Shelter 2: Structural Assessment – Peru SRC

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 2, built by the Spanish Red Cross and IFRC in partnership with the Peruvian Red Cross in Peru. This assessment is based on the input documents listed in Appendix A.

Summary Information:

Location: Peru, Ica Region

Disaster: Earthquake 2007

Materials: Eucalyptus wood poles, bamboo matting, plastic sheeting, wire and nails, concrete slab

Material source: Mats and wood locally available, plastic sheeting imported, staples and staple guns imported.

Time to build: 2 days

Anticipated lifespan: 12 months minimum

Construction team: 4 people

Number built: 3000

Approximate material cost per shelter: 225CHF (2007)

Programme cost per shelter: 340CHF (2007)

Shelter Description:

The structure is a rigid box consisting of braced frames in both directions to provide lateral stability. The eucalyptus timber frame has a flat roof and is covered with stapled plastic sheeting and nailed palm matting on all faces. It is 2m high and 3 x 6m on plan and the bracing consists of crossed twisted wires. The 75mm diameter columns are connected horizontally with 50mm diameter horizontal secondary members. The foundation and floor consists of an unreinforced concrete slab with cast in wire ties and the connections between members are made using bent nails.

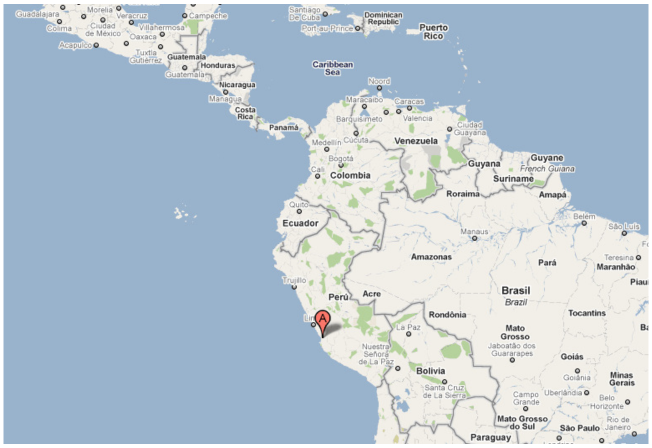
The matting traps dirt and mould and is prone to breakage where stapled, the plastic sheeting may fail due to wear and tear and the timber is untreated. The shelter is not upgradable, but straw mats and frame could be reused. The shelter is demountable and could be easily moved from its foundation by cutting the wire ties. The timber frame can be reused, but the slab cannot, and a new foundation will be required if the shelter is moved.

1.2 Location and Geo-hazards

1.2.1 Location of Shelter

Ica Region, Peru

The shelters were located in Ica, in the Ica Region of Peru. It has been assumed that all sites are in desert coastal regions on flat land. =



1.2.2 Hazards

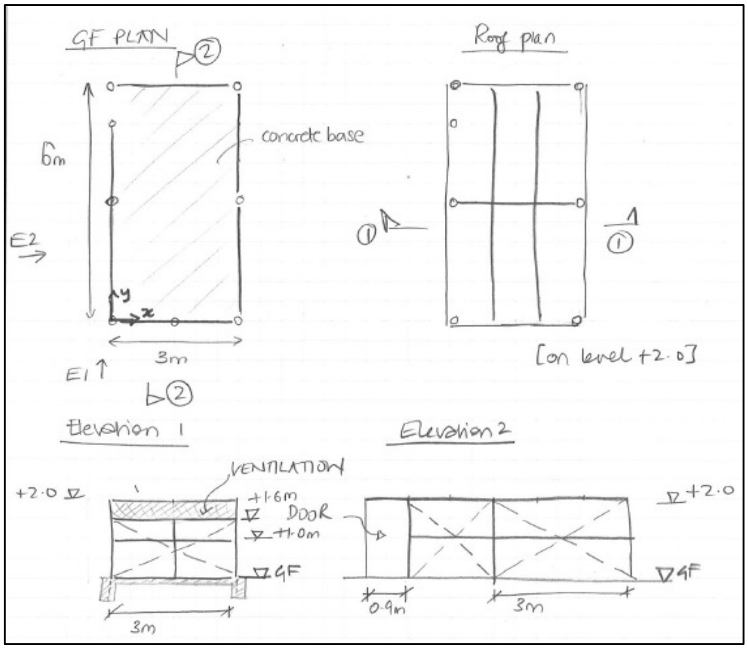
A summary of the natural hazards faced in the Ica Region of Peru are given below[1]:

- HIGH Earthquake Risk. A map from the Peruvian Design Code[2] suggests that the shelters are situated in an area with 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.
- 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 previous 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.

[1] See Appendix A, reference 6 – the Chincha Province lies within the Ica Region. This note therefore provides a good basis for establishing hazard risks in the area.
[2] 'National Building Code, Technical Standard of Building E.030, Earthquake Resistant Design', Lima 02/04/03.

- Arid desert location with high temperature variations. Dry climate with strong winds and regular sandstorms. Rains rarely.

1.3 Geometry



The geometry was determined using the drawings and photographic information provided, see Figure 1.1 for key members and levels. A GSA model detailing the geometry has been created from this data as illustrated in Figure 1.2.

Figure 1.1 – Sketch of Geometry

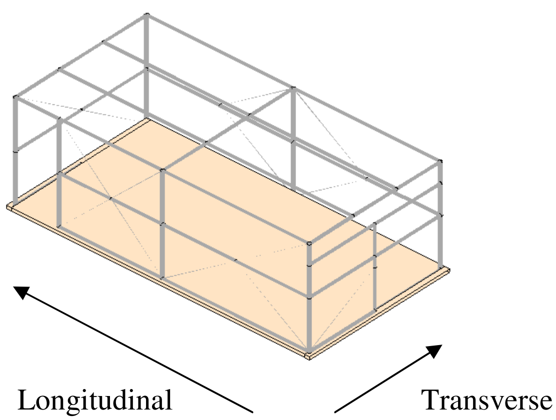


Figure 1.2 – GSA Model

The structure is a rigid box consisting of braced frames in both lateral directions. The structure is 2m high with a flat roof, 6m x 3m on plan, and has a full height door at one side and full width window at one end. There are eight columns laterally connected by horizontal transoms to support the wall cladding. The roof consists of members spanning between the columns in the transverse direction and secondary support members spanning in the longitudinal direction. The roof also contains in plane wire bracing to transmit lateral forces. The bracing in walls and roof consists of two twisted strands of wire. The secondary transoms on walls and roof support the bamboo mat and plastic sheet cladding. The base of the frame is connected to the concrete slab foundation as detailed in Section 1.4.

1.4 Structural System

- Vertical loads are carried by horizontal beam members back to vertical columns which transmit forces into the foundation slab in bearing onto the concrete.
- Global stability in the transverse and longitudinal direction is provided by bracing in the plane of the walls made from two strands of twisted wire (see Figure 1.3) wound around nails at the connections. These forces are transferred back to the foundations by the connections detailed below.

It should be noted that the structure has very little lateral stability since wire braces yield under low lateral forces. This means that they will no longer be elastic and will be permanently deformed from their original shape in a low magnitude earthquake or wind event. In future events they will therefore be unable to absorb wind or seismic energy and may fail. This is not a code compliant seismic or wind resistant lateral system.

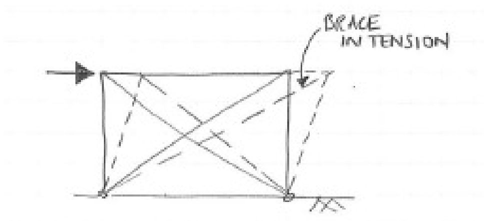


Figure 1.3 – Deflected Shape of Lateral System

The bracing members are to be checked as ties assuming that one brace per bay transmits the lateral forces in tension only at any time. All connections using wire and nails between members have been conservatively assumed to be pinned. The wires have been assumed to be continuous where they cross other structural members.

The foundation consists of a 3m x 6m concrete base slab. The structure is connected to the foundation using one of two options. The second option has been assumed in the analysis.

1. Embedment of columns to 0.5m in concrete pocket in appropriate locations in slab.[1]
2. Embedment of a stick into the slab by 50mm with wire ties twisted around it. These are then fixed to the base members of the rigid frame.[2] Double wires at the 8 column locations have been assumed.

[1] See Appendix A, Reference 2.
[2] See Appendix A, Reference 4.

1.5 Member Sizes

The table below shows the 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 updated Bill of Quantities is given in Appendix B.

Name	**Length (m)**	**Description**	**Number**
Structural Members			
Main Columns	2.0	75mm diameter eucalyptus wood pole	7
Secondary Column[1]	1.6	50mm diameter eucalyptus wood pole	1
Horizontal Beams	6.0	50mm diameter eucalyptus wood pole	6
Horizontal Beams	5.1	50mm diameter eucalyptus wood pole	2
Horizontal Beams	3.0	50mm diameter eucalyptus wood pole	8
Secondary Members			
Vertical Door Members	2.0	50mm diameter eucalyptus wood pole	2
Horizontal Door Members	3.0	50mm diameter eucalyptus wood pole cut into three 0.9m lengths	1
Wire Cross Bracing	2.9/3.4/3.6/4.25	No.16 double wires	2/2/8/4

1.6 Materials

Materials are all locally sourced and transported by truck with the exception of the plastic sheeting for roof and walls. The timber frame consists of Eucalyptus poles, connected using nails and wire loops and connected to concrete slab foundation using nails and wire loops. Twisted wire bracing has been used to provide stability. Walls and roof consist of plastic sheeting and bamboo matting.

[1] The size of this member has been increased and quoted as the larger size in the Bill of Quantities due to the results of the analysis.

1.6.1 Material Assumptions

Type	IFRC Specification	Arup Assumption	Comments
Concrete	Portland cement + common sand/gravel (2 bags cement to 1m3 sand)	Compressive cube strength f_{cu} = 15-20MPa (low strength concrete).	Quantities provided suggest the ratio of cement to sand seems low therefore it is expected that the concrete quality will be very low.
Bamboo mat	'Esteras' bamboo mat (2m x 3m)	Assume 7.5mm thick bamboo mat, density 350kg/m^3.	Material is light weight. Assume that it is stapled at sufficient intervals to prevent breakage under wind loads.
Timber	Eucalyptus lumber	Density 800 kg/m^3, Young's Modulus 7584N/mm^2 (low G2 Timber) – for other properties assume Grade 2 Douglas Fir.	Member dimensions given are assumed to be as cut – no sacrificial allowance has been made. For further information on assumptions made see Section 1.6.2.
Nails	No. 3"/2.5"/1.5"	10d/8d/4d nails respectively. All connections are assumed to be made with two nails bent into a loop.	Nail embedment is assumed to be the minimum to stabilise the joint.
Staples	Steel max 1"	22/25 staples assumed (~0.64mm thick)	Assume that wall mats and sheeting are stapled at sufficient intervals to prevent tearing or breakage under wind loads.
Wire	No.16 galvanised double tie wires	AWG 16, 1.3mm diameter wires – yield strength of steel 275N/mm^2	
Plastic Sheeting	Plastic Sheeting (4m x 6m)	Polyethylene sheet with braided core (HDPE/LDPE) – 200g/m^2	Assume that sheeting is stapled at 150mm intervals to prevent tearing under wind loads.

1.6.2 Timber – Eucalyptus Lumber

In Southern America the most common form is Eucalyptus Globulus or Blue Gum Timber. It has been observed that this type of Eucalyptus wood is similar in its properties to both Douglas Fir and Southern Pine for which design values are tabulated[1]. The material properties of eucalyptus wood are similar to the tabulated Grade 2 values for both wood types – Douglas Fir has been chosen as an equivalent for design since it provides the most conservative strength values.

[1] National Design Specification Supplement – Design Values for Wood Construction, American Forest and Paper Council, 2005 Edition.

1.7 Codes, Standards and References

General

The IBC (International Building Code) 2009 will be 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 may however be referenced where appropriate or where the IBC is thought to be less applicable. This may include the Eurocodes and local codes where appropriate.

Other references used:

- Standards referred to by IBC 2009 including: ASCE 7-10 (2010), NDS for Wood Construction, ACI 318 for Concrete, and AISC for Steel.
- UBC 1997 Volume 2 for preliminary wind calculations.

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 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.

[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.[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

  The resulting wind pressure on the structure was found to be 0.74kPa before modification by the relevant pressure coefficients.

- Modifying the wind pressure by the pressure coefficients gives an uplift pressure of 0.89kPa and a maximum lateral force on the structure of 11.5kN in the transverse direction. Local knowledge of higher wind speeds must be taken into account by using higher design pressures for specific shelter locations where necessary.

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[1] Proyecto de norma tecnica de edificacion, E.020 Cargas, December 2004. The method used in this guidance to calculate wind pressures is very similar to that used by the UBC and gives results that make those described above conservative.
[2] These wind speeds are extended from the Peruvian code. They represent research ito 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 I (Table 1.5-1 low risk to human life in event of failure) in Table 11.6-1	Seismic Design Category D
Importance factor assuming risk category I – Table 1.5-2	$I_e = 1.0$
Assume no codified seismic lateral system – Table 12.2-1[5]	R = 1.0

The equivalent lateral force procedure has been used to calculate horizontal loads for design. The resulting base shear is only 1.65kN due to light weight of the materials used.

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

1.9 Calculation Plan

1.9.1 Design Methodology

The performance of the 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 are made, assumptions and reasoning for lower factors of safety and alternative standards has been justified. Logical reasoning has therefore been used where necessary and upgrades suggested in order for the shelter to meet these criteria.

Assumptions:

- One structural form has been considered – the structure is enclosed and the roof and wall covering has sufficient strength to transmit wind loads to structural members without damage. It is assumed that since the structure has little lateral stability it will fail before the stapled plastic.
- The plastic sheeting is ‘hand-taut’ (not machine fixed) and therefore will not flap in the wind.
- Seismic loads will act on the structure from its own self-weight but wind loads will govern.

1.9.2 Structural Checks

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

2 Results of Structural Assessment

2.1 General key findings

I. All members perform adequately under vertical loads and seismic loads only. The wind loads expected on the structure greatly exceed those that can be taken by the building's lateral stability system.

II. There is no overall uplift of the structure due to overturning in the seismic case. Assuming a tie to the foundation at each column location, under wind loads the ties cannot take maximum uplift force of 4.5kN and number of ties should be increased. This will ensure that there is no uplift under wind loads.

III. Friction under the foundation slab is insufficient to prevent sliding under wind loads, but is sufficient under seismic loads. This assumes the structure is properly tied to the slab but under extreme wind loads more ties are required. A concrete foundation trench could be used as a shear key to prevent sliding.

IV. Bearing of column on to slab under dead and wind loads, and bearing of distributed dead and live loads are both acceptable.

V. Capacity of tension bracing under wind loading is exceeded; 4mm diameter wire or 10 wires per tie is required. Vertical column forces are acceptable but the central column bending strengths are exceeded under wind loads. The spacing of secondary wall and roof members should be decreased and some members increased in diameter to accommodate bending stresses and reduce deflections.

2.2 Other Hazards

- Eucalyptus wood is known to be resistant to termite attack but alternative protection methods should be employed if other wood types are used.

3 Conclusions and Recommendations

3.1 Assumptions

- A stiff soil type (see Site Class D, '2009 International Building Code', ICC, February 2009) has been assumed in analysis of the structure. Softer soil or soil of variable quality may adversely affect the performance of the shallow foundations in service. 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.
- It is assumed that under wind pressures the plastic sheeting will not tear therefore transferring wind forces to the structure. This requires a maximum distance between staples of approximately 150mm on all edges.
- The foundations consist of 8 ties with 10*10*100mm sticks embedded below the 50mm thick concrete slab. The slab has wire mesh reinforcement at 25mm depth and there are 4 wires providing resistance per tie point.

- The roof members are slender and can only support a minimal dead load. It is assumed that there are no additional roof loads such as volcanic ash, sand or snow.
- All connections are sufficient to transfer the required forces between members.
- The plastic sheeting is assumed to be 'hand-taut' (not machine fixed) and will not flap in the wind.

3.2 Conclusions

Performance Analysis	
The performance of the shelter under gravity and seismic loads alone is satisfactory. Under wind loads modifications are required to strengthen the shelter.	
Hazard	**Performance**
Earthquake – HIGH	The shelter attracts low seismic loads and its performance under these is adequate. The resistance of the shelter to lateral loads is low so damage is expected. However since it is lightweight and relatively 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. The structure must be more securely tied down to prevent uplift and the foundation size increased to prevent sliding. More bracing must be added in the walls and roof to provide sufficient lateral stability. Additional columns and roof members are also required.
Flood – MEDIUM	The flood risk increases during El Nino period every 10-15 years. The shelter does not incorporate any flood protection strategies so in the case of flooding the damage would be great.

Notes on Upgrades:

Upgrading the roof with materials of a similar weight, for example lightweight metal sheets would not change the structural performance of the shelter. In order to upgrade the roof or walls with heavier and more substantial materials, such as plywood, the frame member sizes would need to be increased, connections strengthened and foundations upgraded to take the increased gravity and seismic loads. Upgrading the shelter with masonry or other very heavy materials is not recommended as they attract high seismic loads causing the structure to perform poorly in an earthquake. Collapse of a heavy 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: Two double wire ties are required per column, cast into the foundation and tied to the structure to prevent uplift under wind loads.

B. CHANGE: Modify foundations to Type 3 or 4 (see C.1) to prevent problems with uplift/sliding under wind loads. For example the columns could be securely tied into embedded concrete pockets.

C. CHANGE: Use 8 columns to decrease the roof member spacing.

D. CHANGE: Increase bracing to 10 strands of wire on end faces and roof to provide adequate resistance to wind pressures.

E. CHECK: If the roofing is upgraded with a heavier material the roof member sizes must be increased accordingly.

F. CHECK: See Section I.1 for the correct concrete mix for the slab. A layer of mesh reinforcement to increase tie pull out resistance is required.

G. CHECK: In areas known to have high local wind pressures adequate foundations and member sizes must be provided to account for this.

H. CHECK: All member sizes, particularly central columns and bracing, should be increased in accordance with design to local wind pressures.

I. CHECK: The design and detailing of all connections is critical to the stability of the structure and should be checked for individual cases.

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

Appendix A – Source Information

1. 'Summary Information – Transitional shelter data sheet', CT & JA (IFRC), 27th September 2010.
2. 'Transitional Shelter Technical Details, Peru', CHF/IFRC, August 2007.
3. 'Modelo alojamiento temporal – Peru', IFRC/CRP, 2007.
4. 'Trans-shelter guidelines – Self help manual for temporary accommodation', SRC
5. 'Shelter KIT_Peru (English)', Bill of Quantities.
6. IFRC Hazard Assessmnet/Peru Chincha Province – Memorandum, 10th December 2010, Juliet Mian & Sasha Drozd (Arup).

Appendix B – Bill of Quantities

The table of quantities below 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	Material Spec.	Quantity	Total	Unit	Comments
Structure - Foundations					
Portland cement	Concrete	2	2	bags	42.5kg/bag
Sand/Gravel	Concrete	1	1	m^3	Estimate only[1]
Wire mesh reinforcement	-	18	18	m^2	
Main Structure					
Main columns (2m x 75mm dia.)	Timber 2	8	14.0	m	
Window column (1.6m x 75mm dia.)	Timber 2	1	1.6	m	
Beams (6m x 50mm dia.)	Timber 2	6	36.0	m	
Beams (5.1m x 50mm dia.)	Timber 2	2	10.2	m	
Beams (3m x 50mm dia.)	Timber 2	8	24.0	m	
Structure - Door					
Verticals (2m x 50mm dia.)	Timber 2	2	4.0	m	
Horizontals (0.9m x 50mm dia.)	Timber 2	3	2.7	m	
Covering – Wall and Roof					
Plastic sheet (4m x 6m)	Plastic		54	m^2	
Bamboo mats (2m x 3m)	-		54	m^2	
Fixings					
Galvanised AWG16 wire	Wire	130	130	m	Used in double lengths
Nails – 10d	Nails		3	kg	
Nails – 8d	Nails		2	kg	
Nails – 4d	Nails		1	kg	
Staples – 22/25	Staples	2000	2	box	
Hinge – 62.5mm steel	-	3	3	piece	
Knocker – 50mm steel	-	1	1	piece	
Padlock	-	1	1	piece	
Tools Required					
Hand saw	-	1	1	piece	
Shovel	-	1	1	piece	
Hammer	-	1	1	piece	
Pliers	-	1	1	piece	
Clippers	-	1	1	piece	
Wheel barrow	-	1	1	piece	
Industrial stapler	-	2	2	piece	
5m tape measure	-	1	1	piece	
7m plastic level pipe	-	1	1	piece	

[1] Quantities should be modified according to concrete specification (See I.1).

Appendix C

Calculation Plan

1) Loading

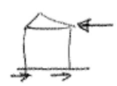
The members have been checked using allowable stress methods in accordance with the NDS for wood construction. The loads described in Section 1.8 have therefore been combined using the un-factored load cases described in the IBC (International Building Code) 2009, Section 1605.

2) Foundations

    a. Bearing pressure

    b. Uplift

    c. Base Shear



The effect of overturning must be included in the vertical force calculations.

3) Stability

    a. Overturning

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

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

4) Primary Members

Check members for a combination of vertical and lateral loads, including columns, beams and bracing. Check ground floor/foundation slab for dead and live loads.

5) Secondary Members

Check members for a combination of vertical and lateral loads, including roof joists and transoms. Check capacity of plastic sheeting and matting with current framing.

6) Fixings – check connections assuming pullout strength of nails in wood and tensile strength of tie wire. Connections will be assumed to be pinned, including at column bases. Key connections include the column base connection and the connection of the horizontal members to the columns.