Shelter 8: Structural Assessment – Peru ARC

1.1 Introduction and Purpose

Arup was commissioned to carry out a structural review to assess and validate nine selected shelter designs for the IFRC. This document summarises the information gathered for and the key outcomes of the verification of the structural performance of Shelter 8, built by the American Red Cross in partnership with the Peruvian Red Cross. This assessment is based on the input documents listed in Appendix A.

Summary Information

Location: Peru, Chincha Province

Disaster: Earthquake 2007

Materials: Bolaina (Bolayna) Timber frame with timber cladding and corrugated metal sheet roofing

Material source: All materials sourced locally and produced in local fabrication workshops

Time to build: 1 day

Anticipated lifespan: 24 months +

Construction team: 2 people

Number built: 1900

Approximate material cost per shelter: UNKNOWN

Approximate programme cost per shelter: 560CHF

Shelter Description

The shelter has a Bolaina (Bolayna) timber braced frame, measuring 3m x 6m on plan with a single pitched roof at four degrees. The shelter is clad with tongue and groove solid timber board walls and a corrugated cementitious sheet roof. It is 2.4m high and stands on a new or existing concrete floor slab. In instances where a new slab has been used, wire ties wrapped around nails have been cast into the slab and attached to the frame at all column locations to resist uplift. Where existing slabs have been used the shelter has been staked to posts installed outside the slab. The frame is constructed in 6 panels which are then nailed together using connecting wooden members, connecting plates and plastic strapping. A main roof beam is attached to the frames and purlins nailed on top of this to support the roof.

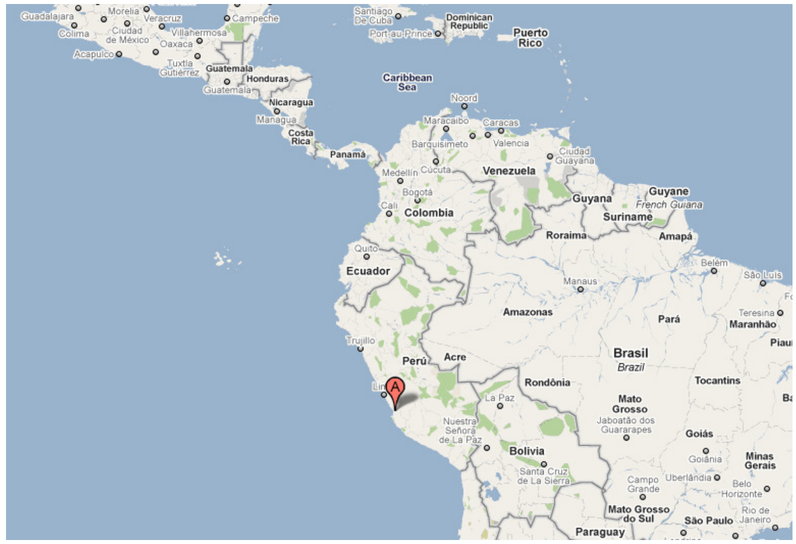
The shelters were intended for upgrade and built with quality materials which were intended to be reused. However, since the timber is untreated, the durability is poor and the members are susceptible to damp and rot.

1.2 Location and Geo-hazards

1.2.1 Location of Shelter

Chincha Province, Peru

Areas including Hoja Redona, Condorillo, Tambo de Mora, Keiko, Sofia and Pueblo Nuevo la Union. It has been assumed that all sites are in desert coastal regions on flat land. An approximate latitude and longitude for the site are 13 deg 17’ S, 76 deg 8’ W.



1.2.2 Hazards

A summary of the natural hazards faced in the Chincha Province of Peru are given below[1]:

- HIGH Earthquake. A map from the Peruvian Design Code[2] suggests that the shelters are situated in Zone 3 which has a high peak ground acceleration (PGA) of 0.4g for an earthquake return period of 475 years.
- MEDIUM wind. The area not prone to tropical storms or cyclones. Wind speeds vary considerably depending on the region and local topography. The coastal location implies that wind speeds may be higher than average but information should be site specific or based on local knowledge. See Section 1.8.3 for wind loading details.
- MEDIUM Flood Risk. During the El Nino phenomenon every 15-20 years heavy rain can fall which causes widespread flooding and mudslides.
- High Liquefaction Risk. Previous history of extensive soil liquefaction in the region during previous earthquakes.
- Landslide Risk. Previous history of landslides in mountainous areas during earthquakes or during heavy rainfall but lower risk in coastal areas.
- Other hazards that will not be designed against include tsunami and volcanoes. There is precedence for tsunamis in the region but there are no active volcanoes.
- Arid desert location with high temperature variations. Dry climate with strong winds and regular sandstorms, rains rarely. Temperatures range from 14^{o}C to 29^{o}C maximum.

---

[1] See Appendix A, Reference 1.
[2] See Appendix A, Reference 6.

1.3 Geometry

The geometry was determined using the photos, drawings and bill of quantities provided. See Appendix A for a list of source information. Figure 1.1 shows sketches of the shelter geometry, and Figure 1.2 shows a 3D image of the shelter.

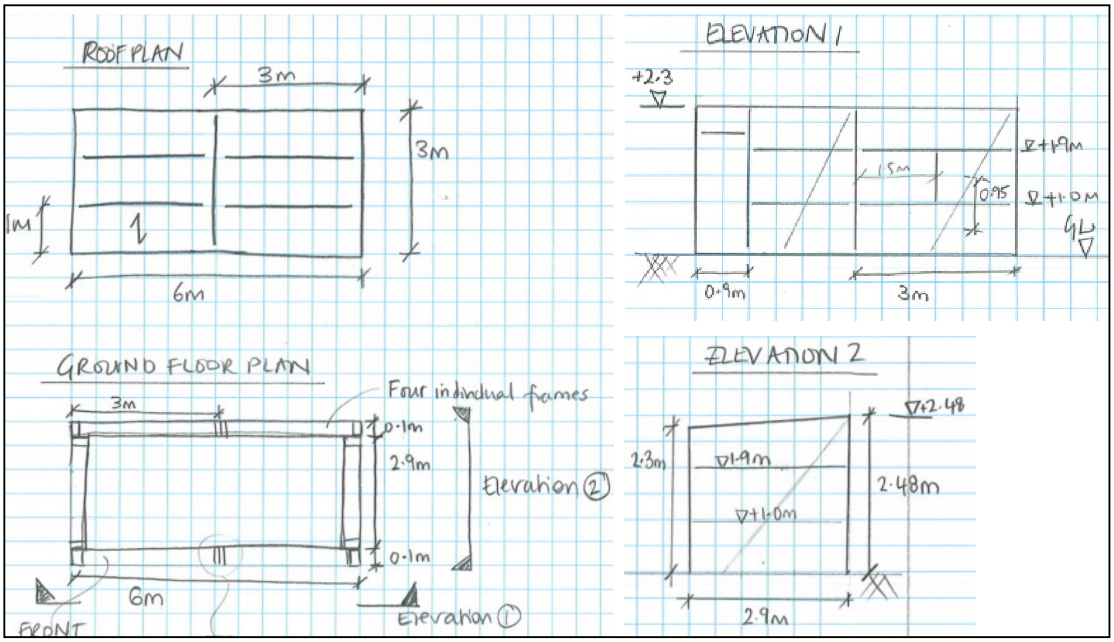


Figure 1.1 – Sketches of Geometry

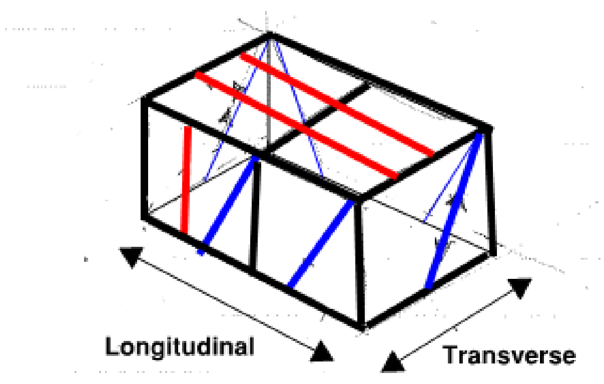


Figure 1.2 – 3D Drawing of Shelter

The shelter has a timber braced frame, measuring 3 x 6m on plan with a shed roof pitched at 3.6 degrees. The shelter is clad with tongue and groove solid timber boards to form the walls and has corrugated cementitious sheet roofing. It is 2.4m high and stands on either an existing or new concrete slab that forms the floor. Where a new slab has been used, wire ties wrapped around a nails have been cast into the slab and attached to the frame at the seven column locations to resist uplift. The frame is constructed in 6 panels which are then nailed together using connecting wooden members, gusset plates and plastic strapping. A main roof beam is attached to the frames and purlins nailed on top of this to support the roof.

Geometrical Assumptions:

- After 500 shelters were built a review was carried out making recommendations to improve the shelters. It has been assumed in this assessment that those recommendations have been put into practice in all further shelters and retrofitted in the existing shelters. This includes:
    - Tying shelter to the slab using wire and nails cast into new slabs or by staking outside existing slabs.
    - Bracing members shown in drawings added into walls.
    - Strapping added at nailed connection joints and gusset plates used where three or more members meet.
    - Support blocks added for roof joists not resting on wall panels, new shelter roof beams to rest on wall panels.
- The shelter is constructed from 6 panels made individually and then connected together. It has been assumed that column members act individually rather than compositely.
- All connections are nailed with two nails and are assumed to act as pinned connections.
- For the purposes of analysis it has been assumed that the shelter has been fixed to a newly cast 100mm thick minimum concrete slab using 7 wire ties at each of the column locations. Each wire tie consists of a single 6d nail with a double AWG 16 wire twisted around to leave the two free ends above the concrete. It has been assumed that the slab has one layer of square A142 mesh reinforcement half way down.
- The primary roof beam in the centre rests directly upon the side frame columns rather than on additional support bracket.

1.4 Structural System

- Vertical loads are transferred from the longitudinally spanning roof purlins back to the primary roof beam and eaves beams and then to the columns. The columns transfer these forces by bearing to the concrete floor slab which is ground bearing on the soil.
- In both the transverse and longitudinal directions stability is provided by timber bracing members. These are single braces in each frame and are not triangulated to the columns as shown in Figure 1.2. They therefore transfer horizontal forces in both tension and compression.
- Bracing is required in the plane of the roof to transfer transverse loads back from the central columns to the edge columns.
- Resistance to uplift and shear is provided by wire ties embedded in the concrete slab at column locations.

The bracing in the shelter is insufficient and it was noted in a review of the design that external propping had been added to the walls. The wall bracing provided is not a code compliant lateral system, therefore the shelter has been assumed to be flexible and an R value of 1.0 has been used in the seismic analysis. Some lateral resistance will also be provided by the timber tongue and groove wall panelling. The roof sheeting provides some diaphragm action and restrains the independent movement of adjacent frames.

1.5 Member Sizes

The table below shows the key timber frame member sizes that have been assumed for the structural assessment. These sizes have been based on information given in the drawings and Bill of Quantities referenced in Appendix A. The table does not include secondary timber framing to form doors or other non-structural elements. The full list of elements is given in the updated Bill of Quantities in Appendix B.

Member Description	**Length (m)**	**Member Size (mm)**
Vertical Columns	2.3 – 2.5	25mm x 50mm
Wall Transoms	3	25mm x 50mm
Roof and Eaves Beams	3	25mm x 50mm
Floor Beams	3	25mm x 50mm
Bracing	3	25mm x 50mm
Purlins	3	25mm x 50mm

1.6 Materials

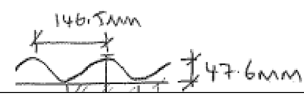
The timber for the frame and cladding was sourced locally and cut into the required lengths and sizes by a local contractor in batches. It has been assumed that the frame and cladding is made from untreated White Bolaina timber connected using 6d or 8d nails.

1.6.1 'Bolaina' Wood

The panels are fabricated from frame and cladding members made using White Bolaina (*Guazuma Crinita*) timber native to Peru. This type of timber is typically found in the natural forests of the Peruvian Amazon riverbank but more recently is being cultivated for timber production in large plantations. The timber is fast growing and needs flat moist and partially flooded soils to thrive, under these conditions the trade turnover of the timber is 7-9 years.

The timber properties are good, lying somewhere between a hard and soft wood. For analysis purposes the closest equivalent from the NDS for Wood Construction 2005 has been chosen: No.2 structural grade South Douglas Fir. The modulus of elasticity of these timbers are very similar, and although data suggests the strength of the Bolaina is higher than the Douglas Fir, it is thought that the tabulated Douglas Fir values provide a good equivalent design level. The timbers are untreated so will therefore be susceptible to moisture absorption and rot near ground level.

1.6.2 Material Assumptions

Type	IFRC Specification	Arup Assumption	Comments
Concrete	Dry mix of sand and cement sprinkled on ground and compacted	Compressive cube strength f_{cu} = 15-20MPa (low strength concrete), see I.1 concrete specification	Specification given suggests that the quality of the concrete is very low
Roof Sheeting	Lightweight cementitious 'fibrocemento' corrugated panels, 1 x 3m sheets, ¼" thick	High strength fibre cement sheet with polypropylene reinforcement strips, 6.7mm thick, 1086x3050mm sheets with 70mm overlap, 0.17kN/m^2 installed weight[1] 	Two fixings per sheet in 2mm oversize holes with 8d nails & washers, 1300mm maximum purlin spacing, 8mm butyl strip required between sheets to seal
Timber Frame	Bolayna timber frame	White Bolaina timber, with properties similar to Douglas Fir South – No.2 Structural Grade, density 410kg/m^3, Young's Modulus 8274N/mm^2, bending strength 5.86N/mm^2	See Timber 2, I.1. Member dimensions are assumed to be as cut – no sacrificial allowance has been made, for further information see Section 1.6.1.
Timber Cladding	Tongue and groove solid Bolayna wood panelling, 3.5" wide, 3/8" thick	Properties of Bolaina timber as for frame, thickness and panel width as given by IFRC	6d nails at 150mm spacing required with 400mm maximum stud spacing
Plastic Strapping	None	See plastic specification, I.1	For joint fixings, not checked in this review
Screws	None	Steel screws, yield strength 275N/mm^2	Location unknown therefore diameter and length not required for check
Wire	14 or 16 gauge	Galvanised AWG16, 1.3mm diameter and 275N/mm^2 yield strength	For foundations, see I.1 specification
Nails	1.5" nails to fix cladding and corners	Galvanised 6d/8d nails in 275N/mm^2 yield strength steel, see I.1 specification for further details	Assume 2 nails for all timber connections and washers are used to nail roof sheet in accordance with guidelines in C.3

[1] It should be noted that in comparison to corrugated steel roof sheeting and other materials used on the other shelters, this is not particularly lightweight.

1.7 Codes, Standards and References

General

The IBC (International Building Code) 2009 has been used as a basis for the design checks on the shelters since it is widely accepted worldwide, particularly for extreme loading cases such as earthquakes or strong winds. Other codes have been referenced where appropriate or where the IBC was thought to be less applicable. This includes national codes where appropriate and the UBC 1997.

Other references used for this shelter:

- Standards referred to by IBC 2009 including: ASCE 7-10 (2010) for loading and the NDS for Wood Construction 2005 for timber design
- UBC 1997 Volume 2 for preliminary wind calculations and parts of seismic calculations.
- Peruvian national codes as referenced in Appendix A for seismic and wind loading data.

1.8 Loads

1.8.1 Dead Loads

- Self-weight of structural materials applied in accordance with the densities specified in Section 1.6.1.

1.8.2 Live Loads

- For IBC compliancy a live loads of 1.92kN/m^2 on the ground floor and 0.96kN/m^2 on the roof should be applied[1]. In this case however, no live load is assumed on the roof since there will be no maintenance access or snow load so it is not applicable. The live load allowance for the ground floor has been reduced to 0.9kN/m^2 since this represents a more realistic loading situation.
- The ground floor of the shelter consists of a ground bearing concrete slab, therefore no loading checks are required.

[1] 'International Building Code', ICC, 2009 – Table 1607.1.

1.8.3 Wind Loads

Wind loads can be calculated using a minimum wind speed for Peru of 75km/hr (20.8m/s)[1] with a gust speed factor of 1.62 to give a basic wind speed of 33.7m/s for a 50 year return period[2]. The UBC[3] method was used with the following values to calculate the design pressures:

Convert basic wind speed to pressure Table 16-F	$q_s = 0.70\ kN/m^2$
Assume exposure class C and height of 0-4.6m – Table 16-G	$C_e = 1.06$
Importance Factor – Table 16-K	$I_w = 1.0$
Pressure coefficients assuming an enclosed structure – Table 16-H	C_q – varies for each element

Modifying the wind pressure by the pressure coefficients gives a maximum uplift pressure in the partially enclosed case of 0.89kPa and a maximum lateral force on the structure of 13.9kN in the transverse direction. The resulting factored pressure on the windward face of the structure was found to be 0.94kPa.

Local knowledge of higher wind speeds must be taken into account by using higher design pressures for specific shelter locations where necessary.

[1] See Appendix A, Reference 7. The method used in this standard to calculate wind pressures is very similar to that used by the UBC and gives results that make those described above conservative.
[2] This wind speed is derived from the Peruvian code and represents research into typical speeds in that region. Site specific speeds are subject to local knowledge.
[3] UBC 1997 – Division III.

1.8.4 Seismic Loads

Seismic Loading has been considered in accordance with the IBC[1] using a short period design acceleration based on the UBC methodology. Stiff soil has been assumed (soil category D or Site Class D). The design response acceleration was determined using the PGA detailed in Section 1.2.2.

Assume Site Soil Category D[2] (20.3-1) and use PGA (Z) in UBC Table 16-Q	$C_a = 0.44N_a$
Assume seismic source type A[3] (UBC Table16-U) and distance to source is >10km[4] (UBC Table 16-S)	$N_a = 1.0$
Assume structure response in 0.5-1.5s period (UBC 16-3) to get S_{DS}	$S_{DS} = 2.5C_a$
Assume Risk Category II[5] (Table 1.5-1) in Table 11.6-1	Seismic Design Category D
Importance factor assuming Risk Category II – Table 1.5-2	$I_e = 1.0$
Assume no codified seismic lateral system – Table 12.2-1[1]	R = 1.0

The equivalent lateral force procedure has been used to calculate horizontal loads for design. The resulting base shear is 4.73kN which is smaller and therefore less critical than the lateral wind load due to the lightweight nature of the structure.

[1] 'ASCE 7-10 – Minimum Design Loads for Buildings and Other Structures', Chapters 11&12.
[2] In locations where liquefaction is a risk the Site Soil Category should be changed accordingly for seismic design.
[3] Type A assumes that relevant faults are capable of producing large magnitude events – see Appendix A, reference 6.
[4] In locations where shelter is located closer to faults this parameter should be modified accordingly for seismic design.
[5] Risk Category II has been assumed for this shelter rather than Risk Category I as assumed for previous shelters. This is because the roof sheeting is heavier and therefore poses more of a risk to the life safety of the occupants in the event of failure. This does not however affect the magnitude of the seismic load the shelter is designed for.

1.9 Calculation Plan

1.9.1 Design Methodology

The performance of each shelter has been assessed by checking that the structure as assumed from the information provided is safe for habitation. Relevant codes and standards have been used as the baseline for identifying appropriate performance/design criteria, but the structure has been checked against code requirements: if variations from this have been made, assumptions and reasoning for lower factors of safety and alternative standards have been justified. Logical reasoning was therefore used where necessary and upgrades suggested in order for the shelter to meet these criteria.

The shelter has been assumed to be enclosed since it has complete timber cladding on all sides and has panelled doors and window shutters. The worst case for wind normal to the front of the shelter is when the door and window are open, in which case the structure acts as a partially enclosed structure since the face is more than 15% open by area (as defined in the IBC 2009). This means that the uplift on the roof is larger for wind from this direction since the internal pressures are higher.

1.9.2 Structural Checks

For a summary of the checks performed to assess the building, refer to Appendix C.

[1] Bracing is not considered sufficient to resist lateral loads due to its low strength.

2 Results of Structural Assessment

2.1 General Key Findings

I. The foundation solution performs adequately in bearing under vertical, seismic and dead loads. In order to resist shear loads in the seismic case twice the number of ties would be needed. Under wind loads, three ties are needed per column (rather than one) to resist uplift and shear forces, or an alternative foundation solution is required.

II. Under dead loads alone the roof of the shelter must be strengthened to take the weight of the roofing by decreasing the spacing of the roof purlins and using larger timber members at the centre and eaves.

III. Under seismic loads in addition to the strengthening of the purlins, roof and eaves beams, the section size of the in-plane wall bracing must be increased to resist compression. The floor beam size must also be increased to resist bending caused by tension in the bracing, or alternatively the wall bracing could be triangulated by connecting it back to the column base rather than to the floor beam.

IV. Under wind loads in-plane steel cross bracing is required in the roof to transfer wind loads on the intermediate frames back to the end braced frames. This could be done using double wires as tension only bracing. In addition to the recommendations made for seismic loading to strengthen the roof, bracing and floor beams, the column sizes should be increased or the spacing decreased to resist the bending moments from the wind pressures. The spacing of the wall transoms should also be decreased to 400mm maximum so that the timber cladding can resist the wind pressures and the transoms themselves can resist the bending moments from the wind loads.

3 Conclusions and Recommendations

3.1 Assumptions

- Cementitious roof sheeting is a relatively heavy high strength fibre cement sheet, with polypropylene reinforcement strips, 6.7mm thick, 1086x3050mm sheets with 70mm overlap, 0.17kN/m^2 installed weight.
- Timber wall panneling is sufficiently fastened and of sufficient strength to transfer wind loads back to the frame without damage to the cladding.
- The connections between the 6 frame panels are of sufficient strength to transfer forces between frames and use the recommended plastic tape strapping and timber wall plates. Columns have been assumed not to act compositely but an adequately nailed connection to facilitate this is recommended.
- The primary roof beams and purlins are supported directly off the top of the wall panels, and not from secondary supports of any kind.
- All connections are nailed with two nails and are assumed to act as pinned connections.
- For the purposes of analysis it has been assumed that the shelter has been fixed to a newly cast 100mm thick minimum concrete slab using a double wire tie at each of the column locations (seven in total). Each wire tie consists of a single 6d nail with a double AWG 16

wire twisted around to leave the two free ends above the concrete. It has been assumed that the slab has one layer of square A142 mesh reinforcement half way down.

- Fixings between members have been made using nails only but are of sufficient strength to transmit forces. The design and detailing of all connections is critical to the stability of the structure and should be checked for individual cases.
- A stiff soil type has been assumed in analysis of the structure. For sites where liquefaction may be a hazard (near river beds, coastal areas with sandy soils and high water tables), the shelters could be seriously damaged if soil liquefies in an earthquake but such damage is unlikely to pose a life safety risk to occupants.

3.2 Conclusions

Performance Analysis	
The performance of the shelter under gravity loads is inadequate. The roof must be strengthened by decreasing the purlin spacing and strengthening roof and eaves beams. Further modifications are required to strengthen the building under seismic and wind loads.	
Hazard	**Performance**
Earthquake – HIGH	The performance of the shelter under seismic loads is inadequate. The roof must be strengthened and the in-plane wall bracing increased in size and connected back to the column base. The shelter must also be more adequately tied to the foundations to prevent sliding. The resistance of the shelter to lateral loads is low so damage is expected. However, as the shelter is relatively lightweight and flexible it poses a low risk to the life safety of the occupants when damaged.
Wind – MEDIUM	The structure has insufficient resistance to wind loads. It must be more securely tied down to prevent uplift and sliding, in addition to the strengthening of the roof and wall bracing. In-plane wire cross bracing is required in the roof, the spacing of the wall transoms must be decreased and the columns strengthened to resist lateral wind pressures.
Flood – MEDIUM	The shelter does not incorporate any flood protection strategies and the connection of the shelter to the slab may be insufficient to hold the shelter during a flood event.

Notes on Upgrades:

The most common upgrade to the shelter is the addition of an internal partition which improves the lateral stiffness if a wooden partition is used. The shelter was also used as an extension or starter room for permanent homes and in many cases has been painted and insulated with polythene or plastic to retain heat in cold weather. If additional insulating material is added on the roof then further strengthening in addition to that already recommended will be required to carry the increased loads. The shelters were built with materials which were intended to be reused. However, since the timber is untreated, the durability is poor and the members are susceptible to damp and rot and are therefore not suitable for reuse in permanent housing.

In some cases modules have been joined together to form larger structures. In this case the internal walls must be retained otherwise the shelter will become unstable. Nailed plywood walls would provide a more durable and stiff solution to the timber planking for little extra cost.

Upgrading the shelter with masonry or other very heavy materials to a high level or on the roof is not recommended as they will attract high seismic loads causing the structure to perform poorly in an earthquake. Collapse of a heavier roof or unreinforced masonry walls poses a serious risk to the life safety of the occupants.

Watch-its for drawings: 'Change or Check'

A. CHANGE: Decrease purlin spacing and change orientation so that the shorter edge is connected to the roofing.
B. CHANGE: Increase the dimension of the nailed face of timber members to 50mm to avoid splitting when nailed.
C. CHANGE: Increase the size of the central roof and eaves beams to take dead and wind loads.
D. CHANGE: Use an alternative foundation solution to provide uplift and sliding resistance (Type 2 or 5, C.1) against wind and seismic loads, or use three double wire ties per column when casting a new slab (28 in total).
E. CHANGE: Treat timber members to prevent rot and insect degradation.
F. CHANGE: Increase roof pitch to over 5 degrees to allow rain water run-off, and prevent deterioration of the roof.
G. CHANGE: Increase size of timber bracing members to take compression forces and move braces to meet at column base instead of floor beam.
H. CHANGE: Use crossed double wire ties as bracing in roof plane to provide stability under lateral loads.
I. CHANGE: Space wall transoms at a maximum distance of 400mm to prevent excessive deflection and failure of the timber plank walls under wind loads and improve lateral stability.
J. CHANGE: Increase column sizes or decrease spacing according to design to local wind pressures to prevent bending failure and excessive deflection.
K. CHECK: Fix roof sheeting using two 8d nails per roof sheet panel in 2mm oversize holes with washers, placed through the crown. Use a 70mm overlap and a seal between sheets for total weatherproofing.
L. CHECK: In areas known to have higher local wind pressures adequate foundations and member sizes must be provided to account for this.
M. CHECK: Fix timber wall planks to transoms using 6d nails at a maximum spacing of 150mm.
N. CHECK: Increase timber plank thickness from 9.4mm to 12.5mm minimum to provide an adequate weather covering, or add plywood sheathing.

O. CHECK: If an existing slab is used for the base, design appropriate anchor system to resist uplift and sliding forces under wind and seismic loads.

P. CHECK: Do not upgrade using masonry due to risk to life safety and increase in seismic force attracted to the structure.

Q. CHECK: The cementitious roof sheeting is heavier than alternative roofing materials such as corrugated steel sheeting. This increases the risk to life safety in the event of an earthquake or strong wind.

R. CHECK: Design member connections for local hazard conditions by connecting members using wall plates and additional wood strips, and securing roof against uplift using plastic strapping.

S. CHECK: Check soil type for shelter location is stiff, otherwise design foundations accordingly.

Appendix A – Source Information

1. IFRC Hazard Assessmnet/Peru Chincha Province – Memorandum, 10th December 2010, Juliet Mian & Sasha Drozd (Arup).
2. 'C.5 Peru – 2007 – Earthquake', Shelter Projects 2008, IASC Emergency Shelter Cluster, UN Habitat, UNHCR & IFRC, 2008.
3. 'American Red Cross Transitional Shelter Program; Peru Earthquake, 2007', ARC, 2007.
4. Drawings: 'ARC Peru frame plans & TS ARC cladding' and relevant photos, IFRC, 2011.
5. 'Peru 2007 Earthquake assessment trip report November 12-19, 2007', LeGrand Malany & Shelley Cheatham, ARC, December 2007.
6. 'National Building Code, Technical Standard of Building E.030, Earthquake Resistant Design', Lima 02/04/03.
7. 'Proyecto de norma tecnica de edificacion, E.020 Cargas', December 2004.

Appendix B – Bill of Quantities

The table of quantities is for the materials required to build the shelter. It does not take into account issues such as available timber lengths and allowances for spoilage in transport and delivery.

Item (Dimensions in mm)	Material Spec.	No.	Total	Unit	Comments
Structure - Foundations					
Portland Cement	Concrete	4	4	bags	42.5kg/bag
Sand	Concrete	-	0.34	m^3	Estimate only
Gravel	Concrete	-	0.68	m^3	Estimate only
Wire mesh Reinforcement	-	18	18.0	m^2	
Nails – 6d	Nails	7	7	Pieces	
Wire (16 AWG)	Wire	6	6.0	m	Estimate only
Main Structure					
Columns – 25 x 50 (L=2.5m)	Timber 4	13	32.5	m	
Roof Beam – 25 x 50 (L=3m)	Timber 4	2	6.0	m	
Eaves Beams – 25 x 50 (L=2.9m)	Timber 4	2	5.8	m	
Eaves Beams – 25 x 50 (L=3.0m)	Timber 4	4	12.0	m	
Floor Beams– 25 x 50 (L=2.9m)	Timber 4	2	5.8	m	
Floor Beams – 25 x 50 (L=3.0m)	Timber 4	4	12.0	m	
Bracing – 25 x 50 (L=3.0m)	Timber 4	6	18.0	m	
Secondary Structure					
Purlins – 25 x 50 (L=3.0m)	Timber 4	4	12.0	m	
Wall Transoms – 25 x 50 (L=3.0m)	Timber 4	6	18.0	m	
Wall Transoms – 25 x 50 (L=2.9m)	Timber 4	4	11.6	m	
Wall Transoms – 25 x 50 (L=2.05m)	Timber 4	2	4.1	m	
Door & window framing – 25 x 50 (L=1.0m)	Timber 4	2	2.0	m	
Covering – Wall and Roof					
Cementitious Roof Sheeting (1 x 3m sheet, 6.25 thick)	Sheet 3	6	6	Pieces	
Timber tongue & groove planks – 87.5 x 9.4 (L=2.48m)	Timber 4	68	169	m	
Timber tongue & groove planks – 87.5 x 9.4 (L=2.30m)	Timber 4	43	98.9	m	
Timber tongue & groove planks – 87.5 x 9.4 (L=0.42m)	Timber 4	10	4.2	m	
Timber tongue & groove planks – 87.5 x 9.4 (L=0.32m)	Timber 4	16	5.1	m	
Timber tongue & groove planks – 87.5 x 9.4 (L=1.01m)	Timber 4	16	16.2	m	
Timber tongue & groove planks – 87.5 x 9.4 (L=2.48 decreasing to 2.30m)	Timber 4	70	70	Pieces	
Fixings					
Nails – 8d roofing nails with protecting cap/washer	Nails	-	0.5	kg	
Nails – 6d	Nails	-	1.6	kg	
Plastic Tape (10 x 150)	-	8	8	Pieces	For joints
Steel hinge 2.5”	-	7	7	Pieces	
Screws	Screws	3	3	Pieces	
Wood strips – 30 x 60 (L=3m)	Timber 4	2	6	m	
Wall plates – 60 x 60 x 9.4 thick	Timber 4	-	7.5	m	Cut corner plate reinforcement
Tools Required					

Hammer	-	1	1	Pieces	

Appendix C

Calculation Plan

1) Loading

The seismic and wind loading has been calculated using data from the Peruvian Design Codes referenced in Appendix A along with methodology from the UBC 1997 and IBC 2009. The timber members have been checked using allowable stress design (ASD) to IBC 2009 which references the National Design Specification for Wood Construction (NDS) 2005 (see Section 1.7).

The loads described in Section 1.8 have therefore been combined using the un-factored load cases described in the IBC (International Building Code) 2009, Section 1605.3.1 for Allowable Stress Design (ASD).

2) Stability

   a. Overturning forces on foundations due to lateral seismic and wind loads

   b. Transverse Stability – key members: columns, primary beams and bracing

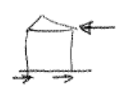
   c. Longitudinal Stability – key members: columns, primary beams and bracing

3) Foundations have been checked for the following cases accounting for the effects of overturning:

   a. Bearing pressure (dead loads + overturning)

   b. Uplift (wind uplift + overturning)

   c. Base Shear (worst case from wind/seismic)



4) Primary Members

Check members for a combination of vertical and lateral loads, including: columns, roof and eaves beams, floor beams and bracing.

5) Secondary Members

Check members for a combination of vertical and lateral loads, including: roof purlins, wall transoms, roof sheeting and wall cladding. Recommend fixing spacing for roof and wall cladding.

6) Fixings – assumed to be strong enough to transmit all member forces. Connections have been assumed to be pinned for analysis, including at column bases.