Construct approaches (ramp or steps);
- Extract personnel.

Typical Cost (including helicopter):

- 9m: \$10,000 materials/fabrication, plus \$5,000 - \$10,000 installation
- 12m: \$15,000 materials/fabrication, plus \$8,000 - \$15,000 installation

It is noted that similar structures can be fabricated from open sections (usually equal angles) and this option has also been used by PWS. This is a practical alternative in some situations, particularly where a more robust and “industrial” looking design is acceptable. The RHS design however has proved popular for the following reasons:

- Best economy in terms of strength and weight;
- More appealing appearance;
- Less accumulation of dirt and debris on member surfaces/corners;
- Difference in fabrication cost between SHS and EA versions is not significant.

Photographic Record – Quinns Creek Bridge Installation 12m Modular Truss



PWS personnel fitting lifting slings



Connection of long line



Lift commences



Monitoring of flight



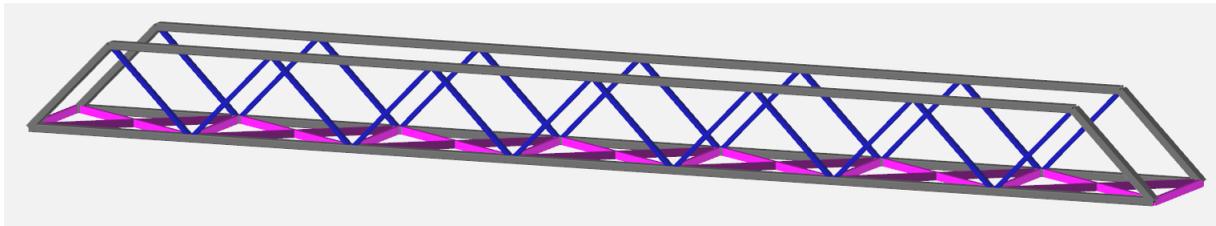
Completed structure



Close view of module connection

Walk-Through Truss Bridge Development

Walk-through truss bridges can be very efficient for longer spans, with up to 24m length being reasonably practical (this can be fabricated in most workshops and transported by road). Generally a truss depth of 1.0 – 1.4m is desirable so that the truss itself becomes part of the safety barrier system but is not so high as to disrupt views or create a sense of enclosure. A “Warren” truss configuration is generally adopted at PWS (alternating tension/compression diagonals), as this provides an attractive and highly economical design that can be readily adapted to different lengths. A “Pratt” configuration (verticals and tension-only diagonals) has also been used successfully.



Walk-through truss bridge, 15m length

At PWS, the majority of bridges of this type have been constructed from welded aluminium hollow sections, to produce very lightweight structures with long spans. It is noted that FRP sections could also be used to fabricate trusses in a similar manner; however this application has not yet been used in Tasmania.

The structures are particularly well suited to coastal applications due to the durability of marine-grade aluminium and also suited to remote locations where the main truss can be delivered directly onto the abutments, in a single heli lift.

Using this style of truss at remote sites has enabled PWS to bridge some significant watercourses with a single span (no piers), with minimal installation time. To date, several 15m spans have been successfully heli lifted, with the longest span installed by helicopter being 19m. Several 24m spans have been installed by crane at coastal sites accessible by vehicles.

Most design work has been undertaken in-house, using AS 1664.1 (limit state design) and analysis by Space Gass. The following notes relate to design issues:

- Outrigger bracing of the top chord is generally not desirable for a number of reasons including aesthetics, weight and increased width (more difficult to transport and manoeuvre between trees etc.). Hence at ULS, top chord buckling is critical in many cases.
- The top chord buckling mode and factor of safety (FoS) can be readily determined using Space Gass, or similar software. It has been found that a minimum FoS for buckling of 1.5 should apply to factored ULS loads, to achieve compliance with AS 1664.1. Generally, a FoS > 2.5 is desirable for reasonably conservative design and where this is achieved, tension failure of the lower chord will generally occur before any buckling effects.

- Tension in the lower chords, particularly at butt welds, must be checked using the reduced welded alloy properties, and this will sometimes govern the design capacity.
- The structures are relatively stiff. For SLS live loads (e.g. 4kPa), deflections are usually within Span/600.
- Resonant frequencies need to be checked using Space Gass or similar software, with lumped masses representing the deck and rails. Based on AS 5100.2-2017, bridges with vertical frequencies >5 Hz do not need to be investigated further (this has increased from 3.5Hz in the 2004 code). The majority of bridges installed by PWS have natural frequencies > 5Hz and this is considered desirable. The torsional mode is generally the lowest frequency and (although not stipulated in AS 5100.2) should also be >5Hz where practical, as this mode is readily excited by deliberate action of users.
- The majority of bridges use alloy 6061-T6 or 6063-T6 for extruded sections (excellent corrosion resistance and weldability).
- Welding is specified as Category B to AS 1665-2004.
- Designs need to allow for thermal expansion/contraction (which is significantly higher for aluminium than for a steel bridge of the same length).

The following table highlights some of the properties of aluminium compared to steel:

| | Aluminium (6061-T6) | Steel | Comparison ratio |
|--------------------------------------|--------------------------------|----------------------|-----------------------------|
| Density (kg/m ³) | 2710 | 7850 | 0.35 |
| Modulus of Elasticity (GPa) | 70 | 200 | 0.35 |
| Coeff Thermal Expansion (/ deg C) | 23 x10 ⁻⁶ | 12 x10 ⁻⁶ | 1.92 |
| Ductility (% elongation, 50mm gauge) | 17 | 10-40 | - |
| Melting Point (deg C) | 660 | 1500 | 0.44 |

Aluminium versus Steel Comparison Table

Low Temperatures

Aluminium performs very well at low temperatures. Unlike steel, which can become brittle, aluminium remains ductile at lower temperatures and the strength (e.g. yield and tensile stress) actually increases. This is a useful property in Tasmania, where winter temperatures often drop to -10 deg C, or lower.

Flood/Hydraulics

The type of structures discussed in this paper are generally installed with the soffit above the estimated 1:100 ARI water level where practical, or the 50 year level as a minimum. Note that for SLS, AS 5100.1 recommends a 10-50 year ARI for pedestrian and cycle bridges. The design water level is determined by either simple hydrological modelling or from site evidence and local observations/knowledge.

Bushfire

Although the situation has not yet occurred in Tasmania, an aluminium structure within the flame zone of a very intense bushfire will likely melt and collapse under self-weight. It is however very unlikely that it will ignite or add to the intensity of the fire. The damaged materials would, in most cases, be relatively easily cleaned up and removed from site.

Based on typical fire ratings of commercially available windows and shutters, it can be inferred that an aluminium structure would likely survive (without collapse) the radiant heat and ember attack of bushfire conditions similar to AS 3959 BAL 40 (heat flux 29-40 kW/m²). Full replacement of the structure would be required after such an event.

Hollow Section Drainage

Although it has not been observed in PWS structures, due to the lower strength of the material it appears likely that aluminium sections could be particularly vulnerable to freeze/thaw splitting in cold climates. For this reason it is critical that all hollow sections used in these structures are free draining where they are installed at sites subject to low temperatures (and this is good design practice in general).

Unlike the hot dip galvanizing process, mandatory (i.e. externally visible) vent holes are not required and these structures may be designed with predominantly concealed drainage paths utilising “hidden” holes at the junction of members. Concealed drainage holes must be clearly documented and subject to a formal holdpoint inspection during the fabrication process, prior to welding.



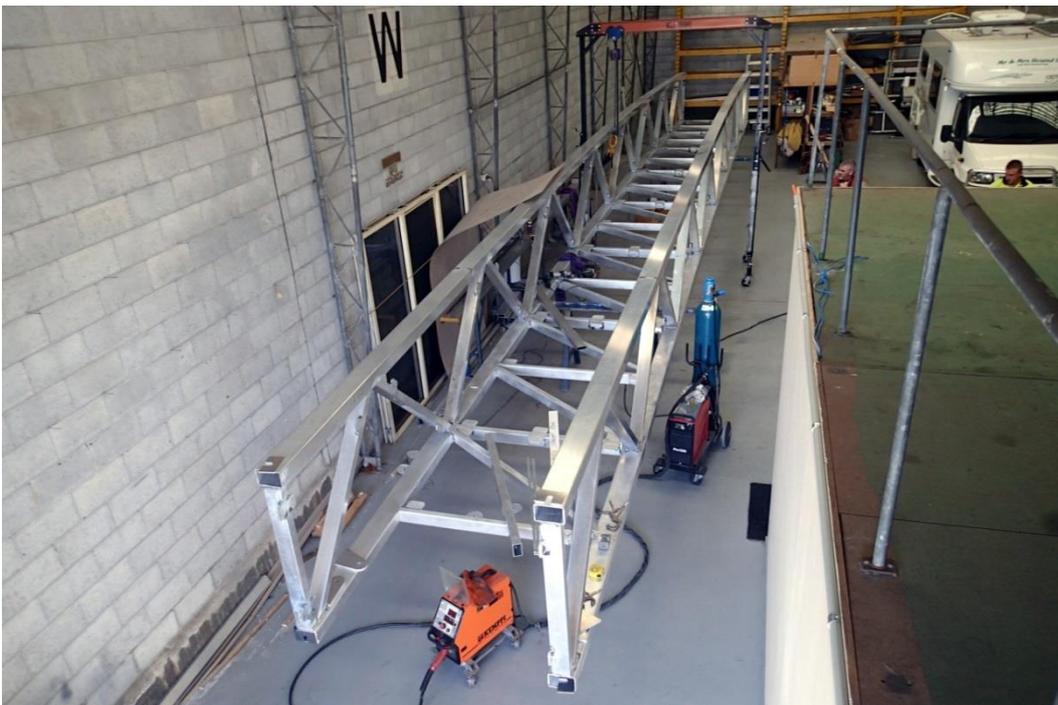
Examples of external drainage holes

Photographic Record – Enchanted Bridge, Cradle Mountain

The previous steel and timber bridge was built in 1995 from welded Duragal SHS. Extensive freeze/thaw splitting was discovered in the truss diagonals in 2014 and this issue, combined with a number of other deficiencies, resulted in a recommendation to replace the structure after only 20 years in service. The new structure was an 18m span, walk-through truss, deck width 1.4m. The project was completed for about \$80,000 and the materials from the previous structure were reused or recycled.



Original steel and timber structure, prior to demolition



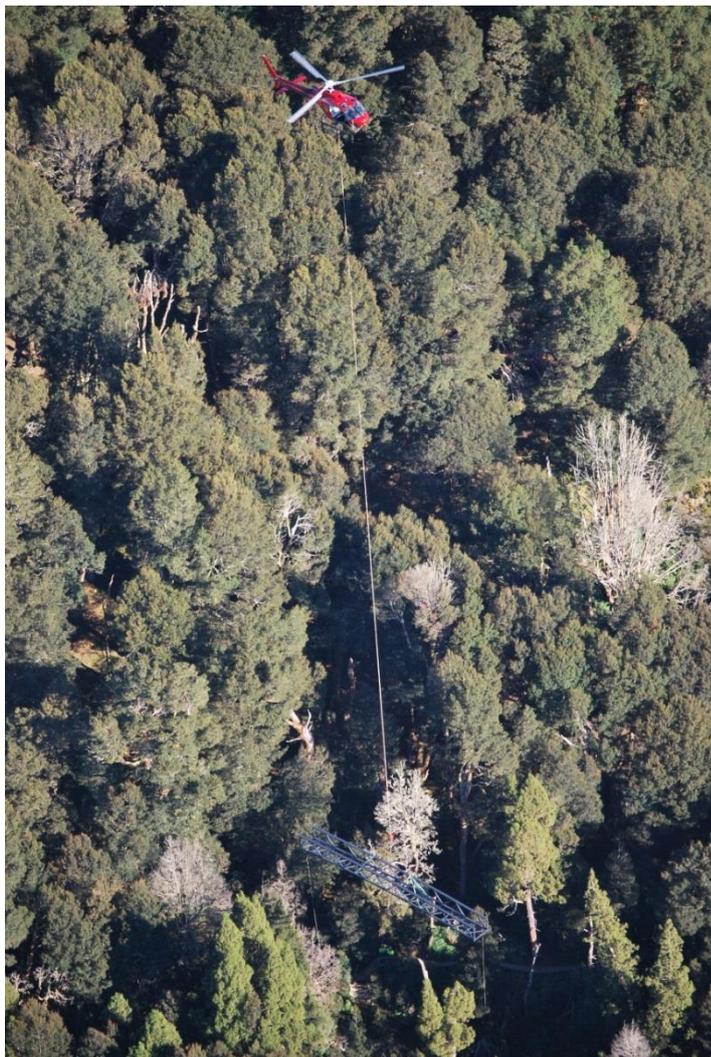
New aluminium walk-through truss under fabrication, length 18m, mass 850kg



Commencement of heli lift from local base



Heli lift in progress



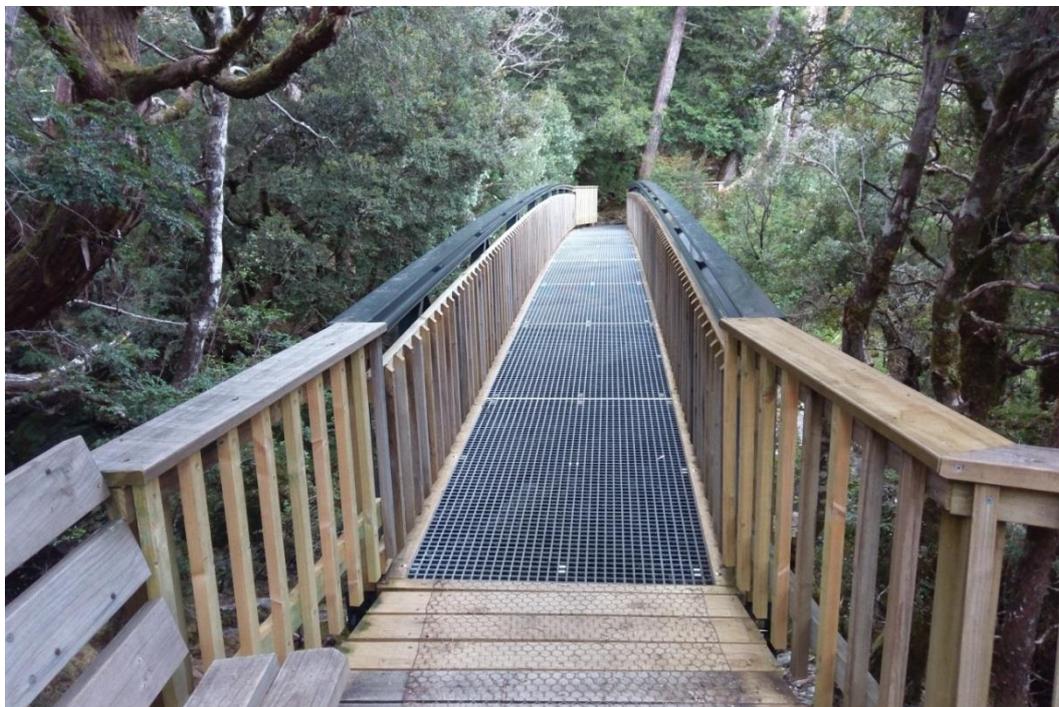
Truss is lowered to tree top level, where ground crew can reach guide ropes



Ground crew (both sides of river) use guide ropes to help manoeuvre structure through tree canopy. Radio communications are maintained with pilot throughout.



The structure is positioned on the existing abutments (with adapter brackets)



Decking and barriers are installed (timber recycled from the previous structure)



The completed structure spans 18m and utilises the previous abutments (the intermediate piers had been vulnerable to flood damage and were demolished).



First snowfall of the season

Case Study – Warrawee Bridge Flood Damage

This site is located on the banks of the Mersey River. It is accessible by vehicle and originally had a range of day use facilities including walkways, toilets and picnic shelters. The bridge was part of a new section of walking track intended to improve the movement of visitors between facilities.

The bridge was designed to span a natural channel that was nearly always dry, and for this reason (and because the structure would clearly be no more flood susceptible than the rest of the site), no hydrological assessment was undertaken.

The 15m span truss bridge was locally fabricated and installed by crane.



Installation of the structure in a single lift, 01 June 2016



The bridge is lowered onto (cast insitu) concrete abutments



End anchor pins are installed and the installation is complete in 2-3 hours



6 June 2016 (5 days later): Northern Tasmania is impacted by a major flood event, with measured rainfall in this catchment indicating at least 100 ARI. The structure was swept about 20m downstream from its original location, still attached to the footings.



Nearly all infrastructure at the site was destroyed, including the access road. Suburban areas in the nearby town of Latrobe were also inundated.

Lessons Learnt

Although only spanning a minor channel, this site was located close to the Mersey River, a major catchment system. This site was well known to be flood susceptible and damage to infrastructure in the area had occurred several times over the last 20-30 years, though not to this extent. It was not practical to build flood immune structures for this type of event at the site and it was acknowledged that new parks infrastructure should be considered “sacrificial”, given that it was likely to be damaged in anything above, say a 20 year ARI event. Despite this, no one would have believed the new structure would only survive 5 days!

It can be tempting to ignore engineering principles and, at the planning stage, to dismiss such issues with arguments like “sacrificial infrastructure is appropriate here”, or “if a large flood occurs the damage here will be insignificant compared to what occurs elsewhere”.

With hindsight, the engineering concerns relating to the low flood immunity for this structure could have been used to initiate a review of the project risks or more broadly to inform a new policy for the site as a whole (e.g. strategic removal of infrastructure and facilities at end of life, with no replacement).

Footnote: Surprisingly, some months later when the structure was salvaged and examined in a workshop, only superficial damage to the balustrade was found. The structure was repaired and will be installed at a more suitable location. Most of the infrastructure at the original site will not be replaced.

Viewing Platforms

Similar to bridges, in many cases there can be significant life-cycle cost benefits in increasing the service life of viewing platforms (e.g. from 20-30 years to 50-100 years). For a longer service life, it is important that planning for future visitor numbers is carefully considered and provision is made, if necessary, to extend platforms or add additional viewing opportunities at a later date. Due to congregation of visitors, the adequacy of viewing platforms is more dependent on usage levels than other walking track infrastructure and, if not carefully planned for, crowding issues can drive the need for upgrades or replacement well before the engineering design life is achieved.

Recent viewing platforms built by PWS use many of the same principles as discussed above for prefabricated bridges (though large spans are generally not required). Prefabricated galvanized or painted steel structures with FRP decking offer robust solutions at inland locations. In coastal areas aluminium, FRP or timber structures (all with Grade 316 stainless steel fasteners) are appropriate.

Case Study - Upgrade of Nelson Falls Viewing Platform

Nelson Falls is a popular 20 minute walk on a wheelchair accessible (Class 1) track, close to the Lyell Highway on the west coast. The original viewing platform dated from the early 1990's and was constructed from treated pine on rudimentary concrete piers. The site is susceptible to flooding and nearly always wet, with slime growth occurring on all timber surfaces. Wire mesh was installed on the decking to reduce the considerable slip hazard, though maintenance of this was problematic.

In 2010 it was found during routine inspections that many of the piers were undermined by scour and that there was extensive decay in the substructure. Given that the structure had only survived about 20 years in service, a decision was made to replace the platform with a larger structure, using more durable materials.

The final design was a galvanized steel structure, prefabricated in modules with an FRP deck. It was designed to be installed at a higher level than the previous structure, with anchorage to large concrete/rock combination foundations. The structure would cantilever over the river to provide an improved view of the falls and a sense of height above the moving water.

The concrete and prefabricated components for the steel structure were transported from the highway along the walking track, using power barrows and manual labour, and installed over about 10 days. This was a major effort by the PWS staff, as no helicopter transport was available to the project due to budget constraints at the time. This project was delivered for a total cost of \$30,000 (excluding PWS site labour).



The original treated pine structure



Side view of the original structure, showing low standard construction



The replacement steel platform provides improved flood immunity and a much larger non-slip viewing area. It has a design life of 75+ years.

Example of Heli lifted Steel and FRP Viewing Platform

This site at Dip Falls in North West Tasmania is located at the base of a waterfall, only accessible by a steep staircase. Prior to this project, no viewing platform had ever been constructed below the falls due to the difficult access and the risk of flood damage.

To withstand floodwaters it was determined that the design must be robust, with well-anchored concrete footings and a heavy duty substructure. Whilst achieving this, all components must still be within the allowable weight limit (850kg) for heli lifting.

The concrete foundations were poured several weeks ahead of the installation date, with materials heli lifted and mixed on site. The prefabricated hot-dip galvanized steelwork and decking were also lifted to the site and the structure was erected over 2-3 days.

As a result of the height and outward cantilever of the “crow’s nest”, the new platform provides a vantage point to view the falls that was not previously available to visitors. It is intended to cater for projected visitor numbers and has a design life of 75+ years, utilising materials that are compatible with the perpetually damp environment.



Assembly of the structure in the workshop, prior to galvanizing



The “crow’s nest” part of viewing platform, as erected on site

CONCLUSION

In the past 20 years there has been a dramatic improvement in the standard of design, inspection and management of walking track infrastructure in Tasmania. Improvements have been largely driven by the adoption of professional engineering services by the Tasmania Parks and Wildlife Service, following an increase in awareness of visitor safety issues since the mid 1990’s.

Routine inspection and maintenance programs have been strategically prioritised to focus resources on higher risk assets. This approach maximises the use of available resources and in doing so ensures compliance with the intents of the *Tasmanian Civil Liability Act 2002*.

From a professional perspective, engineering of national park infrastructure is a small but growing “niche” field of civil/structural engineering. It is a rewarding field in which to work that demands a practical, hands-on approach to problem solving and a good appreciation of environmental, recreational and land management perspectives. The importance of learning and improving from past failures is critical to success and for this reason a number of case studies have been included in this paper, where valuable experience has been gained.

As well as being of general interest to civil/structural engineers, it is hoped that this paper will demonstrate the significant value offered to land management authorities by professional engineers.

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BIOGRAPHY



Tim Chappell is Senior Engineer with Parks & Wildlife Service Tasmania, a position he has held since 2008. With a strong interest in the natural environment and outdoor recreation he enjoys the challenge of managing infrastructure in some of the most remote and wild places in Australia. Tim has a background in structural design and, prior to joining the PWS, worked mostly in the renewable energy sector (hydro and wind generation).