TIMBER STRUCTURE FOR PHOTO-VOLTAIC ARRAY IN SOUTHERN PHILIPPINES

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SUMMARY

Photo-voltaic (PV) panels have reduced in cost in recent years, and the reduction is expected to continue. In 1976 the installed cost was about 100 USD/watt, but by 2017 had dropped to about 0.41 USD/watt.[i] This makes PV panels attractive as a source of electricity in third-world communities blessed with high sunshine hours but low financial ability to import fuels and rotating machinery needed to generate electricity in the traditional manner of moving wires through a magnetic field.

This paper describes a 24 X 24 metre timber structure, capable of supporting 288 PV panels that can generate 90 kilowatts of electricity. Each structure is in the form of a six-legged table with a sloping top, some thirty tables being needed for the first site. We have designed the structure to survive cyclones and earthquakes in accordance with the National Structural Code of the Philippines.[ii]

LAND VALUE

Land for the proposed PV farm is covered with 40-year-old rubber trees (*hevea brasiliensis*) that are beyond their economic latex-producing life. The developer's family owns the land, and as a new tree crop cannot produce a return for a few years, they have agreed to forego rent for a similar period.

TIMBER

Rubber trees are to be felled and sawn on site to produce structural timber, using a diesel-powered bandsaw imported from China. Planks up to 400 millimetres wide and up to 9 metres long will be available. Joints made from timber cheek plates and timber cleats (Figure 8) are to be used wherever possible.

Chemical preservation by vacuum and pressure techniques is not available. The developer plans to build open troughs on site then immerse the sawn timber in solutions of sodium borate. The chemical is not expected to penetrate more than a few millimetres and is expected to be leached out by rain. Hence the design must keep the structure as dry as possible.

STEEL

Steel will be used for joints only where no timber detail can be devised. Light metal-arc welding is possible on-site, and heavier weldments can be made in the nearest city. Weld quality is known to be poor, so welds must be designed to accept many imperfections.

Threaded rods, coach screws, and self-drilling roofing screws are to be used. Nails are avoided, as service experience has shown these perform poorly during cyclones[iii].

CONCRETE

Concrete is commonly used for commercial buildings in the Philippines. Reinforcing is often sparse. We propose concrete foundations, and recognise that vigilance will be needed to ensure specified reinforcing is in place.

MARKET

The developer plans to sell electricity to the national grid. The local load is a new residential and retail area with a demand for day-time air-conditioning.

GENERAL FORM OF STRUCTURE

In wealthy communities, it is feasible to fit PV panels to the roofs of private houses. In poor communities, the houses are of such ramshackle construction they routinely blow down during wind storms, and so are unlikely to survive long enough to amortize the capital costs of the PV panels. Commercial buildings are better, but as it is usual for reinforcing starter bars to be left poking up from walls, to fit another storey later, a PV roof installation may have insufficient economic life.

It is common to build PV arrays at ground level. This developer prefers to build a few metres off the ground, partly to keep dangerous voltages away from local trespassers, partly to allow access for fire trucks, and partly because he hopes to use the under-panel space for other commercial opportunities.

The structure is to be in the form of a table top, sloping down towards the equator at an angle corresponding to the local latitude, increased if necessary to encourage rapid run-off of rain. The orientation is chosen to suit the site boundaries. For the first site, a 10-degree slope is suitable, initially specified along a diagonal. This means slopes of 7 degrees parallel to the edges. Adjacent "tables" are to be as close as possible, limited only by the need to prevent adjacent tables from crashing together as they sway during wind storms or earthquakes.

Each table top is to be supported on six timber legs, three at 8-metre spaces under each of two longitudinal trusses 16 metres apart, as shown in Figure 1.

The legs and trusses above them form portal frames, initially designed with pin joints at the bottom and robust moment-resisting joints at the top. During heavy rain storms water is expected to flow over the surface of the ground. The concrete foundations are required to project 150 mm above the surface, and we have designed the timber to be supported 100 mm above this. Despite this elevation, the lowest extremities of the legs are expected to get wet, leading to the possibility of rot. By requiring only uplift and downthrust resistance at the bottom, pin joints with no moment resistance are sufficient, so some rot may be tolerated.

The PV panels are at best 20% efficient. Hence 80% of the received solar radiation must be reflected, absorbed, or transferred to air by convection, meaning at least 360 kilowatts of heat must be

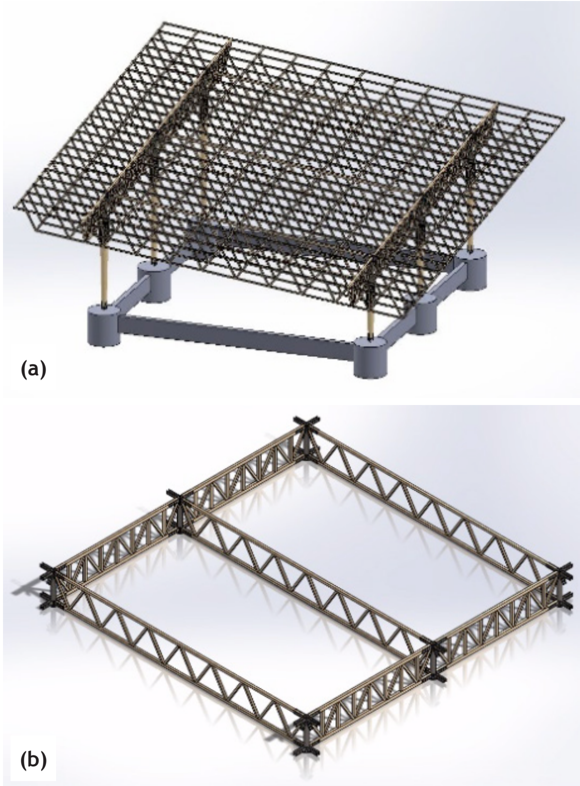

Figure 1: *(a) Timber structure on buried concrete foundation. (b) Main frame comprises two longitudinal and three lateral trusses.*

dispersed. Experiments with a small panel forming the top of an otherwise closed carboard box showed the air temperature under the panel quickly reached 40°C when the ambient shade temperature was 30°C. It is thought a full scale array will reach 50°C, a temperature at which the efficiency will drop 2%[iv] and more heat will need to be dispersed.

The simplest solution to the temperature rise problem is to rely on natural convection, with hot air rising by buoyancy up the slope under the panels. The small slope (7°) means the buoyancy will be weak, and so the space immediately under the panels will need to be kept as clear as possible.

One way of resisting wind drag from severe storms is to make the trusses as open as possible, minimising their solidity. Members need to shield each other. An open design may have its windage reduced by rounding the edges of members wherever possible.

A second way of reducing drag is to enclose the entire superstructure. This adds mass, which reduces the natural frequency, possibly into the wind sensitive region below 1 hertz, but may add stiffness, increasing the natural frequency. Enclosure worsens the temperature rise problem. One solution is to fit an array of water pipes immediately under the panels: if fitted with sprinkler heads they would double as a fire protection system, and if a day-time market for hot water could be found would provide extra cash flow.

Electrical wiring advantages arise if the 288 panels are increased to 300. This would require one extra lateral truss, increasing the area to 24 x 25 metres. Clearly, the wind and earthquakes loads would increase. We plan to study this increase in size after we are satisfied that we have satisfactory details for the 24 x 24 metre array.

BUILDING CODE

The National Structural Code of the Philippines (NSCP) is adapted from American practice. It includes design action clauses that deal with wind storms and earthquakes, provisions for design by prescription, design by engineering analysis, and design by physical testing. This PV array structure is sufficiently unusual that design by analysis is required, with joint performance verified by physical testing. In some respects, the New Zealand requirements are more rigorous, for example in requiring vertical earthquake actions, and these requirements have been adopted.

Design wind speeds have been calculated according to the NSCP. The first site is a little too close to the equator to be subjected to cyclonic winds, and is sheltered by a volcano. On flat ground, the design wind speed would be 39 metres/second, but to allow for topography and possible future deforestation a value of 50 m/s has been adopted. Wind forces parallel to the plane of the panels have been applied in each of four directions.

Uplift wind actions will occur when the wind blows onto an elevated leading edge, and downthrusts when the leading edge is depressed. In all cases the pressures are more pronounced on the half which includes the leading edge. Eddy effects near corners and edges are taken to increase the uplifts and downthrusts by 50% on localised areas that extend to several PV panels.

The NSCP requires wind sensitivity to be checked to guard against the first natural structural frequency coinciding with the frequency of wind gusts. Rayleigh's method is permitted, with computed elastic deflections and masses.

Earthquake actions have been calculated according to the NSCP. There is no requirement to consider the natural frequency of the structure, and enquiries to local building consent officers showed no appreciation of the importance of natural frequency for earthquake actions. The NSCP provides for unknown soil types by specifying parameters appropriate to deep clay. As in NZ a near fault factor is required, but as the nearest known fault is 50 kilometres away, it has no effect in design. The NSCP requires horizontal earthquake actions 75% of the weight, applied in four directions, and following NZ practice[v] a vertical up force of 105% weight has been included.

The NSCP includes long lists of the properties of many species of timber. Plantation rubber trees are amongst the least desirable structural species. Tabulated properties have been used in design, and are to be verified by simple on-site bending tests.

ANALYTICAL METHODS

Initially no structural analysis program was available. Preliminary analysis was accomplished with classical two-dimensional methods, using closed form equations[vi] for the drags, uplifts, and downthrusts on the lateral portal frames, and the theorem of three moments[vii] for the uplifts and downthrusts on the longitudinal trusses. No analytical solution for the horizontal drag forces on the longitudinal (two bay) portals was known, so the conservative approximation of ignoring the contribution from the middle leg was adopted.

The lead author then arranged to use facilities at Wellington Institute of Technology, Weltec. He was to use a structural analysis program in return for providing tuition in its use. Weltec assigned a final year Bachelor of Engineering Technology student to the project, requiring him to learn its rudiments, teach himself to use a drafting program, and assist with experimental work. This work satisfied the academic requirements for the student's BEngTech project.

We constructed a three-dimensional model, Figure 2, in the structural program.

This enabled the analytical forces to be computed for every member, but had no facility for comparing these with the stresses the NSCP allowed. Hence a series of spreadsheets had to be written to convert the forces and moments into stresses and compare these to those allowed.

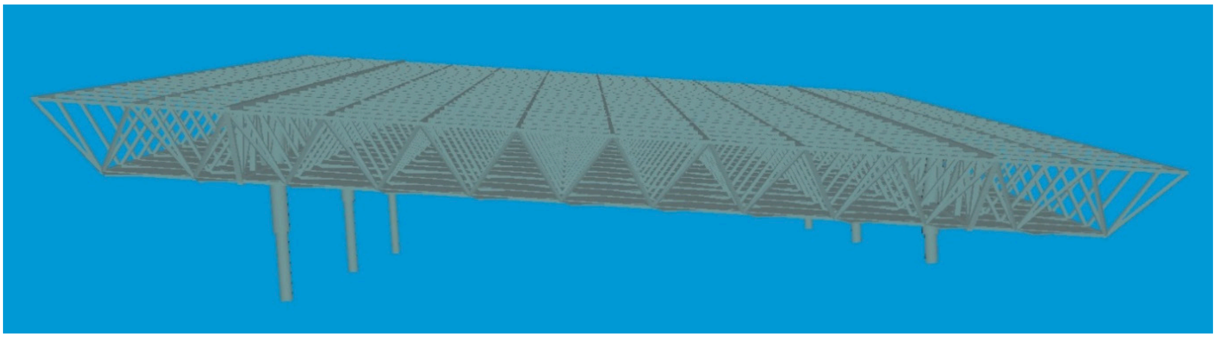

Figure 2: *Initial model of structure.*

THICK TRUNK DESIGN

Two-dimensional analysis showed that legs 600 mm diameter at the top would be necessary, with robust connections to the trusses. The 3-D model confirmed the large diameter. We devised a steel connection, with a system of threaded rods and spherical washers to provide adjustment in angle and height. Adjustment in angle was required to accommodate the developer's changing plans for orientation of each table; adjustment in height to accommodate inaccuracies in the as-built height of the legs.

An initial evaluation indicated the bottom of the leg required only uplift resistance, with adjustment for height. Because it is to be fastened to a solid steel stem 50 mm diameter, which provides moment resistance, the straps connecting the stem to the timber trunk had to be lengthened to accommodate enough coach screws to provide moment resistance.

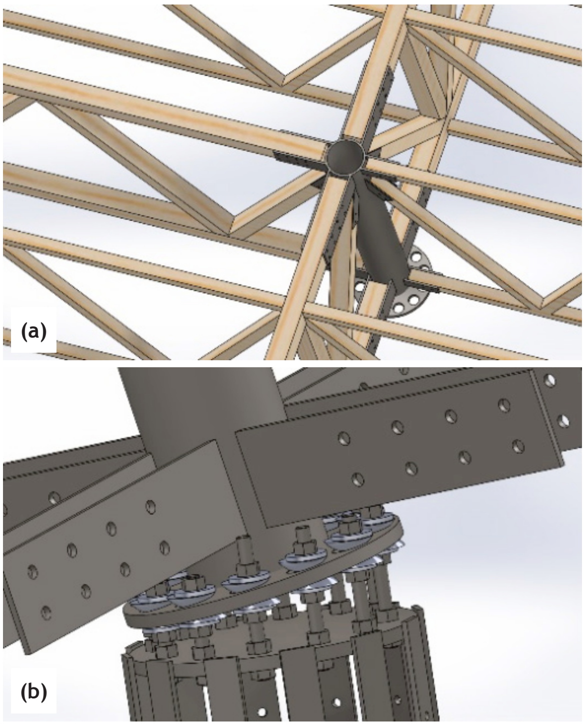

Figure 3: *(a) General arrangement for moment-resisting connection between leg and trusses. (b) Spherical washer detail.*

The developer disapproved of the moment-resisting joint incorporating spherical washers on the ground of local manufacturing difficulty, so asked for a new design.

TRUNK AND BRANCH DESIGN

A new design replaced the severe moment at the top of each leg with forces which could form couples in the longitudinal and lateral planes. The "trunk" could be reduced from 600 to 300 mm diameter, and new 250 mm "branches" were designed to spring from points near the base. The acute angles between trunk and branches lent themselves to joints made from long T-section steel weldments fixed with many coach screws and threaded rods. Weldments made from flat shapes cut with an angle grinder from 3 mm steel plate appeared cheaper than hot-rolled tee-sections. Joints at the top could be made with smaller and thicker plates.

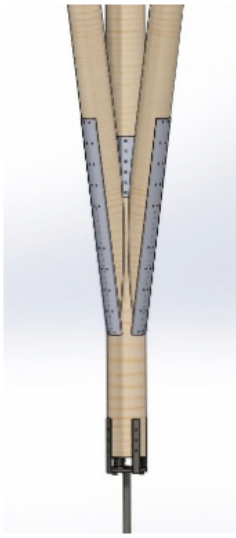

Figure 4: *Joints at bottom of trunk and branch design. Three-hole straps at base were replaced with a five-hole design.*

JOINTS BETWEEN CHORDS AND DIAGONALS IN LONGITUDINAL AND MAJOR LATERAL TRUSSES

We designed these heavily loaded joints with sheet steel plates connected with threaded rods. We trusted the NSCP allowable loads for the rods, but resorted to physical tests to check that the plates would neither buckle nor deform locally in bearing against the rods. We made a test assembly with short diagonals to suit the height capacity of Weltec's Avery test machine.

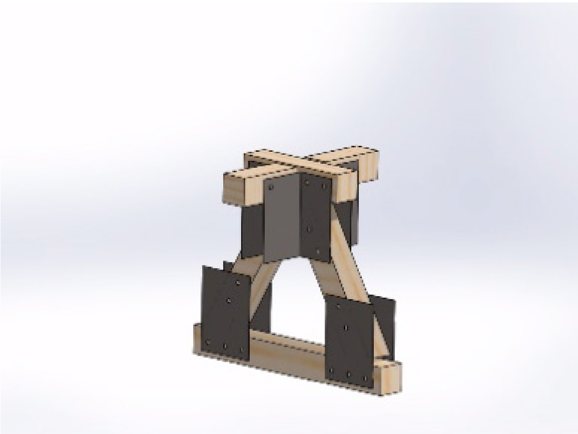

Figure 5: *Test assembly for steel plate joints.*

We found no satisfactory way of making these joints in timber, so developed a system of sheet steel plates. Computer analysis gave the member forces, and the NSCP gave the allowable forces for various sizes of bolts and screws in rubber wood. The assembly shown at the top of Figure 5 was an early conception of a cross-truss joint, later improved.

JOINTS BETWEEN LONGITUDINAL TRUSSES AND CANTILEVERS

Four metre cantilevers are required at one metre intervals along each longitudinal truss. The maximum computed load, which is a 12.5 kN tension, is exerted by a cantilever chord on a longitudinal truss chord. We could devise no timber joint adequate for this, so resorted to a triple-thickness (flat plate + folded L-bracket + folded triangular gusset) assembly as shown in Figure 6. We devised a test assembly that could be loaded in tension in Weltec's Avery test machine to simulate the many hundreds of loading cycles expected during a cyclonic wind storm[viii]. Only one load cycle at 12.5 kN was required, with 626 others at lesser loads as shown in Table 1.

The maximum compressive load at this joint is 9.5 kN. We re-arranged the test assembly to include a gap at the T-joint, thus forcing the load to go through the fasteners and thin plates, then loaded it in compression. The machine applied over 50 cycles of load from 6.5 to 9.5 kN with no signs of distress.

In a real wind storm turbulent eddies are expected to form around the upwind corners of the "table top", producing uplifts and downthrusts in quick succession. These actions would give tensile then compressive loads at the T-joints. The Avery machine cannot provide this action, and an intermediate mechanism which could do so is beyond the scope of this project.

Table 1: Fatigue loading cycles from AS/NZS 1170.2 Figure 2.4.

No of cycles	%age maximum load
285	61.5
25	77
3	92
1	100
3	92
25	77
285	61.5

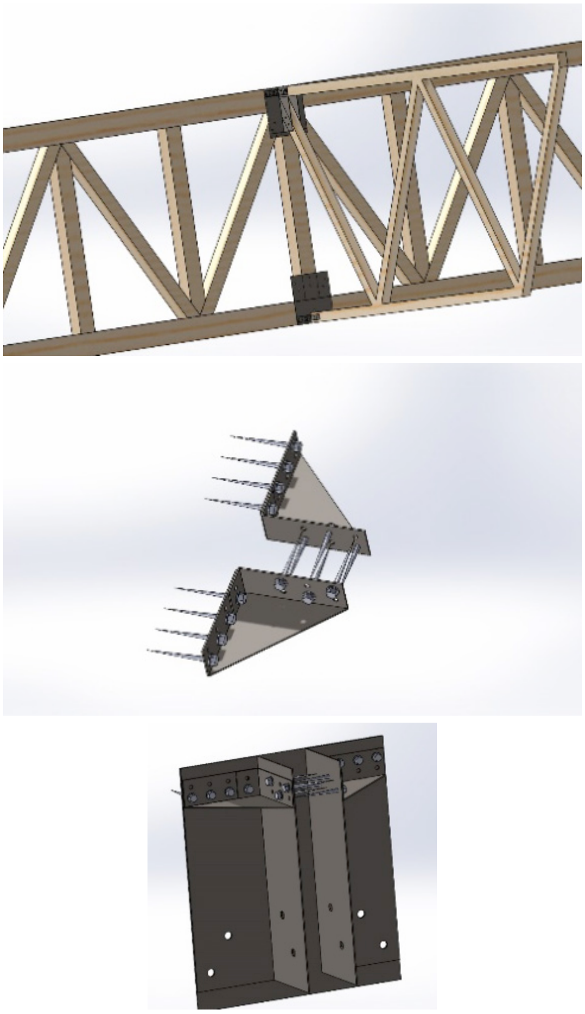

Figure 6: *Sheet steel plates and brackets to join minor lateral trusses to major longitudinal truss.*

JOINTS BETWEEN LONGITUDINAL TRUSSES AND INBOARD LATERAL TRUSSES

These are to be the same as those fixing the cantilevers to the longitudinal trusses.

JOINTS IN TRUSS CHORDS

We did not expect timbers over nine metres long to be available, so at least one joint would be required in each chord of each 16 metre lateral truss. We designed these as steel cheek plates, either thick plate or thinner channel sections. Two plates 610 x 100 x 5 mm thick clamped with eight M12 threaded rods in each end matched the allowable compression in the timber and exceeded the analytical loads. We saw the channel sections to be vulnerable to buckling, so tested them in compression using the cyclonic loading cycles.

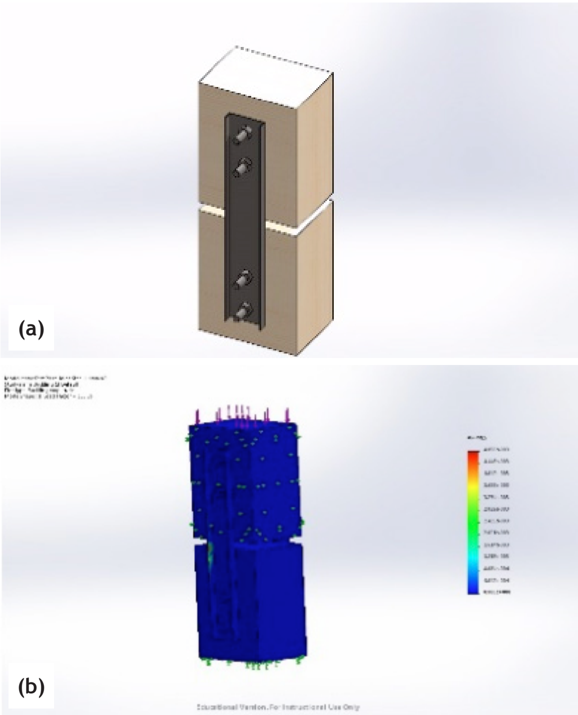

Figure 7: *(a)Test piece for channel section splicing chords. (b) the computer-aided drawing program included a finite element post-processor which was used to choose thickness of channel, later verified by physical test. End gaps were deliberately provided in all compression test assemblies, to simulate along-grain shrinkage of timber.*

JOINTS IN MINOR LATERAL TRUSSES

We designed the joints between the chords and diagonals with timber cheek plates. We chose threaded rods to clamp the plates to the diagonals, and roofing screws to fix the plates to the chords. To satisfy NSCP rules for edge, end, and spacing distances, we discovered planks 400 mm wide would be required, with the grain parallel to the diagonal.

The maximum computed load in any diagonal was 5.8 kN. Making the conservative assumption that this load could exist simultaneously in both diagonals meeting at a node, their vector sum is 10 kN. Hence one test for the joints was to apply a 10 kN test load through a packer that rested only on a chord as shown in Figure 8b. This load is expected only once during the design wind storm, but many hundreds of less severe load cycles are expected. Hence the cheek plates and fasteners in the joint were loaded through 627 cycles as shown in Table 1.

One cheek plate developed an along-grain crack after 80 test load cycles, so we fitted timber cleats, initially planned to house aluminium tubes doing double duty as dwangs and an electrical earthing grid, to suppress crack growth.

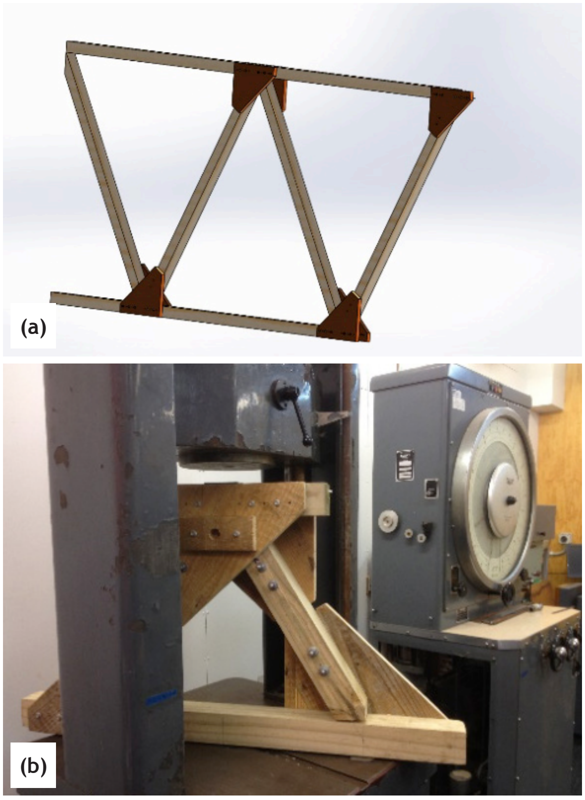

Figure 8: *(a) Timber cheek plates (truncated triangles) on minor lateral trusses. (b) Test assembly showing crack-suppressing cleat parallel to and below chord.*

DWANGS NEAR TOP SURFACE, IMMEDIATELY UNDER PV PANELS

Dwangs, included in the 3-D structural model near the top surface, are required to stop the top chords from buckling sideways. They should not be immediately under the PV panels as there they would impede the natural convection flow of cooling air. They can be slightly lower, where they can suppress buckling but allow cooling air to flow.

One design would be to fit timber dwangs, skew-screwed to the crack-suppressing cleats in light timber framing style. This design would require a system of aluminium earthing straps to electrically join the frames of the PV panels.

Another design uses aluminium tubes located in holes in the crack-suppressing cleats. Metal hose clips of the worm drive type can clamp aluminium strips which pass under the crutch of the truss diagonals to join successive tubes along the line, and to earth the PV panel frames.

DWANGS IN BOTTOM SURFACE

The bottom chords of the trusses must also be prevented from buckling sideways. The 3-D structural model includes timber planks fastened herring-bone fashion to the bottom chords. If an enclosed design is used, these planks can suppress buckling, can act as the lower skin, and can act as a maintenance floor. If an open design is used, the planks can be of square section, similar to the truss diagonals, so would require loose plywood sheets to be used as a maintenance floor. In both cases these planks can rest on suitably shaped crack-suppressing cleats.

FURTHER STRUCTURAL WORK

The nature of a truss is that one chord is loaded in compression while the other is in tension. Tests to reproduce this action are yet to be done, using a more elaborate test assembly in Weltec's Avery machine. This is to form the basis of another student project.

FURTHER WORK ON NATURAL CONVECTION

Dissipating 360 kW of heat has not yet been demonstrated to be possible by natural convection. It may be that half of this can be radiated and convected from the upper surface of the PV panels, but this leaves 180 kW to be removed by convection from the lower surface. The driving force for the air movement is buoyancy provided by the density difference of air at ambient 30°C and 50°C under the panels. This force, diluted by the slope of the table top, must drive the air over a distance of 24 metres. Theoretical work to model this effect is in progress.

REFERENCES

[i] Maycock, *Bloomberg New Energy Finance*, 2017

[ii] National Structural Code of the Philippines, NCSP C101-10, 2010 edition

[iii] Hernandez et al, *Fastener pull-out tests to determine threshold values for roof failure modes observed after Typhoon Haiyan in the Philippines*, 17th Australasian Wind Engineering Society Workshop, 2015

[iv] Manufacturer's data for Yingli Solar YGE 72 photo voltaic panels

[v] NZS 1170.5 *Earthquake actions*, 2016 edition, clause 3.2

[vi] Roark, *Formulas for stress and strain*, 4th edition, p 114

[vii] Roark, p 113

[viii] AS/NZS 1170.2 *Wind actions*, figure 2.4