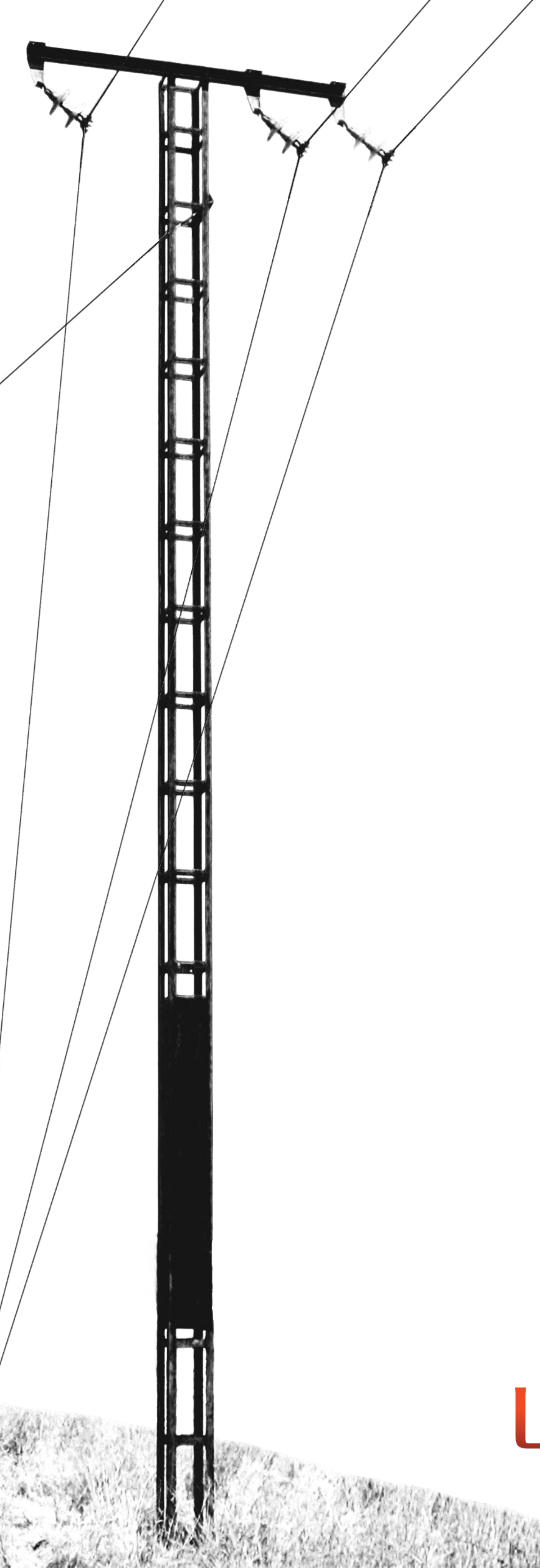

Hillston Feeder

Reinforcement Review

PREPARED FOR

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CLIENT REFERENCE

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1 EXECUTIVE SUMMARY

This report provides a detailed review of the reinforcement criteria currently used by Essential Energy and the expected performance of the preferred OZ-C-Splint reinforcement in general. The Hillston Feeder line is then used as a case study to give an indication of how the reinforcement criteria and performance actually works in reality.

Based on our analysis, the Asset Inspection manual requires some work to remove some contradictions, but more importantly, to make it more useable as a procedural and rule reference up front, with supporting information at the back in appendices (or in a separate document). The manual should also include guidance around accessibility in wet weather when determining if a pole is to be reinforced.

It is recommended that Essential Energy reassess the wall thickness and residual Factor of Safety (FOS) triggers for pole reinforcement/replacement to better differentiate between the two. This will be required to provoke more reinforcement in the network. A higher FOS trigger (i.e. 3 rather than 2) would be a key mechanism to consider. In addition, a move to using a Serviceability index based on limit states design capacity divided by load, rather than the working stress FOS values would be beneficial in aligning the design and inspection/maintenance aspects of the business.

It was found that the C-Splint reinforcement is a good design in general, but the published capacities are above the capacities calculated using current steel design standards and elastic theory. URIE do not recommend using plastic theory based on testing experience and lack of resistance once a plastic hinge is formed. Recommendations are given as to how this should be changed, and the effect this finding may have on the current C-Splint reinforced pole population. In addition, recommendations are given as to additional information that should be sought from the supplier with regard to banding capacity and performance over time.

Existing reinforcements should be re-assessed during the next round of inspections based on the capacities given in this report, and at most they should remain in service for no more than 10 years if the reinforcement capacity is found to be less than the design loads on the pole, due to the increased risk of failure. If the reinforcement is correctly sized and oriented, the reinforcement should have a life expectancy of 10-20 years depending on corrosion and timber decay rates, and the final criteria for replacement that is chosen by Essential Energy. An upper limit of 20 years is recommended unless further study shows this can be extended whilst an acceptable risk of failure is maintained.

The study of the Hillston Feeder line initially showed that only 5% of the reinforcements were adequately sized, while an additional 20% may be adequate if the suppliers' capacities are assumed. However, it was discovered after Version 0 of this report was presented that Essential Energy's asset database was not set up to handle most of the "B" class higher strength splints, and hence the installed splints were not recorded as the correct size. Once the correct splint sizes are considered where data is available; 52% are adequately sized, and the remaining 48% would be adequately sized if the supplier's capacities are used (i.e. 100% are correctly sized to supplier ratings). This has highlighted that the asset database needs to be revised to enable entry of all of the C-Splint sizes. It also shows that the calculations used to size the splints are accurate/conservative.

One unexpected finding was that all of the reinforced poles in the Hillston line appear to have a factor of safety of at least 2.3 for the timber alone based on Essential Energy's own database, or a minimum of 2.8 according to our calculations. If this is typical of reinforced poles, then the reason why Essential Energy has a low failure rate of reinforced poles is likely due in a large part to the timber being able to support the loads without any assistance.

If the reinforcements are correctly sized, they can be used on both distribution and transmission poles, but it is recommended that a dual-pole structure (i.e. Pi structure) should have no more than one pole reinforced. If both poles require reinforcing, they should both be replaced. This is a risk based decision for Essential Energy to make based largely on consequence of failure and confidence in the reinforcement capacity.

If the reinforcements are correctly sized and the poles correctly inspected, there is also no reason to avoid climbing the poles once reinforced.

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REVISIONS

Revision Number	Date	Comments	Approved By
A	10/02/2014	Draft issued for review	NS
0	24/03/2014	Final issue to client	NS
1	05/06/2014	Revised upon receipt of corrected actual reinforcement data	NS

DISCLAIMER

2 INTRODUCTION

Essential Energy engaged URI Engineering to undertake a review of their reinforcement practices with the aim of determining appropriate reinforcement criteria to try and optimise the overall reliability of the network without increasing costs. The Hillston Feeder line is of particular interest to Essential Energy and is the case study for this assessment due to the significant number of poles that were condemned in the line at one time (82 poles). Upon closer review, Essential Energy decided to reinforce around 75% of the condemned poles and replace the other 25% (21 poles).

The Hillston Feeder job has confirmed to Essential Energy that overall network reliability increases may be possible whilst maintaining existing expenditure levels, by increasing the ratio of reinforcement vs. replacement for condemned poles.

Essential Energy recognise the balance that must be achieved with this approach because reinforcement is a mechanism to delay investment to a more appropriate time. In essence, replacement levels still need to remain relatively high to ensure the delay does not increase the required replacements in the future to levels that are unmanageable.

Essential Energy are responsible for determining the appropriate levels of reinforcement in relation to an appropriate investment mix going forward. This report deals with the structural appropriateness of the reinforcement criteria, the reinforcement itself, and the life expectancy of the reinforced poles, with particular attention given to the reinforced poles in the Hillston Feeder line.

A second report will follow dealing with a more general evaluation of the current reinforcement technology employed by Essential Energy vs. Bolt & Ferrule style reinforcements and the applicability of the different reinforcement types to various pole materials.

Note that the terms reinforcing, splint and nail/nailing are interchangeable in the context of this report and refer to engineered structures, normally made from steel, used to strengthen in-situ poles by bridging degraded sections at ground line. The term “wall thickness” refers to an annulus of sound wood around the circumference of the pole, as measured by the inspector.

3 EFFECTIVENESS OF THE CURRENT NAILING SYSTEM

3.1 GENERAL CRITERIA

A summary of the current Essential Energy nailing criteria is given in Table 1. Note that any reference to factor of safety (FOS) is related to working stress design approaches, not reliability based design (limit state) concepts.

Table 1: Summary of current nailing criteria.

No.	Criteria	Section of CEOM7005
1	If the "Safety Factor" determined during inspection is less than 2.0 but greater than 1.0, the pole shall be condemned as unserviceable, but can be "Reinforced if suitable"	4.5
2	The pole must have more than a minimum average wall thickness of 15mm below ground for C-Splint reinforcement. Remaining external diameter must be at least 25% of the original diameter (presumably below ground line). **Note: This is in the inspection criteria for a reinforced pole, not the reinforcement criteria section.**	17.1.2
3	Poles with a wall thickness of less than 45mm may be considered for reinstatement if there are any concerns for the serviceability of the pole.	17.3
4	If the pole has any other defects associated with it that are not expected to last more than one inspection cycle then the pole shall not be considered for pole reinforcement.	17.3
5	Poles shall not be considered for reinforcement if they have less than 15mm of sound wood below ground.	17.3
6	Poles with more than a 15 degree lean shall not be reinforced.	17.3
7	Operational, Private and Bollard poles shall not be reinforced.	17.3
8	Poles with concrete backfill, timber baulks or concrete collars shall not be reinforced.	17.3
9	Poles with spans that cross the rail corridor, or poles within the rail corridor shall not be reinforced.	17.3
10	Poles in swampy, tidal or snowy terrain, or more generally, poles with difficult access for EWP's and nailing machinery shall not be reinforced.	17.3
11	The residual wall thickness at 1m and 1.5m above ground line must be greater than the values in Figure 4.	17.3

In general, the Essential Energy Operational Manual: Asset Inspection (1) is a very comprehensive document obviously written by people with significant experience in the field. However, whilst reviewing the document with fresh eyes, it did appear that a lot of the information was repeated in different sections, and the information did not flow particularly well, making it difficult to follow and to link serviceability criteria to inspection practice. This shows up in Table 1 as some inconsistencies in the wording and general requirements of different sections.

In particular, criteria Number 5 uses the wording "minimum", whereas, Number 2 uses "minimum average" – potentially very different values. In addition, there is a minimum external diameter requirement in Section 17.1.2 "Serviceability Criteria" of the manual, but nothing of the sort in Section 17.3 "Assess Pole for Reinforcement". There are also mixed signals throughout the document as to what should be done if the pole has a Factor of Safety (FOS) of less than 2. In most cases, the document reads that if the FOS is less than 1, the pole should be replaced immediately, but in the

“Safety Factor Assessment Guides” on Page 99 of the manual, there is an indication that poles may be reinforced down to a factor of safety of 0.17 if the only section loss is through loss of external diameter.

These are not significant issues, but in reviewing the document it was apparent that there were no clear-cut rules for when a pole should be reinforced. This makes it difficult for an inspector to nominate a pole for reinforcement, as most people will take the more conservative replace option. Rearranging the manual to centralise the requirements for serviceability and the actions to be taken that depend on the serviceability measures may improve this, along with some changes to the criteria that will be discussed further in this report. This should also be closely related to the inspection requirements, and may be better off within the actual “Pole Inspection” section.

There is also no information in the inspection manual to allow an inspector to determine the required reinforcement size during the inspection. It is understood that the reinforcement is sized using a software program. However, there does not appear to be any documentation on what specific inputs the software requires, and the calculations that are performed to give the required capacity. This is not to say that it is being done incorrectly, but there are some issues when sizing and installing the reinforcements that should be considered by the inspector/installation crew that should be documented, whether it be in CEOM7005 or another document is up to Essential Energy. The reasons for this will be discussed further in Section 3.3.

3.2 USER SATISFACTION

As part of this project a short, 10 question survey was produced and sent out to each region within Essential Energy. The idea behind this stems from past experience where field staff and regional managers can often provide a more practical account of product performance than what can be achieved by just looking at database reports and engineering calculations.

An example of the survey that was sent out and a summary of the results is included in Appendix B. In this case, the reported performance of the reinforcing systems employed by Essential Energy are as expected; there are little or no failures of reinforced poles in the network each year, and all respondents allocated C-Splints as their only reinforcement type having ever been used. This may not be correct given the data provided from Essential Energy’s database for the number of poles of different reinforcement types – shown in Figure 1 – but it has been the only one used for some time.

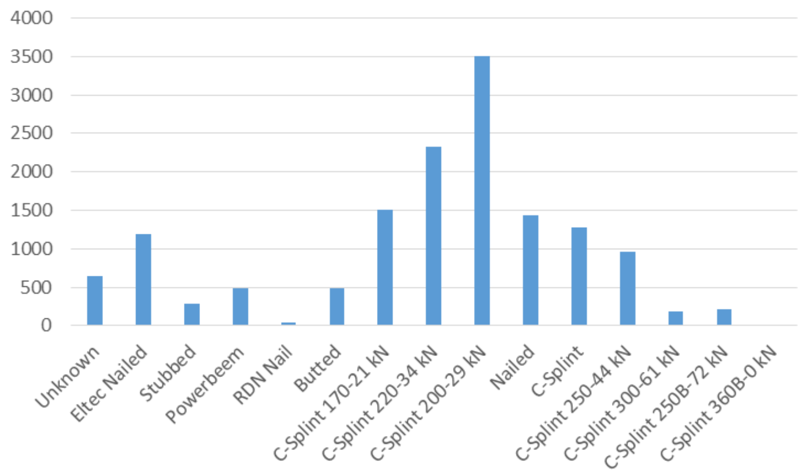

Figure 1: Number of poles reinforced with the various reinforcement types in Essential Energy’s network.

The number of reinforcements used in each region is less than 150 in each case, ranging from <50 per year to <150 per year. The average time that reinforced poles remain in service was quite variable, ranging from 0-5 years in some regions, to 20+ years in the Riverina area. This is particularly interesting because the Riverina area is known for its high salinity and hence corrosive soils, but the confidence level that the respondent had in their answer was none, so it could just be a bad guess.

Interestingly, only one respondent thought that there was a fixed time limit that a reinforced pole was allowed to remain in service, and that time limit was 8 years with high confidence, all other respondents did not believe there was a time limit. We also could not find a published time limit in the literature that was provided.

All respondents reported 0-5 reinforced pole failures per year, which is expected based on Essential Energy only being able to provide evidence of 1 reinforced pole failure in recent times. Only the northern region provided a response for the location of failure, noting all failed in the timber at ground line when they did fail.

The most interesting survey results actually came in the form of the more open Questions 8-10. A summary of the relevant comments are as follows;

- Because line crews are not allowed to climb reinforced poles, there are concerns about poles being reinforced in locations that do not have all-weather access for elevated work platforms (EWP's). This can be an issue when the pole-top hardware needs to be worked on during wet weather and outages caused by storms.
- A similar concern was expressed around nailing switching poles, in that they cannot be climbed to use the switches or replace fuses.
- If reinforcement volumes are low, some regions just replace the poles because it is logistically beneficial if they only get one more inspection cycle out of the pole before it is flagged for replacement.
- There are safety & environmental concerns over the strength and suitability of the reinforcement to prevent the pole failing and or leaning.
- The time period between the determination that a pole should be nailed, and installation of the nail (seemingly due to sourcing times) is too long (> 6 months).

These are very important considerations for Essential Energy when deciding on their final reinforcement policies, particularly the practicalities of working on reinforced poles if they are not allowed to be climbed, and the logistical efficiency issues with low volumes of reinforcements spread across large areas.

Some regions appear to avoid using reinforcements as a local rule due to the above issues.

It is noted that if the reinforcement is correctly sized and installed, there should be no issue with climbing reinforced poles when compared with unreinforced timber poles. However, it would be important to check that the banding is sufficiently tensioned.

3.3 STRENGTH

3.3.1 Current Knowledge

The main reason for reinforcing a pole is to increase the strength of the structure such that replacement of a low-strength pole can be delayed without a significant difference in likelihood or consequence of failure. How reinforcement provides this is quite complicated.

Until recently, it has been thought that the reinforcement and the pole work together to resist the loads on the structure. However, recent testing for another utility has shown in simple terms, this depends on how tightly the reinforcement is attached to the pole. If the connection between the two is 'loose', such as a loose fitting bolt or band, the reinforcement takes practically no load until the timber has begun to fracture. Once the timber begins to fracture and sheds load to the reinforcement there can be some minor increase in the overall capacity from the two working together, but then the capacity is solely dependent on the capacity of the reinforcement.

This may be different for tightly banded reinforcement, but the amount of composite action is expected to be limited given the relatively high stiffness of even a very decayed pole, and observed relative movement between the pole and the reinforcement. Loads on structural elements are always apportioned as to the stiffness of the members that are sharing the load, and the connection between them. This is important for Essential Energy reinforcements given that Essential Energy criteria requires a minimum good wood thickness in the pole at ground line for the entire period the pole is reinforced, and because there is no mechanical shear connection between the pole and the reinforcement other than friction, which is not considered sufficient to provide full composite action between the pole and the reinforcement. It is also important to note that once the timber does fail, it may induce a shock load in the reinforcement.

Given the doubt about how the loads are shared between the reinforcement and the pole, the most appropriate design method would be to assume that the strength of the reinforced pole in the reinforced zone is solely based on the capacity of the reinforcement itself. Hence the use of reinforcement needs to consider the following;

1. If the reinforcement is only a temporary solution to make sure the structure does not catastrophically fail before it can be replaced, then the reinforcement needs to be strong enough to simply “catch” the failing pole and prevent it from causing damage or harm.
2. The structural adequacy of the reinforcement cannot be assumed adequate by looking at the number of reinforced poles in the network and comparing to the number of failures, because if the timber has not failed, the chances are the reinforcement has not been significantly loaded, and the relationship is more reliant on the occurrence of high wind events than it is the strength of the reinforcement.
3. It must be assumed that there is no timber strength and that the reinforcement takes the full design load, particularly when the reinforcement is intended to reinstate the full design capacity of the pole.

At this stage, it is understood that the recommendation from the OZ-C-Splint supplier (the current preferred reinforcement type for Essential Energy), is that the C-Splint will take a pole from a FOS of 2, back up to its original FOS of 4, and that the capacity of the reinforcement can be added to the residual capacity of the pole. Based on what we now know, it would be more appropriate to consider a newly reinforced pole to have a FOS of 2, rather than 4 (in simple terms), assuming that the reinforcement is correctly sized.

It is noted that compression and shear loads are not considered in reinforcement design currently. In general, it is assumed that the effect of these forces will be minimal with regard to reinforcement capacity. It may be reasonable to assume that compressive loads are always taken by the timber, but it is noted that there will be additional second order bending moments induced into the reinforcement as the pole starts to bend and fail. This is particularly important for poles with heavy attachments such as transformers.

3.3.2 C-Splint Strength

The OZ-C-Splint capacities given by the supplier are shown in Figure 2, with the typical Essential Energy pole size ranges lying within the coloured boxes. The diameter ranges for the overlay are based on Grade A and B sizes from EAS111, and the bending capacities shown in the notes are based on a FOS of 1 and working stress design capacities, and an embedment of 10% of the pole length +600mm. This assumes that the pole is at 100% of its design capacity using working stress design. Hence, the size 135 splint may be suitable for some very lightly utilised 2kN poles (i.e. some street lights). It is assumed that the capacities are based on only one splint member per pole.

Figure 2 shows that there are some larger poles with high kN (kilonewton) ratings that may be unsuitable for single splint reinforcement. However, if two splints are used and it is assumed they share the load equally, then double splint systems should be able to support the majority of the working stress design loads that Essential Energy poles will be subjected to, assuming the actual capacity is as stated. Dual splints may require additional banding.

The following is a list of the positive aspects of the OZ-C-Splint from a structural perspective;

- The splints extend a good distance above and below ground, which will help reduce the loads on the banding.
- The banding option is a good one assuming there is no shear transfer requirement between the pole and the reinforcement to give the desired/calculated capacities.
- The banding option will minimise the number of catastrophic pole failures that occur at the top of the reinforcement in bolted systems due to the reduced cross section at the bolt locations.
- The section shape is reasonably efficient and for most types, especially the larger ones, there is little difference between section capacity and member capacity in bending (as per AS 4100-1998 (2)).

However, our assessment also highlighted the following concerns;

- There were a number of different documents provided that nominated material strengths, size ranges and bending moment capacities of the C-Splint nails. A minor concern stemming from this was that the reported steel yield strength for the normal splints ranges from 420 MPa for the older documents (circa 2001), to 380 MPa in the more recent (2006 on) documents. This is not a significant change, but Essential Energy need to ensure that their assumed capacities use the latest information.
- Based on calculations in accordance with AS 4100—1998, the capacities expressed by the supplier appear to correspond somewhat with the section capacity in the strong direction only, however this is not documented anywhere in the literature that we reviewed.
- The difference between section capacity (section yield) and member capacity (yield along the length of the member, usually due to lateral torsional buckling) is small for the larger poles, but based on the calculations shown in Appendix C, the member capacity can be as low as 62% of the section capacity for the 220B Splint, up to 92% for the 300 Splint. The higher steel strength "B" splints have a greater reduction than the normal splints, and the smaller sizes have a larger reduction than the larger sizes. In most cases with steel, calculations will prove conservative when compared to test results, however based on our experience with similar systems, testing is unlikely to show the C-Splints to have the supplier nominated capacities (with the FOS of 2).
- When comparing the unfactored capacity in the strong direction from the calculations in Appendix C to the nominated capacities (see Table 2), the nominated capacities are well above the calculated member capacities, and the section capacities in some cases. Upon discussion with the supplier, this is due to the original calculations being based on the plastic section capacity, rather than the elastic member capacity. Based on our experience, once a plastic section failure occurs, the pole will fall to the ground under its own weight, unless the conductors hold it up, which is unlikely to occur for long rural spans. In our experience, the elastic member capacity is much more appropriate and predicts the strength much closer than the plastic section capacity, and in some cases can over-predict strength by small amounts depending on the geometry of the splint/timber arrangement.
- It is assumed that the banding is strong enough, and that the banding does not loosen and slip down the pole during the life of the reinforcement (i.e. through livestock rubbing on it, hot/cold cycles, creep, etc.). There was nothing provided (calculations or testing) that addressed these questions, but this does not mean the information is not available and the supplier should be consulted.

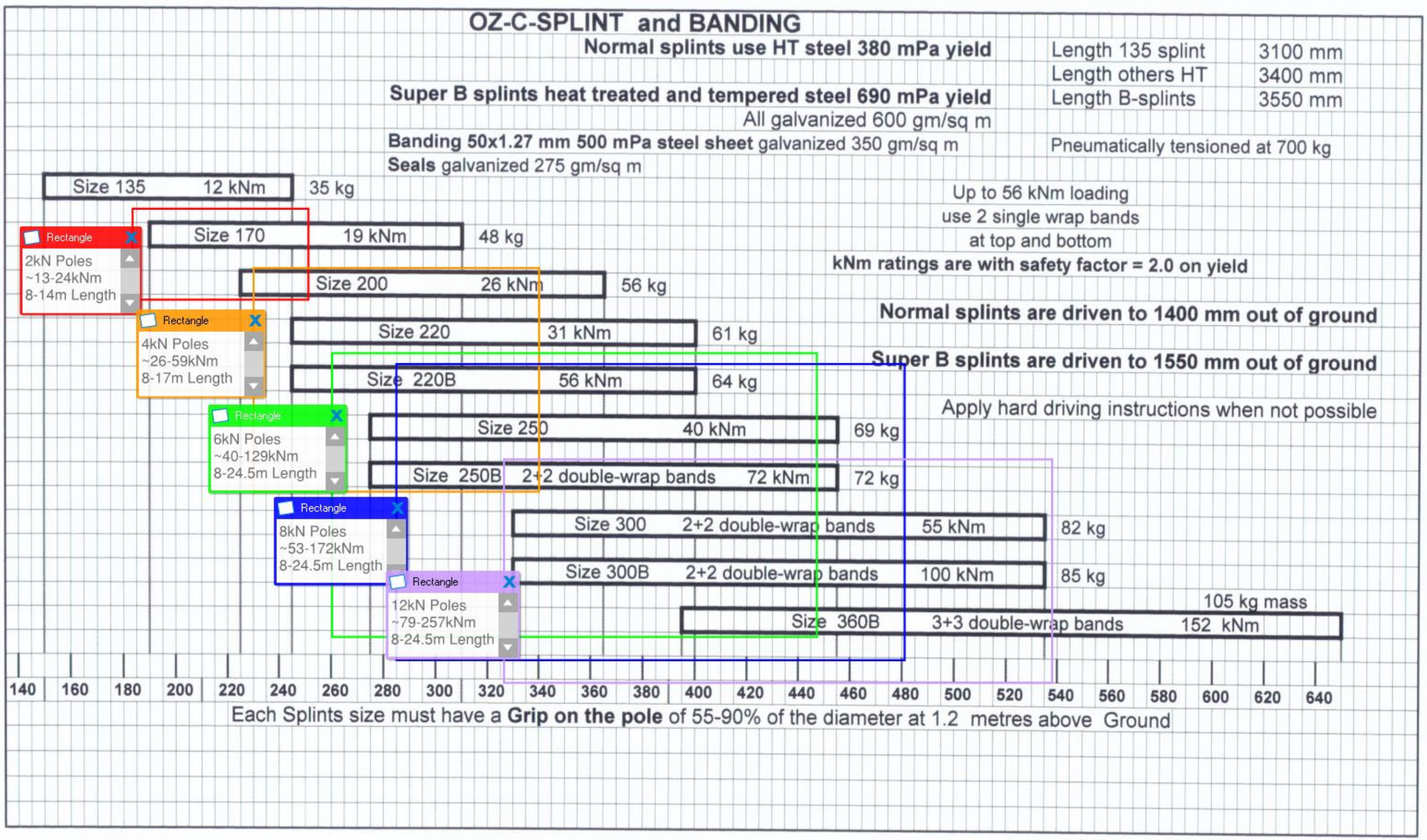

Figure 2: OZ-C-Splint published capacities with pole size and capacity ranges overlain.

- The literature provided appears to assume that the C-Splints are always installed so that the strong axis is always aligned with the axis of the critical load. However, there is no mention of this as being a requirement in any of the documentation we came across. This is a cause for significant concern if the installers are not aware of this requirement, because they are likely to install it in other orientations to avoid other attachments and clearance issues without knowing the effect this will have on the rated capacity. The weak axis capacities (the minimum rating for the two directions shown in Figure 3) are significantly below (approximately 33%–50%) the strong axis member capacities, which are already below the published capacities. This becomes critical, because a linear relationship between orientation and strength will give a 7% reduction in capacity with just a 10° misalignment with the load direction. A 30° misalignment will give a 22% strength reduction.

Table 2: Calculated vs. reported capacities.

OZ-C-Splint Size	URIE Calculated Unfactored Capacity (kNm) Strong Axis Section Capacity (M_{sx})	Strong Axis Member Capacity (M_{bx})	Weak Axis Member Capacity (M_{bx}=M_{sx})	Reported Capacity (kNm, FOS = 1)
135	25.1	18.0	8.6	24
170	38.1	28.5	13.1	38
200	51.1	40.6	17.3	52
220	59.8	49.5	20.1	62
220B	102.0	64.0	33.2	112
250	75.1	65.8	25.0	80
250B	126.9	87.1	40.4	144
300	100.7	95.9	32.7	110
300B	167.5	131.6	51.3	200
360B	221.7	204.2	64.9	304

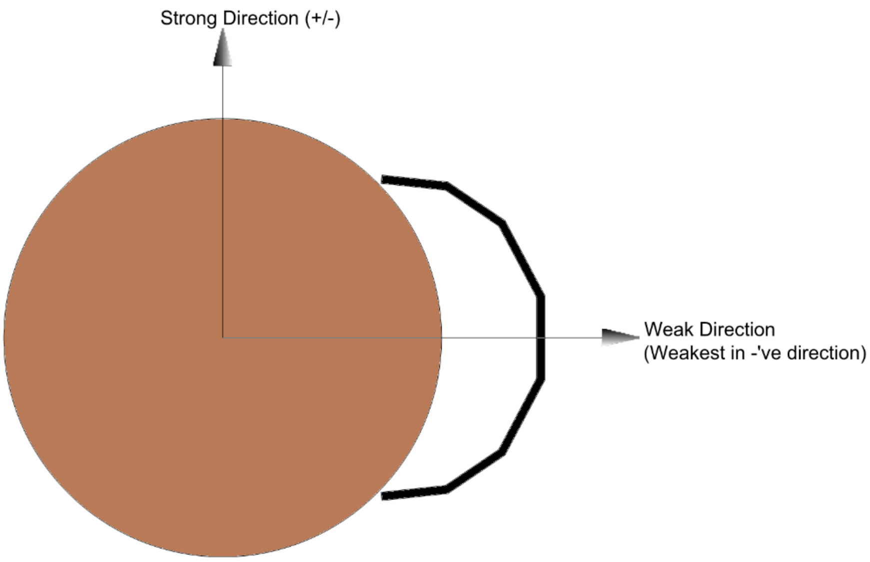

Figure 3: Reinforcement orientation.

- Due to the significant difference between the capacity of the reinforcement in the weak and strong directions, there is a risk that even if the reinforcement is sized appropriately for the load in the strong-direction, any loads that occur in other directions may cause failure if they are large enough. Similar issues are seen in New Zealand where in-line loads from snow or even failure of an adjacent pole have led to over-stressing of the rectangular concrete poles, which has led to failure about the weak axis. This would be more of an issue for un-stayed terminations and angle poles within Essential Energy's network. If there are numerous reinforced poles adjacent to each other, the risk of cascade failure is also heightened.

Based on the findings, we have concerns that the C-Splints may be under-sized for the nominated capacities, particularly for the larger splints, and for splints that are not installed so that their strong axis is in line with the highest load direction.

Even though the reinforcement may be undersized, it is worth noting that if the reinforcement is too-strong for the pole, the pole is likely to fail catastrophically above the reinforcement because the timber at this point will have a lower capacity than the reinforcement. This will normally only occur under extreme load events that are higher than the design load on the original pole. If the reinforcement is sized so that its capacity is just above the design load at ground line and just below the strength of the timber at the top of the reinforcement, it would be an ideal design. In this case the steel would yield during a higher than design event and lean the pole with a very low chance of causing damage or harm, and the timber would not fail catastrophically above the reinforcement.

Unfortunately, the large number of variables and unknowns in this scenario mean that Essential Energy can only try and maximise the chance of the reinforcement failing above the design load and before the timber at the top of the reinforcement, it can never be guaranteed.

3.3.3 Pole Capacity – Top of Splint

The strength of a reinforced pole not only relies on the capacity of the reinforcement at the ground line, it also relies on the strength of the timber at the top of the reinforcement. At the top of the reinforcement the timber needs to have a minimum strength so that the pole can transfer the load into the reinforcement, and so the pole does not fail prematurely above the reinforcement. Figure 4 shows the C-Splint criteria for timber wall thickness towards the top of the reinforcement.

Figure 4: Minimum wall thickness criteria at the top of the splint for C-Splint reinforcement.

It is assumed that the diameters in Figure 4 are the diameter at the point of wall thickness measurement. After doing some basic checks on the numbers in Figure 4, it is apparent that at 1m above ground line the pole is allowed to be reinforced with a timber FOS of 2.6 for a 150mm diameter pole, down to 1.9 for a 650mm diameter pole. This is based on loss of section and not fibre strength, and the assumption of a FOS = 4 to begin with.

At 1.5m above ground line, the FOS ranges from 3.25 for the 150mm diameter pole to 2.73 for the 650mm diameter pole.

Essential Energy have obviously made a decision at some stage in the past to use wall thickness measurements as triggers for pole reinforcement/replacement, regardless of the calculated factor of safety. This can have advantages in removing the particularly risky poles even if the inspection passes them due to some error in the inspectors load measurements or the calculations in general.

Figure 5 shows a representation of the relationship between external diameter, wall thickness, and residual FOS for the pole (based on a FOS = 4 for the new pole). Using the Essential Energy reinforcement and replacement inspection criteria, the shaded red box outlines the situations in which a pole may be reinforceable. Note that using a wall thickness for replacement of 30mm will not see the FOS for the pole drop below 2 for poles less than 370mm diameter. If the pole isn't fully utilised strength-wise when new, this will be even worse. This explains why Essential Energy poles are rarely earmarked for reinforcement and are replaced instead.

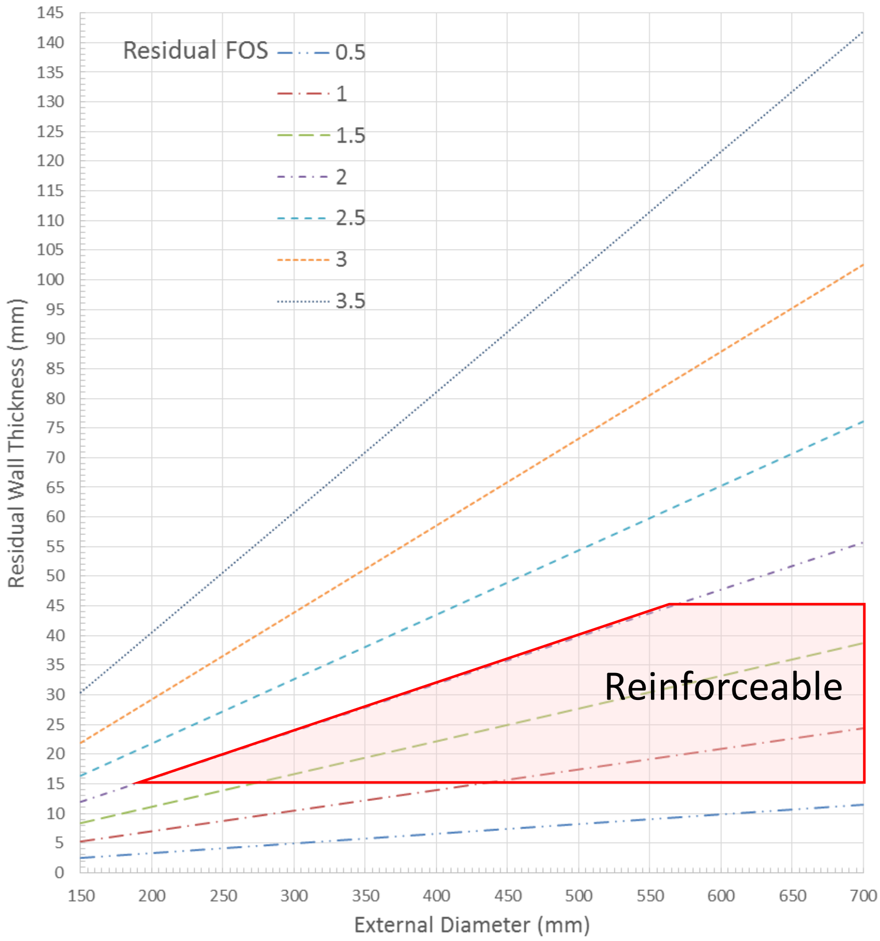

Figure 5: Residual factor of safety for a range of original diameter and wall thickness combinations. Red box outlines the ground line dimensions of poles that can be reinforced (does not consider capacity requirements).

3.3.4 Foundation Capacity

When considering the capacity of the reinforced pole system, it is important to consider the capacity of the foundations as well as the pole and nail themselves. Unfortunately, there have been few studies on the foundation capacity of reinforced poles, as the testing needs to be done full-scale and cannot be calculated. In general, based on the minimal testing that we have witnessed, even loose sandy soils appear to be able to support the reinforcement such that the reinforcement will yield before the foundation undergoes significant rotation. In addition, the report and photos of the reinforced pole failure on the Nyngan to Cobar line (*132kV Pole Failure Nyngan to Cobar Pre autopsy Details* – see Appendix C) suggest that the foundation capacity was not a factor in the failure of the structure. Hence, we do not expect there to be any issues with foundation capacity within the normal range of soils. However, reinforcement should not be used in particularly boggy or weak soils where specific foundations are required.

3.3.5 Using Wall Thickness Readings

Further to the above caution is required when using wall thickness as the sole trigger for further action to be taken on a weak pole. The main reasons for this are;

- The wall thickness can be variable around the pole and hence the measured value may not be the critical value, especially when the inspections are mainly conducted on the neutral axis of the pole.

- The measured wall thickness can be variable between inspectors with both over and under-estimation. Fortunately, from our observations and anecdotal observations from Essential Energy, the inspectors appear to be conservative in most cases.
- The actual bending capacity at the section will change depending on the external diameter as well as the wall thickness. In this regard, the allowable wall thickness should at least be reduced by the reduction in external radius, or some function thereof.
- The required bending moment capacity varies based on the length of the pole for the same diameter.
- Currently poles are reinforceable with a wall thickness of 15mm, but the reinforced pole is no longer suitable once the pole has less than 15mm of sound wood. This means that if the next inspector looks at the pole and the wall thickness has decreased by 1mm they are likely to flag the pole for replacement after only one inspection cycle.

We do note however, that the minimum wall thicknesses for unreinforced poles are over-ruling criteria, in that if the wall thickness is less than these values the pole needs to either be removed or replaced regardless of the calculated safety factor.

Essential Energy criteria is based on both residual FOS and wall thickness, but the two types of criteria do not appear to interact to allow many poles to be reinforced, i.e. the minimum wall thickness may be reached before the FOS for reinforcement is reached and the pole is replaced rather than reinforced.

Hence, Essential Energy may want to consider aligning the capacity triggers differently. Possible limits that would currently not produce an insurmountable number of poles to be reinforced might be to have poles earmarked for reinforcement once they reach a FOS of 3, whilst keeping the wall thickness triggers the same or increasing slightly to 20-25mm, with an above ground wall thickness minimum of 10-15mm for replacement of a reinforced pole. This would remove some of the confusion that arises between the criteria for unreinforced and reinforced poles. Ultimately, Essential Energy will set the limits so as to maximise asset reliability and minimise expense, but there needs to be better definition between reinforcement and replacement criteria if Essential Energy are to increase the number of pole reinforcements.

Note that it may be worthwhile changing the terminology to replace Factor Of Safety (FOS – working stress capacity divided by load) with Serviceability Index (SI – Limit states capacity divided by load) to align the inspection and design aspects of the business. It is noted that even relatively minor terminology changes can cause change management issues, so Essential Energy will have to approach this with some care.

3.4 ESTIMATED ASSET LIFE

The remaining life of a reinforced pole is a hard thing to predict due to the many influencing factors. Even assuming that the inspectors collected all their data and are 100% accurate, there are random variables that cannot be directly measured such as fibre strength, foundation strength, corrosion and decay rates, etc.

However, we can make some assumptions and derive ranges for the life expectancy of the reinforced poles based on durability and wind load probability.

3.4.1 Durability

The durability of the reinforced pole is influenced by the durability of the timber (primarily above ground) and the steel (primarily in-ground). Based on the C-Splint specifications (Figure 2), the splint is galvanised to 600 g/m^2. Based on AS/NZS 4680 (3), this gives a minimum average thickness of 0.085mm (local minimum of 0.07mm). The Galvanizers Association of Australia website (4) states that galvanised steel to this level "will give an additional life of about 10 years to steel pipes" in best performing alkaline and oxidising soils.

Our experience from reports on steel street lights, service poles and other steel pole structures suggests that this is on the conservative side for reasonable soils, but overall it varies between 5 years for highly reducing soils, and 25+ years for more benign soils. AS/NZS 7000 (5) suggests 9-100+ years in above ground environments, but does not suggest any in-ground expected performance. The piling code AS 2159—2009 (6) suggests corrosion rates for bare steel of between 0.01-0.04mm/year, but does not give any values for particular coatings such as galvanising.

Another variable that needs to be considered is the proximity of any earthing stakes, as they may cause an accelerated level of corrosion due to dissimilar metals or perhaps even stray current corrosion (if there is a DC component).

Given that there is no corrosion allowance in the design of the reinforcement, it is recommended that the 80% rating within the top 300mm of soil be used as a visual guide for the replacement of the reinforced pole, as it is unlikely to be associated with a significant strength loss, but is easily detectible visually.

If the reinforcement is to be re-used on another pole after removal, it is recommended that the galvanising thickness be measured using an appropriate Ultrasonic Thickness device, and if the galvanising thickness is more than 90% of the minimum thickness in AS/NZS 4680 (3) in the ground line region (a minimum of 5 readings to be taken), then the reinforcement can be re-used. Bands should be discarded.

Table 3: Meuller corrosion visual classification chart (7).

Rating Percentage	Description of steel surface condition
95.0%	Galvanizing like new
92.5%	Galvanizing dull
90.0%	Galvanizing very dull
87.5%	Pin-point rust spots
85.0%	Galvanizing entirely gone
80.0%	Light rust film
70.0%	Shallow pitting
60.0%	Scaly rust or pits less than 50% penetration of metal
45.0%	Heavy rust or pits half way through metal
30.0%	Heavy rust or pits approx. three quarters of the way through the metal
15.0%	Localised complete perforation
0.0%	**General complete perforation**

The durability of timber above ground is normally significantly higher than the timber in contact with the ground. However, if there is a moisture source close to the timber, no matter how far above ground it is, decay rates can be just as fast as at ground line. In most cases for poles, there is minimal decay more than 1m above ground level, and it is expected that the wall thickness limits at ground line and the steel durability will govern the durability of the pole rather than the timber above ground if reasonable residual strength constraints are used during assessment for reinforcement.

Given the decay rates for timber have a coefficient of variation of around 90% or more (8), it is hard to give a definite life expectancy for the timber portion, but if there is a large enough allowance of residual strength at the top of the reinforcement, the timber can be expected to last 5-20 years or more.

Therefore, if the reinforcement criteria is set up in a similar manner to that suggested in Section 3.3.5, the life expectancy based on the materials would be an average of 10+ years. It would be recommended for the reinforced poles to remain in service for no more than 20 years. Ultimately, with confidence in the inspection accuracy and reinforcement strength, there is potential to push this further.

3.4.2 Wind Load Probability

Based on the current nominated capacities assuming a FOS of 2 against a design wind pressure of 500 Pa on the conductors, and assuming the wind load on the poles are proportional to the conductors, converting to limit states design gives a factor of 1.8 (FOSxϕ = 2x0.9) to be applied to the nominated strengths, and a factor of 1.8 to be applied to the wind loads. The limit states design wind pressure on the conductors is assumed to be 900 Pa (1/50 annual probability of exceedence) for the sake of this assessment, based on Region A values from HB 331—2012 (9), which gives 900/500 = 1.8.

Given that the increase in capacity matches the increase in wind loads when doing the simple limit states conversion, if the published C-Splint strengths are adequate, the annual probability of exceedence (APE) for the design wind speed would be 0.02, which is the same as what distribution poles are currently design to. If sub-transmission poles are design to a 0.01 APE then the reinforced pole would have twice the probability of failure in any given year compared with the original pole design. However, if it is expected that the pole will only remain in service for 20 years, then the cumulative probability[1] of failure over the full service life expectancy would be less than 33%, compared with 39% for a 0.01 APE over a 50 year design life. Note that the cumulative probability of a 0.02 APE over a 50 year design life (i.e. distribution poles) is 64%.

The question remaining is; if the C-Splint capacities are not as high as currently assumed, what is the risk of failure within the maximum expected service life of 20 years, and what would be the maximum time in service to give the same cumulative probability of failure. It is important to note that this assessment is based on the cumulative probability only, on a per-year basis, the risk of failure for the understrength reinforcements will be higher than assumed for an unreinforced pole.

The worst case between calculated and reported capacity is 57.1% for the 220B reinforcement. Hence, the equivalent limit states pressure on the conductor that would see this reinforcement 100% utilised would be 513.9 Pa. Based on the equation for V_R from AS/NZS 1170.2—2011 (10), and assuming that 900 Pa relates to a wind velocity of 39 m/s, this equates to an equivalent wind speed of;

$$V_R = \sqrt{\frac{39^2 \times 513.9}{900}} = 29.5m/s$$

Given that this is less than the velocity of 30 m/s for the V_1 wind speed, it means that there is a 100% chance of this wind speed occurring in a given year. Which also means the cumulative probability of exceedence over any time period is also 100%.

For the worst case normal reinforcement that is applicable to Essential Energy, the 170 Splint is calculated at 75% of its reported capacity. Doing the same calculation gives an equivalent pressure of 675 Pa, and a wind speed of 33.8 m/s. This equates to an APE of 0.12. In turn, this equates to a 73% cumulative probability of exceedence within a ten year design life, which is slightly higher than that for a normal distribution pole over its life.

These calculations are theoretical, so there will always be a difference between the calculations and reality, particularly regarding that most reinforcements would not be 100% utilised when they were

[1] Note that the applicability of using the APE to calculate a cumulative probability of exceedence over the life of the asset is still disputed by some engineers.

originally selected, and testing will normally show a higher capacity compared with the calculations (although not as big as the strength deficiencies noted in Section 3.3.2).

3.5 PREVIOUS FAILURES

Despite this report highlighting a significant deficiency in assumed strength for the C-Splint reinforcements, there are very few reported occurrences of reinforced pole failures in the Essential Energy network. This could be due to a number of factors including, but not limited to;

- The timber poles never get weak enough that they cannot support the loads themselves.
- The design wind loads are conservative based on local effects or just based on the design wind speeds themselves.
- The reinforcements are selected so that they are under-utilised when compared to the design loads.
- Failures may not always be correctly reported.

Essential Energy provided the reinforced pole failure report shown in Appendix C upon request for all information relating to reinforced pole failures. Based on this report, the wind loads were likely to be getting close to the ultimate limit states design wind loads, at least for the 0.02 APE event.

In this instance the structure was a ‘pi’ structure with both timber poles having C-Splint reinforcement installed. Based on the photos in the report, the failure mechanism for the reinforcement was a typical member capacity failure; lateral torsional buckling for both splints (twist around the pole). This failure mechanism is exacerbated by the eccentricity between the centreline of the pole and the shear centre of the reinforcement profile.

It is likely that the larger diameter bottom pole failed first, and it is apparent that the reinforcement was not strong enough the support the failed pole. This is likely to have been made worse by the in-line stays given that they seem to still be taught even after the poles have come to ground. In this case the stays would have applied additional vertical force, which once the pole started to fail, would have added to the bending moment and basically continued to pull the pole to the ground.

Note that after some basic checks, using a working stress wind pressure of 500 Pa and assuming the wind is perpendicular to the conductors, our calculations give approximately half the load calculated in the report; ~120kNm for the whole structure. This would be shared approximately 60% to the larger pole and 40% to the smaller pole based on both poles having the same modulus of elasticity and wall thickness. Hence, if the wind pressure was at 500 Pa at the time of failure the reinforcement would have been required to support approximately 72kNm. From Table 2, the member capacity in the strong axis for the 300 Splint is 95.9kNm (unfactored). There did not appear to be any corrosion on the reinforcement, and the reinforcement appeared to have been oriented correctly to give close to the maximum capacity. Hence it is expected that the load was close to the ultimate capacity of the reinforcement, and the second-order effects from the stays were likely to have provided the extra force necessary to fail the reinforcement and pull the pole all the way to the ground.

At some stage the strapping has failed on the larger pole. This may have been due to fatigue of the strapping over time, corrosion or just a sudden shock load on the system above its capacity. Whether this contributed to the failure and the inability of the reinforcement to stop the pole from hitting the ground or not is uncertain. We have not done the calculations on the capacity of the straps for this report due to the unknowns around fatigue performance, prestressing levels and accuracy, thermal effects on the prestressing and whether the strapping loosens over time and what effect this has on strength. It is assumed that the straps are adequate, but it would be prudent for Essential Energy to ask for information on these aspects from the supplier.

Based on the Nyngan to Cobar line failure, the following can be inferred;

- If the reinforcement had been adequately sized, the poles are unlikely to have fallen all the way to the ground (assuming the straps were also adequate).
- Pi structures with two reinforced poles are not recommended because if one pole fails, the other reinforcement will not be able to support the additional load, as it only has the same capacity as the reinforcement that failed. If both poles need reinforcing, they should both be replaced due to the increased risk of catastrophic failure. If the poles are double nailed, and the combined capacity is assured to be higher than the design loads, then reinforcing both poles may be a viable option.
- Larger lines and poles would benefit from having two nails per pole to reduce the eccentricities between the pole and reinforcement. This may be required to give the desired capacity anyway.
- The member capacity should be used in the design of reinforcement because lateral torsional buckling is a potential failure mode.
- Unless required for termination poles or strain poles, in-line stays should not be used even for un-reinforced structures, as they are unlikely to prevent failure from the across-line loads, and they almost assure that the pole will hit the ground if it does fail. A better solution is to use stays at least 30° to the line, preferably 45°, that way they give some support for in-line and across-line loads.

4 EFFECTIVENESS OF THE NAILS SELECTED FOR THE HILLSTON POLES

4.1 METHOD

To determine the effectiveness of the C-Splints used on the Hillston line, we have completed the following exercise using Essential Energy data, with the results summarised in Appendix A;

- Calculated the loads on each structure from the as-built line drawings, using working stress design wind pressures of 500 Pa on the conductors and 750 Pa on the poles, and approximate conductor tension loads for angle poles based on a 15% stringing tension.
- Calculated an "actual" FOS at ground line based on the load calculated in the first step, and the capacity of the pole from the inspection measurements.
- Selected the appropriate reinforcement size from the supplier capacity ratings.
- Selected the appropriate size from the calculated member capacity in the strong axis (Table 2), divided by 2.
- Compared our required reinforcement capacities with the sizes that were installed based on the provided revised actual installed splints.

4.2 FINDINGS

Based on the results of the assessment, the following observations were made;

- All the poles that were chosen to be replaced (the salmon shaded rows in Appendix A) have a residual factor of safety greater than 2.3, based on the diameter and wall thickness readings only.
- Given the reinforcements that were used (yellow shaded rows in Appendix A are reinforced poles) are shaded light green if they are strong enough based on the supplier strength ratings, or darker green if they are strong enough based on the calculated member capacities, there are 27 of 52 that are suitable based on calculated strengths, and 52 out of 52 that are suitable based on the supplier strength ratings. Note that nine poles had no reinforcement data or were replaced rather than reinforced.
- In other words, the calculations used to calculate the loads appear to be accurate or conservative. The conservativeness means that just over half the splints used were suitable based on the elastic member capacity calculated by URIE.
- The minimum of the 'actual' residual timber pole FOS values was calculated as 2.8. This means that even though some reinforcements are potentially under-strength, the poles have a low risk of failure.
- Even using the Essential Energy FOS values, the minimum of the reinforced poles is 2.28, which confirms that the reinforced poles have a low risk of failure.
- The green shaded rows show poles that appear to have been overlooked for reinforcement despite having some of the lowest FOS values of all the data provided. Some of these may actually be steel poles that have replaced the timber, but the database has not yet been updated. Essential Energy should verify this.

These observations give a good idea of why reinforced poles appear to perform reasonably well despite the potential insufficient capacity; the timber poles themselves may have enough residual capacity such that the reinforcement is never really required.

In recent years, there have been a number of studies commissioned by EANSW and individual utilities that are beginning to suggest that the strength of the sound timber decreases with age for poles, potentially by up to 40-50%. However, even if this loss of strength occurs and hasn't been considered by Essential Energy, the factor of safety is still more than 1 against working stress wind pressures for all the reinforced poles in the Hillston line. This means that there is a low risk of failure of the timber pole, even up to a wind speed of approximately 30 m/s (108 km/h).

The reinforcement was not inspected during the course of this project. However based on their location and time since the reinforcement was installed (less than 1 year), corrosion is not expected to be an issue. It would however be worthwhile for Essential Energy to confirm that the reinforcements are oriented so that the strong axis of the reinforcement aligns as closely as possible to the bisector of the line (or the direction of the highest load).

If Essential Energy were to accept that the reinforcements are in fact under sized in most cases, the poles can have additional splints installed to bring them up to the required capacity.

When considering the discussion in Section 3, the Hillston feeder reinforcements are expected to give acceptable reliability for around 10 years as long as;

- There is only one reinforced pole per structure and the un-reinforced pole has adequate capacity as per Essential Energy's serviceability requirements (1).
- The splints are oriented to align the strong axis of the splint with the worst case load direction for the pole.
- The poles are inspected and maintained as per the current requirements of CEOM 7005 (1).
- There are no defects above the level of the reinforcement that may cause the pole to fail above the reinforcement.

The value of 10 years is due to half the poles having a higher risk of failure due to being sized only to suit the supplier capacities. If this reinforcement capacity is adequate this value would be 20+ years.

Note that the as-built drawings may be incorrect or the correlation between pole numbers on the drawings and Essential Energy asset ID's may be incorrect. However, based on the type of line the configurations and dimensions assumed when calculating the loads, they are considered reasonable for the sake of this assessment.

Based on the calculated residual capacity of the timber poles themselves, the risk of failure of the reinforced poles will be similar or slightly improved if there is some composite action. However, the risk of catastrophic failure where the conductors go to ground or where minimal safety clearances are maintained will decrease.

5 CONCLUSIONS

Based on the general review of the provided literature and data, we conclude that;

- The current OZ-C-Splint strength ratings appear to be higher than they should be based on steel design calculations in accordance with AS 4100—1998 (2), this is due to the use of plastic section capacity rather than elastic member capacity.
- The Essential Energy Asset Inspection manual (1) has a large amount of really good information, but it does also contradict itself in some cases and does not provide enough information on the considerations around whether a pole can be reinforced based on access, reinforcement orientation, defects above the reinforcement level, or even the calculations that are required to check which reinforcement to use.
- The use of wall thickness values as triggers for when a pole can be reinforced and when it should be replaced is difficult given the range of diameters and pole lengths, and the variability in measurements between inspectors. Realignment of the wall thickness and FOS triggers is necessary.
- The orientation of the reinforcement is of particular importance as the capacities given by the supplier are only in relation to the strong axis, and the capacity about the weak axis could be as much as 70% lower.
- There is limited explanation given as to Essential Energy's performance requirements for the reinforcement. In other words, what does Essential Energy need the reinforcement to do in terms of life extension and failure mechanisms?
- Shear, compression, and second order effects are not currently accounted for in the design/selection of the reinforcement types. The effect of high compressive loads and second order effects need to be considered for poles with transformers, stays, etc. as they will affect the ability of the reinforcement to stop the pole from falling to the ground if it does fail.
- The orientation of the C-Splint is critical, but there is no recognition of this in any of the literature that we were provided.
- There are significant advantages to using a banded reinforcement system. In our experience they are less likely to fail above the reinforcement, however the reinforcement must be sized correctly to support the design loads, whilst trying to keep it less than the capacity of the pole at the top of the reinforcement.
- It is critical that the banding be able to support the design loads, and that it does not slip down the pole during service.
- Whilst there is no documentation proving such, the foundation capacity of the C-Splint reinforcement is expected to be sufficient, except in poor, very loose sands or organic/boggy soils.
- There needs to be limits on the capacity of the timber at the top of the reinforcement that ensure the timber is stronger than the reinforcement at that point. This will avoid failures above the reinforcement. This should be based on a residual FOS or serviceability index basis, not just minimum wall thicknesses.
- The pole should not be reinforced if there is evidence of significant termite attack towards the top of the reinforcement. This also applies to any significant strength reducing defects above the top of the reinforcement, including unsound knots, fruiting bodies, etc.
- The life expectancy of the C-Splint reinforcements will depend on the actual utilisation of design strength for each reinforcement in service currently, and the allowable probability of failure that Essential Energy are willing to accept. In general, existing reinforcements should not be left in service for any more than 10 years, and some may require replacement at the next inspection cycle if the correct capacity is used in assessing their adequacy for the design loads.
- If the reinforcements are sized correctly moving forward, the life expectancy would be 10-20 years on average. This will be governed by steel corrosion and timber durability, and 20 years

would be considered the maximum service life of correctly reinforced poles unless further studies determine an extension is possible.

- 'Pi' structures should only be reinforced if at least one of the poles has enough residual capacity that it doesn't need reinforcing. If both are reinforced the low variability of strength may cause failure to ground of the entire structure if one pole fails.
- Larger poles (>14m length) and higher importance lines would benefit from having two splints per pole, not just for extra strength that may be required, but also to increase the likelihood that the reinforcement can catch the failed pole and prevent the failure from causing harm or damage.
- In-line stays are not recommended to be used on intermediate structures. If they are required, they should be placed at an angle of 30-45° to the line to assist with cross-line capacity and prevent them exacerbating the pole failure.

In terms of the Hillston line, we conclude that;

- 27 poles have a suitable level of reinforcement for the actual design loads and calculated reinforcement capacity, assuming that the reinforcement must deliver a FOS of 2 over the working stress design loads.
- The remaining 25 poles with available data have suitable reinforcement capacity based on the suppliers published strength values.
- Hence, even though the nominated capacities may be inadequate, the sizing calculations and practicalities of available stock, etc. mean that over 50% of the poles were adequately sized to the elastic member capacity calculations.
- Given that the residual FOS for all the reinforced poles in the Hillston line is greater than 2.8 using the calculated loads (2.28 if the Essential Energy assessment is correct), failure of the poles is considered unlikely, mainly due to the timber being strong enough without any reinforcement.
- Hence, the risk of failure is similar to that of an adequately reinforced pole, but the risk of catastrophic failure may reduce as the reinforcement may be able to stop the poles from falling to the ground or becoming an immediate safety issue.
- If desired, additional splints could be used to give the required capacity (i.e. double splint poles).
- If the criteria in Section 4.2 is fulfilled, the life expectancy of the reinforced poles on the Hillston line is likely to be in the order of 10 years for the under-strength reinforcements and 20+ years for the reinforced poles that have adequate reinforcement strength.

6 RECOMMENDATIONS

Based on the results of this study, it is recommended that;

- The asset inspection manual be revised to give clear guidance for inspection requirements at the start of the document, and then provide supplementary information, descriptions, pictures, etc. towards the end or in appendices.
- The inspection manual should also either reference other documents that describe the residual factor of safety calculations that are done on the poles, or include the calculations in the inspection manual in an appendix.
- The inspection manual should also provide guidance on the detection of defects above the reinforcement zone that could cause failure above the reinforcement. If a pole has such defects it should be replaced rather than reinforced.
- Essential Energy should review the minimum wall thickness and FOS requirements to better separate them from replacement criteria. Consideration should also be given to using a Serviceability Index instead of FOS to better align with limit states design practices.
- There needs to be more definite limits around when a pole can/should be reinforced. Guidance is given in the report, but more work is required to define exact rules. This will need consideration by Essential Energy as to what they require from the reinforcement, such as full strength reinstatement, life extension at a similar reliability, a temporary risk reduction method where it just needs to stop the pole from hitting the ground if it fails, or a combination of these.
- During the next round of inspections, reinforced poles should be checked against the reinforcement capacities noted in Table 2, with any poles that do not have enough capacity replaced within 2 years.
- Any existing reinforcements should be replaced after 10 years in service due to their risk of failure.
- New reinforcements should be sized as per the member capacities in Table 2 unless testing proves otherwise.
- The supplier should be asked to provide data that shows the banding performance against;
  - Fatigue loads,
  - Heating and cooling cycles and its ability to stay in position,
  - The tension loads that would be imparted during a full design load (limit states preferred).
- The design software/process used to select the reinforcement should be updated to the correct strength values.
- Reinforcement that does not have its strong axis in-line with the worst-case load should be checked for capacity including the influence of orientation on strength. Or the pole should be replaced.
- Corroded reinforcement that has a Meuller rating of less than 80% be replaced (refer to Table 3).
- If new reinforcements are sized correctly as noted above, the reinforcement should remain in service for no more than 20 years, unless further studies show that this can be increased.
- Access to the pole in wet weather and orientation of the reinforcement in relation to the load direction need to be considered when selecting poles that can be reinforced, and the reinforcement type to be used.
- If the reinforcement is correctly sized to consider the weight and potential forces from maintenance line crews, there is no reason why they would not be safe enough to climb. However, existing installations would still be questionable.
- Essential Energy personnel should inspect the Hillston line to ensure the correct orientation of the reinforcement, and to verify the size of the reinforcements used on each pole.

- If there are any significantly under-strength reinforcements (unfactored capacity is less than 90% of the design loads), or both poles of the structure are reinforced, they should be earmarked for replacement within 1 year.

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APPENDIX A HILLSTON FEEDER RESULTS SUMMARY

Asset ID	Asset Label	Pole Install Date	Critical Zone Diameter	Above Ground Diameter	Wall Thickness	EE Safety Factor	Pole No	Blank	WSD Ground Line Moment (kNm)	Max GL Actual FOS	Pole Condition	Suitable Reinforcement Size Based on Supplier Nominated Capacity	Actual Reinforcement installed	Required Size Based on URIE Calculated Member Capacity
3714597	CE202665	01-Jan-70	330	330	20	2.3	653		55.0	2.2	Condemned	220B	None	300B
3718014	CE202809	01-Jan-81	350	350	20	2.31	582		38.7	2.9	Condemned	220B	None	250B
3708401	CE202353	01-Jan-90	380	380	20	2.49	807		69.7	2.4	Condemned	250B	None	360B
3714580	CE202648	01-Jan-80	380	380	20	2.54	662		70.0	2.3	Condemned	250B	None	360B
3714737	CE202680	01-Jan-70	350	350	20	2.63	645		55.6	2.5	Condemned	220B	None	300B
3711273	CE202608	25-Dec-81	400	400	20	2.66	681		69.2	2.7	Condemned	250B	None	360B
3720585	CE202553	01-Jan-75	350	350	20	3.02	708		59.2	2.3	Condemned	250B	None	300B
3720555	CE202505	01-Jan-79	370	370	20	3.14	732		63.5	2.4	Condemned	250B	None	300B
3708027	CE202307	01-Jan-01	350	350	20	3.49	830		56.6	2.4	Condemned	250B	None	300B
3932358	2202500		380	380	25	1.51			41.9	4.7	Condemned	300B	#N/A	250B
3720593	CE202561	01-Jan-01	350	350	25	2.85	704		72.7	1.9	Condemned	300B	None	360B
3709098	CE202378	01-Jan-79	360	360	25	3.04	795		71.3	2.0	Condemned	250B	None	360B
3707334	CE202285	01-Jan-81	370	370	30	2.03	841		57.0	3.8	Condemned	250B	None	300B
3710366	CE202473	01-Jan-79	370	370	30	2.03	748		63.6	3.4	Condemned	250B	None	300B
3717997	CE202793	25-Dec-79	370	370	30	2.03	590		47.2	4.5	Condemned	300	None	300
3932359	2022501		370	370	30	2.03	deleted asset		41.8	5.1	Condemned	300B	#N/A	250B
3720594	CE202562	01-Jan-81	350	350	30	2.11	703		73.6	2.1	Condemned	300B	None	360B
3714598	CE202666	01-Jan-70	320	320	30	2.26	653		55.0	2.8	Condemned	220B	None	300B
3707327	CE202278	01-Jan-01	390	390	30	3.87	845		71.2	2.8	Condemned	250B	None	360B
3707325	CE202276	01-Jan-79	390	390	30	4.03	846		66.0	3.0	Condemned	250B	None	360B
3708369	CE202322	01-Jan-01	380	380	30	4.59	823		78.0	2.9	Condemned	300B	None	360B
3717017	CE202708	01-Jan-01	420	420	40	2.28	632		46.2	7.6	Condemned	300	#N/A	300
3706227	CE80278	25-Dec-82	400	400	40	2.36	75		74.8	3.5	Condemned	300B	360B	360B
3718961	CE202496	25-Dec-79	400	400	40	2.36	736		55.3	5.7	Condemned	220B	#N/A	300B
3710390	CE202577	01-Jan-79	400	400	40	2.36	697		58.7	5.4	Condemned	250B	300B	300B
3717019	CE202710	01-Jan-01	400	400	40	2.36	633		46.8	6.7	Condemned	300	300B	300
3707272	CE202111	25-Dec-82	380	380	40	2.45	129		50.6	5.5	Condemned	300	300B	300B
3708760	CE206025	25-Dec-81	370	370	40	2.49	210		78.0	3.4	Condemned	300B	#N/A	360B
3720584	CE202552	01-Jan-75	370	370	40	2.49	708		57.3	4.6	Condemned	250B	Replaced	300B
3720592	CE202560	25-Dec-81	370	370	40	2.49	704		72.8	3.0	Condemned	300B	300B	360B
3706206	CE110889	25-Dec-82	360	360	40	2.54	64		73.9	2.8	Condemned	300B	300B	360B
3707686	CE202133	25-Dec-82	360	360	40	2.54	140		57.3	4.3	Condemned	250B	300B	300B
3710431	CE206254	25-Dec-81	360	360	40	2.54	337		36.0	5.6	Condemned	250	220B	250B
3708038	CE202318	01-Jan-82	360	360	40	2.54	825		56.2	4.4	Condemned	250B	300B	300B
3707718	CE202165	25-Dec-82	350	350	40	2.58	156		57.2	4.0	Condemned	250B	300B	300B
3708476	CE202233	25-Dec-80	350	350	40	2.58	189		57.2	4.0	Condemned	250B	300B	300B
3710115	CE206252	25-Dec-81	350	350	40	2.58	336		36.0	5.3	Condemned	250	250B	250B
3720556	CE202506	01-Jan-79	350	350	40	2.58	731		69.3	3.3	Condemned	250B	300B	360B
3720563	CE202513	01-Jan-79	350	350	40	2.58	728		70.1	3.3	Condemned	250B	Replaced	360B
3717924	CE202722	25-Dec-71	350	350	40	2.58	625		57.9	4.0	Condemned	250B	300B	300B
3709505	CE206170	25-Dec-81	340	340	40	2.63	296		51.7	3.4	Condemned	300	250B	300B
3712097	CE206536	25-Dec-81	330	330	40	2.68	478		35.9	4.6	Condemned	250	250B	250B
3720573	CE202541	01-Jan-81	400	400	45	2.56	714		68.1	5.0	Condemned	250B	300B	360B

Asset ID	Asset Label	Pole Install Date	Critical Zone Diameter	Above Ground Diameter	Wall Thickness	EE Safety Factor	Pole No	Blank	WSD Ground Line Moment (kNm)	Max GL Actual FOS	Pole Condition	Suitable Reinforcement Size Based on Supplier Nominated Capacity	Actual Reinforcement installed	Required Size Based on URIE Calculated Member Capacity
3718962	CE202497	25-Dec-79	380	380	45	2.64	736		55.2	5.5	Condemned	220B	#N/A	300B
3710365	CE202472	01-Jan-79	360	360	45	2.73	748		63.6	4.2	Condemned	250B	300B	300B
3708428	CE202185	25-Dec-75	440	440	50	2.57	166		65.4	7.0	Condemned	250B	300B	300B
3709512	CE206177	25-Dec-81	420	420	50	2.65	299		52.1	6.5	Condemned	300	300B	300B
3714603	CE202671	01-Jan-82	420	420	50	2.65	649		94.1	4.4	Condemned	300B	300B	360B
3708427	CE202184	25-Dec-82	390	390	50	2.78	165		72.7	4.7	Condemned	300B	300B	360B
3710478	CE206301	25-Dec-80	380	380	50	2.82	361		36.1	7.4	Condemned	250	220B	250B
3714579	CE202647	01-Jan-81	380	380	50	2.82	662		70.0	4.6	Condemned	250B	300B	360B
3708459	CE202216	25-Dec-80	370	370	50	2.87	181		57.3	5.3	Condemned	250B	300B	300B
3713725	CE202629	01-Jan-81	370	370	50	2.87	671		77.4	3.9	Condemned	300B	300B	360B
3714742	CE202685	01-Jan-70	370	370	50	2.87	643		55.4	5.5	Condemned	220B	300B	300B
3717985	CE202782	01-Jan-80	370	370	50	2.87	595		39.5	7.7	Condemned	250	Replaced	250B
3708755	CE206020	25-Dec-80	360	360	50	2.91	207		57.3	4.9	Condemned	250B	300B	300B
3710397	CE202584	25-Dec-81	360	360	50	2.91	693		67.3	4.2	Condemned	250B	300B	360B
3708449	CE202206	25-Dec-80	350	350	50	2.96	176		57.2	4.6	Condemned	250B	300B	300B
3711218	CE206407	25-Dec-81	350	350	50	2.96	414		44.8	4.9	Condemned	300	220B	300
3709085	CE202365	01-Jan-79	350	350	50	2.96	802		71.6	3.7	Condemned	250B	300B	360B
3710021	CE202451	01-Jan-79	350	350	50	2.96	759		69.0	3.8	Condemned	250B	300B	360B
3720588	CE202556	01-Jan-01	350	350	50	2.96	706		54.8	4.8	Condemned	300	300B	300B
3718048	CE202841	01-Jan-81	350	350	50	2.96	566		39.0	6.8	Condemned	250	250B	250B
3706073	CE110872	25-Dec-82	340	340	50	3.01	57		70.9	3.5	Condemned	250B	300B	360B
3710102	CE206239	25-Dec-80	340	340	50	3.01	330		36.6	5.5	Condemned	250	220B	250B
3711687	CE206451	25-Dec-81	340	340	50	3.01	435		35.9	6.9	Condemned	250	220B	250B
3711699	CE206463	25-Dec-81	340	340	50	3.01	441		35.9	5.7	Condemned	250	#N/A	250B
3712076	CE206515	25-Dec-81	340	340	50	3.01	467		35.9	5.7	Condemned	250	300B	250B
3712082	CE206521	25-Dec-81	340	340	50	3.01	470		35.9	5.7	Condemned	250	300B	250B
3712100	CE206539	25-Dec-81	340	340	50	3.01	479		35.9	5.7	Condemned	250	220B	250B
3707673	CE202120	25-Dec-82	330	330	50	3.06	134		57.1	4.7	Condemned	250B	300B	300B
3707685	CE202132	25-Dec-82	330	330	50	3.06	140		57.1	4.0	Condemned	250B	250B	300B
3712474	CE206597	25-Dec-81	330	330	50	3.06	507		35.2	5.4	Condemned	250	220B	250B
3718047	CE202840	01-Jan-81	330	330	50	3.06	566		38.9	4.9	Condemned	250	220B	250B
3711698	CE206462	25-Dec-81	320	320	50	3.11	441		35.8	4.9	Condemned	250	#N/A	250B
3711249	CE206438	25-Dec-81	310	310	50	3.16	429		35.8	4.5	Condemned	250	220B	250B
3713024	CE206692	25-Dec-80	390	390	60	3.08	554		36.1	10.5	Condemned	250	220B	250B
3709495	CE206160	25-Dec-81	380	380	60	3.12	291		51.9	5.7	Condemned	300	300B	300B
3711225	CE206414	25-Dec-80	370	370	60	3.17	417		36.1	10.9	Condemned	250	250B	250B
3711682	CE206446	25-Dec-80	370	370	60	3.17	433		36.1	7.6	Condemned	250	220B	250B
3708426	CE202183	25-Dec-82	330	330	60	3.34	165		72.4	3.5	Condemned	300B	300B	360B
3712477	CE206600	25-Dec-81	330	330	60	3.34	509		36.8	5.6	Limited Life	250	250B	250B

Asset ID	Asset Label	Pole Install Date	Critical Zone Diameter	Above Ground Diameter	Wall Thickness	EE Safety Factor	Pole No	Blank	WSD Ground Line Moment (kNm)	Max GL Actual FOS	Pole Condition	Suitable Reinforcement Size Based on Supplier Nominated Capacity	Actual Reinforcement installed	Required Size Based on URIE Calculated Member Capacity
3710019	CE202449	02-Apr-09	400	400	30	1.5	#N/A		#N/A	#N/A	Serviceable - As New			
3717967	CE202764	08-Apr-09	350	350	20	1.6	#N/A		#N/A	#N/A	Serviceable - As New			
3711266	CE202601	02-Apr-09	380	380	40	1.86	#N/A		#N/A	#N/A	Serviceable - As New			
3711283	CE202618	02-Apr-09	370	370	30	2.03	#N/A		#N/A	#N/A	Serviceable - As New			
3709740	CE202409	02-Apr-09	400	400	40	2.36	#N/A		#N/A	#N/A	Serviceable - As New			
3711274	CE202609	02-Apr-09	390	390	40	2.4	#N/A		#N/A	#N/A	Serviceable - As New			
3709122	CE202401	02-Apr-09	380	380	40	2.45	#N/A		#N/A	#N/A	Serviceable - As New			
3710010	CE202440	02-Apr-09	380	380	40	2.45	#N/A		#N/A	#N/A	Serviceable - As New			
3711265	CE202600	02-Apr-09	380	380	40	2.45	#N/A		#N/A	#N/A	Serviceable - As New			
3707335	CE202286	02-Apr-09	370	370	40	2.49	#N/A		#N/A	#N/A	Serviceable - As New			
3710020	CE202450	02-Apr-09	370	370	40	2.49	#N/A		#N/A	#N/A	Serviceable - As New			
3714572	CE202640	02-Apr-09	370	370	40	2.49	#N/A		#N/A	#N/A	Serviceable - As New			
3708408	CE202360	02-Apr-09	410	410	45	2.52	#N/A		#N/A	#N/A	Serviceable - As New			
3713718	CE202622	03-May-09	400	400	45	2.56	#N/A		#N/A	#N/A	Serviceable - As New			
3714578	CE202646	02-Apr-09	350	350	40	2.58	#N/A		#N/A	#N/A	Serviceable - As New			
3709736	CE202405	02-Apr-09	340	340	40	2.63	#N/A		#N/A	#N/A	Serviceable - As New			
3709106	CE202385	02-Apr-09	370	370	45	2.69	#N/A		#N/A	#N/A	Serviceable - As New			
3708022	CE2O2302	01-Jan-82	390	390	50	2.78	#N/A		#N/A	#N/A	Serviceable - Fair			
3717957	CE202755	08-Apr-09	310	310	40	2.79	#N/A		#N/A	#N/A	Serviceable - As New			
3706873	CE202012	02-Apr-09	370	370	50	2.87	#N/A		#N/A	#N/A	Serviceable - Good			

APPENDIX B SURVEY RESULTS

Your job title
Your location

URI ENGINEERING — UTILITY • RELIABILITY • INNOVATION

Pole Reinforcement Performance Survey

Essential Energy is evaluating the performance of its pole reinforcement systems to consider the most efficient reinforcement/replacement strategy to maximise reliability of the network.
Aside the data that we will utilise from the EE asset database, we wish to also utilise the knowledge and experience of field staff and managers to complete the picture.
Please answer as many of the questions as possible. Please be honest and use actual data where possible, however gut feel is OK if no hard data is available. Please feel free to have this survey completed by members of your group who you think would know the most about reinforced pole performance in your area.
Please return the completed survey to Design Standards (design.construction.standards@essentialenergy.com.au) by 6 December 2013.

1. How many poles have been reinforced in your area each year over the last ten years, on average?
Confidence level? None (could be 100%+ out)

<50
50-99
100-149
150-199

2. How many years on average do the poles remain in service after reinforcement?
Confidence level? None (could be 100%+ out)

0-5
5-10
10-15
15-20

3. What do you think is the maximum time (years) a reinforced pole has been in service after reinforcement without failure.
Confidence level? None (could be 100%+ out)

0-5
5-10
10-15
15-20

4. Is there a fixed limit on the time a reinforced pole is allowed to remain in service? Written or otherwise. Enter the time period below the yes/no selection
Confidence level? None (could be 100%+ out)

Yes
No

years

5. How many reinforced poles experience failures in your area on average per year?
Confidence level? None (could be 100%+ out)

0-5
5-10
10-15
15-20

6. If you have had reinforced poles fail in your area in the past, what percentage fail at;

Timber only at ground line
Timber and reinforcement buckling/twisting/bending at ground line
Timber at top of the reinforcement
Elsewhere along the pole (i.e. more towards the pole tip, cross-arm, etc.)
Failure of the connection between pole and reinforcement (i.e. loose bands)
Other (please elaborate in the comments section)

Confidence level? None (could be 100%+ out) Total **0%**

7. Please use ctrl+click to select **all** the reinforcement types you are aware have been used in your area in the past.
Confidence level? None (could be 100%+ out)

C-Splint
Powerbeam
UAM / RFD
Multi Rib

8. Please describe any particular concerns you have with the current inspection criteria for reinforcing

9. Please comment on the general performance of the different pole reinforcement systems in your network from a reliability perspective? Are you happy? Feel free to suggest potential for improvement.

10. Are there any other comments that you think are important regarding the installation, inspection, removal, reliability of the reinforcement?

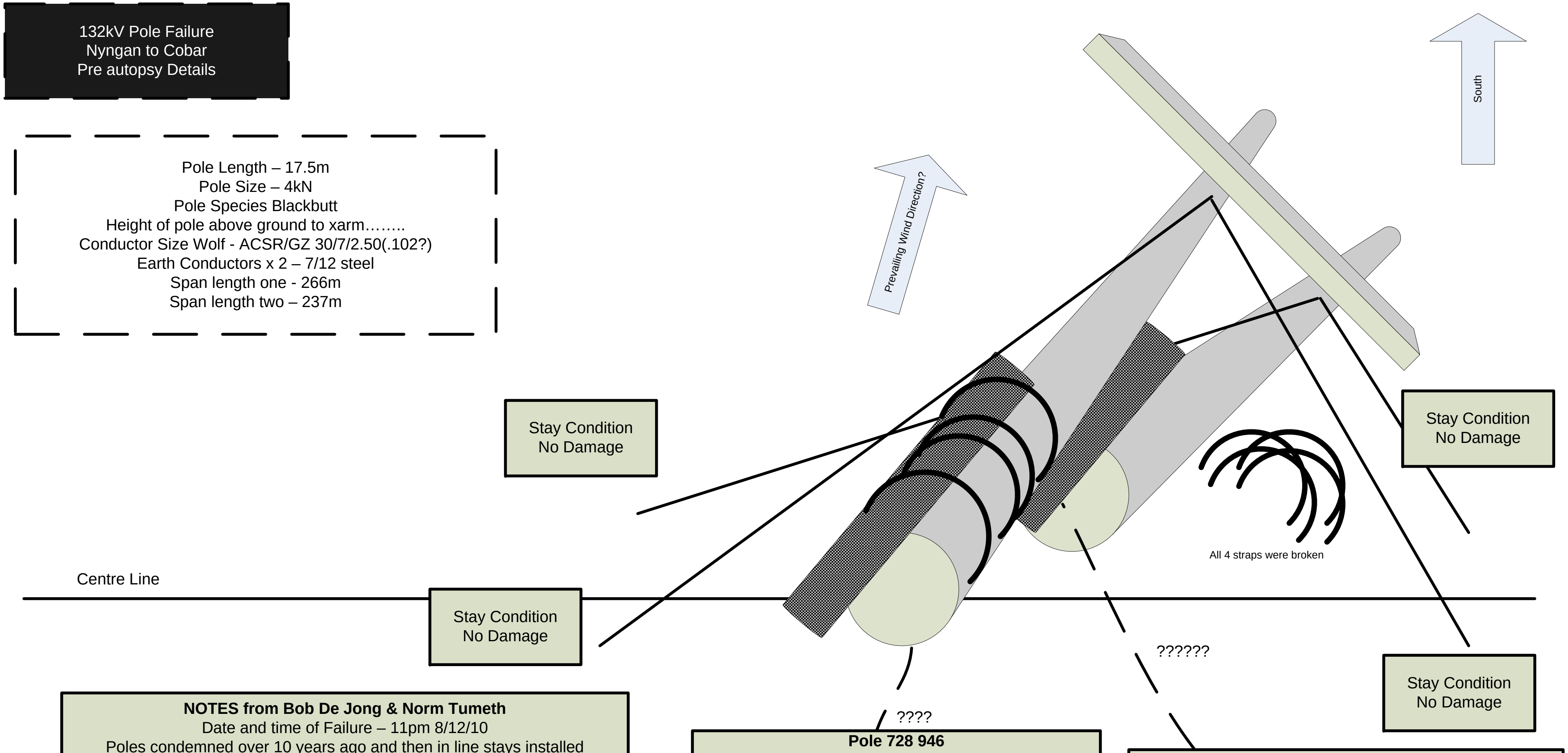
Title	Location	Q1. How many poles have been reinforced in your area each year over the last ten years, on average?	Q1 Confidence	Q2 How many years on average do the poles remain in service after reinforcement?	Q2 Confidence	Q3 What do you think is the maximum time (years) a reinforced pole has been in service after reinforcement without failure.	Q3 Confidence	Q4 Is there a fixed limit on the time a reinforced pole is allowed to remain in service? Written or otherwise. Enter the time period below the yes/no selection	Q4 Time (Years)	Q4 Confidence	Q5 How many reinforced poles experience failures in your area on average per year?	Q5 Confidence
Area Manager	Riverina	50-99	High	20+	None	10-15	Medium	No		None	0-5	High
Area Manager	Central Tablelands	<50	High	0-5	Medium	10-15	None	No		None	0-5	High
Area Manager	Western Slopes and Plains	50-99		5-10		5-10		No			0-5	
Area Manager	Buronga (FW Southern)	<50	None	10-15	None	0-5	None	No		None	0-5	None
Area Manager	Gunnedah	100-149	High	5-10	High	10-15	Medium	No		None	0-5	High
Resource Supervisor	Northern	<50	Medium	0-5	Medium	5-10	High	Yes	8	High	0-5	High
Acting Area Manager	Cooma	<50	Medium	10-15	Medium	15-20	Medium	No		None	0-5	High
Area Manager	Broken Hill	100-149	None	0-5	From Data	0-5	None	No	Nil	None	0-5	None

Title	Location	Q6 If you have had reinforced poles fail in your area in the past, what percentage fail at;						Q6 Confidence	Q7 Please use ctrl+click to select all the reinforcement types you are aware have been used in your area in the past.	Q7 Confidence	Q8 Please describe any particular concerns you have with the current inspection criteria for reinforcing	Q9 Please comment on the general performance of the different pole reinforcement systems in your network from a reliability perspective? Are you happy? Feel free to suggest potential for improvement.	Q10 Are there any other comments that you think are important regarding the installation, inspection, removal, reliability of the reinforcement?
		Timber only at ground line	Timber and reinforcement buckling/twisting/bending at ground line	Timber at top of the reinforcement	Elsewhere along the pole (i.e. more towards the pole tip, cross-arm, etc.)	Failure of the connection between pole and reinforcement (i.e. loose bands)	Other (please elaborate in the comments section)						
Area Manager	Riverina							None	C-Splint	None	There are poles being nailed without all weather access from an EWP. If there is a X-arm failure in wet weather then we cannot replace the are. Some of these poles are Sub transmission	Happy with poles being nailed if we can access them in all weather conditions.	
Area Manager	Central Tablelands							None	C-Splint	Medium		They seem to perform well in my area	The time frame from when they are asset as requiring reinforcement to when they are actually nailed can be too long
Area Manager	Western Slopes and Plains								C-Splint		the only concern is why are we nailing switching pole as we can not climb them if wet we can not get to them with and EWP to do switching of replace fuse etc . Plus not sure if this is the best way to go for cost saving	we only use the C-Splint type	When poles are put down to be nailed it take way to long to get them before they are nailed most of the time It goes beyond the 6 month backlog and this could cause a fire hazard if some thing went wrong When we all have to meet the 6 month backlog target
Area Manager	Buronga (FW Southern)							None	C-Splint	None	FW Southern Area traditionally hasn't reinstated poles because it hasn't been viable. Poles are now being identified to be reinstated but unless there are more then ten per maintenance area then we just replace them.		
Area Manager	Gunnedah							None	C-Splint	High	Expense of visiting the asset twice when we mostly only get an extra inspection cycle anyhow. Does the extra expense and associated costs warrant nailing. Also creates extra issues as you must always have an EWP to access a re-inforced pole creating additional servicability expense and hassle.	c-splints seem to be quite effective, however still have theissues as above.	Replace rather than reinforce.
Resource Supervisor	Northern	100%						None	C-Splint	Medium	If a pole is bad enough to be nailed it is bad enough to be replaced. I have issues with the safety and environmental factors involved with nailing a pole we are aware of being not up to standards	Nailed poles have a tendancy to move in the ground and lean. They also aren't able to be climbed thus when a defect or fault occurs on these poles, it is necessary to have EWP access. This often isn't the case, as EWP access isn't always available.	Please consider disbanding the nailing program. It isn't cost effective, as we are replacing the nailed poles on the next inspection cycle, it's a safety concern and it makes defect and fault repair difficult.
Acting Area Manager	Cooma							None	C-Splint	Medium			it is preffered that no poles be reinforced in the snowy mountains area , we have had issues in the past with gaining access to these poles in snow conditions with our bucket trucks . As a rule staff do not climb reinforced poles so we prefer to replace the poles instead to allow staff to access in light vehicles and conduct their work off ladders.
Area Manager	Broken Hill	NA	NA	NA	NA	NA	NA	None	C-Splint	None			

APPENDIX C NYNGAN – COBAR REINFORCEMENT FAILURE REPORT

132kV Pole Failure
Nyngan to Cobar
Pre autopsy Details

Pole Length – 17.5m
Pole Size – 4kN
Pole Species Blackbutt
Height of pole above ground to xarm……..
Conductor Size Wolf - ACSR/GZ 30/7/2.50(.102?)
Earth Conductors x 2 – 7/12 steel
Span length one - 266m
Span length two – 237m



NOTES from Bob De Jong & Norm Tumeth
Date and time of Failure – 11pm 8/12/10
Poles condemned over 10 years ago and then in line stays installed
4 years later they were condemned and splinted
1 pole broke 18 inches below ground pole number ……, the other at 12 inches…….
1 pole hollow about 4m from GL pole number…….
Both splints were 1900 below GL

Weather Conditions at time of Failure
Caravan owner approx 400m from this site at a camping ground thought their van was going to tip over
Some branches on the road, nothing major

Site
This structure is in the middle of around 6 others on a flat plain between 2 hills, surrounding ground was boggy but the pole site was firm, approx 600 mm loose soil in the pole holes,
Was able to extract 1 pole butt however the other took 2 hours to bore out

All items transported to Cobar FSC for autopsy

Pole 728 946
At time of inspection 3/12/10
Critical Diameter - 340mm
Above Ground Diameter - 340
Wall thickness at inspection hole – 40mm
Construction - 132kV 3 Wire Suspension
2 x GY2 in line

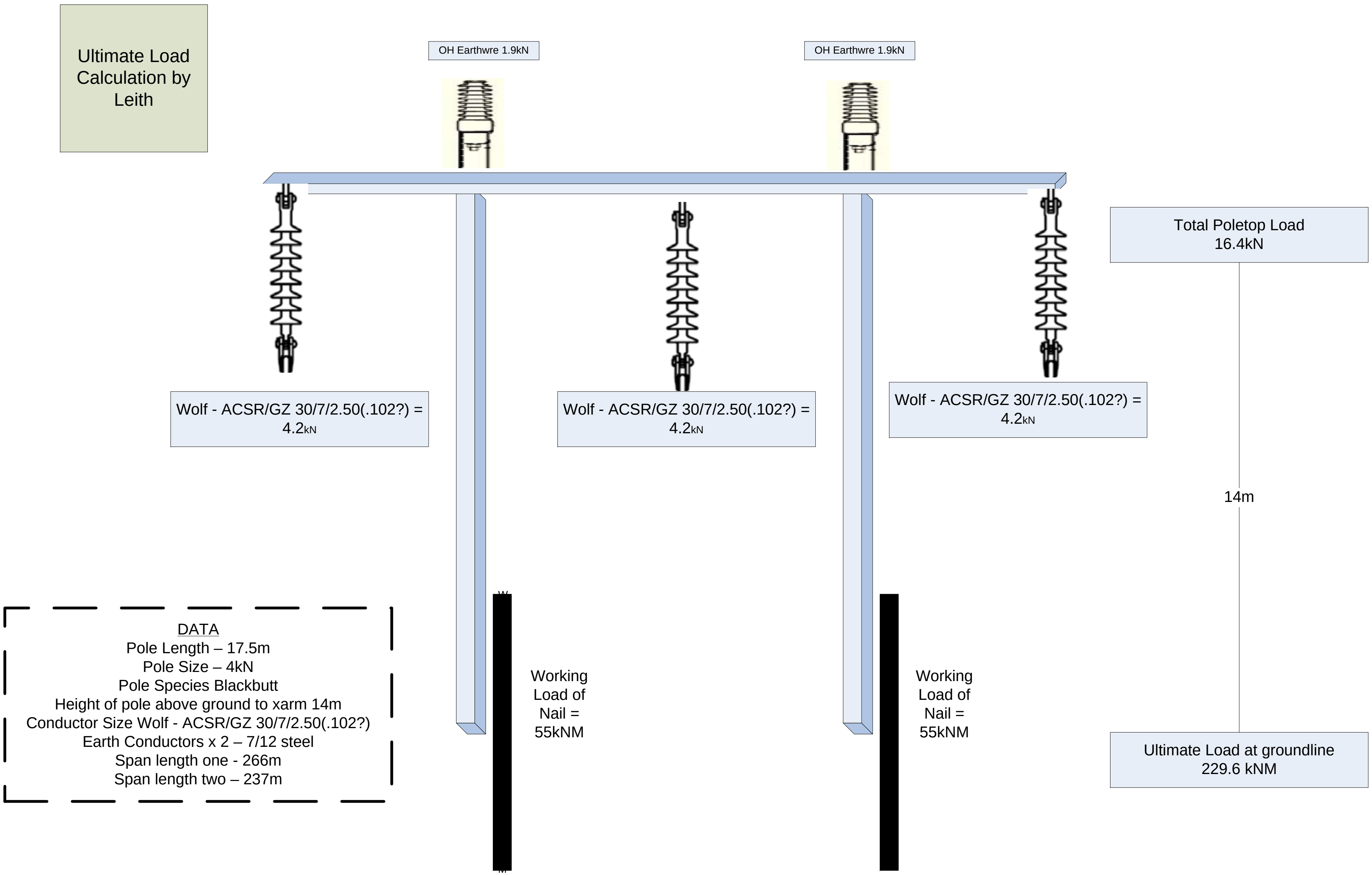
Splint
C Splint - 300 61kNM – de-rated to 55kNM
Date of installation August 2006 (8-6)
Bent and slight twist

Bands
4 x double wrapped
All intact at time of failure

Pole 729 946
At time of inspection 3/12/10
Critical Diameter - 380mm
Above Ground Diameter - 380 mm
Wall thickness at inspection hole – 50mm
Construction - 132kV 3 Wire Suspension
2 x GY2 in line

Splint
C Splint - 300 61kN– de-rated to 55kNM
Date of installation August 2006 (8-6)
Bent, twisted and deformed upon failure?

Bands
4 x double wrapped
All 4 bands failed



Ultimate Load Calculation by Leith
OH Earthwre 1.9kN
OH Earthwre 1.9kN
Total Poletop Load 16.4kN
Wolf - ACSR/GZ 30/7/2.50(.102?) = 4.2kN
Wolf - ACSR/GZ 30/7/2.50(.102?) = 4.2kN
Wolf - ACSR/GZ 30/7/2.50(.102?) = 4.2kN
14m
DATA
Pole Length – 17.5m
Pole Size – 4kN
Pole Species Blackbutt
Height of pole above ground to xarm 14m
Conductor Size Wolf - ACSR/GZ 30/7/2.50(.102?)
Earth Conductors x 2 – 7/12 steel
Span length one - 266m
Span length two – 237m
Working Load of Nail = 55kNM
Working Load of Nail = 55kNM
Ultimate Load at groundline 229.6 kNM



APPENDIX D OZ-C-SPLINT CALCULATED CAPACITIES

Note that the following capacity calculations are based on a segment length of 1300mm for all. It was not considered necessary to change the value for all reinforcement types for the sake of this exercise, as it did not make a significant difference.

The section properties used in the analysis for each splint size were found by drawing the section in DraftSight V1R4 (using the dimensions from drawing A3-1160 revC), saving the drawing as a dxf, and importing into Shapebuilder v7.0, which then determines the section properties automatically.

The calculated capacities also ignore the capacity of the banding. It is recommended that Essential Energy ask the supplier to provide verification that the banding is sufficient for the design loads.

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 135

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	3.8E+6 mm^4	Z_x (Strong Axis)	50.5E+3 mm^3	S_x (Strong Axis)	65.9E+3 mm^3		
I_y (Weak Axis)	716.9E+3 mm^4	Z_y (Weak Axis)	15.1E+3 mm^3	S_y (Weak Axis)	27.0E+3 mm^3		
r_x	52.091 mm	J	20.5E+3 mm^4	f_y	380 MPa	G	80000 MPa
r_y	22.594 mm	I_w	615.2E+6 mm^6	E	200000 MPa	t	6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 36 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25

$\lambda_s=\lambda_e=$ 7.5

∴ Section is **Compact**

2.2 Effective Section Modulus, Z_e

Z_{ex} = 65.9E+3 mm^3

Z_{ey} = 22.6E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 25.1 kNm

M_{sy} 8.6 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Strong Axis		Weak Axis	
Segment length (Strong Axis)	1300 mm	Segment length (Weak Axis)	1300 mm
Segment restraints (Strong Axis)	PU	Segment restraints (Weak Axis)	PU
k_{tx}	1.01	k_{ty}	1.01

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2	k_{ly}	2
Ends with lateral rotation restraints	One	Ends with lateral rotation restraints	One
k_{rx}	0.85	k_{ry}	0.85
Effective Length, l_{ex}	2239 mm	Effective Length, l_{ey}	2239 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1	β_{my}	-1
Length for FLR (Strong Axis)	367 mm	Length for FLR (Weak Axis)	845 mm
Therefore:	Not FLR	Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25		α_{my}	1.25	
M_{oax}	23.1 kNm	Eqn. 5.6.1.1(3)	M_{oay}	19.1 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.57		α_{sy}	0.80	
M_{bx}	18.0 kNm		M_{by}	8.6 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 18.0 kNm

Bending capacity in the weak axis **(Unfactored)** 8.6 kNm

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 170

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	7.2E+6	mm^4	Z_x (Strong Axis)	77.7E+3	mm^3	S_x (Strong Axis)	100.7E+3	mm^3
I_y (Weak Axis)	1.4E+6	mm^4	Z_y (Weak Axis)	23.1E+3	mm^3	S_y (Weak Axis)	41.3E+3	mm^3
r_x	64.336	mm	J	25.3E+3	mm^4	f_y	380	MPa
r_y	27.91	mm	I_w	1.8E+9	mm^6	E	200000	MPa

G 80000 MPa

t 6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 45 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 9.3

2.2 Effective Section Modulus, Z_e

Z_{ex} = 100.2E+3 mm^3

Z_{ey} = 34.4E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 38.1 kNm

M_{sy} 13.1 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Segment length (Strong Axis)	1300 mm		Segment length (Weak Axis)	1300 mm
Segment restraints (Strong Axis)	PU		Segment restraints (Weak Axis)	PU
k_{tx}	1.02		k_{ty}	1.02

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2		k_{ly}	2
Ends with lateral rotation restraints	One		Ends with lateral rotation restraints	One
k_{rx}	0.85		k_{ry}	0.85
Effective Length, l_{ex}	2246 mm		Effective Length, l_{ey}	2246 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1		β_{my}	-1
Length for FLR (Strong Axis)	453 mm		Length for FLR (Weak Axis)	1044 mm
Therefore:	Not FLR		Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25			α_{my}	1.25	
M_{oax}	37.9 kNm	Eqn. 5.6.1.1(3)		M_{oay}	44.0 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.60			α_{sy}	0.88	
M_{bx}	28.5 kNm			M_{by}	13.1 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 28.5 kNm

Bending capacity in the weak axis **(Unfactored)** 13.1 kNm

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 200

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	11.5E+6 mm^4	Z_x (Strong Axis)	107.1E+3 mm^3	S_x (Strong Axis)	138.1E+3 mm^3		
I_y (Weak Axis)	2.2E+6 mm^4	Z_y (Weak Axis)	31.7E+3 mm^3	S_y (Weak Axis)	56.7E+3 mm^3		
r_x	75.346 mm	J	30.6E+3 mm^4	f_y	380 MPa	G	80000 MPa
r_y	32.691 mm	I_w	3.9E+9 mm^6	E	200000 MPa	t	6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 53 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 10.9

2.2 Effective Section Modulus, Z_e

Z_{ex} = 134.4E+3 mm^3

Z_{ey} = 45.6E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 51.1 kNm

M_{sy} 17.3 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Segment length (Strong Axis)	1300 mm	Segment length (Weak Axis)	1300 mm
Segment restraints (Strong Axis)	PU	Segment restraints (Weak Axis)	PU
k_{tx}	1.02	k_{ty}	1.02

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2	k_{ly}	2
Ends with lateral rotation restraints	One	Ends with lateral rotation restraints	One
k_{rx}	0.85	k_{ry}	0.85
Effective Length, l_{ex}	2253 mm	Effective Length, l_{ey}	2253 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1	β_{my}	-1
Length for FLR (Strong Axis)	530 mm	Length for FLR (Weak Axis)	1222 mm
Therefore:	Not FLR	Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25		α_{my}	1.25	
M_{oax}	57.8 kNm	Eqn. 5.6.1.1(3)	M_{oay}	82.2 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.64		α_{sy}	0.92	
M_{bx}	40.6 kNm		M_{by}	17.3 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 40.6 kNm

Bending capacity in the weak axis **(Unfactored)** 17.3 kNm

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OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 220

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	14.9E+6 mm^4	Z_x (Strong Axis)	127.4E+3 mm^3	S_x (Strong Axis)	164.0E+3 mm^3
I_y (Weak Axis)	2.8E+6 mm^4	Z_y (Weak Axis)	37.6E+3 mm^3	S_y (Weak Axis)	67.3E+3 mm^3
r_x	82.089 mm	J	34.5E+3 mm^4	f_y	380 MPa
r_y	35.62 mm	I_w	5.9E+9 mm^6	E	200000 MPa
G	80000 MPa	t	6 mm		

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 58 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25

$\lambda_s=\lambda_e=$ 11.9

∴ Section is **Non-Compact**

2.2 Effective Section Modulus, Z_e

Z_{ex} = 157.3E+3 mm^3

Z_{ey} = 52.9E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 59.8 kNm

M_{sy} 20.1 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Strong Axis			Weak Axis		
Segment length (Strong Axis)	1300	mm	Segment length (Weak Axis)	1300	mm
Segment restraints (Strong Axis)	PU		Segment restraints (Weak Axis)	PU	
k_{tx}	1.02		k_{ty}	1.02	

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2		k_{ly}	2	
Ends with lateral rotation restraints	One		Ends with lateral rotation restraints	One	
k_{rx}	0.85		k_{ry}	0.85	
Effective Length, l_{ex}	2257	mm	Effective Length, l_{ey}	2257	mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1		β_{my}	-1	
Length for FLR (Strong Axis)	578	mm	Length for FLR (Weak Axis)	1332	mm
Therefore:	Not FLR		Therefore:	Not FLR	

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25			α_{my}	1.25		
M_{oax}	74.2	kNm	Eqn. 5.6.1.1(3)	M_{oay}	115.3	kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.66			α_{sy}	0.94		
M_{bx}	49.5	kNm		M_{by}	20.1	kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 49.5 kNm

Bending capacity in the weak axis **(Unfactored)** 20.1 kNm

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OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 220B

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	14.9E+6	mm^4	Z_x (Strong Axis)	127.4E+3	mm^3				S_x (Strong Axis)	164.0E+3	mm^3
I_y (Weak Axis)	2.8E+6	mm^4	Z_y (Weak Axis)	37.6E+3	mm^3				S_y (Weak Axis)	67.3E+3	mm^3
r_x	82.089	mm	J	34.5E+3	mm^4	f_y	690	MPa	G	80000	MPa
r_y	35.62	mm	I_w	5.9E+9	mm^6	E	200000	MPa	t	6	mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 58 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 16.1

2.2 Effective Section Modulus, Z_e

Z_{ex} = 147.8E+3 mm^3

Z_{ey} = 48.1E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 102.0 kNm

M_{sy} 33.2 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Strong Axis			Weak Axis		
Segment length (Strong Axis)	1300	mm	Segment length (Weak Axis)	1300	mm
Segment restraints (Strong Axis)	PU		Segment restraints (Weak Axis)	PU	
k_{tx}	1.02		k_{ty}	1.02	

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2		k_{ly}	2	
Ends with lateral rotation restraints	One		Ends with lateral rotation restraints	One	
k_{rx}	0.85		k_{ry}	0.85	
Effective Length, l_{ex}	2257	mm	Effective Length, l_{ey}	2257	mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1		β_{my}	-1	
Length for FLR (Strong Axis)	429	mm	Length for FLR (Weak Axis)	988	mm
Therefore:	Not FLR		Therefore:	Not FLR	

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25			α_{my}	1.25		
M_{oax}	74.2	kNm	Eqn. 5.6.1.1(3)	M_{oay}	115.3	kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.50			α_{sy}	0.88		
M_{bx}	64.0	kNm		M_{by}	33.2	kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 64.0 kNm

Bending capacity in the weak axis **(Unfactored)** 33.2 kNm

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 250

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	21.8E+6 mm^4	Z_x (Strong Axis)	164.4E+3 mm^3	S_x (Strong Axis)	211.0E+3 mm^3		
I_y (Weak Axis)	4.1E+6 mm^4	Z_y (Weak Axis)	48.4E+3 mm^3	S_y (Weak Axis)	86.6E+3 mm^3		
r_x	93.1 mm	J	37.6E+3 mm^4	f_y	380 MPa	G	80000 MPa
r_y	40.403 mm	I_w	11.1E+9 mm^6	E	200000 MPa	t	6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 66 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 13.6

2.2 Effective Section Modulus, Z_e

Z_{ex} = 197.6E+3 mm^3

Z_{ey} = 65.7E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 75.1 kNm (Unfactored)

M_{sy} 25.0 kNm

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Segment length (Strong Axis)	1300 mm	Segment length (Weak Axis)	1300 mm
Segment restraints (Strong Axis)	PU	Segment restraints (Weak Axis)	PU
k_{tx}	1.02	k_{ty}	1.02

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2	k_{ly}	2
Ends with lateral rotation restraints	One	Ends with lateral rotation restraints	One
k_{rx}	0.85	k_{ry}	0.85
Effective Length, l_{ex}	2263 mm	Effective Length, l_{ey}	2263 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1	β_{my}	-1
Length for FLR (Strong Axis)	655 mm	Length for FLR (Weak Axis)	1510 mm
Therefore:	Not FLR	Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25		α_{my}	1.25	
M_{oax}	107.4 kNm	Eqn. 5.6.1.1(3)	M_{oay}	189.6 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.70		α_{sy}	0.96	
M_{bx}	65.8 kNm		M_{by}	25.0 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 65.8 kNm

Bending capacity in the weak axis **(Unfactored)** 25.0 kNm

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 250B

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	21.8E+6	mm^4	Z_x (Strong Axis)	164.4E+3	mm^3	S_x (Strong Axis)	211.0E+3	mm^3
I_y (Weak Axis)	4.1E+6	mm^4	Z_y (Weak Axis)	48.4E+3	mm^3	S_y (Weak Axis)	86.6E+3	mm^3
r_x	93.1	mm	J	37.6E+3	mm^4	f_y 690 MPa	G	80000 MPa
r_y	40.403	mm	I_w	11.1E+9	mm^6	E 200000 MPa	t	6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 66 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 18.3

2.2 Effective Section Modulus, Z_e

Z_{ex} = 183.9E+3 mm^3

Z_{ey} = 58.5E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 126.9 kNm (Unfactored)

M_{sy} 40.4 kNm

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Segment length (Strong Axis)	1300 mm		Segment length (Weak Axis)	1300 mm	
Segment restraints (Strong Axis)	PU		Segment restraints (Weak Axis)	PU	
k_{tx}	1.02		k_{ty}	1.02	

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2	k_{ly}	2
Ends with lateral rotation restraints	One	Ends with lateral rotation restraints	One
k_{rx}	0.85	k_{ry}	0.85
Effective Length, l_{ex}	2263 mm	Effective Length, l_{ey}	2263 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1	β_{my}	-1
Length for FLR (Strong Axis)	486 mm	Length for FLR (Weak Axis)	1121 mm
Therefore:	Not FLR	Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25		α_{my}	1.25	
M_{oax}	107.4 kNm	Eqn. 5.6.1.1(3)	M_{oay}	189.6 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.55		α_{sy}	0.92	
M_{bx}	87.1 kNm		M_{by}	40.4 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 87.1 kNm

Bending capacity in the weak axis **(Unfactored)** 40.4 kNm

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 300

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	35.7E+6 mm^4		Z_x (Strong Axis)	229.3E+3 mm^3		S_x (Strong Axis)	293.3E+3 mm^3	
I_y (Weak Axis)	6.7E+6 mm^4		Z_y (Weak Axis)	67.3E+3 mm^3		S_y (Weak Axis)	120.3E+3 mm^3	
r_x	109.76 mm	J	44.8E+3 mm^4	f_y	380 MPa	G	80000 MPa	
r_y	47.639 mm	I_w	25.3E+9 mm^6	E	200000 MPa	t	6 mm	

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 78 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 16.1

2.2 Effective Section Modulus, Z_e

Z_{ex} = 265.0E+3 mm^3

Z_{ey} = 86.1E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 100.7 kNm

M_{sy} 32.7 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Strong Axis		Weak Axis	
Segment length (Strong Axis)	1300 mm	Segment length (Weak Axis)	1300 mm
Segment restraints (Strong Axis)	PU	Segment restraints (Weak Axis)	PU
k_{tx}	1.03	k_{ty}	1.03

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2	k_{ly}	2
Ends with lateral rotation restraints	One	Ends with lateral rotation restraints	One
k_{rx}	0.85	k_{ry}	0.85
Effective Length, l_{ex}	2274 mm	Effective Length, l_{ey}	2274 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1	β_{my}	-1
Length for FLR (Strong Axis)	773 mm	Length for FLR (Weak Axis)	1781 mm
Therefore:	Not FLR	Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25		α_{my}	1.25	
M_{oax}	184.4 kNm	Eqn. 5.6.1.1(3)	M_{oay}	362.8 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.76		α_{sy}	0.99	
M_{bx}	95.9 kNm		M_{by}	32.7 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 95.9 kNm

Bending capacity in the weak axis **(Unfactored)** 32.7 kNm

OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 300B

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	35.7E+6 mm^4	Z_x (Strong Axis)	229.3E+3 mm^3	S_x (Strong Axis)	293.3E+3 mm^3		
I_y (Weak Axis)	6.7E+6 mm^4	Z_y (Weak Axis)	67.3E+3 mm^3	S_y (Weak Axis)	120.3E+3 mm^3		
r_x	109.76 mm	J	44.8E+3 mm^4	f_y	690 MPa	G	80000 MPa
r_y	47.639 mm	I_w	25.3E+9 mm^6	E	200000 MPa	t	6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 78 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Non-Compact**

$\lambda_s=\lambda_e=$ 21.7

2.2 Effective Section Modulus, Z_e

Z_{ex} = 242.7E+3 mm^3

Z_{ey} = 74.4E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 167.5 kNm

M_{sy} 51.3 kNm

(Unfactored)

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Segment length (Strong Axis)	1300 mm	Segment length (Weak Axis)	1300 mm
Segment restraints (Strong Axis)	PU	Segment restraints (Weak Axis)	PU
k_{tx}	1.03	k_{ty}	1.03

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2	k_{ly}	2
Ends with lateral rotation restraints	One	Ends with lateral rotation restraints	One
k_{rx}	0.85	k_{ry}	0.85
Effective Length, l_{ex}	2274 mm	Effective Length, l_{ey}	2274 mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1	β_{my}	-1
Length for FLR (Strong Axis)	574 mm	Length for FLR (Weak Axis)	1321 mm
Therefore:	Not FLR	Therefore:	Not FLR

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25		α_{my}	1.25	
M_{oax}	184.4 kNm	Eqn. 5.6.1.1(3)	M_{oay}	362.8 kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.63		α_{sy}	0.96	
M_{bx}	131.6 kNm		M_{by}	51.3 kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 131.6 kNm

Bending capacity in the weak axis **(Unfactored)** 51.3 kNm

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OZ-C-Splint Moment Capacity to AS 4100-1998

Reinforcement Size 360B

1. Section properties (from ShapBuilder or similar):

I_x (Strong Axis)	63.8E+6	mm^4	Z_x (Strong Axis)	338.8E+3	mm^3	S_x (Strong Axis)	431.7E+3	mm^3
I_y (Weak Axis)	12.0E+6	mm^4	Z_y (Weak Axis)	99.2E+3	mm^3	S_y (Weak Axis)	177.2E+3	mm^3
r_x	133.16	mm	J	53.6E+3	mm^4	f_y 690 MPa	G	80000 MPa
r_y	57.807	mm	I_w	66.5E+9	mm^6	E 200000 MPa	t	6 mm

2. Section Capacity in Bending

2.1 Section Slenderness (Cl. 5.2.2)

Plate element outstand (b) 95 mm

$\lambda_{sp}=\lambda_{ep}=$ 9

$\lambda_{sy}=\lambda_{ey}=$ 25 ∴ Section is **Slender**

$\lambda_s=\lambda_e=$ 26.4

2.2 Effective Section Modulus, Z_e

Z_{ex} = 321.3E+3 mm^3

Z_{ey} = 94.0E+3 mm^3

2.3 Section Capacity in Bending

M_{sx} 221.7 kNm (Unfactored)

M_{sy} 64.9 kNm

3 Member Capacity in Bending

3.1 Effective Length (l_e)

Assuming the longitudinal position of the loads is "At segment end"

Strong Axis			Weak Axis		
Segment length (Strong Axis)	1300	mm	Segment length (Weak Axis)	1300	mm
Segment restraints (Strong Axis)	PU		Segment restraints (Weak Axis)	PU	
k_{tx}	1.03		k_{ty}	1.03	

Assuming that the load application to the reinforcement does not provide stability against lateral torsional buckling.

k_{lx}	2		k_{ly}	2	
Ends with lateral rotation restraints	One		Ends with lateral rotation restraints	One	
k_{rx}	0.85		k_{ry}	0.85	
Effective Length, l_{ex}	2287	mm	Effective Length, l_{ey}	2287	mm

3.2 Check for full lateral restraint

Based on an equal flanged channel

β_{mx}	-1		β_{my}	-1	
Length for FLR (Strong Axis)	696	mm	Length for FLR (Weak Axis)	1603	mm
Therefore:	Not FLR		Therefore:	Not FLR	

3.2 Member capacity in bending

Moment modification factor based on Table 5.6.1 of AS 4100

α_{mx}	1.25			α_{my}	1.25		
M_{oax}	365.0	kNm	Eqn. 5.6.1.1(3)	M_{oay}	777.2	kNm	Eqn. 5.6.1.1(3)
α_{sx}	0.74			α_{sy}	0.99		
M_{bx}	204.2	kNm		M_{by}	64.9	kNm	

4. Summary

Bending capacity in the strong axis **(Unfactored)** 204.2 kNm

Bending capacity in the weak axis **(Unfactored)** 64.9 kNm