GUIDELINES FOR POST-EARTHQUAKE TEMPORARY STRUCTURAL STABILISATION OF MONUMENTS OF BAGAN

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1 Introduction

The current document has been developed as Guidelines for Post-Earthquake Temporary Structural Stabilisation of Historical Monuments with specific reference to the archaeological monuments of Bagan, Myanmar. In addition, the document has been prepared for UNESCO in the aftermath of the **M 6.8** Chauk earthquake of 24th August 2016 that severely affected the Bagan region in Myanmar.

A disaster management cycle, as shown in Fig. 1.1, proceeds in a sequential manner with post-disaster ***emergency response***, followed by the ***recovery*** and ***restoration*** phase. These two activities are part of the immediate response to the event that has just occurred, and constitute ***managment*** of the disaster in the short-term to medium-term. This phase could extend typically from zero to five years, and in the case of historical monuments, even longer. The second phase involves activities aimed at ***mitigating*** risks of the identified hazard (earthquakes in this case) and building ***preparedness*** in the region in the medium- to long-term. Retrofit of structures and preparation of disaster management plans are carried out.

The current document addresses the mechanisms of ***post-earthquake safety evaluation*** and identification of potential risks of partial or complete collapse through rapid visual surveys and selection, design, detailing and execution of ***short-term counter measures*** for ***temporary stabilisation and strengthening*** aimed at mitigating the identified risks.

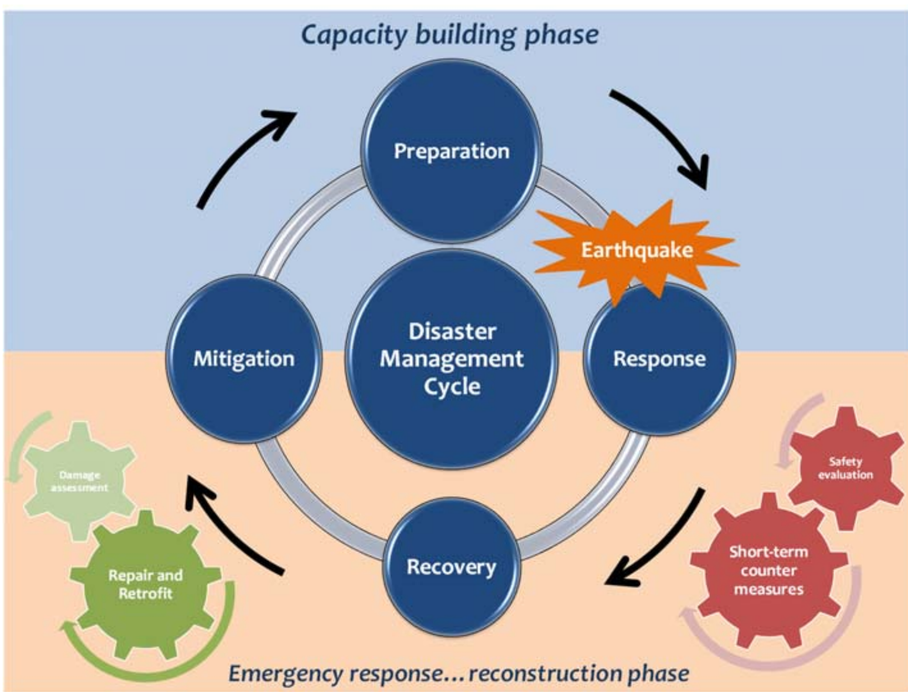


Figure 1.1 Disaster Management Cycle (after FEMA)

Emergency response as well as restoration and reconstruction efforts in seismic- prone areas is generally the responsibility of public authorities. The post-earthquake investigations are usually performed by means of building surveys. These assessments are of two types:

a) ***Post-Earthquake Safety Evaluation***

Post-Earthquake Safety Inspection and Evaluation is the technical activity performed after the earthquake to classify buildings according to their possible use in the short term. It is a quick and temporary, limited assessment, based on expert judgment, visual screening and on data easily collectable, aimed at detecting if the buildings damaged by the earthquake can be used during in the immediate aftermath of the earthquake, and in a period during which seismic aftershocks must be expected. The primary goals in this process are prevention of sudden collapse, thereby safeguarding human life and eventual complete or partial loss of the historical monument.

b) ***Post-Earthquake Damage Assessment***

Post-Earthquake Damage Assessment is the detailed technical activity performed after short-term counter measures are put in place to secure the structure. The aim of the activity is to accurately map the condition of the structure after the earthquake, and develop the basis for selection and design of repair and strengthening measures. Damage measurement includes two stages of measurement, namely subjective mearurements and objective measurements. Subjective measurement includes visual inspection and taxonomy which identifies the maximum damage grade, the most-widespread damage grade and the extent of each damage grade. Objective measurements involve usage of instruments and measurement of selected quantities such as, maximum or residual crack widths, displacements, accelerations, etc.

It is essential to establish the following before the occurrence of earthquakes:

- ***Methodology*** or workflow for safety and damage assessment.
- ***Forms*** for **safety evaluation** (level-1) and **damage assessment** (level-2).
- ***Criteria and procedures*** of risk elements removal, temporal support and temporal shoring for ***short- term counter measures***.

The current document provides inputs on the methodology for safety evaluation, a model form of safety evaluation (level-1) and criteria and procedures for short-term counter measures. In the following sections, an overview of the structural typology classification of monuments encountered in Bagan is provided (Section 2) followed by a discussion on the most recurrent seismic damage observed in these structures (Section 3). These sections would provide the basis for the selection of the short-term counter measures.

2 Structural Typology Classification

Pierre Pichard's exhaustive Inventory of Monuments at Pagan (Pichard, 1992), published by UNESCO in eight volumes, has been the basic source for the classification of structural typologies for an understanding of their earthquake behaviour, and subsequent selection of short-term counter measures. Broadly, monuments of Bagan can be classified into three categories, viz. ***temples***, ***stupas*** (see Fig. 2.1 for a typical stupa) and ***monasteries*** (see Fig. 2.2 for a typical monastery) Most of the monuments are constructed in burnt clay brick masonry with mud or lime mortars, with the use of stone in construction rather limited. Reinforced concrete and steel structural elements have been used in post-earthquake reconstructions and repairs (after the 1975 earthquake).

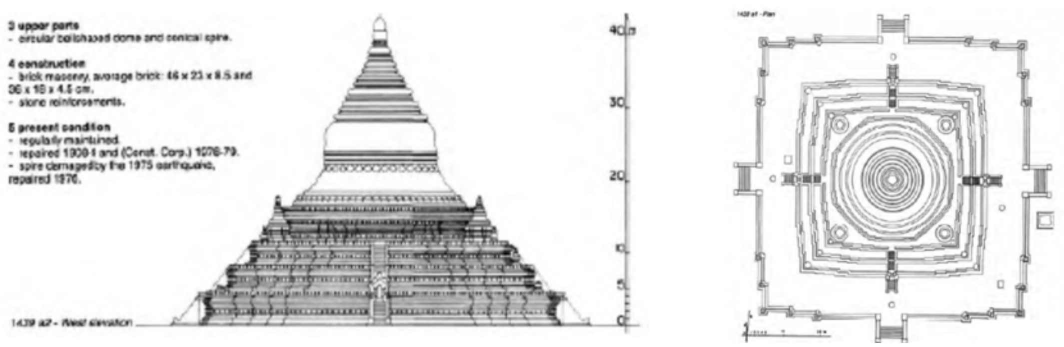


Figure 2.1: Elevation and plan views of a typical Stupa (Pichard, 1992)

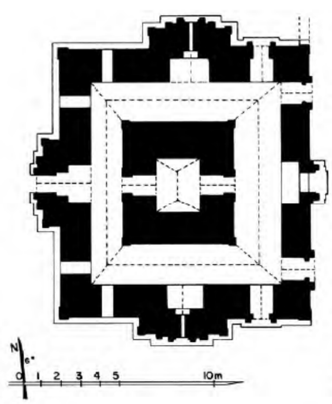


Figure 2.2: Plan of a typical Monastery (Pichard, 1992)

Based on their ***plan configuration***, the temples can be classified as:

1. **Temple with central shrine:** These structures are typically smaller in scale, but with their central spire resting directly on the main vault (see Fig. 2.3).
2. **Temples with central shrine with corridor around:** Larger temples with central shrines had corridors running concentrically around the central shrine (see Fig. 2.4). However, even in this configuration, the central spire rests directly on the main vault.

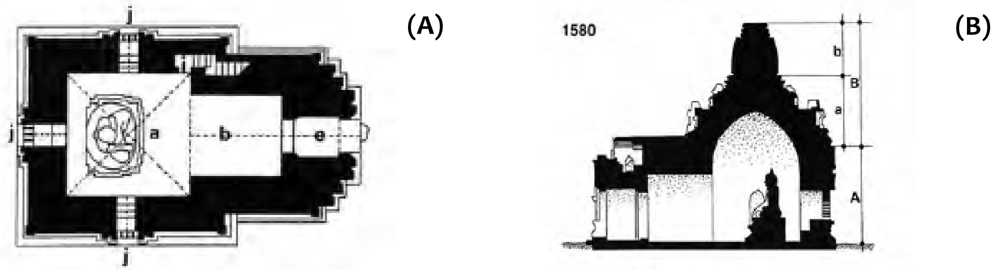


Figure 2.3: Plan and Sectional Elevation of a Temple with a Central Shrine (Pichard, 1992)

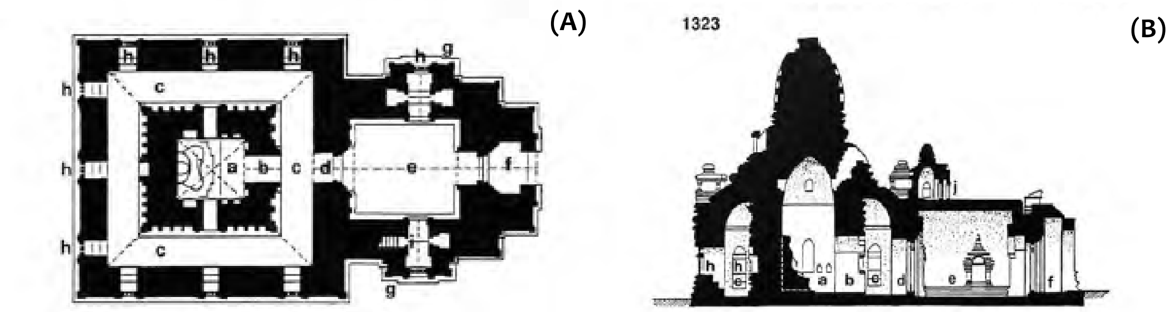


Figure 2.4: Plan and Sectional Elevation of a Temple with a Central Shrine and Corridor Around (Pichard, 1992)

3. **Temples with solid core:** As shown in Fig. 2.5, large temples with prominent central spires or those constructed with multiple levels were provided with a central solid core that ensured vertical continuity of the central spire and a more stable configuration. However, in some configurations, the solid central core terminates in the lower storey, while in the upper storey, the central spire is supported on the vaulted roof. In these configurations, there is significant lateral stiffness irregularity due to the termination of the solid core (see Fig. 2.6-2.7).

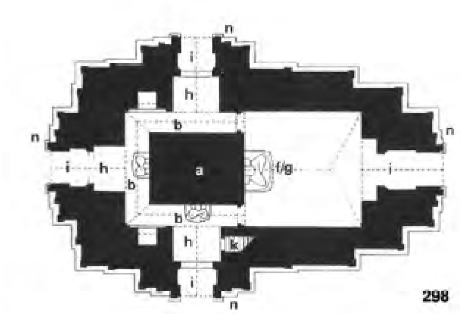


Figure 2.5: Temple with a Central Solid Core (Pichard, 1992)

Based on the **vertical configuration**, the temples can be classified as:

1. **With continuous or discontinuous solid core:** As discussed earlier, the vertical continuity of the solid core, provided greater stability to the structure due to the continuous load path and lesser stiffness irregularity along the height. Significant differences in seismic performance is noticed when the main central spire rests on the solid core, as against on a cross vault (see Fig. 2.6).

2. **Single-storied:** Many of the shrines were limited to a single storey (see Fig. 2.3).
3. **Two-storied or multi-storied:** Temples with two or more storeys typically were provided with the central solid core owing to the scale of the structure (see Fig. 2.6 and 2.7). The multi-storied varieties are massive and attract large inertial forces under seismic action.

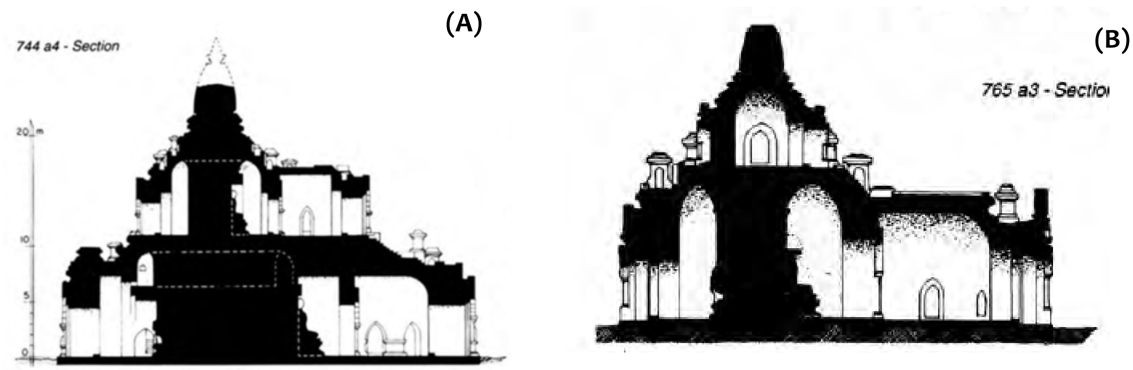


Figure 2.6: Sectional Elevations of Temples with a Central Solid Core (A) Continuous and Supporting the Central Spire; (B) Limited to the Ground Storey (Pichard, 1992)

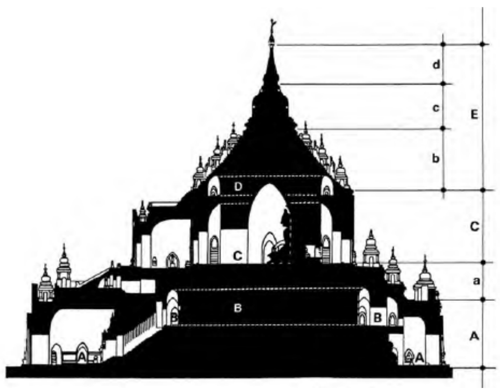


Figure 2.7 Sectional Elevation of Multi-Storied Temple with a Discontinuous Central Solid Core (Pichard, 1992)

Both ***corbelled*** and ***"true arch" vaulting construction systems*** have been used in the Bagan monuments. The corbelled system typically was adopted to construct the bulky structural layer of the vault, and the true arch system of vaulting were adopted apparently in the ornamental layers that were used to provide an aesthetic facing to the otherwise rugged construction of the vault by corbelling. The true arch vault was constructed in one or more independent layers, and often maintaining gaps between the layers and the corbelled vault.

As shown in Fig. 2.8, both cross vaults and barrel vaults were common with varying angles of repose depending on the space to be spanned. The vaults were supported on walls in square, rectangular or circular configurations. Cross vaults were either supported on all walls as seen in Fig. 2.8 A, or barrel vaults had crossing on one end as seen in Fig. 2.8 B. Barrel vaults in the corridors were either complete, three-quarter or half in the vaulting (see Fig. 2.8 D). Crossing of the vaults could either be at equal heights or unequal heights (see Fig. 2.8 E and F). The lateral load behaviour of the vaults varied with different support conditions, spans, aspect ratios and interaction with adjacent vaults.

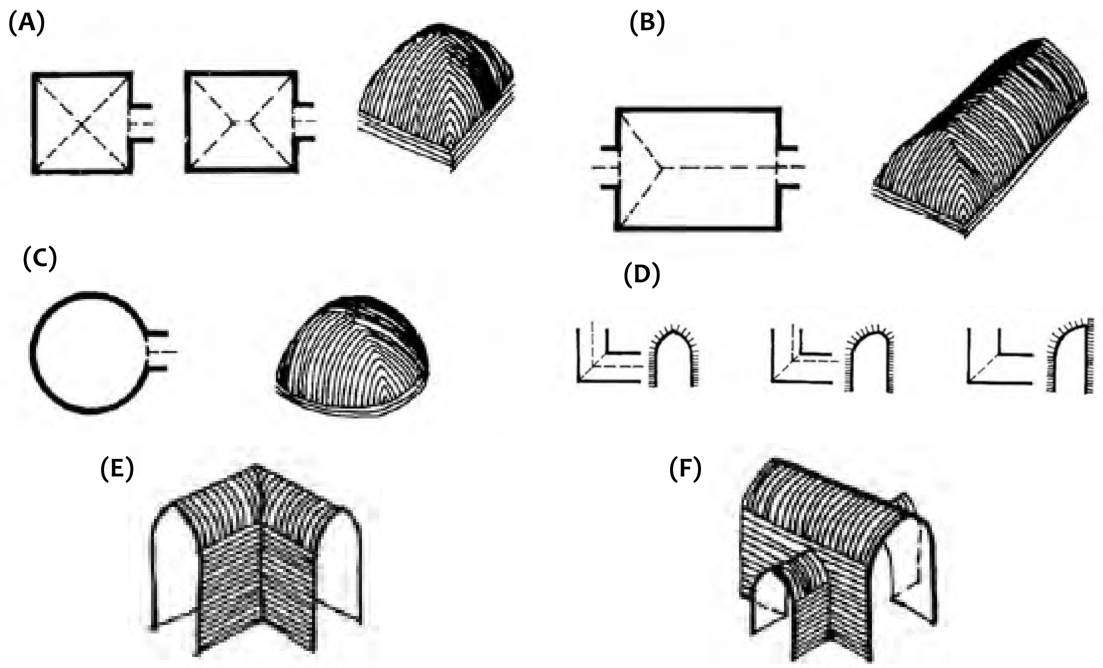


Figure 2.8: (A) Cross Vaults Supported on Square or Rectangular Wall Configurations; (B) Barrel Vaults with Crossing on One End; (C) Vault Supported on Circular Wall Configuration; (D) Configurations of Full, Three-Quarter to Half Vaults in Corridors and their Crossing at Right-Angles; (E) Crossing of Equal Height Vaults and (F) Crossing of Unequal Height Vaults (Pichard, 1992)

3 Overview of Seismic Damage

Every earthquake is unique and presents new learning on the earthquake mechanisms and on structural behaviour of buildings. They provide insights on the desirable and undesirable structural features in the buildings of a region. It is this understanding that provides the basis for structural interventions aimed at restoration and seismic retrofit in historical monuments, as they would for modern seismic design guidelines in new constructions. The recent Chauk earthquake in 2016 was of a lower intensity than the earthquake of 1975, yet affected several hundreds of monuments in Bagan to varying degrees of damage and distress. The following points provide a summary of the important lessons from the earthquake effects:

1. **Absence of Effects due to Aftershocks**: Since the earthquake mechanism in the region is governed by a deep subduction process, there have been no perceptible aftershocks. This has clearly been a favourable scenario for the emergency stabilisation period. However, this should be no basis for delaying or avoiding short-term counter measures aimed at temporary stabilisation and strengthening of the affected structures. The monuments have been weakened in the earthquake, and hence are vulnerable to partial or total collapse, particularly in portions where failure mechanisms have formed, and the structure is in a ***state of unstable equilibrium between the failure mechanism and collapse***. In such situations, elements triggering collapse could vary from temperature variations, rainfall and related water seepage or incorrect counter measures.
2. **Predominant Damage Mechanisms:** Predominant seismic damage is related to first mode mechanisms, otherwise known as local or out-of-plane mechanisms, which are notorious in historical masonry structures. These mechanisms have particularly affected the main or subsidiary spires, parapet walls along stairways of pagodas, and pediments on the façade of vestibules, all of which are free-standing or cantilevered structural elements. These mechanisms do not compromise the global stability of the structure, but are capable of causing grievous structural damage to other parts of the monument due to impact as they fall from considerable heights.
3. **Moderate Intensity of Earthquake:** With a moderate intensity in the current earthquake, i.e. Modified Mercalli Intensity (MMI) of V at Bagan, many of the monumental structures in brick masonry have remained almost elastic, thereby leading to increased acceleration and displacement demand on free-standing and non-structural components, due to amplification. The amplified response in the spires, parapet walls and pediments is the evidence to this behaviour.
4. **Global Damage Mechanisms:** Second mode mechanisms, otherwise referred to as global mechanisms are limited, with shear cracks in walls noticed in few places, and cracks due to support movement of vaults or due to pounding of the spire on the vaulted roofs. The latter is prominently observed, possibly due to the significant

vertical component of the ground shaking in the current earthquake. An understanding of the geometry and construction system of the vaults is essential to correctly define the observed damage. A correlation between their geometry and damage will be necessary to verify strategies for their protection. However, the damage to the vaults is complicated by previous damage or interventions. Dilation of masonry walls is another recurrent seismic damage observed in the earthquake.

5. **Role of Previous Repair Interventions:** In most cases of damage to spires, newly constructed portions in cement or lime mortar have remained intact with crushing or sliding occurring at the interface with the original construction in brickwork with mud mortar. In a few cases, Sula-mani-gu-hpaya (Monument no. 748) for example, this is complicated by the presence of RC elements at such interfaces. Availability of the detailed documentation of previous repair and seismic strengthening interventions, carried out in response to the 1975 earthquake that affected the region, are essential in comprehending the response of the structures to the current earthquake. In addition, verification on the ground whether the proposed strengthening interventions post-1975 were fully implemented is crucial. In several cases, undesirable earthquake behaviour has been observed where the entirety of the proposed interventions were not executed, or affected by poor workmanship.

A classification of predominant damage mechanisms in different elements are reported in the ensuing pages, before appropriate counter measures are identified in each scenario. Predominant ***local collapse or damage mechanisms*** observed can be enumerated as follows:

1. **Damage to spires:** Damage is caused due to amplified accelerations and displacements at higher elevations in the structure (see Fig. 3.1). Sliding failure has been observed in smaller spires (see Fig. 3.2B), whereas heavier and larger spires show evidence of shear cracking (diagonal tension cracks) as seen in Fig. 3.3 A and flexural crushing or compression failure due to rocking as seen in Fig. 3.2 A.
2. **Damage to pediments at entrance gateway to vestibules:** Overturning and partial collapse of free-standing elements such as pediments at entrance gateway to vestibules is a recurrent damage mechanism (see Fig. 3.3 B, 3.4 A and 3.5 A).
3. **Damage to parapets:** Overturning and partial collapse of free-standing elements such as parapets walls is a recurrent damage mechanism (see Fig. 3.4 B and 3.5 B).

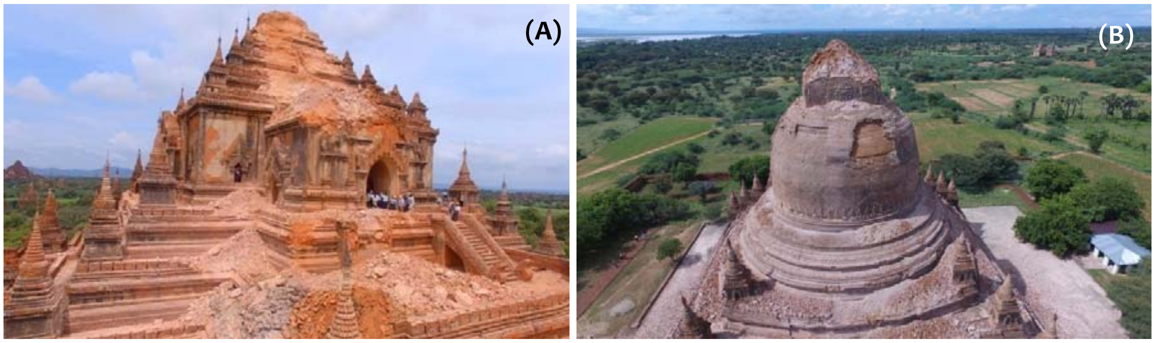


Figure 3.1: Collapse of spire

Figure 3.2: (A) Crushing of masonry due to flexural rocking at the base of the pinnacle; (B) Shear sliding of the pinnacle of the spire

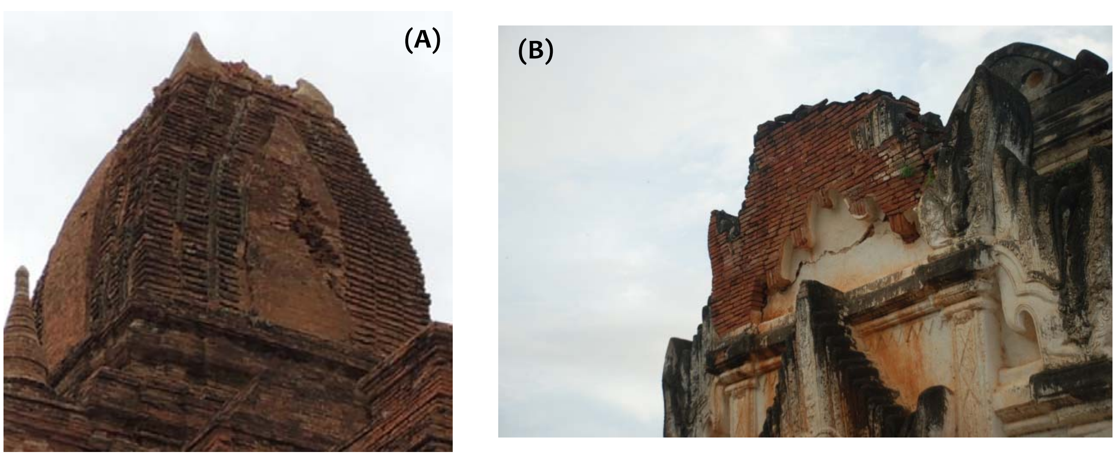


Figure 3.3: (A) Shear damage to the base of the spire; (B) Detachment and overturning of the free-standing pediment

Figure 3.4: (A) Detachment and overturning of the free-standing pediment; (B) Detachment and overturning of parapet walls along the stairway

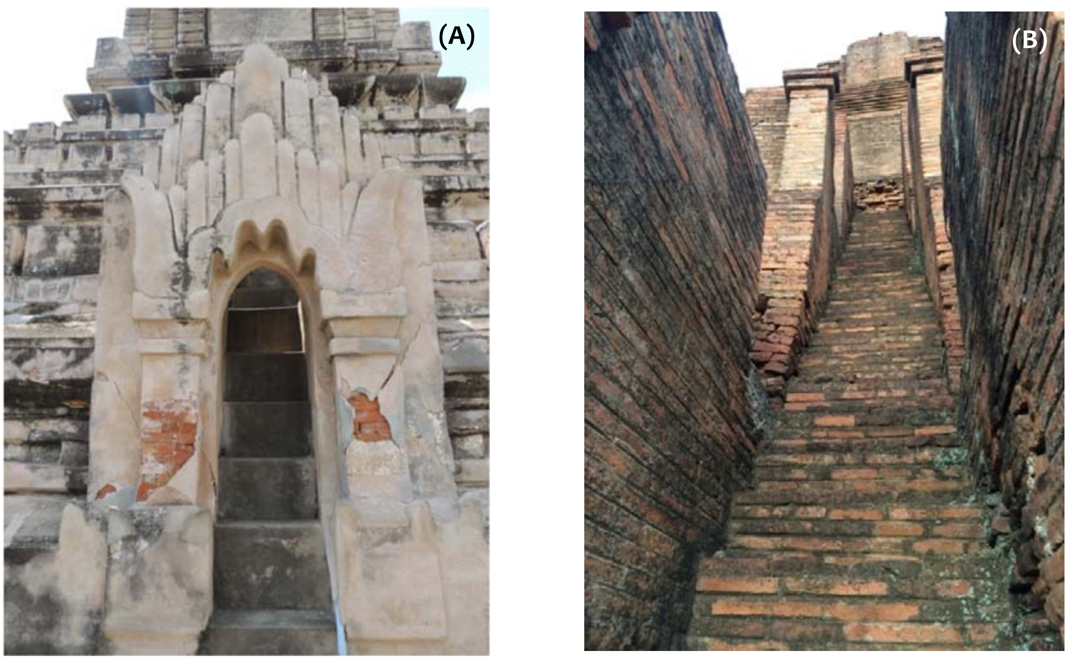


Figure 3.5: (A) Shear cracking at the supports of the entrance structure to the stairway; (B) Overturning of parapet walls along the stairway

4. **Damage due to pounding from falling debris:** Significant damage has also been observed due to the impact of massive brick masonry blocks of the spire or other free-standing structural elements (e.g. pediments) on other roof structures such as vaults (see Fig. 3.1).

5. **Overturning of facade:** Detachment and partial or total overturning of the facade wall at the entrance to the vestibule (see Fig. 3.6 A). This is primarily due to poor connection of the facade of the vestibule to the rest of the structure.

   Significant ***global damage mechanisms*** can be enumerated as:

1. **Dilation of masonry walls:** Outward movement of masonry walls due to absence of a rigid diaphragm action (horizontal connectivity between walls) and poor

interconnections between the walls, leading to partial overturning and bulging of walls and formation of vertical cracks (see Fig. 3.6 B). In turn, such dilation of the masonry walls results in formation of separation in the true arch portion of the vaults, particularly when they are constructed in distinct layers without interlocking (see Fig. 3.7 A), and dislodgement of bricks (see Fig. 3.7 B and Fig. 3.8 A).

2. **Damage to vaults:** Vaults are damaged either due to dilation of the masonry walls, leading to lateral movement of the supports of the vaults. Dislodgement of brick units or voussoirs in the true arch vaults is a direct result of the dilation of the walls (see Fig. 3.7 B and Fig. 3.8 A). Damage to vaults could also occur due to pounding (reversed cyclic vertical action) of the heavy spires on the supporting vaults. This is particularly expected when the structure does not have a solid core.
3. **Damage to solid core:** Shear damage to the solid core with the formation or diagonal cracks can compromise the vertical load carrying capacity of the core. Damage to the walls of the core could also occur due to pounding (reversed cyclic vertical action) of the heavy spires on the supporting solid core with bulging or the walls and formation of vertical cracks (see Fig. 3.8 B).

Primary behaviour of structures and counter measures required are summarised as follows:

1. ***Dilation of masonry walls*** due to poor interconnections between walls and poor diaphragm action, making them vulnerable to out-of-plane deformations and overturning. Counter measures require ***provision of confinement and tying of walls at critical locations along the height***.
2. ***Loss of true arch mechanism*** in the ***vault with dislodgement of bricks or voussoirs*** due to dilation of masonry walls or due to vertical impact. Counter measures require ***provision of confinement and tying of walls*** at critical locations along the height to prevent further lateral movement of supports. Damaged sections of the ***vault*** also have to be provided with ***vertical supports or props***.
3. ***Dilation and bulging of solid core walls***, or shear damage with loss of vertical load carrying capacity. ***Circumferential confinement*** of the solid core can restore or enhance its shear and axial load capacity.
4. ***Overturning of free-standing elements*** (e.g. pediments, small spires, parapets, etc.) including facade walls would require ***lateral bracing*** to prevent further overturning.

4. **Figure 3.6: (A) Detachment and overturning of the facade; (B) Dilation of masonry walls and formation of vertical cracks**

Figure 3.7: (A) Damage to vault due to dilation of masonry walls; (B) Deformation of brick masonry vault and dislodgement of bricks

Figure 3.8: (A) Deformation of brick masonry vault and dislodgement of bricks; (B) Bulging and cracking in the solid core supporting the central spire

4 Need for Short-Term Counter Measures

Short-term counter measures, with the goal of risk reduction, allow buildings to become usable in the immediate post-event period, and hence, speed up the restoration of the normal everyday life.

In the risk reduction, there is no intention to return to the initial safety level of the building; this would be achieved after final interventions and strengthening measures are adopted in the detailed assessment and restoration and/or retrofit phase. Reduced safety levels are assumed as the risk in case of aftershocks are to be reduced and not for the design earthquake. Their validity is limited to only the post-event period, and they have to be replaced, as soon as possible, with long term counter measures.

The aims of short- term counter measures are:

1. Reduce the risk of citizens in public areas;
2. Reduce the risk of citizens in private buildings; and
3. Preserve the building from further damage in case of historical heritage structures.

Figure 4.1: Aims of short-term counter measures (A) Reducing risk of citizens in public places, (B) Reducing risk of citizens in private buildings, (C) Preserving heritage structures against further damage

5 Workflow for Short-Term Counter Measures

Departments involved in coordinating and executing the post-earthquake emergency interventions must have a practicable ***methodology*** or ***workflow*** (sequence of activities) in place. Such a workflow has to be clearly identified in the period prior to the occurrence of a natural disaster, and constitutes the activity aimed at disaster risk mitigation. It is essential that necessary drills are performed time to time to ensure that the workflow is achievable without any roadblocks, and required changes are brought to the system to ensure smooth progress of short-term counter measures.

An important aspect to be recognised is that any short-term intervention on the historical monument aims at minimising potential losses of human life and property in the aftermath of the natural disaster. Any wrong interventions (inappropriate type of intervention, inadequate design or improper execution), due to errors of omission or even errors of commission, can potentially lead to loss of life, grievous injury and loss of properties of historical value. Hence, there are ***liabilities*** involved. This calls for sufficient ***skill and training of personnel*** directly involved in the process of post-emergency safety evaluation and interventions.

Particularly, there is the indispensable requirement of ***qualified structural engineers*** in different stages of the process. Such personnel should have a deep understanding of the structural typology at hand, earthquake effects on structures and have prior training in design and execution of short-term counter measures.

In the following table, a workflow compatible with the current procedures followed by the Department of Archaeology in conjunction with the Ministry of Construction is outlined, specifically with the expected skill level of each personnel involved.

Table 5.1: Workflow for Temporary Structural Stabilisation and Other Temporary Interventions

S. N.	ACTIVITY	PERSONNEL	SCOPE	EXPECTED SKILL LEVEL	OUTCOME
1.	**Initial field evaluation**	Engineers, Archaeologists, Architects (DoA, MoC, National and International experts)	Reconnaissance survey and/or rapid visual survey (RVS); Identification of broad issues for stabilisation	Expertise/prior experience in post-earthquake assessment and rehabilitation of historical monuments	Identification of criticality: R-Y-G tagging; RVS form; Photo documentation of damage
2.	**Detailed field assessment**	Engineers and architects (DoA, MoC, National and International experts available for extended period)	Detailed damage recording; Identification of critical zones/issues for stabilisation	Team (of min. 2 persons) should have a structural engineer; rest of the team can be given training prior to field process	Detailed survey form (with measurements, if possible); Identification of locations for temporary stabilisation; Identification of level of difficulty; Level of risk; Identification of necessary scheme of temporary support
3.	**Design of temporary stabilisation**	Structural engineer; engineer; architect; draughtsman (DoA, MoC)	Preparation of schematic (conceptual) drawings; Preparation of design calculations (selection of material; dimensioning); Preparation of CAD drawings; Preparation of specifications and method statement details; Preparation of quantity estimate (BOQ)	Architect (scheme) Structural engineer (design); draughtsman (drawings); Specifications and BOQ (engineers)	Schematic drawings (for iterations); Detailed working drawings; Technical specifications; Bill of quantities; and Cost estimate

4.	**Verification of temporary stabilisation scheme**	DoA, MoC (Executive Engineer/Chief Engineer) National/International experts	Approval of selected scheme and details for a given monument	…	Verification; Alterations; Green signal for execution
5.	**Procurement or mobilisation of resource**	DoC/MoA (Executive Engineer/Chief Engineer)	Mobilisation of resources for execution (need not be monument by monument)	…	Field execution ready
6.	**Execution of temporary stabilisation**	Site engineer (DoA and MoC) Labour	Execution; Quality control and quality assurance; Adherence to technical method statement	Supervisory engineer must have prior experience; Labour to be trained or oriented in field	Temporary structural stabilsiation and other temporary interventions (e.g. weather protection) put in place
7.	**Verification of temporary stabilisation execution**	DoC/MoA (Executive Engineer/Chief Engineer)	Acceptance of executed temporary stabilisation		Completion of process
8.	**Monitoring of stabilsiation**	DoC/MoA engineers	Performance of temporarily stabilised structure	Supervisory engineer (ideally member of implementation team)	Alterations, if necessary

6 Precautions for Seismic Safety Inspection

6.1 Personal Protective Equipment (PPE)

Adequate protective equipment shall be used by the safety inspection members to minimise any risk to their health or safety. Members accessing the affected structure should be equipped with hard hat, metal-edged safety shoes, safety cords, reflective jackets and whistle.

Figure 6.1: Personal Protective Equipment (PPE)

6.2 Personal Safety Protocol

Prior to the access of any earthquake affected structure, it should be visually examined from the exterior to identify any potential danger of further collapse or falling objects. Entry should be made only through those parts of the structure where it is safe against falling hazards. The team should comprise of at least one member who stays outside the structure keeping an account of the number of people accessing the structure. Entry to the structure shall be made only as a team or at least in pairs with adequate personal safety gear. A staff may be used to tap on the floor prior to stepping to identify the presence of any loose floor elements or hollow space beneath. In the event of tagging the structure, the tag should be placed suitably at conspicuous locations. Unauthorized persons should be denied access to the affected area.

6.3 Use of Barriers and Signage

Unsafe areas should be cordoned off by means of barricading or by providing signage. Barricading is very useful in risk reduction as it prevents access of unauthorized persons and those without adequate protection and awareness of the risks to the unsafe areas. It also helps persons keep off falling hazards. Barricades also help secure pre-designated pathways important for the rescue workers. Signage must be provided in local and foreign language/s to inform trespassers of the risk involved.

Figure 6.2: Use of Barriers and Signage

In case of imminent collapse, ensure that a circular area of radius equal to height of the structure, around the structure, be cordoned off from public access. This area is normally the extent of debris fall in ancient brittle masonry structures (see Fig. 6.3).

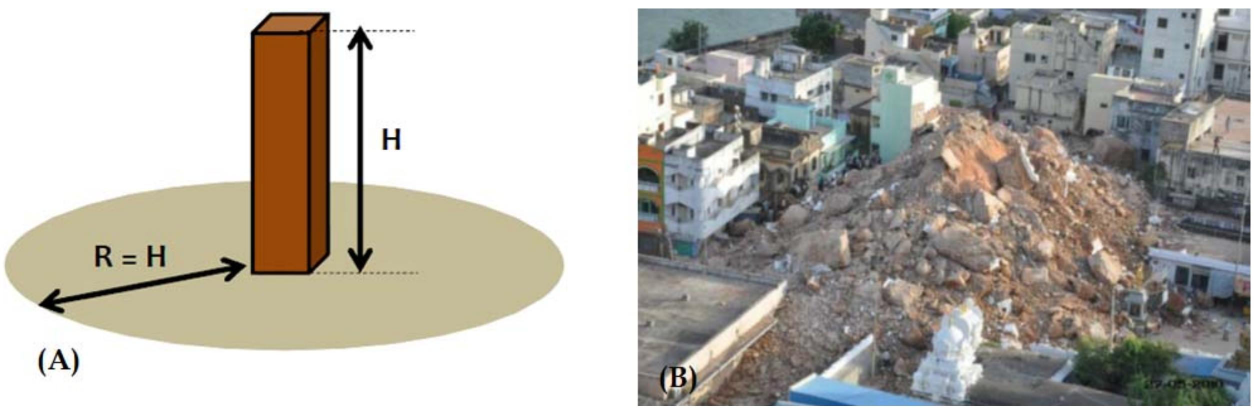


Figure 6.3: (A) Circular Area with Radius Equal to Height of the Structure to be Cordoned off (B) 7-Storied Temple tower collapse in Srikalahasti, India

6.4 Removal of falling hazards

Unstable elements should be identified and removed by controlled demolition if possible, when they pose a falling hazard to passersby and to inspection and rescue teams. This is with particular reference to non-structural components, such as overhanging portions of roofs, tiling, chimneys, turrets/pinnacles/merlons, projecting balconies and other light cantilevered structural components that have been damaged.

In case of heavy damage, the whole building can be hazardous. Such buildings must be distinguished from other buildings as their repair and restoration is not be feasible.

Figure 6.4: Falling Hazards

7 Rapid Visual Inspection for Post- Disaster Seismic Safety

Prior to accessing any earthquake-damaged structure, a Rapid Visual Screening (RVS) from the exterior of the structure has to be carried out for hazard identification. A damaged structure may be 'at rest' but does not mean it is 'stable'. What remains after the triggering event may have come to rest, but the danger of further collapse or falling objects is present.

The ***objectives of RVS*** are as follows:

1. To ***identify the condition of the structure*** and ***tag the structure*** as:
    a. Safe to access;
    b. Unsafe without sufficient precautions; or
    c. Marked for demolition.
2. To ***gauge the accessibility*** of the structure ***and identify access route/s***;
3. To ***identify*** potential ***falling hazards***; and
4. Carry out adequate ***photo-documentation*** from exterior.

The Rapid Visual Screening should be carried out ideally by a three-member team, where an ***RVS Form*** should be filled up individually by two members, and verified by the third. This is to ensure that there is broad consensus on the type of tag appropriate for the building, but by independent, unbiased examination. RVS of a building should not take more than 15-30 minutes.

It does not guarantee safety of the structure in a future event comparable to the main shock, but only addresses safety under existing gravity loads and small aftershocks expected in the post-earthquake period. The most critical question that the RVS addresses is whether or not the vertical load path in the building for gravity loads is intact. ***Imminent danger of partial or complete collapse of structure due to an aftershock*** (or even otherwise) ***must be considered***.

Based on the responses to questions on specific structural damage, a building is tagged GREEN, implying usable, YELLOW, implying usable with necessary temporary interventions, and tagged RED, implying unusable in the immediate post-earthquake period. It is noted that a RED tag does not imply demolition; this would require further detailed evaluation, which may also lead to strategies for salvaging the structure.

A ***sample RVS form*** for ***historical masonry structures*** in the Bagan Archaeological Zone is proposed here. Similar reference RVS forms for masonry and reinforced concrete structures prepared for safety evaluation in the aftermath of the Nepal earthquake of 2015 are provided in Appendix-2.

Post-Earthquake Rapid Visual Screening Form
Historical Masonry Buildings in Bagan Archaeological Zone

Inspection Details

Inspector ID: *Inspection Date and Time:* *Earthquake: Chauk, 24th August 2016*

Organisation: *Areas inspected:* ☐ *Exterior only* ☐ *Exterior & Interior*

Monument Identifier

Monument Name: *Monument Number:* *GPS Coordinates:*

Reconstructions: ☐ *Partial* ☐ *Complete* *Seismic Retrofit:* ☐ *Yes* ☐ *No*

Type of Construction

☐ *Stone masonry* ☐ *Brick masonry* ☐ *Brick masonry with stone cladding*

Mortar type: ☐ *Mud mortar* ☐ *Lime mortar* ☐ *Cement mortar*

Brick type: ☐ *Burnt clay* ☐ *Sun-dried clay*

Architectural Configuration

☐ *Structure with solid core* ☐ *Without solid core:* *Continuous over height* ☐ *Discontinuous* ☐

☐ *Single-storied* ☐ *Two-storied* ☐ *Multi-storied*

Type of Floor/Roof ☐

☐ *Masonry Vault* ☐ *Timber* ☐ *Timber with mud plaster* ☐ *RC Slab*

Primary Occupancy

☐ *Temple* ☐ *Stupa* ☐ *Monastery* *Others:* ____________________

☐ *Used for religious activities* ☐ *Used only for tourist purposes*

S.no.	***Structural Safety Parameters***			***Tag***
1.	***Site***	1.1	*Ground failure: a. Landslide/Fissures, b. Liquefaction, c. Tilt*	Red
		1.2	*Unsafe/tilted adjoining or uphill building*	Red
2.	***Out-of-plane failure***	2.1	*Separation of walls at junctions/corners, dilation of walls*	Red
		2.2	*Floor-wall junction separation, with wall out-of-plumb*	Red
		2.3	*Gable collapse*	Red
		2.4	*Separation of wythes*	Red
		2.5	*Damage to masonry plinth*	Red
3.	***In-plane damage***	3.1	*Horizontal sliding at spire base, or any storey*	Red
		3.2	*Diagonal shear cracking in wall piers, spandrels, spire base*	Red
		3.3	*Crushing of masonry at wall base or spire base*	Red
		3.4	*Bulging of solid core with formation of vertical cracks*	Red
4.	***Vaults***	4.1	*Dislodged voussoirs and/or severe vertical deflection of vault*	Red
5.	***Columns***	5.1	*Crushed or out-of-plumb column*	Red
6.	***Out-of-plane damage***	6.1	*Overturning of small spire/s*	Yellow
		6.2	*Pediment or parapet collapse*	Yellow
7.	***Vaults***	7.1	*Minor deformations in vault*	Yellow
8.	***Material***	8.1	*Disintegration of masonry constituents: Unit, mortar or assembly*	Yellow
9.		None of the above		Green

Tagging

GREEN	***YELLOW***	***RED***
Usable	*Usable with temporary interventions*	*Unusable*
	Suggested temporary interventions:	

Further Actions: *Detailed Visual Screening Recommended* ☐

Areas to be barricaded:

8 Logistics for Debris Removal

In several monuments, the situation of the monument after the earthquake was rendered highly precarious due to the falling hazard of endangered massive portions of the structure, mostly at its pinnacle. As shown in Fig. 8.1, in several cases of pinnacles in stupas, structural safety was compromised due to extensive damage to the base of the pinnacle portion. A similar situation was encountered at the base of central or subsidiary spires in temples.

The complication in all these cases was due to the size and weight of the damaged portion, the position in which these were located and the requirement of removal of these large pieces of debris.

Figure 8.1: Structures with Heavily Damaged Portions Requiring Careful Removal

The complication in all these cases was due to the size and weight of the damaged portion, the position in which these were located and the requirement of removal of these large pieces of debris. A quick ***feasibility study of using an automated crane*** (see Fig. 8.2 A) was undertaken for possible procedure to be adopted for removal of the large compromised portions of the structure of Monument no. 1439 - Mingala-zedi.

The affected portion of the pinnacle is approximately 1.75 m in diameter, 3.5 m height at the top and approximately, 3.0 m in diameter and 2.0 m height immediately below (see Fig. 8.2 B and 8.3). The volume (assuming cylinders: *pr2h*) works out to approximately 43 metric tonnes.

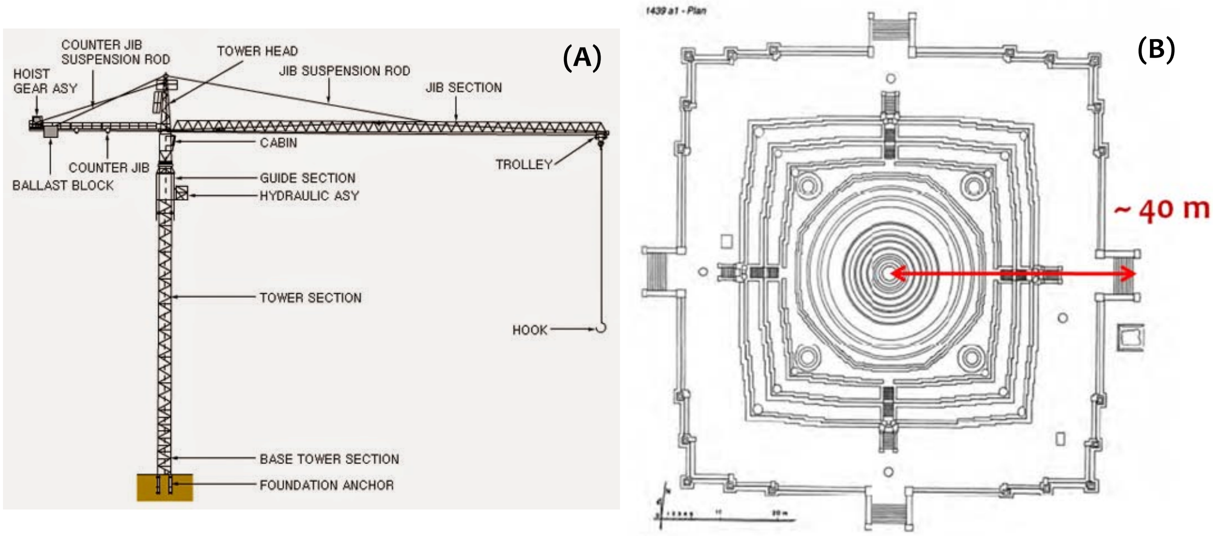


Figure 8.2: (A) Parts of a Tower Crane; (B) Plan of Monument no.: 1439 - Mingala-zedi

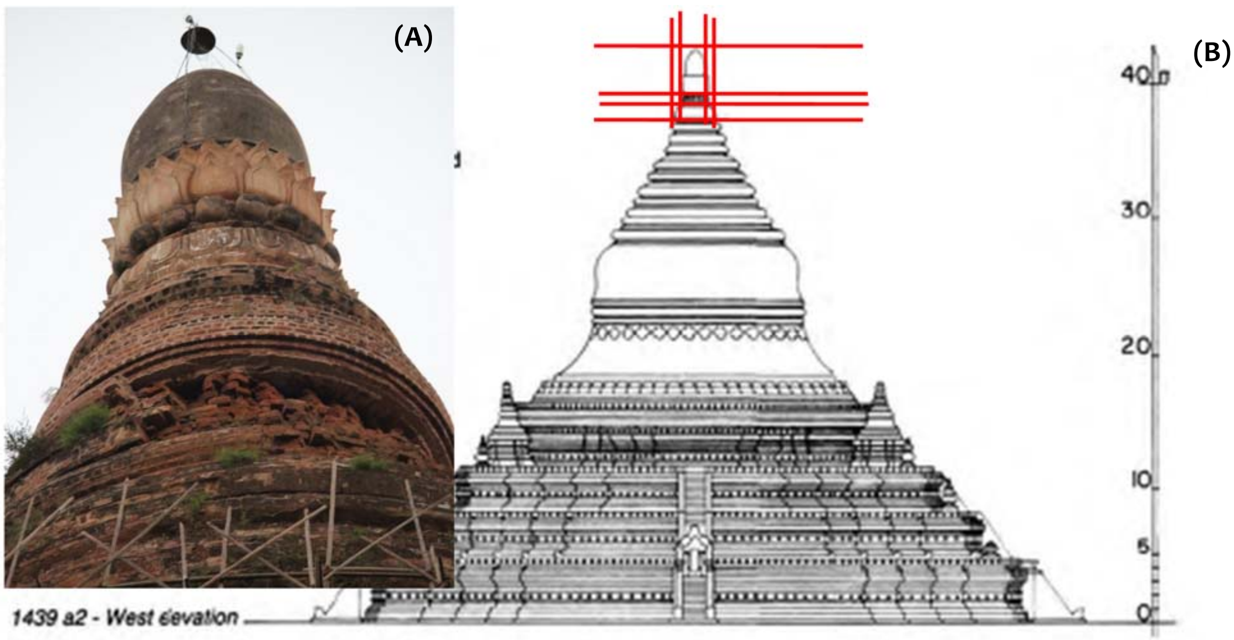


Figure 8.3: View of Pinnacle of Monument no.: 1439 - Mingala-zedi and elevation of the structure

Considering the height of the structure and the base dimensions, the required tower crane height would be a minimum of 45.0 m with a minimum jib length of 45.0 m. With these constraints, a standard 18T tower crane would not be able to handle more than 5-7 T at a time (see table in Fig. 8.4). This essentially would render the use of the tower crane impractical, as it would call for slicing of the precarious mass into smaller blocks with wire cutter and then handling them.

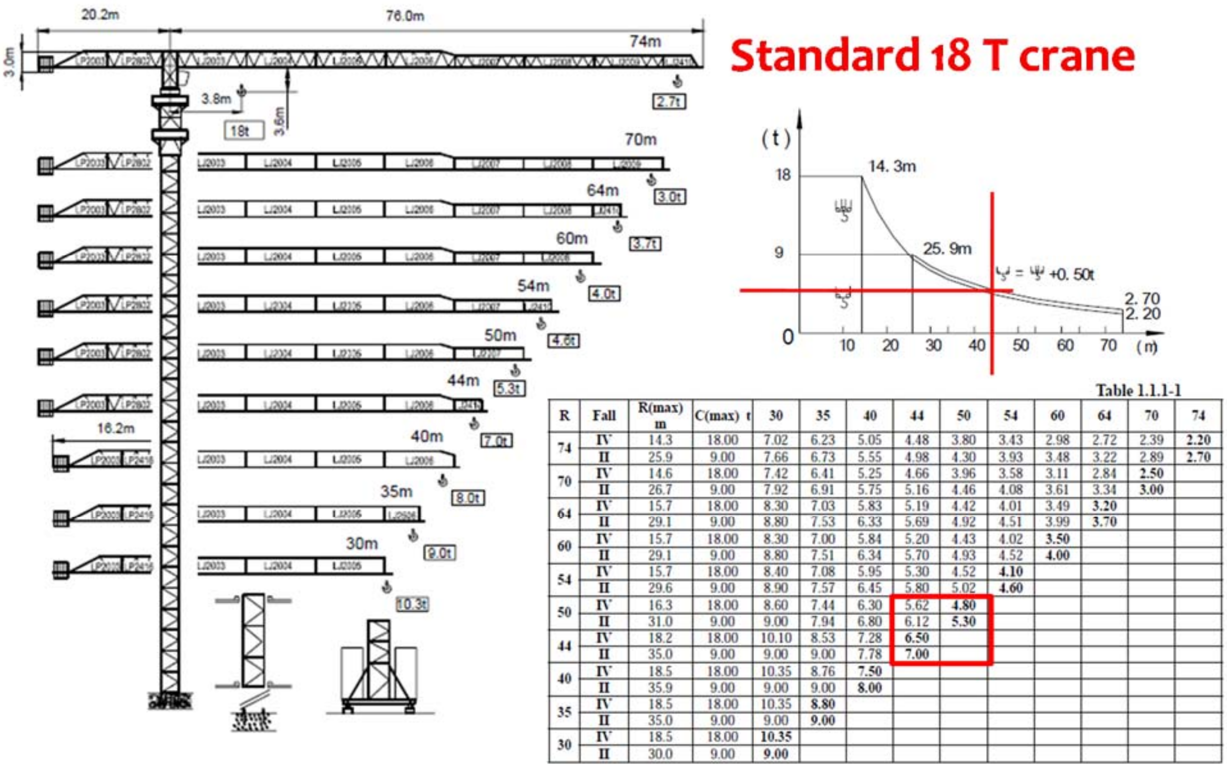


Table 1.1.1-1

R	Fall	R(max) m	C(max) t	30	35	40	44	50	54	60	64	70	74
74	IV	14.3	18.00	7.02	6.23	5.05	4.48	3.80	3.43	2.98	2.72	2.39	**2.20**
	II	25.9	9.00	7.66	6.73	5.55	4.98	4.30	3.93	3.48	3.22	2.89	**2.70**
70	IV	14.6	18.00	7.42	6.41	5.25	4.66	3.96	3.58	3.11	2.84	**2.50**	
	II	26.7	9.00	7.92	6.91	5.75	5.16	4.46	4.08	3.61	3.34	**3.00**	
64	IV	15.7	18.00	8.30	7.03	5.83	5.19	4.42	4.01	3.49	**3.20**		
	II	29.1	9.00	8.80	7.53	6.33	5.69	4.92	4.51	3.99	**3.70**		
60	IV	15.7	18.00	8.30	7.00	5.84	5.20	4.43	4.02	**3.50**			
	II	29.1	9.00	8.80	7.51	6.34	5.70	4.93	4.52	**4.00**			
54	IV	15.7	18.00	8.40	7.08	5.95	5.30	4.52	**4.10**				
	II	29.6	9.00	8.90	7.57	6.45	5.80	5.02	**4.60**				
50	IV	16.3	18.00	8.60	7.44	6.30	5.62	**4.80**					
	II	31.0	9.00	9.00	7.94	6.80	6.12	**5.30**					
44	IV	18.2	18.00	10.10	8.53	7.28	**6.50**						
	II	35.0	9.00	9.00	9.00	7.78	**7.00**						
40	IV	18.5	18.00	10.35	8.76	**7.50**							
	II	35.9	9.00	9.00	9.00	**8.00**							
35	IV	18.5	18.00	10.35	**8.80**								
	II	35.0	9.00	9.00	**9.00**								
30	IV	18.5	18.00	**10.35**									
	II	30.0	9.00	**9.00**									

Figure 8.4: Operating Range of Interest of a Standard 18T Tower Crane

The procedure described here provides a basic framework to verify the feasibility of utilising a tower crane in the process of debris removal. The methodology could still be feasible for smaller blocks that have to be removed. Standard protocols for installation and operation of such tower cranes will have to be followed, and should not be used where such requirements compromise the safety of ruins or structures (subterranean too) within the archaeological site.

9 Temporary Weather Protection

Protection of damaged monuments from the elements of nature, particularly precipitation is critical. Ingress of rainwater through cracks and separation in masonry walls and roofs, and deterioration of frictional resistance due to presence of moisture can be a trigger for collapse of sections of a distressed structure. Wet compressive strength in masonry is significantly lower than the dry compressive strength. Moreover, brick masonry with mud mortar can also soak up a large quantity of water, increasing the weight of the structural components. All these factors could become critical for a part of a structure whose stability that has already been compromised due to earthquake damage.

In this regard, the indigenous practice of rapid assembly of scaffolding with locally available bamboo is noteworthy and appropriate (see Fig. 9.1). The scaffolding is then covered with palm leaf mats, over which tarpaulin sheets are fitted.

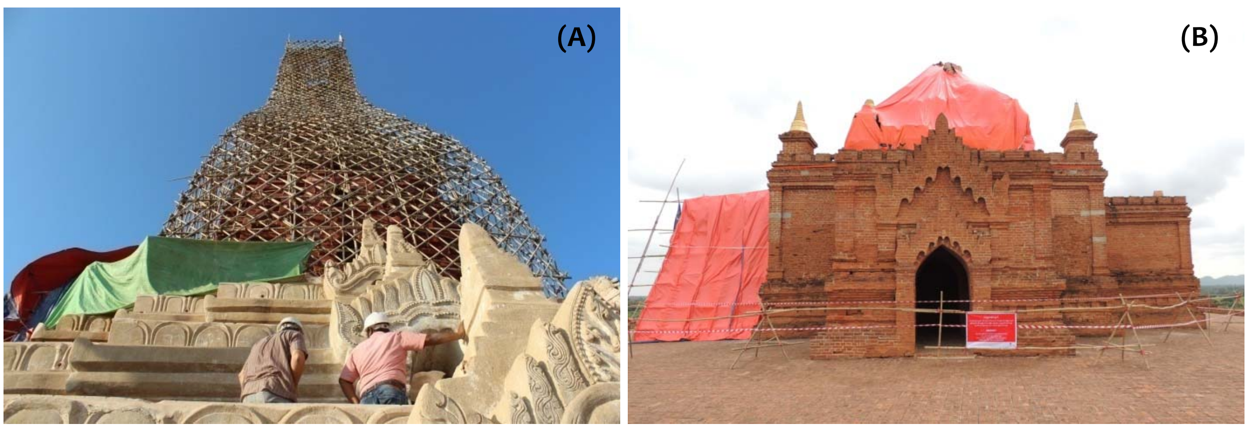


Figure 9.1: (A) Indigenous Practice of Rapid Assembly of Scaffolding; (B) Tarpaulin Sheets Fitted over Bamboo Scaffolding

Figure 9.2: (A) Deterioration of Bamboo and Tarpaulin Sheets; (B) Wind Effect on Scaffolding

There are a few concerns regarding this weather protection system:

1. The connections between bamboo elements, typically in coir ropes, deteriorate rather rapidly, compromising the stability of the scaffolding. Hence, these need to be monitored and replaced from time to time. Nylon ropes could also be used, however, even nylon deteriorates rather quickly on continuous exposure to sunlight (and is less environmental friendly, vis-à-vis this indigenous scaffolding and weather protection system). Even the tarpaulin sheets are affected by sunlight, and strong winds (see Fig. 9.2 A). In this regard, a regular maintenance protocol must be in place in order to ensure that the system is effective throughout the wet period.
2. The scaffolding system, while it attempts to become airtight, is severely affected by strong winds, and in many instances blown away (see Fig. 9.2 B). The most efficient way of handling effects of wind is to provide the weather protection system with a sufficient number of air vents, which are in turn protected from ingress of rainwater (with necessary overhanging eaves). In this regard, the vernacular bamboo constructions for residences could be a model to be followed (see Fig. 9.3 A).
3. In cases where resources are available for a more formal system, modular steel towers (e.g. HD Towers with built-in stairs, manufactured by companies such as Larsen and Toubro) can be erected around the periphery of the structure to be protected. The peripheral framework can then be mounted with a temporary steel roofing structure with a fabric membrane. Such a formal weather protection system (see Fig. 9.3 B)can be in place for the entire period of implementation of short-term counter measures and execution of final restoration and strengthening interventions, which could extend up to 5 years.

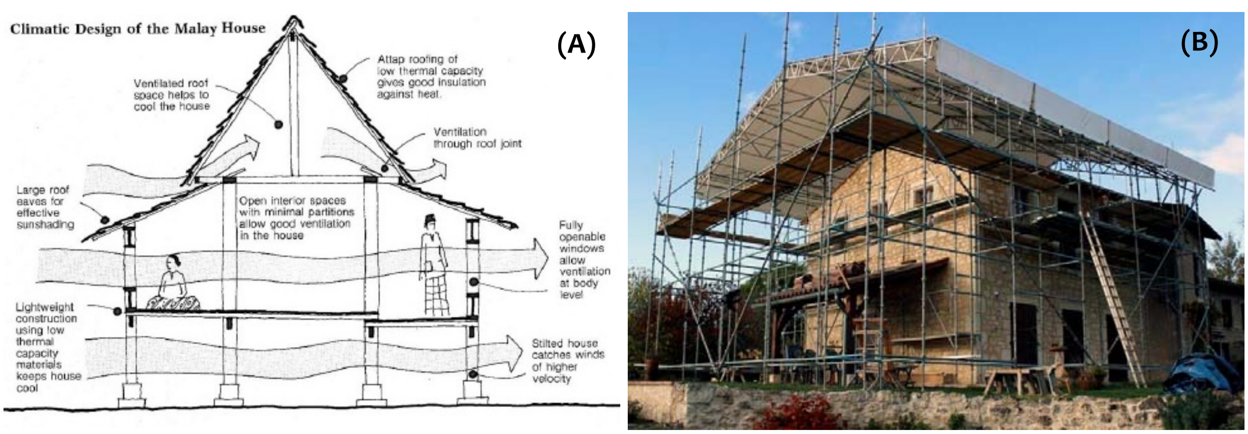


Figure 9.3: (A) Climatic Design of a Traditional Malay House (Pei Hong Ng; 2017) (B) Formal Temporary Weather Protections Systems Structurally Design in Steel and Fabric
(Ref: http://www.trevormorrisrenovation.com)

10 Types of Temporary Supports

Temporary supports are inserted to bear the vertical loads of the damaged structure, mainly due to gravity. Interventions must ensure prevention of collapse and life safety of technical personnel and labour accessing the structure. The intervention must be fully reversible and removable, and it should not damage the structural fabric. The capacity of members/system used should be greater than expected demand on the supporting system, and member dimensioning and structural configuration of supports will require engineering input from a structural engineer. A large number of manuals for temporary stabilisation and strengthening interventions have been prepared by organisations and institutions involved in post-disaster emergency management. The current section attempts to provide an overview of general methods of propping, bracing, shoring and tying, appropriate for the context of Bagan monuments.

10.1 Propping

Propping is providing vertical or inclined supporting elements against vertical movements. They can be height-adjustable steel modular single elements or scaffolding. Casuarina or bamboo may also be used but these might require the use of more number of members, which in turn leads to instability. The props require timber planks/ top plate to receive the supported element and base plate/planks for the uniform transfer of loads to the floor.

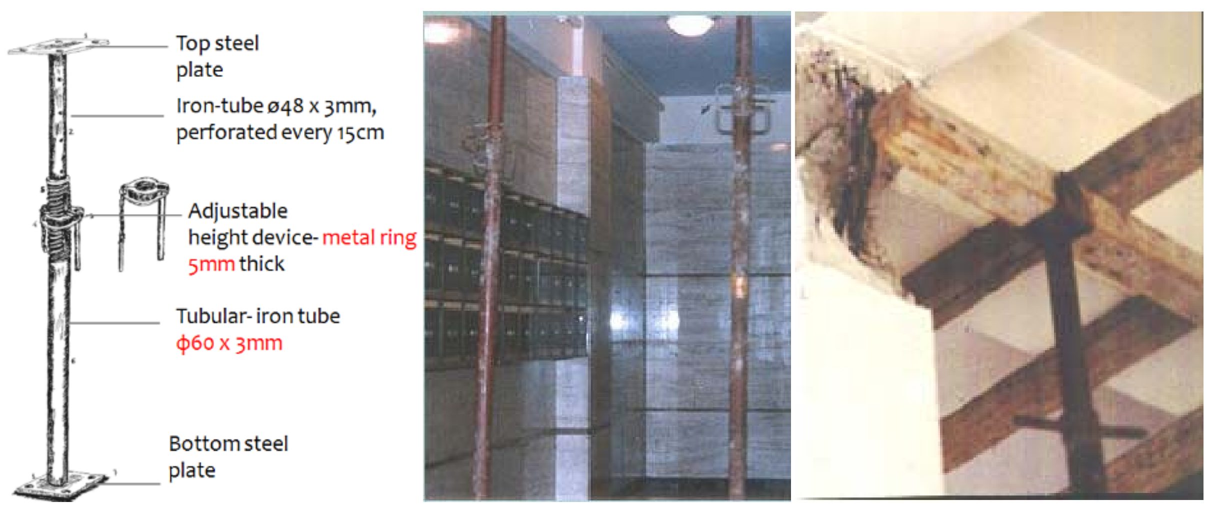


Figure 10.1: Single Metal Adjustable Height Prop (Ref.: EPPO, 2000)

10.1.1 Propping with single metal columns

In case of bearing light load or small damages, the metal columns of adjustable height can be used (see Fig. 10.1). The bearing load of these 3.0 m long columns is about 2.0 ton. Wedging is done by using special turning jacks, which are available on every column. The shore with single metal columns of adjustable height is easy and fast. Metal columns are basically used for small height propping.

10.2 Propping and Bracing

10.2.1 Propping with metal industrial scaffold

Industrial metal pipe scaffolding is used in order to bear light vertical loads but to a greater extent of area i.e. slab loads or relieving horizontal elements. Integrated vertical, horizontal and inclined (bracing) elements with adjustable heights, assembled by pairs into a tower using diagonal bars, and in combination with wooden beams are used as typical elements for propping mainly slabs, beams, etc. (see Fig. 10.2).

Figure 10.2: (A) Propping using Metal Industrial Scaffold (EPPO, 2000); (B) Larsen and Toubro's Doka Formwork System

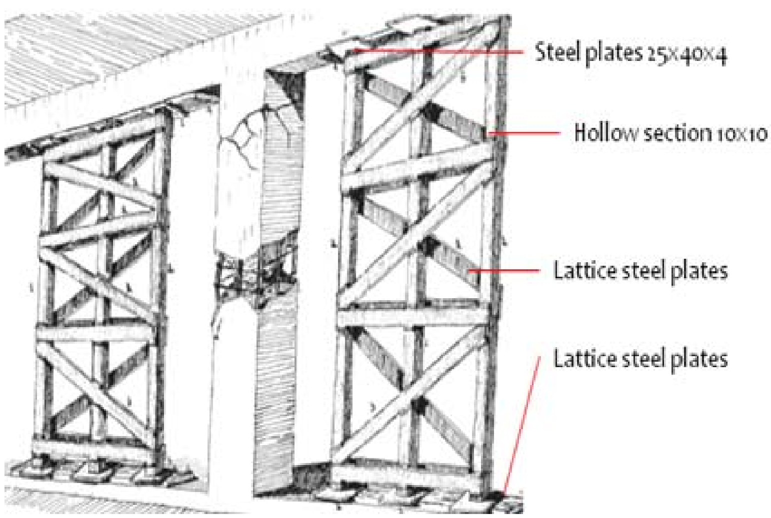


Figure 10.3: Propping with hollow section beams on either side of damaged column (EPPO, 2000)

10.2.2 Propping with assembly of metal sections

Steel plates 25x40 cm of sufficient thickness are welded at the upper and lower parts of double T-section. Batten plates or lattice plates of sufficient thickness tie the hollow beams in order to form a separate column. On either side of the damaged pillar or column, at approximately 30 cm distance, a new hollow beam column is created. In order to secure stability of the propping a wooden supporting piece 25 x 40 cm and thickness 4 cm is used on the upper and lower parts. Channel sections may also be used instead of hollow beams or double T- sections. Wedging is done with wooden wedges at the upper part between the steel plate and the wooden supporting piece.

10.2.3 Vertical propping with timber beams

Timber beams can be used in place of steel columns independently in cases of light damages or to receive smaller loads. The timber beams can be combined in a supporting tower formation or an independent wooden shore similar to the steel scaffolding case. It is important to connect with x or z connectors the wooden beams at all the above mentioned cases.

Vertically positioned wooden beams, wooden supporting pieces, top horizontal wooden beams and triangular wooden wedges are required. The beams must always be seated on solid base.

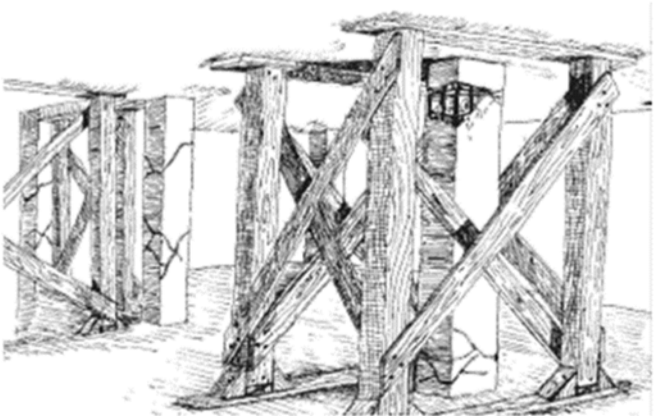


Figure 10.4: Propping with wooden beams around a damaged column (EPPO, 2000)

10.2.4 Triangular propping with wooden beams

The triangular propping is normally used to support heavy horizontal roofs or bridge decks, which have been damaged by earthquakes and when there is enough space on either sides of the base. This type of propping needs wooden "hat“, wooden buttress beams, horizontal connectors, support base and triangular wedges (see Fig. 10.5).

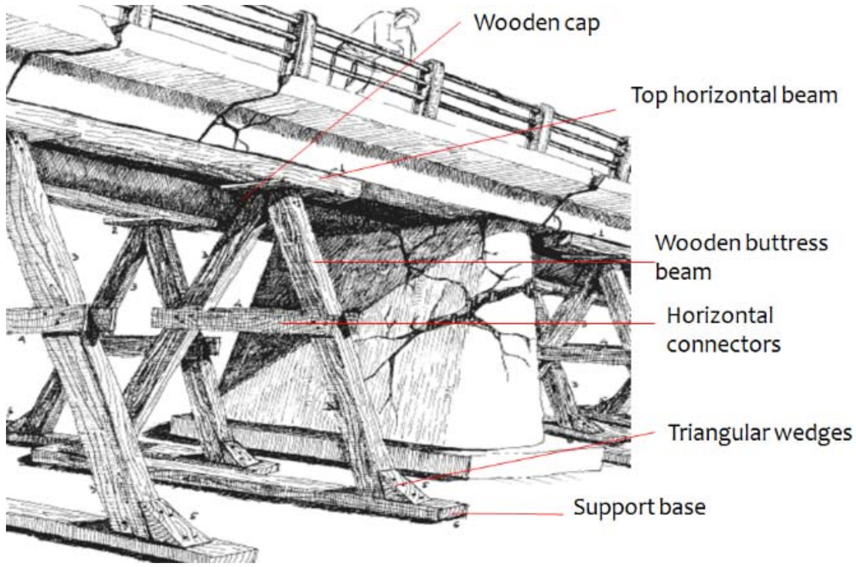


Figure 10.5: Multiple triangular propping with wooden beams (EPPO, 2000)

10.2.5 Propping with tree trunks

The beams must be straight, in one piece, with constant diameter, without nodes, from hard and healthy timber such as beech and oak etc. The trunks of each group are connected with: at least 4 boards (thickness 2 cm and width 4 cm) which are nailed in 45° angle; steel brackets Ø10, placed in X-formation in the middle of the beam. Hard wooden pieces of at least 4 cm thickness and 25 x 40 cm dimensions, sufficient enough to ensure that the base will withstand the load, are placed on top and bottom of every tree trunk (see Fig. 10.6).

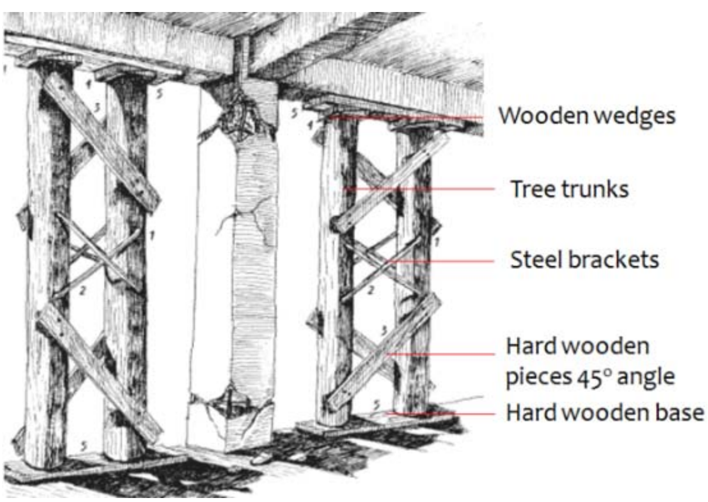


Figure 10.6: Propping using tree trunks either side of damaged column (EPPO, 2000)

10.2.6 Propping with railway timber sleepers

The timber sleepers are placed in layers intersecting each other and on either side of the damaged column. On the top of the railway timber propping formation, wide flange metal I-sections are used (see Fig. 10.7). The wedging is provided between the top surface of the I-section and the bottom of the beam either side of the damaged column.

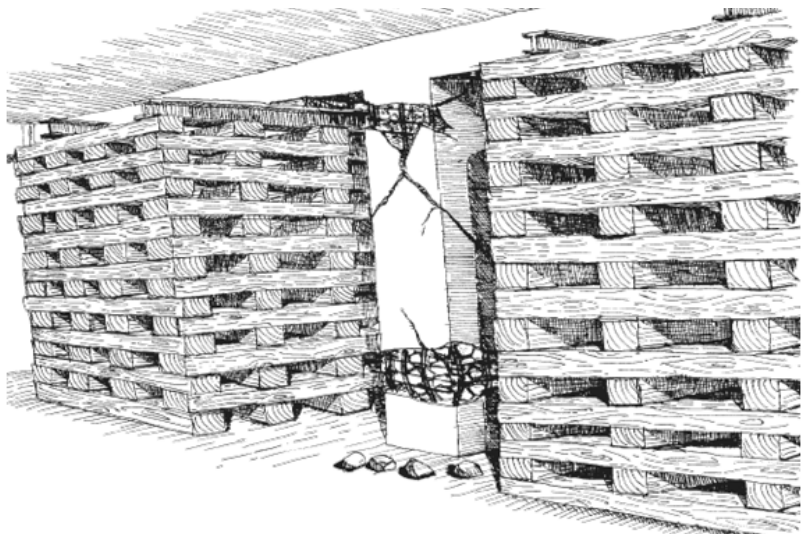


Figure 10.7: Strong propping using railway timber beams either side of a damaged column (EPPO, 2000)

10.3 Propping and bracing door and window openings

Since windows and doors are the weakest parts of a wall, structural movement occurs at or around their locations first. Placing bracing and shoring in the opening stabilizes this weak area. Diagonal bracing has been used successfully to prevent a structure from further deforming (see Fig. 10.8). Inclined elements are required for wedging.

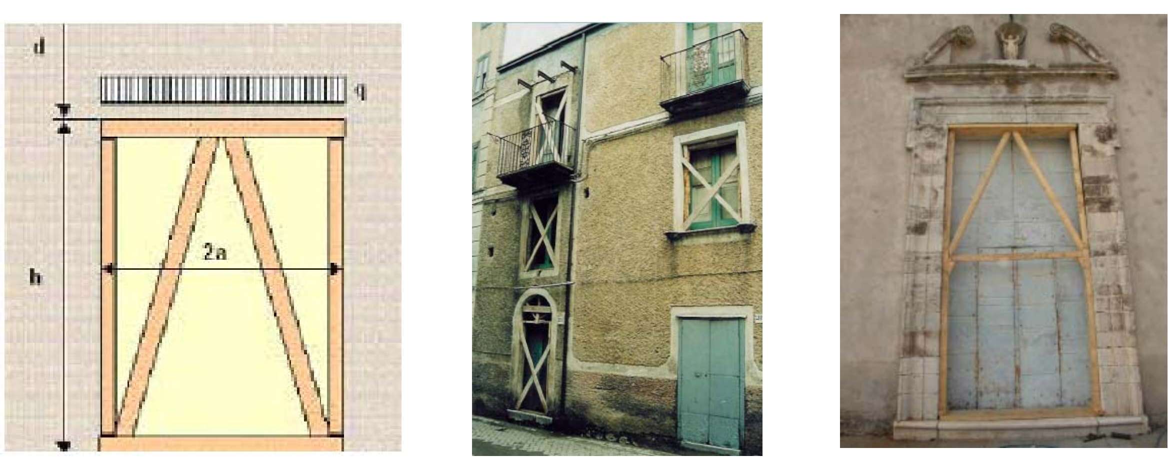


Figure 10.8: Propping and bracing door and window openings (SSN-DPC, 2005)

10.4 Propping and bracing arched openings

Timber elements profiling the intrados of the arch are essential for uniform and stable transfer of the loads. Also, inclined elements are required for wedging. Care should be taken to avoid supporting the arches directly at the centre or along the curves without the use of a profiling element (see Fig. 10.9).

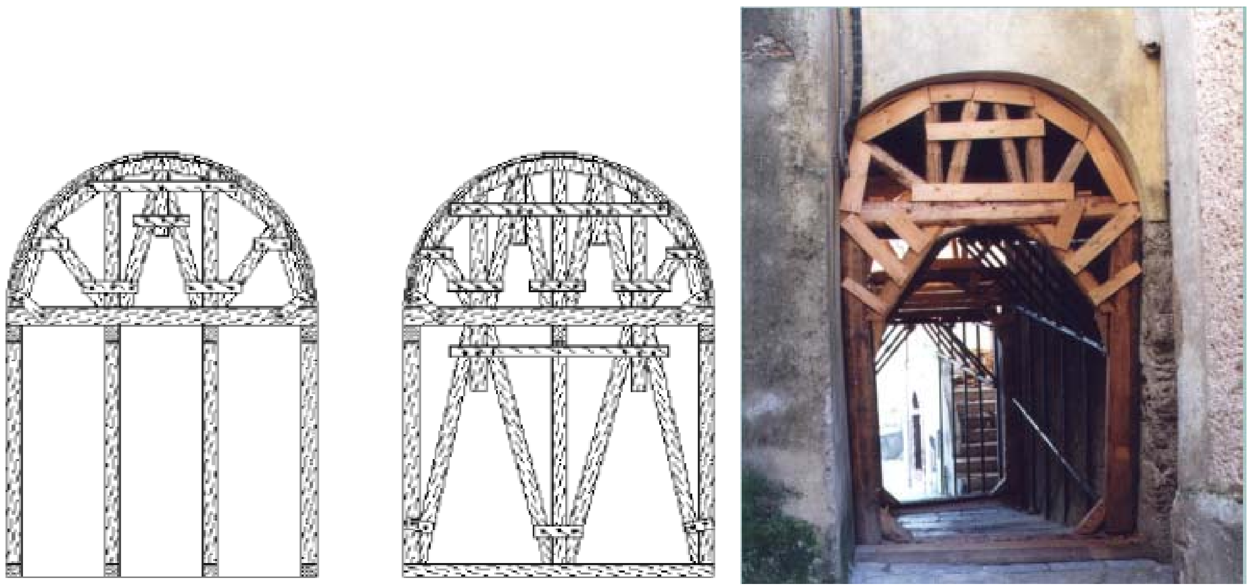


Figure 10.9: Propping and bracing arched openings (SSN-DPC, 2005)

10.5 Shoring: Single and Multiple Rakes

Temporary shoring is provided to support the structurally damaged and unstable elements by receiving and collecting loads from the damaged elements, and transmitting and distributing these loads to the structural elements in the remaining stable parts of the building that are capable of resisting additional loads without getting damaged themselves.

The rakes can be of timber or steel sections. Braces (horizontal members) are required to reduce slenderness and to prevent buckling of the members. Single point of contact between the rake and the wall should be avoided to prevent further damage. Uniform transfer of loads can be ensured by placing horizontal timber planks on which the shoring rakes shall be supported. In case of single-storied structures, the shoring members shall be supported at the ceiling level. The damaged wall should never be supported at the parapet level as it may cause further damage. Proper anchorage is to be provided on the ground to prevent the horizontal sliding. The vertical portion is to be connected to the building to prevent relative sliding.

Different configurations of single or multiple rakes are possible, few of which are discussed here. The choice is dependent on the size of the structural potion that requires protection, the space available adjacent to the portion to be protected.

10.5.1 Shoring using Single Raking shores

A single rake is used to transmit the loads from the damaged elements to structurally sound elements. The horizontal forces are developed due to inclination of the building either because of damage of the vertical elements or because of foundation subsidence.

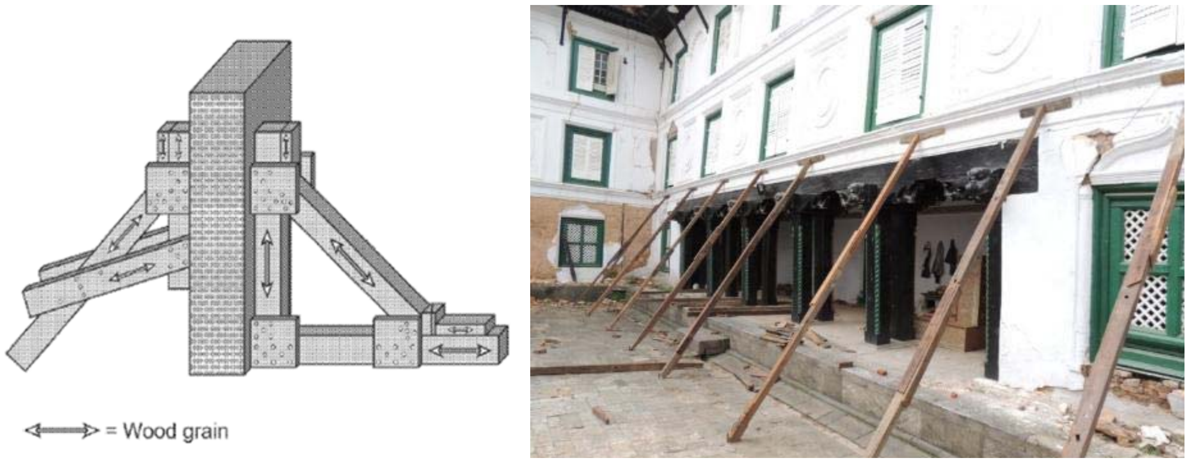


Figure 10.10: (A) Single Raker Shoring (Connel, 1953); (B) A single raker shoring executed in a heritage structure (Nepal, 2015)

10.5.2 Shoring using Multiple Raking shores

Double rakes are used to shore a wall that has severe out-of-plane deformation, and where shoring is required for more than one floor.

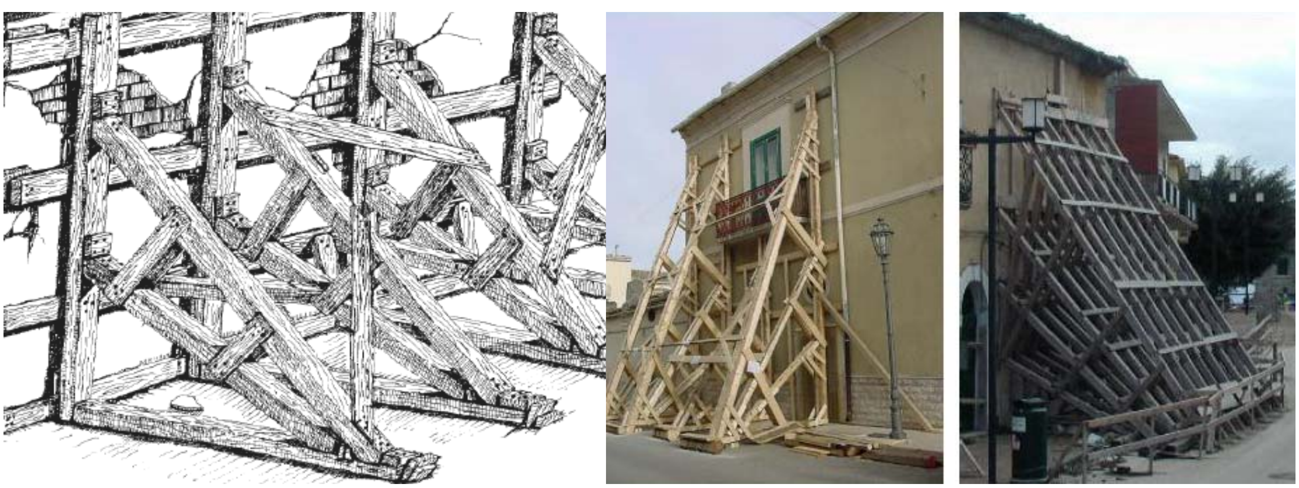


Figure 10.11: Multiple raking shores (EEFO, 2002 and DPC: Molise, Italy, 2002)

Further examples of rakes with multiple parallel insertions (see Fig. 10.12) or converging insertions (see Fig. 10.13) and details of connections and anchoring at the base are provided here. For further details, readers are referred to the STOP Vademecum (Grimaz et al., 2010).

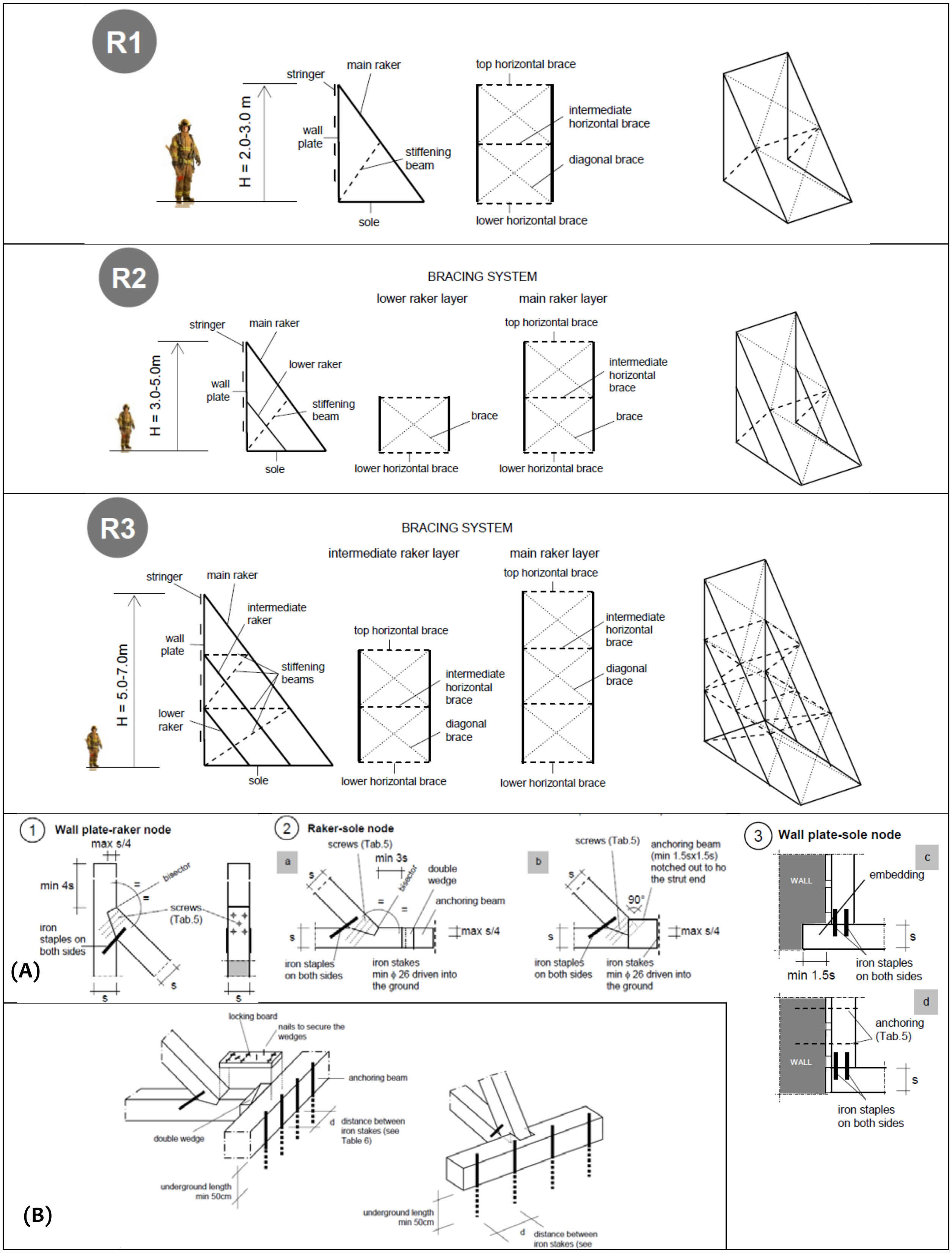


Figure 10.12: Solid sole timber rakes for different heights (R1, R2 and R3) with multiple, parallel insertions; (A) Details of connections (1-3), and (B) Anchoring solutions for the base (Grimaz et al., 2010)

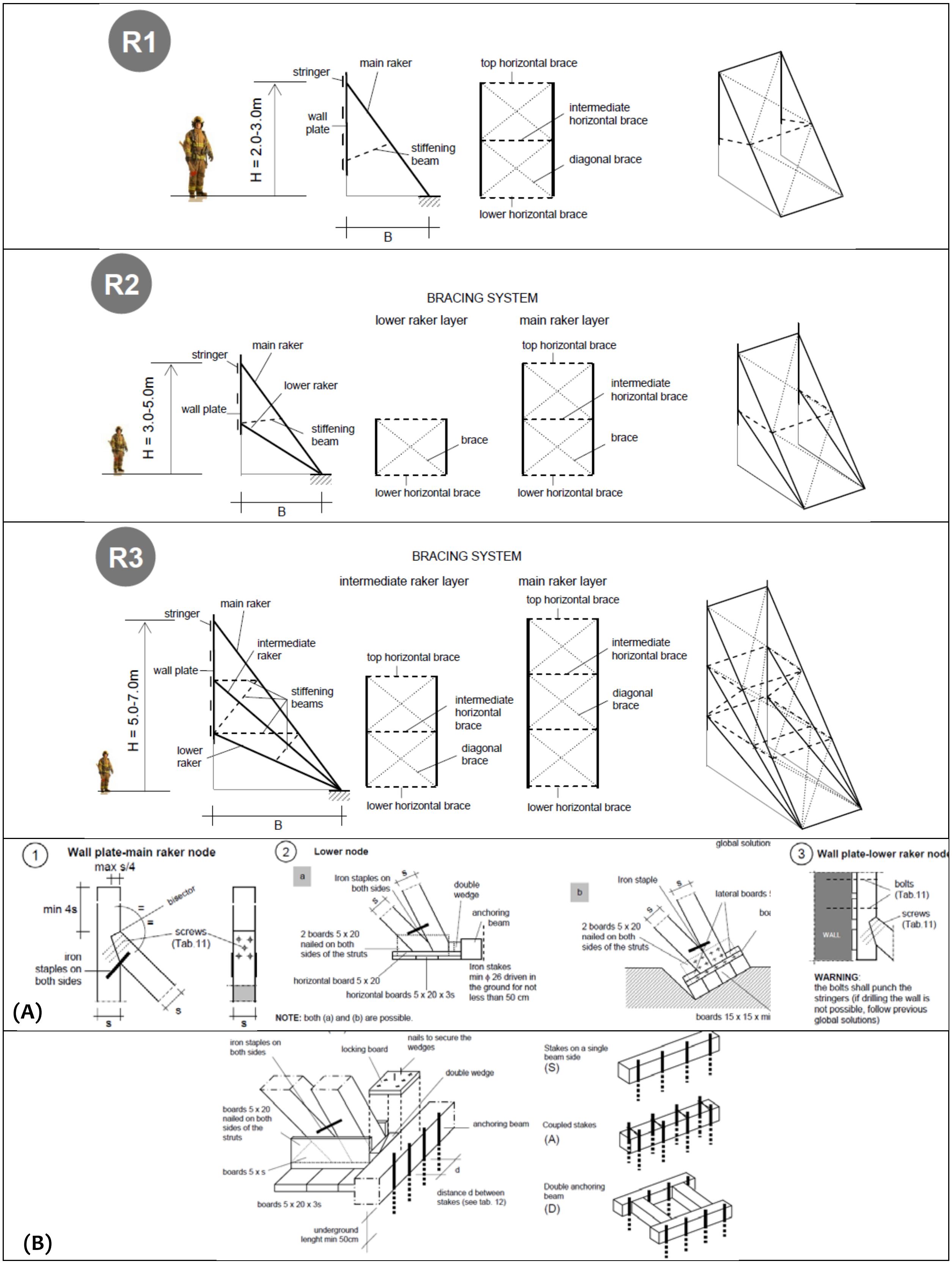


Figure 10.13: Single point base rakes for different heights (R1, R2 and R3) with multiple, converging insertions; (A) Details of connections (1-3), and (B) Anchoring solutions for the base (Grimaz et al., 2010)

10.6 Corner rakes

Corner rakes are made at the building corner in case of risk of corner collapse or separated facade walls.

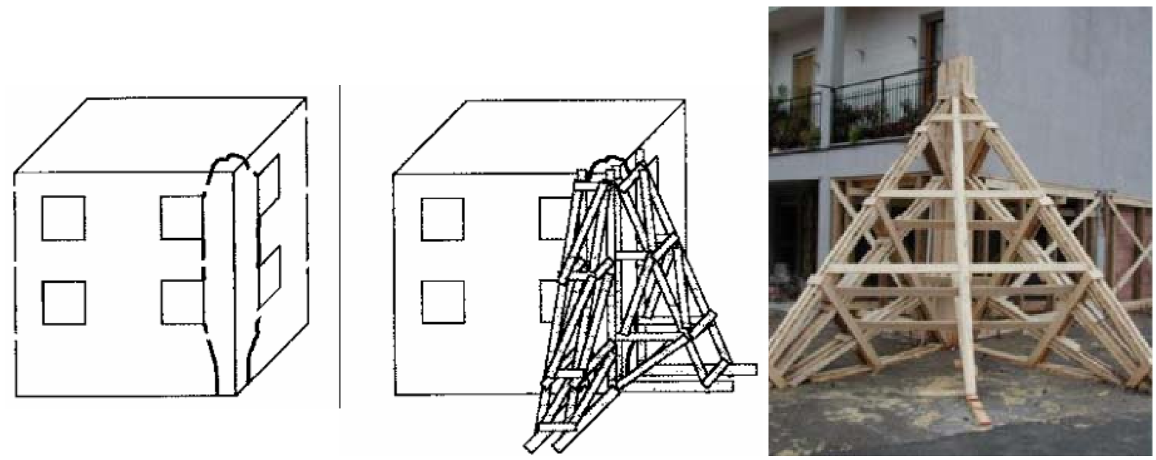


Figure 10.12: Corner Shoring (DPC: Molise, Italy, 2002)

10.7 Contrasting Shores

Contrasting shores are provided between buildings, and along narrow passages, especially when space constraints are present (see Fig. 10.13 A). Additional horizontal and inclined braces may be used to stiffen the arrangement. Flying shores are provided at a certain height when passages cannot be blocked due to space constraints (see Fig. 10.13 B).

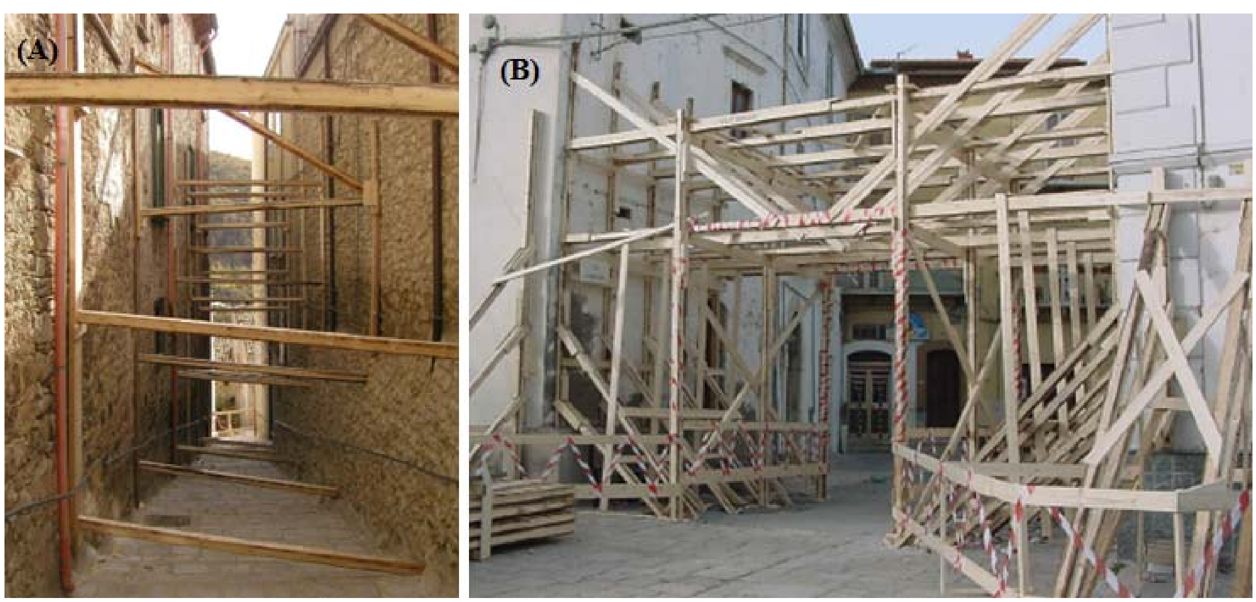


Figure 10.13: (A) Contrasting shores; (B)Flying shores (DPC, Molise earthquake 2002)

10.8 Tying and Anchoring Walls

Load-bearing walls which deformed out-of-plane, separated from return walls perpendicular walls) and dilated can be tied together with steel anchors, which are post-tensioned with jacks (see Fig. 10.14). Tying is carried out to connect orthogonal walls and ensure box action against lateral movement. Steel pre-stressing tendons/cables can be used for the purpose (see Fig. 10.15 and Fig. 10.16). Possible configurations and details for tying walls with tendons or tendons and beams (steel or wooden) can be seen in Fig. 10.17. For further details, readers are referred to the STOP Vademecum (Grimaz et al., 2010).

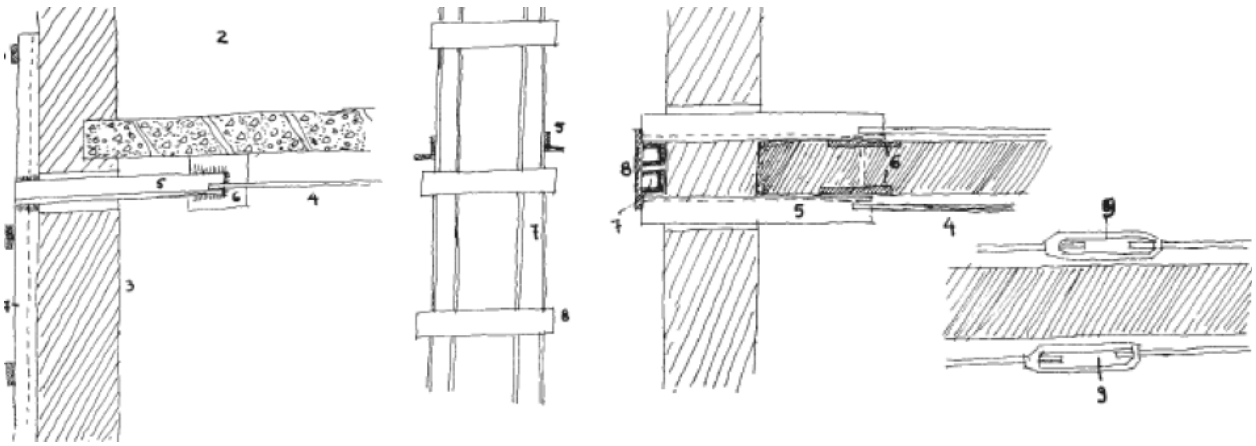


Figure 10.14: Shoring with internal anchorage (EPPO, 2000)

Figure 10.15: Connecting orthogonal walls using steel pre- stressing cables (DPC, Molise earthquake, 2002)

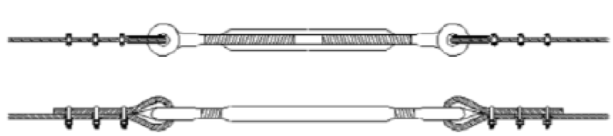


Figure 10.16: Steel pre-stressing cables used for tying connected with turn-buckles

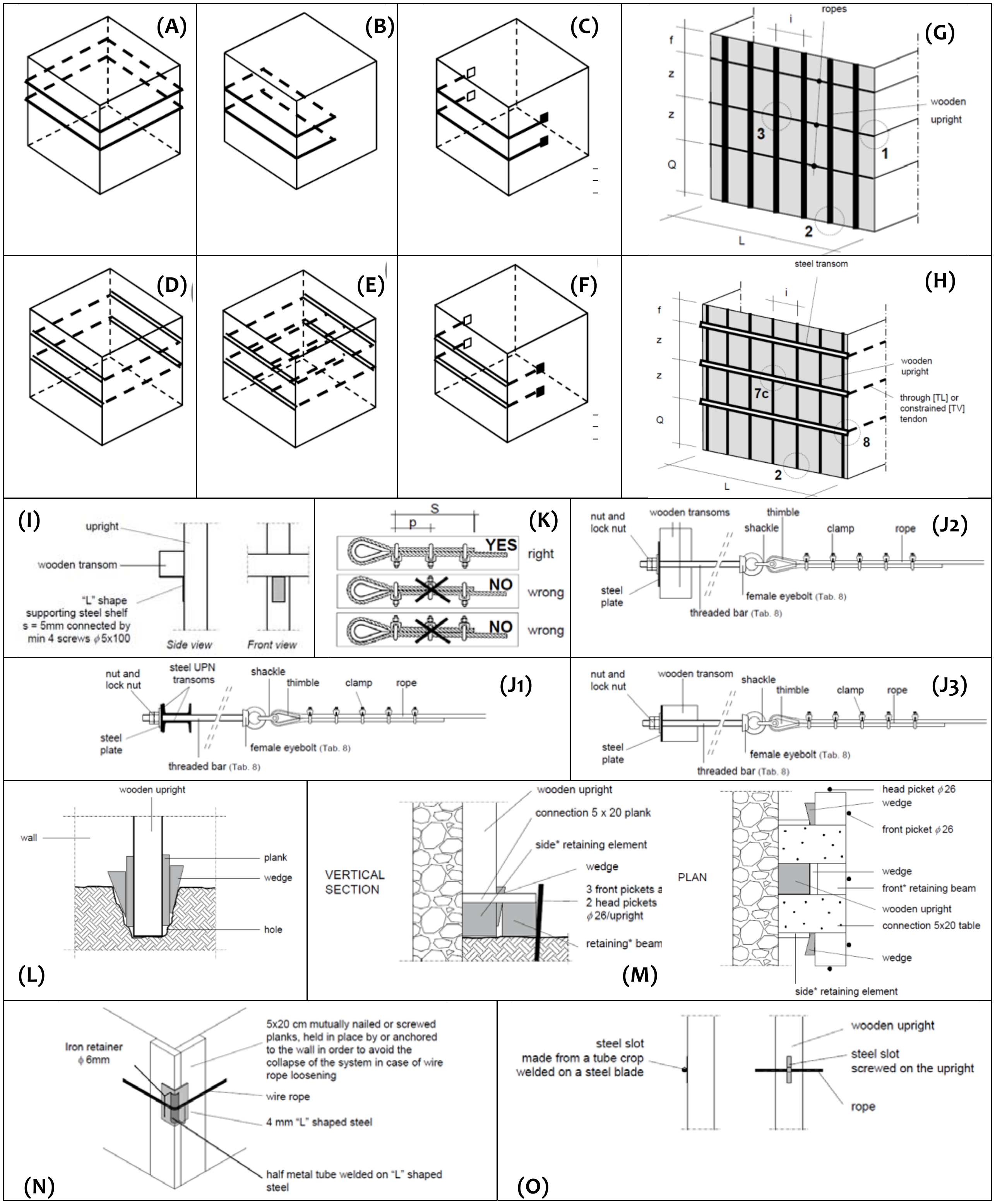


Figure 10.17: Possible configurations for tying: (A) Full external tendons; (B) Partial through tendons; (C) Anchored through tendons; (D) Beam with external side tendons; (E) Beam with through internal distributed tendons; (F) Beam with anchored side tendons; (G) Tendons with wooden uprights; (H) Steel beam with wooden uprights; (I) Wooden beam to upright connection; (J1-J3) Beam to tendon connections; (K) Clamp tightening; (L)-(M) Upright base details; (N) Corner detail; (O) Tendon-upright crossing (Grimaz et al., 2010)

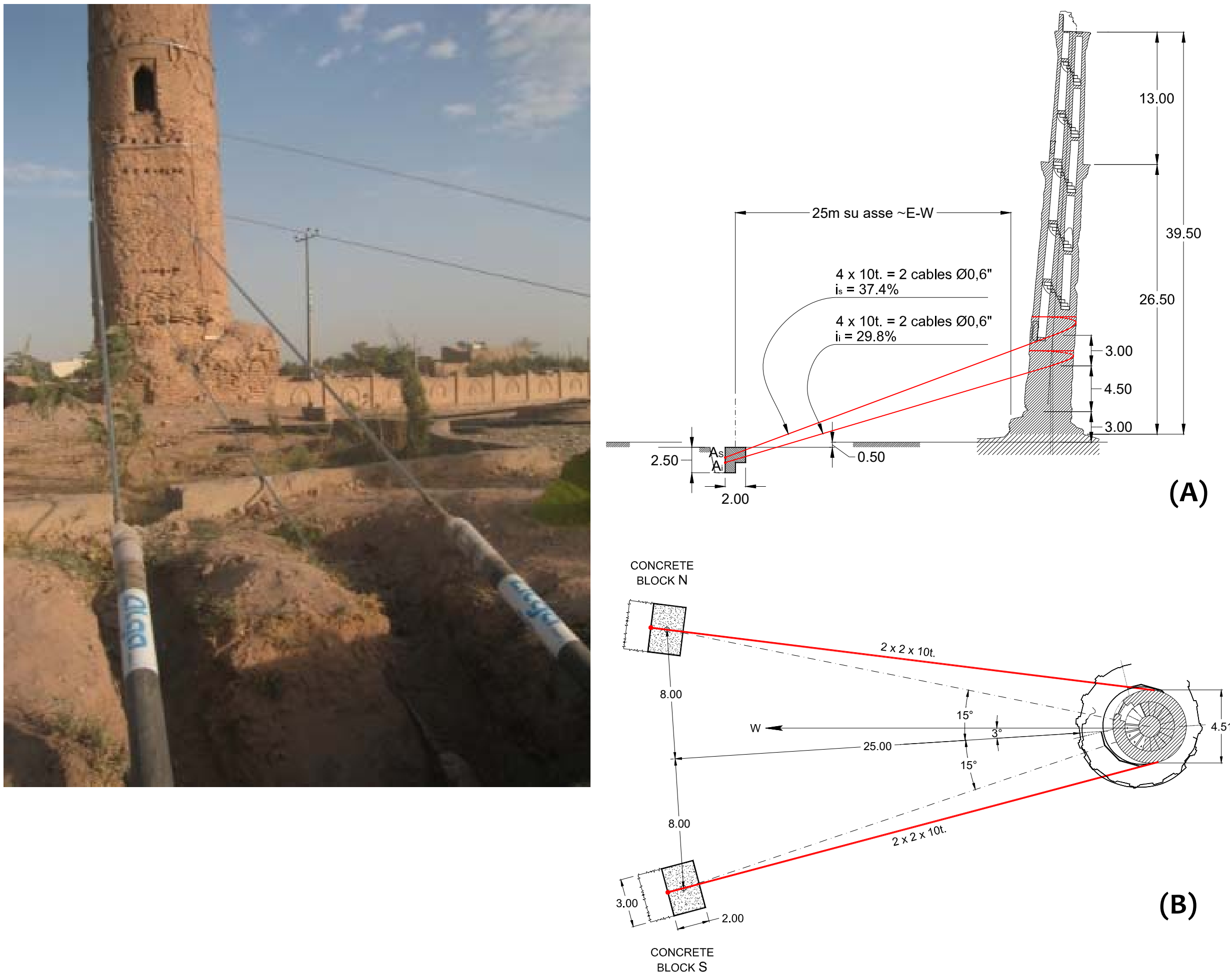


Figure 10.18: Temporary stabilization of the Minaret of Herat, Afghanistan by means of steel cables, anchored to the floor in RC anchor block (Courtesy: Prof. Giorgio Macchi, University of Pavia, Italy).

This procedure is also applied to situations where temporary structural stabilisation is required. An example of such an application for stabilising an inclined tower as a temporary removable measure is shown in Fig. 10.18.

10.9 Ratchet Lashing

Ratchet lashing is carried out to connect orthogonal walls and ensure box action against lateral movement by means of synthetic (nylon or polypropylene) belts with buckles. These nylon lashings are available in different sizes and load ratings, and have been successfully used in a few of the earthquake-damaged monuments in the Bagan archaeological zone.

Figure 10.18: (A) Ratchet lashing to connect orthogonal walls (Langenbach, 2003); (B) Lashing and buckle

11 Temporary Strengthening

11.1 Circumferential post-tensioning

Circumferential post-tensioning is the provision of lateral confinement with steel cables, timber sections, synthetic belts or steel sections welded or bolted together (see Fig. 11.1). This significantly improves factor of safety for vertical load carrying capacity, while preventing any further lateral deformation of the member. Belts or ties must be snug fit with some tightening. In case of sharp corners, as in rectangular or polygonal cross sections, corner element in wood or synthetic material is essential to prevent local crushing.

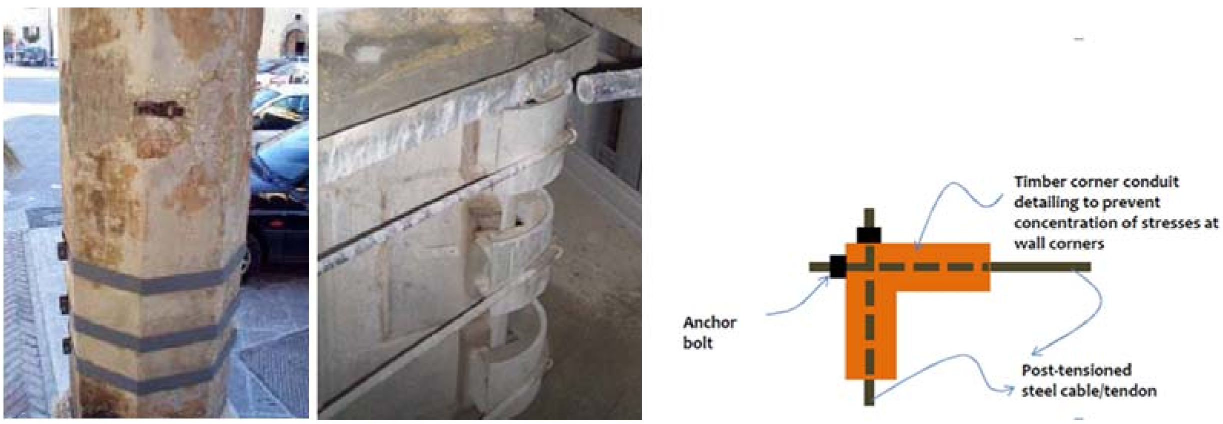


Figure 11.1: Lateral confinement with metallic belts or tendons and detailing at the corner

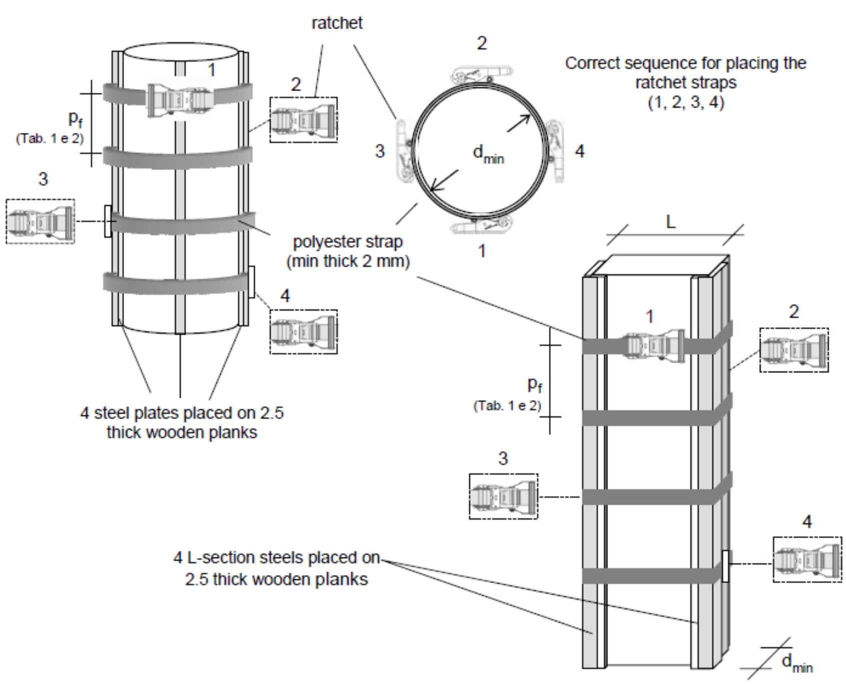


Figure 11.2: Possible configurations for circumferential confinement (Grimaz et al., 2010)

Possible configurations for load-bearing members with belts (ratchet lashing) can be seen in Fig. 11.2. For further details, readers are referred to the STOP Vademecum (Grimaz et al., 2010). The spacing between the belts and the number of belts is arrived at depending on the severity of the damage in the member, and the axial load acting on the member.

Circumferential post-tensioning or confinement is also used extensively as a longer-term temporary measure after detailed assessment. In Fraccaro Tower in Pavia, Italy, desired confinement of masonry walls was achieved through a removable intervention (Balio, 1993). Uniform lateral confinement all along the height of the critical section was provided with central steel rigs holding 12 steel ties running through the masonry wall, introduced in pre-existing scaffolding holes, and anchored on the exterior surface (see Fig. 11.3).

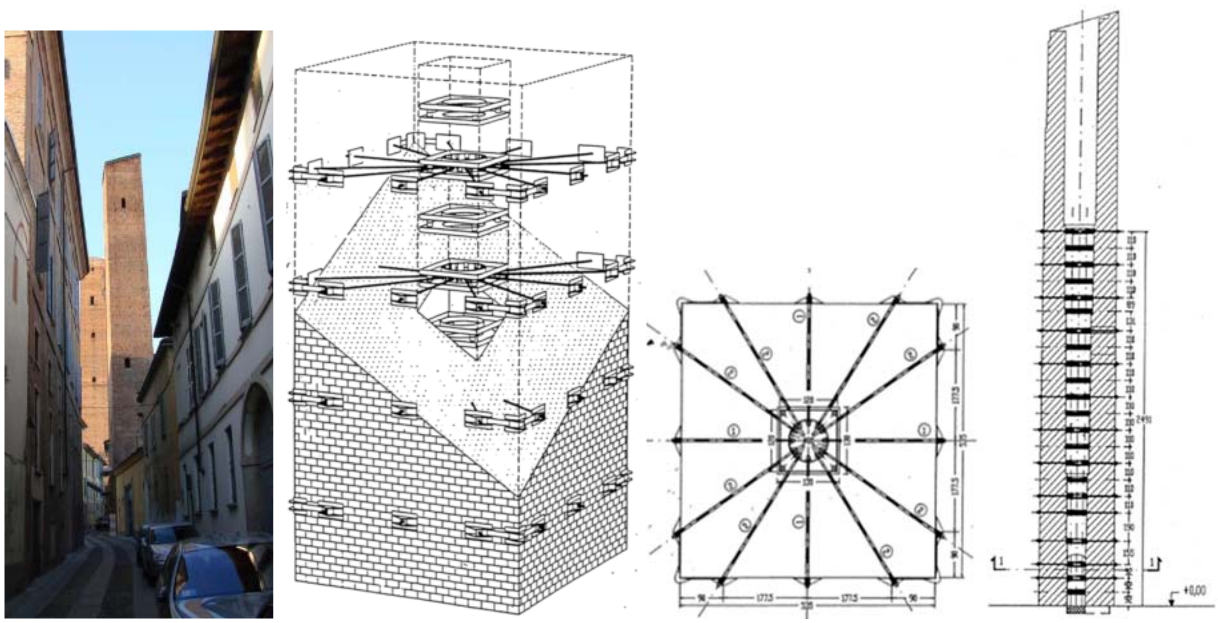


Figure 11.3: Details of intervention at Fraccaro Tower in Pavia, Italy for circumferential confinement (Ref: Balio, 1993)

11.1.1 Tightening of columns using metal section

Angular elements (at least 100 x 100 x 10 mm) are placed at the four corners of the damaged column, covering the full height top to bottom. Beyond these corner supports, and at a spacing of 60cm distance, pairs of transverse angular elements (L ≥ 120 x 120 x 12 mm) are placed on both sides of the column in alternating steps. Every pair of these transverse angular elements are tightened together using wires and bolts. After the first tightening of the bolts, steel batten plates 50 x 10 mm are welded on the vertical angular elements at 60 cm increments and the bolds are finally retightened. The wedging of the upper part of column can be done by as many flat thin steel plates as required. The formation of excessive friction between the angular elements and the material of the damaged column makes it capable of transferring the whole or part of the vertical load to the angular tightening steel elements (see Fig. 11.4).

Further tightening can be applied when there is necessity for the structural member to bear part of the axial load of the damaged column. ***Caution***: such interventions require a detailed assessment followed by selection of intervention from different alternatives.

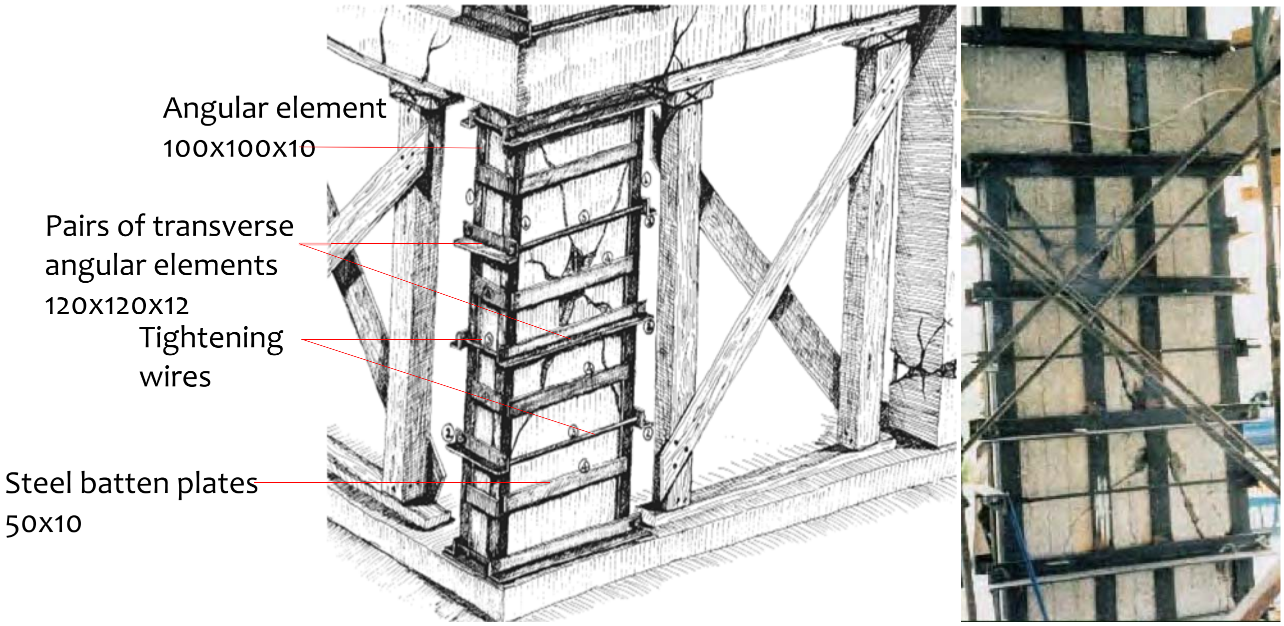


Figure 11.4: Tightening of column with angular elements (EPPO, 2000)

12 Details during Execution of Short-Term Measures

Specific constructional details have to be adhered to avoid longitudinal and transversal instability of structural members used to prop, brace or shore damaged portions of the structure. The various methods adopted include wedging of propping and shoring, anchoring to avoid sliding, and increasing joint stiffness to reduce slenderness.

12.1 Anchoring Methods

The shoring members must be anchored by any of the following methods (also refer to details in Fig. 10.12 B and 10.13 B):

a. Using embedded steel bars for horizontal restraint against sliding;
b. Embedding members directly into the soil, where possible, for horizontal restraint against sliding; and
c. Anchoring members to the wall for vertical restraint against sliding (this is essential when upright members are not present connected to the rake).

Figure 12.1: Anchoring by (A) Steel bars as horizontal restraint (B) Embedding into soil (C) Rake restrained from vertical sliding

12.2 Wedging Methods

The total or partial transfer of loads from the damaged element of the building to the shoring formation is successful when efficient wedging is done. This is achieved by wooden twin wedges or jacks (mechanical, hydraulic).

The wedges must be of dry timber, should not contain nodes and should be safe from sliding by oblique nails. The wedging must be applied slowly and carefully and the beam behaviour must be observed at the wedging position. Hydraulic or mechanical jacks are used for bearing heavy loads, ensuring homogeneous transfer of loads.

Figure 12.2: Wedging methods (EPPO, 2000)

12.3 Protection of Murals and Incised Decorative Plasters on Walls

A large proportion of monuments in Bagan are endowed with valuable mural paintings in the interiors and with incised decorative plasters or glazed tiles on the exterior as part of the architectural renders. Any damage to these valuable pieces of art and sculpture will be irreplaceable. Temporary or permanent structural intervention on these monuments must take into account necessary techniques and protocols for the protection of these architectural features. As these elements are fragile, they will not be able to remain undamaged under bearing stresses introduced by structural members in contact with walls with murals or decorative plaster. A region of potential hazard to a mural painting is shown in Fig. 12.3.

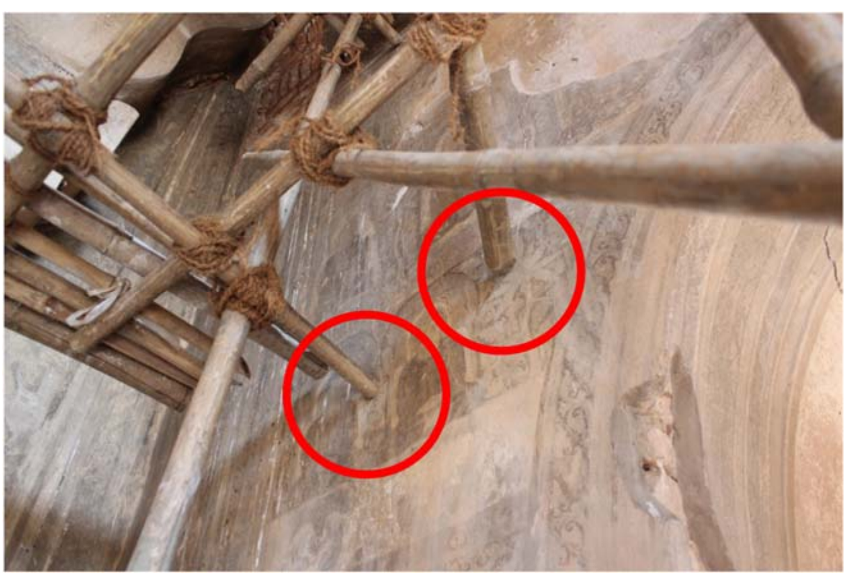



Figure 12.3: Ends of wooden structural members of the temporary stabilisation in direct contact with a wall with an ancient mural painting in Bagan

Care must be taken to ensure that no ***structural member*** that constitutes a temporary stabilisation and strengthening intervention should ever come in ***direct contact*** with a location with a ***mural***, ***incised decorative plaster*** or ***glazed tiles***. Sufficient distance must be maintained between the end or edge of the steel or wooden structural member. Or in situations where contact is inevitable, sufficiently thick ***cushioning material*** (neoprene rubber pads or synthetic foam pads) must be introduced between them, with necessary detailing such that the cushioning material is held in position and does not slide down in case of potential structural movements (e.g. during an aftershock).

In case of mouldings on walls, glazed tiles, incised plaster work or murals that are present in niches or offset into a wall, necessary precaution to prevent any direct contact of temporary structural members (or even elements of scaffolding) must be taken. For instance, expanded polyethylene (EPE) foam sheets, available in different thicknesses, cut to size can be inserted into such niches to protect glazed tiles from bearing stresses that could be transferred from the horizontal wooden member seen in Fig. 12.4 and Fig. 12.5.

Figure 12.41: Potential hazard posed by wooden scaffolding and temporary propping members to glazed tiles

Figure 12.52: Introduction of EPE foam sheet over the niche with the glazed tile

Another strengthening or stabilisation activity of ***potential danger*** to murals or incised plaster work could come from ***grouting***. Excess grouts could runoff on such surfaces and stain them forever. Hence, care has to be adopted if grouting has to be carried out in areas with valuable architectural details. ***Selection of materials*** for protection of surfaces (e.g. EPE foam) and of grouts must be in ***consultation with the art conservation team***.

Prevention of damage to such valuable components of the structure is feasible only if the ***workflow protocol of short-term counter measures*** ensures vetting of proposed interventions in locations with such artefacts by qualified team of art conservators and archaeologists. More importantly, the presence of at least one member from the art conservation and archaeology team must be guaranteed during the execution of temporary interventions by the team of engineers and labour from the Ministry of Construction. This will ensure that unintentional errors by the labour are avoided, and in case potential danger to the artefacts is identified, necessary decisions on the field can be taken immediately.

Protection of sculpted columns and other idols during execution of temporary stabilisation works in the aftermath of the Nepal earthquake, at the Swayambunath World Heritage Site is shown in Fig. 12.6.

Figure 12.63: Sculpted stone idols and ornamented columns covered with EPE foam before installation of temporary stabilisation measures at Anantpur Temple at Swayambunath World Heritage Site, Nepal in the aftermath of the earthquakes of 2015: (A) Before installation of short-term measures; (B) After installation of short-term measures

12.4 Access for Detailed Assessment and Further Interventions

An aspect that needs to be taken into account while selecting and designing temporary stabilisation and strengthening schemes is the access to the damaged location after the implementation of the short-term measure for detailed assessment, for preparatory works for the execution of the final intervention and the execution of the latter itself. The problem becomes evident with the photographs in Fig. 12.7 A and B, and Fig. 12.8.

An attempt should be made to verify if a heavy intervention with several temporary structural members is really essential, and whether a leaner and more efficient system is possible. The latter requires an in depth understanding of the expected behaviour of the structure being protected. The latter approach will also adequate lesser hindrances during the detailed assessment and execution phase of final interventions. In this regard, where applicable, methods based on tying and anchorage (section 10.8) can be advantageous over props and rakes (sections 10.1-10.7).

(B)

Figure 12.74: (A) Congested temporary intervention; (B) Lean temporary intervention

Figure 12.85: Congested temporary intervention

12.5 Additional Good Practices

Based on the observations from the sites of damaged monuments in Bagan's archaeological zone, the following aspects have been highlighted as good practices that one can follow in the designing and execution of short-term counter measures.

It is essential to ensure that elements such as synthetic belts and metallic straps meant to serve for circumferential post-tensioning or confinement are always snug fit and taut (see Fig. 12.9). Otherwise, they will cease of be effective in their intended action, with disastrous consequences in the event of significant aftershocks. The same holds true for post-tensioning tendons used for circumferential post-tensioning (see Fig. 12.10), which should never become slack. To achieve their intended effectiveness, all such sites must be regularly inspected and the effectiveness of the intervention verified.

Figure 12.96: Monitoring time to time if tautness of synthetic belts and metal straps used for tying and confinement

Figure 12.107: Circumferential post-tensioning will be rendered ineffective if the tendons are slack

Termite treatment of timber structural elements, which form part of a temporary stabilisation system, and which are in direct contact with the soil is a good practice (see Fig. 12.11).

Figure 12.118: Termite treatment of timber structural members in contact with soil

Instrumentation of damaged locations with simple measuring devices such as crack width gauges, tiltmeters or load cells, wherever feasible, will provide crucial information on effectiveness of a temporary intervention (see crack-width gauge installed at cracked location in a structural member). Such measures can potentially provide valuable information for decision making in the detailed assessment phase, which could extend from a few months to a even a few years in the context of historical monuments.

Figure 12.129: Quantitative monitoring of damaged locations with crack-width gauge

13 Tables for Selection and Dimensioning of Structural Members

The current document concludes with an exercise aimed at estimating the maximum permissible length for the raker members used for shoring for a given lateral load to be supported and a given cross section. Following cross sections have been adopted for which length calculations have been carried out and demonstrated:

i. Square and circular solid sections of *Sal wood* (a variety of hardwood timber); and
ii. Square and circular hollow sections of *Tata Structura Steel*, Grade 310.

For each of the sections listed above, the following configurations for supporting the lateral load from the walls are considered:

iii. Single rake shores inclined at 45°;
iv. Single rake shores inclined at 60°;
v. Two rake shores with members inclined at 45° and 26.5°; and
vi. Two rake shores with members inclined at 60° and 40°.

Table 13.1 Index to Tables for Selection and dimensioning of structural members

Configuration	Material	Shape of c/s	Angle of inclination	Table No.
Single rake	Timber (Sal)	Solid Square and Solid Circular	45°	13.2
			60°	13.3
	Steel	Hollow square	45°	13.4
			60°	13.5
		Hollow Circular	45°	13.6
			60°	13.7
Two Rake	Timber (Sal)	Solid Square and Solid Circular	45° and 26.5°	13.8
			60° and 40°	13.9
	Steel	Hollow square	45° and 26.5°	13.10
			60° and 40°	13.11
		Hollow Circular	45° and 26.5°	13.12
			60° and 40°	13.13

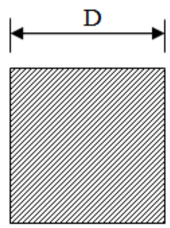
The design of the timber members is carried out as per specifications of IS 883 (1994), and that of the steel members, as per IS 800 (2007). Details of the design calculations and demonstrative examples can be found in Appendix-1.

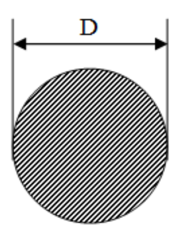
Table 13.2 Loads supported by Single rake shore in Timber (Sal wood) at 45˚

Maximum length of member for Sal wood at 45 degrees (m)																										
Diameter or width (mm)	Lateral Load Supported(kg)																									
	1000		1250		1500		1750		2000		2250		2500		2750		3000		3250		3500		3750		4000	
	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square
100	4.26	5	3.81	4.85	3.48	4.43	3.22	4.1	3.01	3.84	2.84	3.62	2.7	3.43	2.57	3.27	2.46	3.13	2.36	3.01	2.27	2.9	2.2	2.83	2.13	2.71
125	5.54	6.25	5.54	6.25	5.44	6.25	5.04	6.25	4.71	6	4.44	5.65	4.21	5.36	4.02	5.11	3.85	4.9	3.69	4.7	3.56	4.53	3.44	4.38	3.33	4.24
150	6.65	7.5	6.65	7.5	6.65	7.5	6.65	7.5	6.65	7.5	6.4	7.5	6.07	7.5	5.78	7.36	5.54	7.05	5.32	6.78	5.12	6.53	4.95	6.31	4.79	6.11
175	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.24	8.75	6.98	8.75	6.74	8.59	6.52	8.31
200	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.8	10	8.52	10
225	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25
250	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5

Table 13.3: Loads supported by Single rake shore in Timber (Sal wood) at 60˚

Maximum length of member for Sal wood at 60 degrees (m)																										
Diameter or width (mm)	Lateral Load Supported(kg)																									
	1000		1250		1500		1750		2000		2250		2500		2750		3000		3250		3500		3750		4000	
	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square
100	3.59	4.57	3.21	4.08	2.93	3.73	2.71	3.45	2.54	3.23	2.39	3.04	2.27	2.89	2.16	2.75	__	2.64	__	2.53	__	2.44	__	__	__	__
125	5.54	6.25	5.01	6.25	4.57	5.82	4.23	5.39	3.96	5.04	3.73	4.75	3.54	4.51	3.38	4.3	3.23	4.12	3.11	3.96	2.99	3.81	2.89	3.68	2.8	3.57
150	6.65	7.5	6.65	7.5	6.59	7.5	6.1	7.5	5.7	7.26	5.38	6.85	5.1	6.5	4.86	6.19	4.66	5.93	4.47	5.7	4.31	5.49	4.17	5.3	4.03	5.14
175	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.32	8.75	6.94	8.75	6.62	8.43	6.34	8.07	6.09	7.76	5.87	7.47	5.67	7.22	5.49	6.99
200	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.86	10	8.65	10	8.28	10	7.95	10	7.67	9.76	7.41	9.43	7.17	9.13
225	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.7	11.25	9.37	11.25	9.08	11.25
250	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5	11.07	12.5





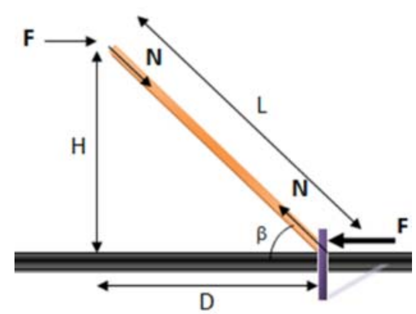
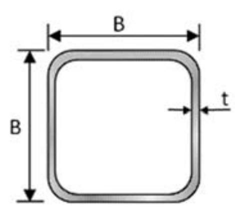


Table 13.4: Loads supported by Single rake shore in steel- square hollow section at 45°

Maximum length of member for Tata Structura Steel (Square hollow sections) at 45 degrees (m)																					
B X B (mm)	**t(mm)**	**Lateral Load Supported(kg)**																			
		500	**1000**	**1500**	**2000**	**2500**	**3000**	**3500**	**4000**	**4500**	**5000**	**5500**	**6000**	**6500**	**7000**	**7500**	**8000**	**8500**	**9000**	**9500**	**10000**
40 X 40	**4**	2.59	2.59	2.36	2.02	1.78	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_
50 X 50	**4.5**	3.28	3.28	3.28	3.12	2.76	2.5	2.29	2.12	1.97	1.84	_	_	_	_	_	_	_	_	_	_
60 X 60	**4**	4.06	4.06	4.06	4.06	3.63	3.29	3.02	2.79	2.61	2.44	2.3	2.17	2.04	_	_	_	_	_	_	_
80 X 80	**4**	5.53	5.53	5.53	5.53	5.53	5.33	4.9	4.55	4.27	4.02	3.8	3.61	3.44	3.29	3.15	3.01	2.89	2.77	2.65	2.54
100 X 100	**4**	7	7	7	7	7	7	7	6.58	6.17	5.82	5.52	5.26	5.03	4.82	4.62	4.45	4.29	4.14	4	3.86
	6	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.59	6.29	6.02	5.78	5.57	5.37	5.19	5.03	4.87	4.73
150 X 150	**4**	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.66	10.17	9.73	9.35	9	8.68	8.4	8.13	7.89	7.66
	6	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.19	9.88	9.59	9.33
220 X 220	**6**	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65
	10	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31

Table 13.5: Loads supported by Single rake shore in steel- square hollow section at 60°

Maximum length of member for Tata Structura Steel (Square hollow sections) at 60 degrees (m)																					
B X B (mm)	**t(mm)**	**Lateral Load Supported(kg)**																			
		500	**1000**	**1500**	**2000**	**2500**	**3000**	**3500**	**4000**	**4500**	**5000**	**5500**	**6000**	**6500**	**7000**	**7500**	**8000**	**8500**	**9000**	**9500**	**10000**
40 X 40	**4**	2.59	2.44	1.96	1.66	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_
50 X 50	**4.5**	3.28	3.28	3.02	2.58	2.28	2.04	1.85	_	_	_	_	_	_	_	_	_	_	_	_	_
60 X 60	**4**	4.06	4.06	3.97	3.4	3	2.67	2.46	2.26	2.08	1.91	1.75	_	_	_	_	_	_	_	_	_
80 X 80	**4**	5.53	5.53	5.53	5.5	4.87	4.41	4.04	3.74	3.49	3.27	3.07	2.89	2.73	2.56	2.41	2.24	2.07	1.88	_	_
100 X 100	**4**	7	7	7	7	7	6.37	5.86	5.44	5.09	4.79	4.53	4.29	4.08	3.89	3.71	3.54	3.38	3.22	3.06	2.9
	6	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.5	6.09	5.75	5.46	5.2	4.97	4.76	4.57	4.4	4.24	4.09	3.96	3.83
150 X 150	**4**	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.51	9.86	9.3	8.83	8.41	8.02	7.71	7.4	7.13	6.88	6.64	6.43	6.22
	6	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.21	9.77	9.38	9.04	8.72	8.43	8.17	7.92	7.69
220 X 220	**6**	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.24	14.8	14.39
	10	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31



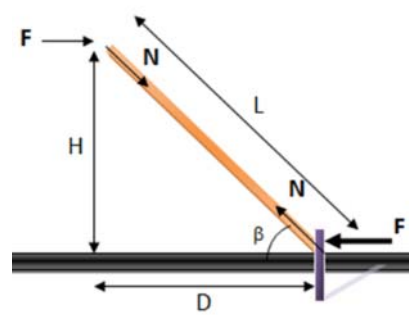


Table 13.6: Loads supported by Single rake shore in steel- circular hollow section at 45°

Maximum length of member for Tata Structura Steel (Circular hollow sections) at 45 degrees (m)																						
Nominal Diameter (mm)	**Outer Diameter D(mm)**	**t(mm)**	**Lateral Load Supported(kg)**																			
			500	**1000**	**1500**	**2000**	**2500**	**3000**	**3500**	**4000**	**4500**	**5000**	**5500**	**6000**	**6500**	**7000**	**7500**	**8000**	**8500**	**9000**	**9500**	**10000**
40	**48.3**	**4**	2.83	2.83	2.64	2.26	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_
50	**60.3**	**4.5**	3.56	3.56	3.56	3.44	3.05	2.76	2.53	2.34	2.18	_	_	_	_	_	_	_	_	_	_	_
80	**88.9**	**4**	5.4	5.4	5.4	5.4	5.4	4.94	4.54	4.22	3.95	3.72	3.51	3.33	3.17	3.02	2.88	2.75	2.63	2.51	2.39	2.26
100	**114.3**	**4.5**	7	7	7	7	7	7	7	6.7	6.29	5.93	5.63	5.36	5.13	4.91	4.72	4.54	4.38	4.23	4.09	3.95
150	**165.1**	**4.5**	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.14	9.68	9.26	8.9	8.57	8.26	8	7.74	7.5	7.29
200	**219.1**	**6**	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.28
		8	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44
250	**273**	**6**	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17
		8	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87
300	**323.9**	**8**	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11

Table 13.7: Loads supported by Single rake shore in steel- circular hollow section at 60°

Maximum length of member for Tata Structura Steel (Circular hollow sections) at 60 degrees (m)																						
Nominal Diameter (mm)	**Outer Diameter D(mm)**	**t(mm)**	**Lateral Load Supported(kg)**																			
			500	**1000**	**1500**	**2000**	**2500**	**3000**	**3500**	**4000**	**4500**	**5000**	**5500**	**6000**	**6500**	**7000**	**7500**	**8000**	**8500**	**9000**	**9500**	**10000**
40	**48.3**	**4**	2.83	2.73	2.19	1.85	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_	_
50	**60.3**	**4.5**	3.56	3.56	3.33	2.85	2.51	2.26	2.05	1.87	_	_	_	_	_	_	_	_	_	_	_	_
80	**88.9**	**4**	5.4	5.4	5.4	5.1	4.52	4.08	3.74	3.46	3.22	3	2.81	2.63	2.46	2.29	2.11	1.92	_	_	_	_
100	**114.3**	**4.5**	7	7	7	7	7	6.49	5.97	5.55	5.19	4.89	4.62	4.39	4.17	3.98	3.8	3.63	3.47	3.32	3.16	3.01
150	**165.1**	**4.5**	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10	9.38	8.85	8.4	8	7.65	7.33	7.05	6.78	6.54	6.32	6.11	5.91
200	**219.1**	**6**	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.37	12.88	12.43	12.02	11.65	11.31	10.99
		8	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.06	12.7
250	**273**	**6**	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	16.54	16.06	15.61
		8	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87
300	**323.9**	**8**	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11

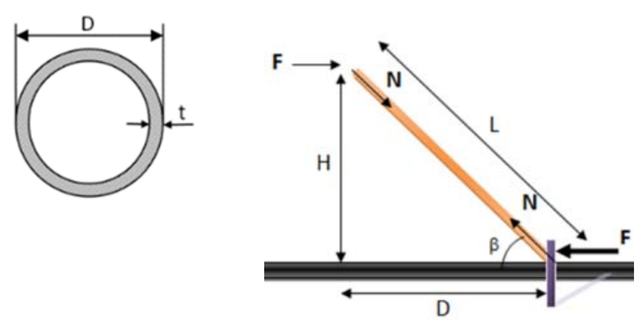
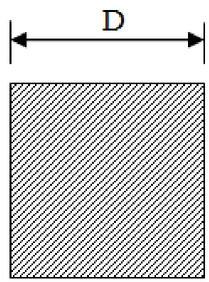
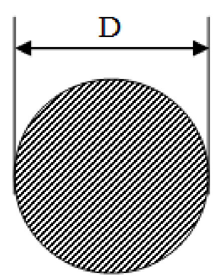


Table 13.8: Loads supported by Two rake shore in Timber (Sal wood) at 45° and 26.5°

Diameter or width (mm)		Maximum length of member for Sal wood at 45 and 26.5 degrees (m)																	
		Lateral Load Supported(kg)																	
		1000		1500		2000		2500		3000		3500		4000		4500		5000	
		Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square
75	45°	3.32	3.75	2.77	3.53	2.40	3.05	2.15	2.73	1.96	2.49	1.81	2.31	1.70	2.16	_	2.04	_	1.93
	26.5°	3.32	3.75	3.12	3.75	2.70	3.44	2.41	3.07	2.20	2.81	2.04	2.60	1.91	2.43	_	2.29	_	2.17
100	45°	4.43	5.00	4.43	5.00	4.26	5.00	3.81	4.85	3.48	4.43	3.22	4.10	3.01	3.84	2.84	3.62	2.70	3.43
	26.5°	4.43	5.00	4.43	5.00	4.43	5.00	4.29	5.00	3.92	4.99	3.63	4.62	3.39	4.32	3.20	4.07	3.03	3.86
125	45°	5.54	6.25	5.54	6.25	5.54	6.25	5.54	6.25	5.44	6.25	5.04	6.25	4.71	6.00	4.44	5.65	4.21	5.36
	26.5°	5.54	6.25	5.54	6.25	5.54	6.25	5.54	6.25	5.54	6.25	5.54	6.25	5.30	6.25	5.00	6.25	4.79	6.04
150	45°	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.40	7.50	6.07	7.50
	26.5°	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50
175	45°	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75
	26.5°	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75
200	45°	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00
	26.5°	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00
225	45°	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25
	26.5°	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25
250	45°	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50
	26.5°	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50





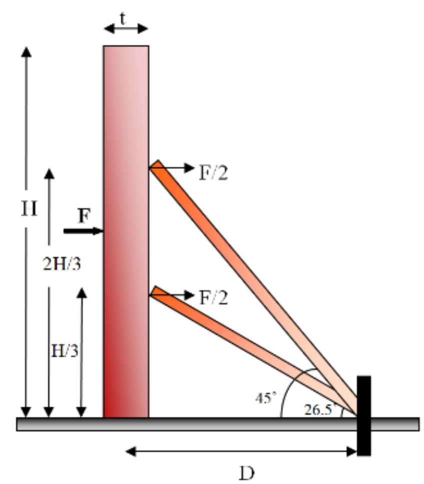
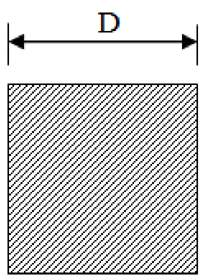
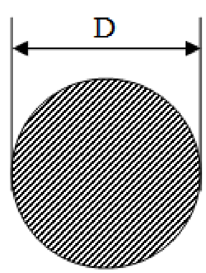


Table 13.9: Loads supported by Two rake shore in Timber (Sal wood) at 60˚ and 40˚

Maximum length of member for Sal wood at 60 and 40 degrees (m)																			
Diameter or width (mm)	Lateral Load Supported(kg)																		
		1000		1500		2000		2500		3000		3500		4000		4500		5000	
		Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square	Round	Square
75	60°	2.85	3.63	2.33	2.97	2.02	2.37	1.80	2.30	1.65	2.10	_	1.94	_	1.82	_	_	_	_
	40°	3.32	3.75	2.88	3.67	2.50	3.18	2.23	2.84	2.04	2.59	_	2.40	_	2.25	_	_	_	_
100	60°	4.43	5.00	4.14	5.00	3.59	4.57	3.21	4.08	2.93	3.73	2.71	3.45	2.54	3.23	2.39	3.04	2.27	2.89
	40°	4.43	5.00	4.43	5.00	4.43	5.00	3.97	5.00	3.62	4.61	3.35	4.27	3.14	4.00	2.96	3.77	2.81	3.57
125	60°	5.54	6.25	5.54	6.25	5.54	6.25	5.01	6.25	4.57	5.82	4.23	5.39	3.96	5.04	3.73	4.75	3.54	4.51
	40°	5.54	6.25	5.54	6.25	5.54	6.25	5.54	6.25	5.54	6.25	5.24	6.25	4.90	6.24	4.62	5.89	4.39	5.58
150	60°	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.59	7.50	6.10	7.50	5.70	7.26	5.38	6.85	5.10	6.50
	40°	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.65	7.50	6.32	7.50
175	60°	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.32	8.75	6.94	8.75
	40°	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75	7.75	8.75
200	60°	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00
	40°	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00	8.86	10.00
225	60°	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25
	40°	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25	9.97	11.25
250	60°	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50
	40°	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50	11.07	12.50





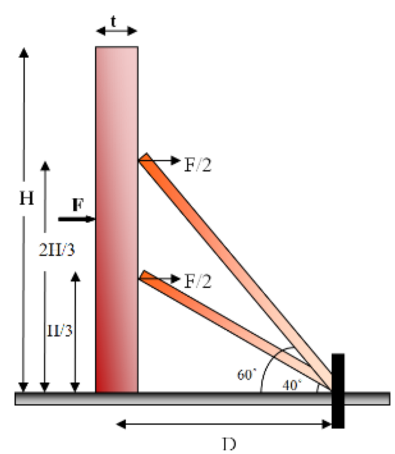
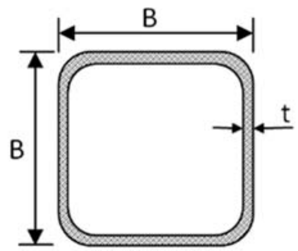


Table 13.10: Loads supported by Two rake shore in steel- square hollow section at 45° and 26.5°

Maximum length of member for Tata Structura Steel (Square hollow sections) at 45 and 26.5 degrees (m)												
B X B (mm)	**t(mm)**	**Angle**	**Lateral Load Supported(kg)**									
			1000	**2000**	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
40 X 40	**4**	**45°**	2.59	2.59	2.36	2.02	1.78	_	_	_	_	_
		26.5°	2.59	2.59	2.59	2.3	2.04	_	_	_	_	_
50 X 50	**4.5**	**45°**	3.28	3.28	3.28	3.12	2.76	2.5	2.29	2.12	1.97	1.84
		26.5°	3.28	3.28	3.28	3.28	3.14	2.84	2.61	2.43	2.27	2.13
60 X 60	**4**	**45°**	4.06	4.06	4.06	4.06	3.63	3.29	3.02	2.79	2.61	2.44
		26.5°	4.06	4.06	4.06	4.06	4.06	3.74	3.44	3.19	2.99	2.81
80 X 80	**4**	**45°**	5.53	5.53	5.53	5.53	5.53	5.33	4.9	4.55	4.27	4.02
		26.5°	5.53	5.53	5.53	5.53	5.53	5.53	5.53	5.17	4.85	4.58
100 X 100	**4**	**45°**	7	7	7	7	7	7	7	6.58	6.17	5.82
		26.5°	7	7	7	7	7	7	7	7	7	6.61
	6	**45°**	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83
		26.5°	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83
150 X 150	**4**	**45°**	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68
		26.5°	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68
	6	**45°**	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51
		26.5°	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51
220 X 220	**6**	**45°**	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65
		26.5°	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65
	10	**45°**	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31
		26.5°	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31



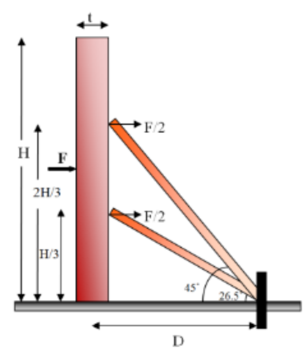
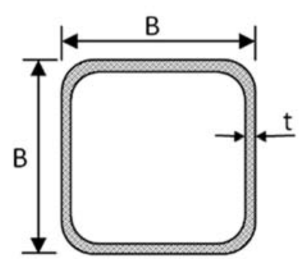


Table 13.11: Loads supported by Two rake shore in steel- square hollow section at 60° and 40°

Maximum length of member for Tata Structura Steel (Square hollow sections) at 60 and 40 degrees (m)												
B X B (mm)	**t(mm)**	**Angle**	**Lateral Load Supported(kg)**									
			1000	**2000**	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
40 X 40	**4**	**60°**	2.59	2.44	1.96	1.66	_	_	_	_	_	_
		40°	2.59	2.59	2.47	2.11	_	_	_	_	_	_
50 X 50	**4.5**	**60°**	3.28	3.28	3.02	2.58	2.28	2.04	1.85	_	_	_
		40°	3.28	3.28	3.28	3.26	2.89	2.61	2.4	_	_	_
60 X 60	**4**	**60°**	4.06	4.06	3.97	3.4	3	2.67	2.46	2.26	2.08	1.91
		40°	4.06	4.06	4.06	4.06	3.79	3.43	3.15	2.92	2.73	2.57
80 X 80	**4**	**60°**	5.53	5.53	5.53	5.5	4.87	4.41	4.04	3.74	3.49	3.27
		40°	5.53	5.53	5.53	5.53	5.53	5.53	5.12	4.76	4.46	4.2
100 X 100	**4**	**60°**	7	7	7	7	7	6.37	5.86	5.44	5.09	4.79
		40°	7	7	7	7	7	7	7	6.86	6.44	6.08
	6	**60°**	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.5	6.09	5.75
		40°	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83	6.83
150 X 150	**4**	**60°**	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.51	9.86	9.3
		40°	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68	10.68
	6	**60°**	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51
		40°	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51	10.51
220 X 220	**6**	**60°**	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65
		40°	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65	15.65
	10	**60°**	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31
		40°	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31	15.31



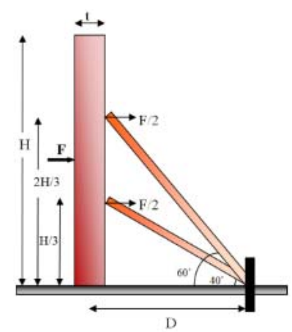
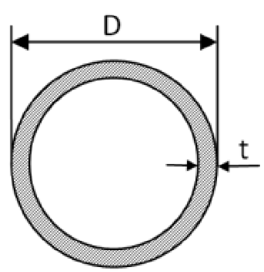


Table 13.12: Loads supported by Two rake shore in steel- circular hollow section at 45° and 26.5°

Maximum length of member for Tata Structura Steel (Circular hollow sections) at 45 and 26.5 degrees (m)													
Nominal Dia (mm)	Outer Dia(mm)	t(mm)	Angle	Lateral Load Supported(kg)									
				1000	2000	3000	4000	5000	6000	7000	8000	9000	10000
40	48.3	4	45°	2.83	2.83	2.64	2.26	_	_	_	_	_	_
			26.5°	2.83	2.83	2.83	2.57	_	_	_	_	_	_
50	60.3	4.5	45°	3.56	3.56	3.56	3.44	3.05	2.76	2.53	2.34	2.18	_
			26.5°	3.56	3.56	3.56	3.56	3.46	3.14	2.88	2.68	2.5	_
80	88.9	4	45°	5.4	5.4	5.4	5.4	5.4	4.94	4.54	4.22	3.95	3.72
			26.5°	5.4	5.4	5.4	5.4	5.4	5.4	5.16	4.8	4.5	4.25
100	114.3	4.5	45°	7	7	7	7	7	7	7	6.7	6.29	5.93
			26.5°	7	7	7	7	7	7	7	7	7	6.74
150	165.1	4.5	45°	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23
			26.5°	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23
200	219.1	6	45°	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57
			26.5°	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57
		8	45°	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44
			26.5°	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44
250	273	6	45°	17	17	17	17	17	17	17	17	17	17
			26.5°	17	17	17	17	17	17	17	17	17	17
		8	45°	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87
			26.5°	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87
300	323.9	8	45°	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11
			26.5°	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11



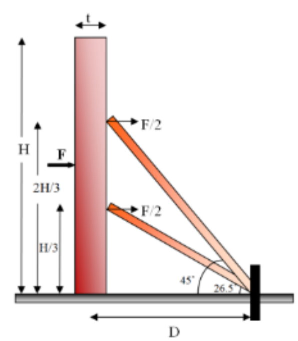
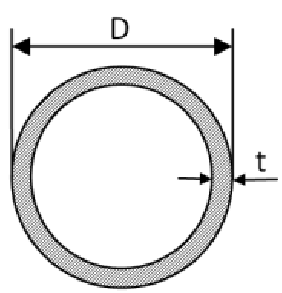
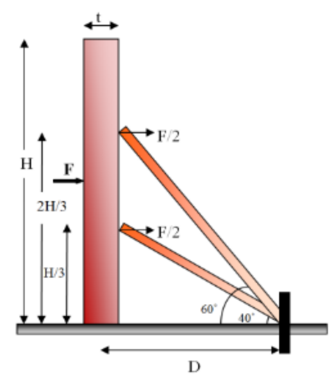


Table 13.13: Loads supported by Two rake shore in steel- circular hollow section at 60° and 40°

Maximum length of member for Tata Structura Steel (Circular hollow sections) at 60 and 40 degrees (m)													
Nominal Dia (mm)	Outer Dia(mm)	t(mm)	Angle	Lateral Load Supported(kg)									
				1000	2000	3000	4000	5000	6000	7000	8000	9000	10000
40	48.3	4	60°	2.83	2.73	2.19	1.85	_	_	_	_	_	_
			40°	2.83	2.83	2.76	2.36	_	_	_	_	_	_
50	60.3	4.5	60°	3.56	3.56	3.33	2.85	2.51	2.26	2.05	1.87	_	_
			40°	3.56	3.56	3.56	3.56	3.18	2.88	2.64	2.45	_	_
80	88.9	4	60°	5.4	5.4	5.4	5.1	4.52	4.08	3.74	3.46	3.22	3
			40°	5.4	5.4	5.4	5.4	5.4	5.16	4.75	4.41	4.13	3.89
100	114.3	4.5	60°	7	7	7	7	7	6.49	5.97	5.55	5.19	4.89
			40°	7	7	7	7	7	7	7	7	6.56	6.2
150	165.1	4.5	60°	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10	9.38	8.85
			40°	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23	10.23
200	219.1	6	60°	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57
			40°	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57	13.57
		8	60°	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44
			40°	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44	13.44
250	273	6	60°	17	17	17	17	17	17	17	17	17	17
			40°	17	17	17	17	17	17	17	17	17	17
		8	60°	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87
			40°	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87	16.87
300	323.9	8	60°	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11
			40°	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11	20.11





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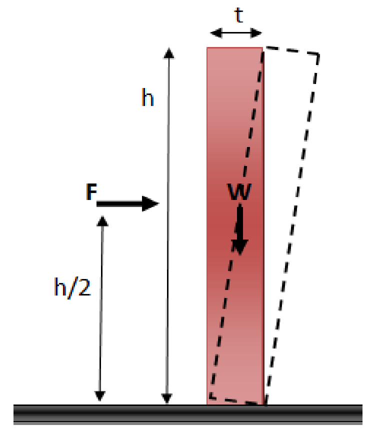
Appendix-1: Basic Calculations for Shoring Rakes

Prepared by: Samson, S., Shravan, V.S.M, Shukla, S. Chacko, N. and Arun Menon, NCSHS, IIT Madras

Single Rake Shores



The lateral load required to overturn the masonry wall about its base is calculated as below:

Considering moment equilibrium about the base of the wall,

$$F\,\frac{h}{2} = W\,\frac{t}{2}$$

Horizontal force, $F = W\,\frac{t}{h}$

where,

W = Self weight of the wall

t = thickness of the wall

h = height of the wall

The calculation of axial force (N) on the rake is described below:

Considering moment equilibrium about the base of the rake (point B),

$$FH - R1(D) - w\frac{D}{2} = 0$$

$$R1 = F\tan\beta - \frac{W}{2}$$

Considering force equilibrium at joint A,

$$Nsin(\beta) = R1$$

$$N = F\sec\beta - \frac{W}{2}cosec(\beta)$$

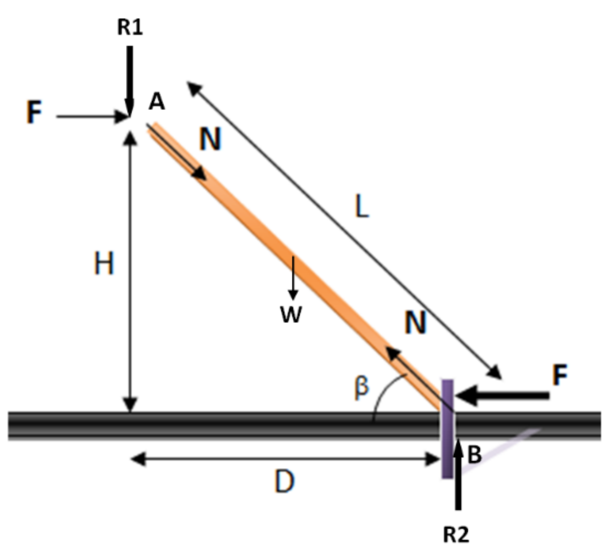
where,

β = angle of inclination of the raker

L = Length of the raker

D = Distance of bottom of raker from the wall

w = Self weight of the raker



Case a) Timber member

The material properties assumed for timber (Sal wood) for calculations as per IS:883 (1994) are given below.

Young's Modulus (E) = 12670 N/mm^2

Allowable stress (f_{cp}) = 7.7 N/mm^2

Depending on the slenderness ratio (L/d), the rakers are classified into three categories, namely short, intermediate and long rakers as per Cl. 7.6.1, IS: 883 (1994).

For short rakers, max permitted length, $L = 11d$

For intermediate, permissible length, $L = K_8 d \left[3\left(1 - \frac{f_c}{f_{cp}}\right)\right]^{1/4}$

For long rakers, permissible length, $L = d\left(0.329\,\frac{E}{f_c}\right)^{1/2}$

where,

L= Length of the raker (maximum permissible limit)

d = Dimension of equivalent square section

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}}$$

f_c = Compressive stress under axial load (N / A)

Case b) Steel members

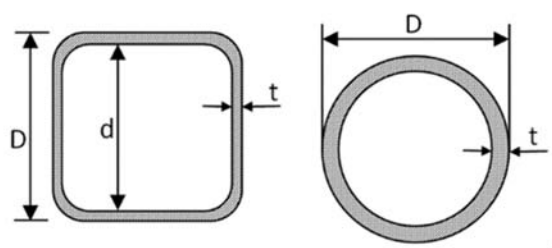
Check for limiting width to thickness ratio of the section

For the section to be semi- plastic,

$d/t < 42\varepsilon$ →Box sections

$D/t < 88\varepsilon^2$ →Circular hollow tubes

where, $\varepsilon = \sqrt{\frac{250}{f_y}}$



Elements which exceed semi-compact limits are to be taken as of slender cross-section. The section should be converted to semi- plastic by taking the semi- compact limiting values and recalculating the width, area and moment of inertia.

An initial length of the member, L_1 is assumed.

The corresponding effective slenderness ratio, $\frac{KL_1}{r_{xx}}$ is calculated.

where, r_{xx} is the radius of gyration

Following the design procedure for compression members as per IS 800: 2007,

Buckling class	= 'a', for hollow sections
Imperfection factor, α	= 0.21 for buckling class 'a'
Euler buckling stress, f_{cc}	$= \frac{\pi^2 E}{(KL/r_{zz})^2}$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$$

$$\phi = 0.5(1 + \alpha(\lambda - 0.2) + \lambda^2)$$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

As per Cl. 7.1.1 of IS: 800 (2007), the permissible axial force in compression members is given by

$P_{dz} \;=\; A\, f_{cd}$

where,

A = Area of cross section of the member

f_{cd} = Design allowable compressive stress (Table 9a, IS:800 (2007))

The section is safe if $N<P_{dz}$

The same procedure is repeated for different lengths of the member and the maximum length for which the member remains safe can be determined. The effective slenderness ratio (KL/r_{xx}) shall not exceed 180.

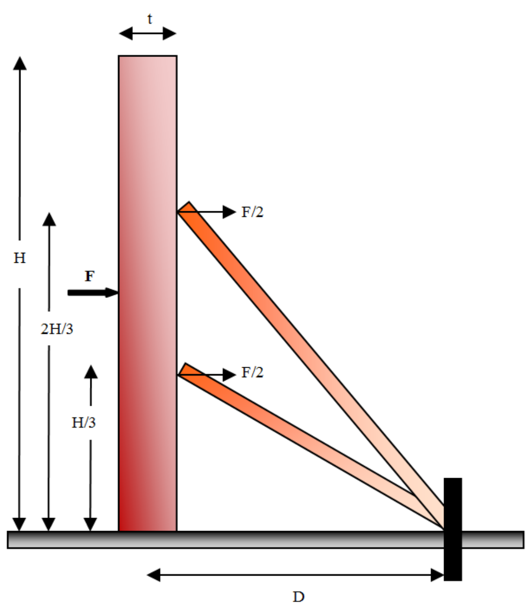
Two-Rake Shores

Assuming the two rakes are placed at the same point on the ground such that the heights at which they are in contact with the wall are H/3 and 2H/3, the horizontal forces acting on the rakers are given by:

Horizontal force in each raker = F / 2

Using the same procedure as detailed above (for single rakers), the permissible lengths for the double rakes for the following two cases are calculated.

i. Rakers are inclined at 45° and 26.5°
ii. Rakers are inclined at 60° and 40°



Calculation of maximum permissible length of shoring members

1. Timber (Sal wood)

Young's Modulus (E) : 12670 N/mm^2

Allowable stress (f_{cp}) : 7.7 N/mm^2

Diameter of the section, D : 150 mm

Angle of inclination : 60°

a) Lateral load supported, F : 1000 kg

Width of equivalent square section , $d = \sqrt{\frac{\pi D^2}{4}} = \sqrt{\frac{\pi\ 150^2}{4}}$ = 132.93 mm

Axial Force in the raker ,$N = F \sec \beta$

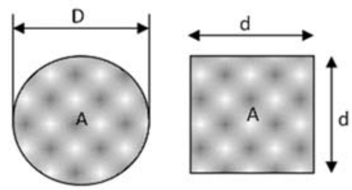
= 1000 x 10 x sec(60)

= 2 x 10^4 N

Compressive stress in the section, $f_c = \frac{N}{A}$

$= \frac{2\ X\ 10^4}{132.93^2} = 1.1318$ N/mm^2

Assuming the raker to be long,



Length of the member, $L = d\left(0.329\frac{E}{f_c}\right)^{1/2}$

$= 132.93\left(0.329\frac{12670}{1.1318}\right)^{1/2}$

= 8.07 m

L/d = 8070/132.93 = 60.71

But L/d cannot be greater than 50. Hence, taking L/d=50,

Maximum permissible length of the member, L = 50 d = 50 x 132.93 = 6.65m

b) Lateral load supported, F : 2000 kg

Axial Force in the raker ,$N = F \sec \beta$

= 2000 x 10 x sec(60)

= 4 x 10^4 N

Compressive stress in the section, $f_c = \frac{N}{A}$

$= \frac{4\ X\ 10^4}{132.93^2} = 2.2635$ N/mm^2

Assuming the raker to be long,

Length of the member, $L = d\left(0.329\frac{E}{f_c}\right)^{1/2}$

$= 132.93\left(0.329\frac{12670}{2.2635}\right)^{1/2}$

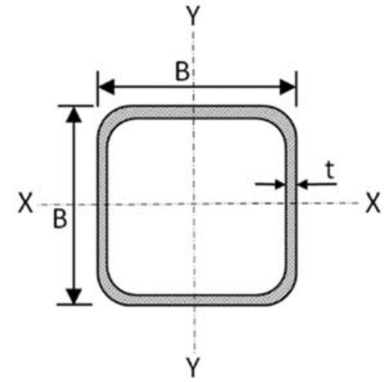
= 5.7 m

L/d = 5700/132.93 = 42.88

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}}$$

$= 0.584\sqrt{\frac{12670}{7.7}}$

$= 23.69$



Since $L/d > K_8$, the assumption that the raker is long is correct.

Therefore, Maximum permissible length of the member, L = 5.7 m

2. Steel

Young's Modulus (E) : 2×10^5 N/mm²

Yield stress, f_y : 310 N/mm²

Section Dimensions, B : 150 mm

t : 4 mm

Area of cross section, A : 22.95 cm²

Unit weight, W1 : 18.01kg/m

Moment of inertia, I :807.82 cm⁴

Angle of inclination : 60°

Radius of gyration, r_{xx} $= \sqrt{\frac{I}{A}} = \sqrt{\frac{807.82}{22.95}} = 5.93$ cm

Checking the limiting width to thickness ratio of the section,

$d = B - 2t = 150 - 2 \times 4 = 142$ mm

$d/t = 142/4 = 35.5$

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{310}} = 0.898$$

$42\varepsilon = 37.72$

$d/t < 42\varepsilon \rightarrow$ Section is semi plastic

Assuming the minimum height at which the raker is placed as 0.75 m , length of the member,

L1 = 0.75/ sin(60) = 0.866 m

$\frac{KL}{r_{xx}} = \frac{866}{59.3} = 14.6$

Self-weight of the member = 0.866 x18.01 = 15.59 kg

Factored self-weight, w = 15.59 x 10 x 1.5 = 233.85

a) Lateral load to be supported = 1000 kg

Factored load, F = 1000 x 10 x 1.5 = 15000 N

$$N = F\sec\beta - \frac{W}{2}cosec(\beta)$$

$= 15000 \sin(60) - \frac{233.85}{2} \operatorname{cosec}(60)$

= 12855.37 N

= 128.55kN

Following the design procedure for compression members as per IS 800: 2007,

Buckling class = 'a', for hollow sections

Imperfection factor, α = 0.21 for buckling class 'a'

Euler buckling stress, f_{cc} $= \frac{\pi^2 E}{(KL/r_{zz})^2} = \frac{\pi^2 X\ 2\ X\ 10^5}{(14.6)^2} = 9260.28$ N/mm²

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{310}{9260}} = 0.183$$

$$\phi = 0.5(1 + \alpha(\lambda - 0.2) + \lambda^2)$$

$= 0.5\ (1+0.21(0.183-0.2)+0.183^2)$

= 0.515

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$

$= \frac{310/1.1}{0.515+(0.515^2-0.183^2)^{0.5}}$

= 282.84 N/mm²

Compressive strength of the member, $P_{dz} = f_{cd}A$

=282.84 x 2295

=649 kN

Since, $N<P_{dz}$ the section is safe.

The same procedure is repeated for different lengths of the member and the maximum length for which the member remains safe can be determined. The effective slenderness ratio (KL/r_{xx}) shall not exceed 180.

Here, the maximum permissible length, L = 180 r_{xx} = 180 x 59.3 = 10679.2 mm

= 10.68 m

Appendix-2:
RVS Forms and Manual for Masonry and RC Buildings, Nepal 2015

Prepared by: Arun Menon and C.V.R. Murty, IIT Madras

Post-Earthquake Rapid Visual Screening Form
Masonry Buildings

Inspection

Inspector ID: *Inspection Date and Time:*

Organisation: *Areas inspected: Exterior only* ☐ *Exterior & Interior* ☐

Building Description

Building Name: *Address:*

Contact Phone no.: *District:*

Municipality/VDC:

Ward No.: *Tole:*

Type of Construction

Adobe ☐ *Stone masonry in mud mortar* ☐ *Stone masonry in cement mortar* ☐

Timber Frame ☐ *Burnt brick masonry in mud mortar* ☐ *Burnt brick in cement mortar* ☐

Sun-dried brick masonry in mud mortar ☐

Type of Floor

Timber ☐ *RC Slab* ☐ *Timber with mud plaster* ☐ *Other* ________________

Type of Roof

Timber ☐ *RC Slab* ☐ *Timber with mud plaster* ☐ *Other* ________________

Primary Occupancy

Residential ☐ *Hospital* ☐ *Government Office* ☐ *Police Station* ☐

Educational ☐ *Industry* ☐ *Office* ☐ *Mixed Use* ☐

Commercial ☐ *Hotel* ☐ *Others*

S.no.	***Life Threatening Parameters***			***Tag***
1.	***Site***	*1.1*	*Ground failure: a. Landslide/Fissures, b. Liquefaction, c. Tilt*	*Red*
		1.2	*Unsafe/tilted adjoining or uphill building*	*Red*
2.	***Out-of-plane failure***	*2.1*	*Separation of walls at junctions or corners*	*Red*
		2.2	*Floor-wall junction separation, with wall out-of-plumb*	*Red*
		2.3	*Gable collapse*	*Red*
		2.4	*Separation of wythes*	*Red*
		2.5	*Damage to masonry plinth*	*Red*
3.	***In-plane damage***	*3.1*	*Horizontal sliding at any storey*	*Red*
		3.2	*Diagonal shear cracking in wall piers and/or spandrels*	*Red*
		3.3	*Crushing of masonry at wall base*	*Red*
4.	***Arches***	*4.1*	*Dislodged keystone or wide crown cracks*	*Red*
5.	***Columns***	*5.1*	*Crushed or out-of-plumb column*	*Red*
6.	***Out-of-plane damage***	*6.1*	*Parapet collapse*	*Yellow*
		6.2	*Chimney collapse*	*Yellow*
7.	***Arches***	*7.1*	*Crown cracking*	*Yellow*
8.	***Material***	*8.1*	*Disintegration of masonry constituents: Unit, mortar or assembly*	*Yellow*
9.			*None of the above*	*Green*

Posting

GREEN	**YELLOW**	**RED**
Usable	*Usable with temporary interventions*	*Unusable*
	Suggested temporary interventions:	

Further Actions

Areas to be barricaded:

Conceptual Visual Screening Recommended ☐

Post-Earthquake Rapid Visual Screening Form
Reinforced Concrete Buildings

Inspection

Inspector ID: Inspection Date and Time:

Organisation: Areas inspected: Exterior only ☐ Exterior & Interior ☐

Building Description

Building Name: Address:

Contact Phone no.: District:

Municipality/VDC:

Ward No.: Tole:

Type of Construction

In-situ ☐ Precast ☐

Type of Roof

Flat RC slab ☐ Sloped RC slab ☐ Others ____________________

Primary Occupancy

Residential ☐ Hospital ☐ Government Office ☐ Police Station ☐

Educational ☐ Industry ☐ Office ☐ Mixed Use ☐

Commercial ☐ Hotel ☐ Others

S.no.	Life Threatening Parameters			Tag
1.	Site	1.1	Ground failure: a. Landslide/Fissures, b. Liquefaction, c. Tilt	Red
		1.2	Unsafe adjoining or uphill building	Red
2.	Form	2.1	Open ground storey frame with shear cracks in columns or beam-column joints	Red
		2.2	Frame with floating columns, with cracked supporting beams	Red
		2.3	Short columns with shear cracks in columns	Red
		2.4	Extensive cracking or out-of-plane collapse of infills	Red
		2.5	Collapse or damage to staircase or any blockade of staircase	Red
3.	Strength	3.1 with	Shear cracks in columns , beam-column joints or shear walls and without spalling of cover concrete	Red
		3.2	Flat slab with punching shear failure	Red
4.	Strength	4.1	Shear cracks in beams, with and without spalling of cover concrete, and infill cracks	Yellow
5.	Form	5.1	Infill-frame separation cracks with no damage to columns	Yellow
		5.2	Dislodging/sliding of rooftop water tanks, lift machine rooms	Yellow
		5.3	Parapet wall collapse	Yellow
6.	Material	6.1	Longitudinal cracks and/or spalling in members with reinforcement corrosion	Yellow
7.		None of the above		Green

Posting

GREEN	YELLOW	RED
Usable	Usable with temporary interventions	Unusable
	Suggested temporary interventions:	

Further Actions

Areas to be barricaded:

Conceptual Visual Screening Recommended ☐

Manual for Post-Earthquake Rapid Visual Screening Forms

Preface:

The purpose of the Rapid Visual Screening (RVS) form is to be able to identify whether or not the building is suitable for immediate occupancy in the post-earthquake period. The RVS does not guarantee safety of the structure in a future event comparable to the main shock, but only addresses safety under existing gravity loads and small aftershocks expected in the post-earthquake period. The most critical question that the RVS addresses is whether or not the vertical load path in the building for gravity loads is intact or not.

Based on the responses to questions on specific structural damage, a building is tagged GREEN, implying usable, YELLOW, implying usable with necessary temporary interventions, and tagged RED, implying unusable in the immediate post-earthquake period. It is noted that a RED tag does not imply demolition; this would require further detailed evaluation, which may also lead to strategies for salvaging the structure.

The Rapid Visual Screening should be carried out ideally by a three-member team, where the form should be filled up individually by two members, and verified by the third. This is to ensure that there is broad consensus on the type of tag appropriate for the building, but by independent, unbiased examination. RVS of a building should not take more than 15 minutes.

A. Description of Life Threatening Parameters for Masonry Structures

If any of the items in points 1 to 5 are observed in the building, then the masonry building being surveyed is RED-tagged.

1. Site-Level Issues:

1.1 Ground failure: a. Landslide or Fissures

Features such as formation of fissures on the ground due to surface manifestation of fault rupture or due to ground displacements, or landslides and failure of hill slopes, in the vicinity of the structure, render the structure susceptible to further damage or collapse, hence potentially unsafe.

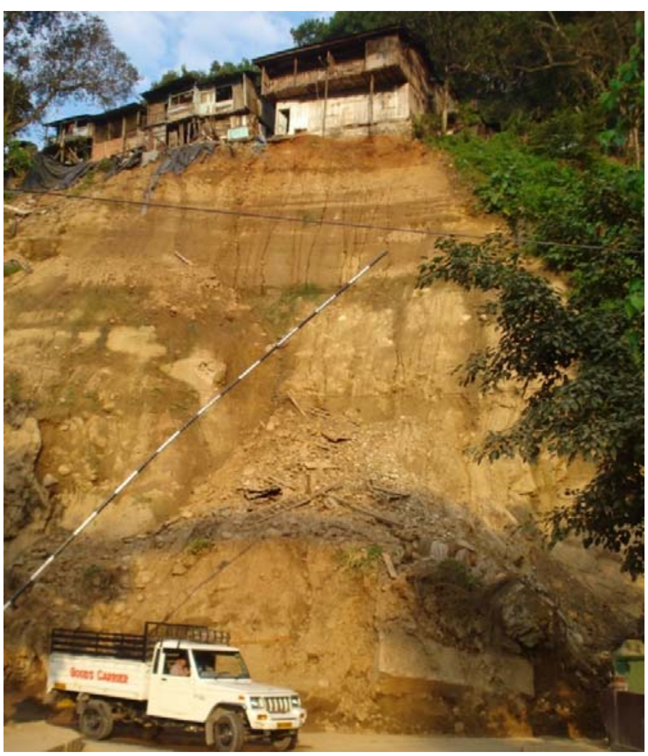


Figure 10: (A) Buildings precariously placed in landslide affected areas; (B) Ground fissures due to loss of slope stability in hill slopes or surface rupture due to fault movement

b. Liquefaction

When the phenomenon of liquefaction is observed at the site of a building, wherein saturated or partially saturated loose or cohesionless soilssignificantly lose their bearing capacity under earthquake groundmotion, then the building situated on such soils should be tagged red.

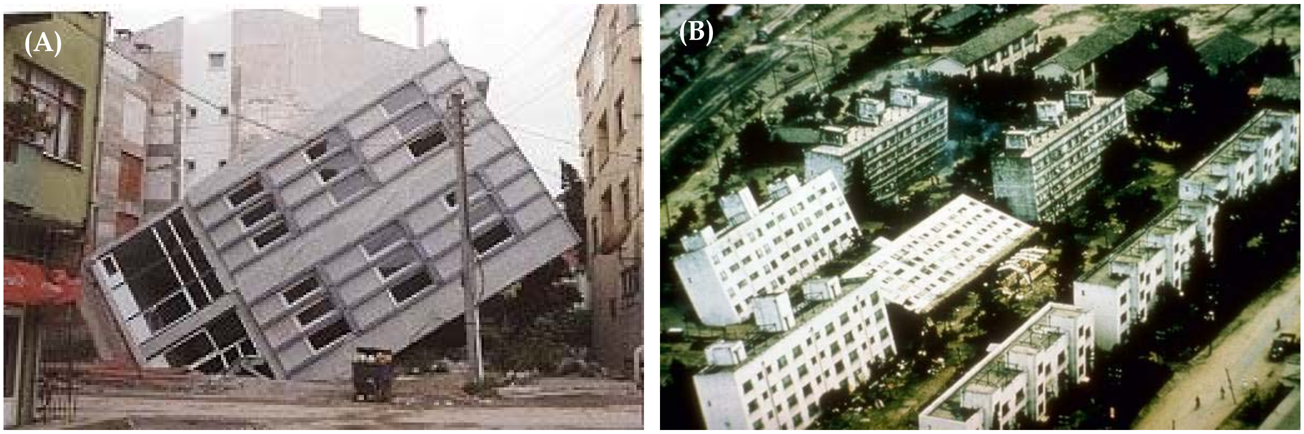


Figure 11: (A) Tilting of building due to liquefaction in the Kocaeli earthquake of August 1999; (B) Tilting of buildings due to liquefaction in the Nigata earthquakeJapan, 1964

c. Tilt

A building which has perceptible tilt due to localised foundation settlement or due to structural damage is considered to be unusable.

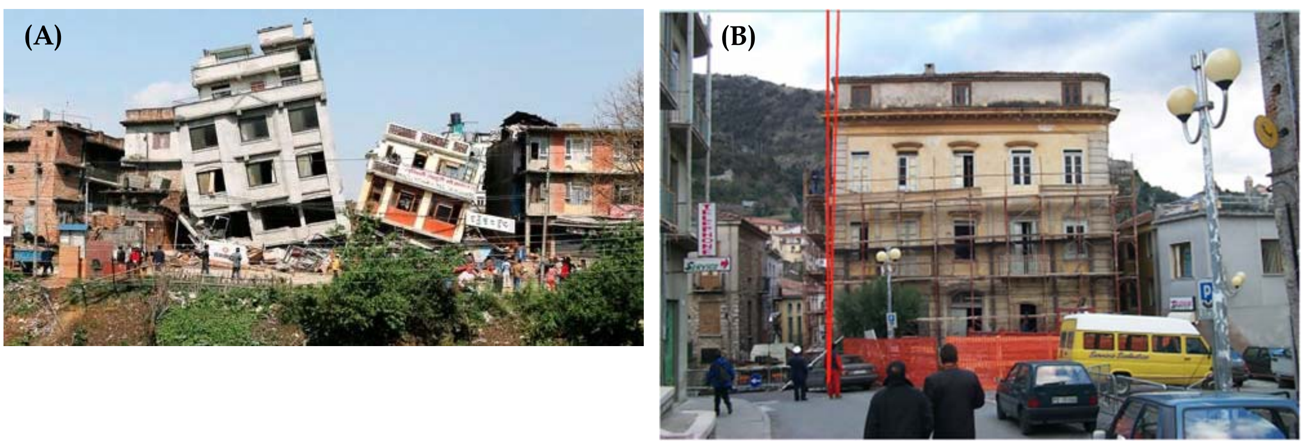


Figure 12: (A) Tilting of building due to structural damage at lower stories; (B) Tilting of building due to soil settlement

1.2 Unsafe adjoining or uphill building

Any building that is sitting either adjoining one that has collapsed or heavily damaged or below one that is uphill and has collapsed or heavily damaged is vulnerable to structural damage in the event of any further movement or collapse of the adjoining or uphill building.

Figure 13: (A) Life safety hazard to building due to collapsed adjoining structure; (B) Life safety hazard to buildings downslope due potential sliding or collapse of heavily damaged uphill building

2.Out-of-Plane Mechanisms: Level-1

2.1 Separation of walls at junctions or corners

Out-of-plane mechanisms are typically observed at the uppermost portions of a structure, where the overturning resistance is the least due to decreasing gravity loads coming from self-weight.Separation of masonry walls at the junctions of walls or corners of walls is due to poor interconnections between walls. Box action in the masonry structure is lost as a result, and the free-standing walls are vulnerable to further movement out-of-plane and collapse. The roof or floor slab is no longer being supported uniformly by the walls and poses a serious threat.

Figure 14: Buildings showing separation at the corners of perpendicular walls and out-of-plumb walls

2.2 Floor-wall junction separation with out-of-plumb wall

Separation of masonry walls at the junction of walls with the floor or roof system, showing pull out of timber joists or rafters or RC slab, is due to poor interconnections between walls and the roof/slab system. If the walls are perceptibly out-of-plumb, the structure has to be red-tagged. Box action in the masonry structure is lost as a result, and the free-standing walls are vulnerable to further movement out-of-plane and collapse. The roof or floor slab is no longer being supported uniformly by the walls and poses a serious threat.

(A)

(B)

Figure 15: (A) Separation of roof slab and joists from supporting walls with out-of-plumb walls; (B) Separation and sliding of roof system from the supported façade wall, which is also out-of-plumb

2.3 Gable collapse

The tall free-standing masonry gable walls, typically at the ends of a building with sloped roof are susceptible to out-of-plane movement and severe damage or collapse due to poor interconnections with the perpendicular return walls and the roof or floor slab system. Such a damage or collapse implies loss of box action of the masonry and non-uniformly supported roof or slab systems.

(A)

(B)

Figure 16: (A) Building showing a gable wall that has partially collapsed inwards with heavy debris loading the floor and posing a threat to the structure; (B) A building showing outwards collapse of the gable wall, leaving the supported roof system and return walls further vulnerable

2.4 Separation of wythes

When multi-wythe (or multi-leaf) walls are subjected to out-of-plane shaking there could be collapse of the exterior wythe. This is primarily due to the poor quality of masonry walls. The out wythe collapses because the floor or roof slab system is typically supported on the inner wythe and not along the full thickness of a wall. Hence, restraint against lateral translation and rotation is offered to the inner wythe and not the external wythe, thereby causing it to fail out-of-plane. Such collapse of the external wythe results in an increase in the slenderness ratio (height to thickness ratio) with potential buckling of the wall and loss of support to the floor or roof system.

Figure 17: Separation and collapse of the external wythe of the multi-wythe wall

2.5 Damage to masonry plinth

Partial or complete collapse of the plinth of the masonry structure (typically made in random rubble masonry), renders the structure unsafe, due to the significant disturbance of the supporting base of the structure.

Figure 18: (A) Tilted structure due to partial collapse of RR masonry plinth; (B) Partial collapse of the structure due to disintegration of the RR masonry plinth of the structure

3.In-Plane Mechanisms:Level-1

3.1 Horizontal sliding at any storey

When interconnections between walls and between walls and roof/floor system are adequate, the in-plane shear resisting capacity of the masonry walls can be garnered. Horizontal sliding resulting in relative displacement between walls at any storey indicates that the in-plane shear capacity of the structure has been exceeded and the in-plane walls have failed in shear. The vertical load-carrying capacity is compromised after shear failure in the in-plane walls, which typically support the roof or floor system.

Figure 19: (A) Shear-sliding of the upper storey relative to the lower one; (B) Sliding-shear cracks noticed at the wall base or at the junction of the wall with the floor slab

3.2 Diagonal shear cracking in wall piers and/or spandrels

When interconnections between walls and between walls and roof/floor system are adequate, the in-plane shear resisting capacity of the masonry walls can be garnered. Diagonal shear cracking showing the classical "x-cracking" is a predominant shear failure mechanism in in-plane walls. The vertical load-carrying capacity is compromised after shear failure in the in-plane walls, which typically support the roof or floor system. Such failure is typically noticed in the lower stories of the structure where the shear demand due to the ground motion is expected to be the greatest.

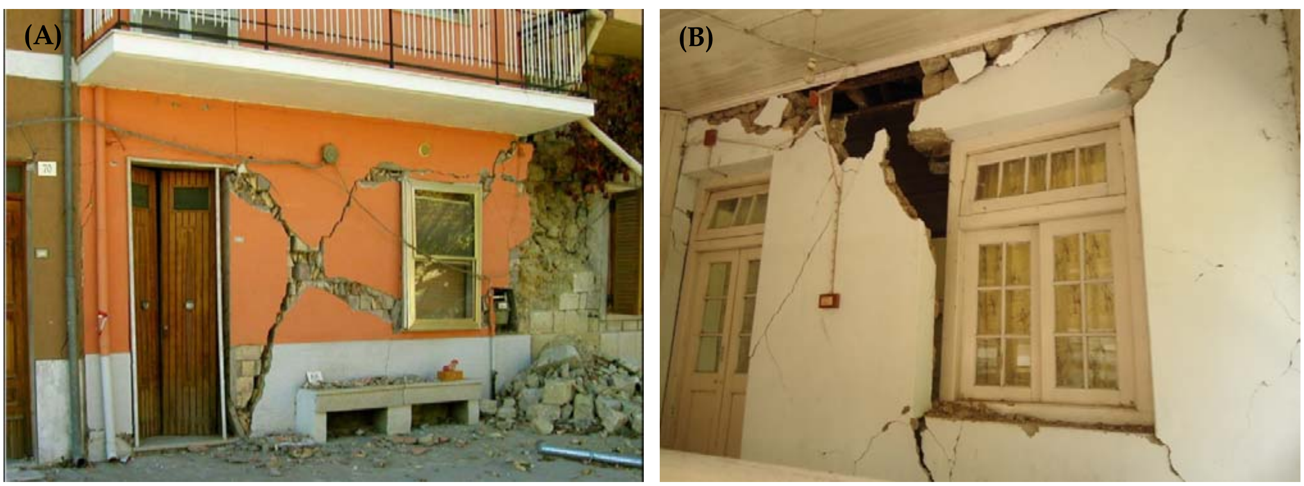


Figure 20: (A) Life safety hazard to building due to collapsed adjoining structure; (B) Life safety hazard to buildings downslope due potential sliding or collapse of heavily damaged uphill building

3.3 Masonry crushing at wall base due to rocking of piers

Crushing of the compressed toe of a masonry wall is indication of the ultimate state of a masonry wall pier underdoing in-plane rocking response, an in-plane shear-resisting mechanism. The vertical load-carrying capacity is compromised after shear failure in the in-plane walls, which typically support the roof or floor system.

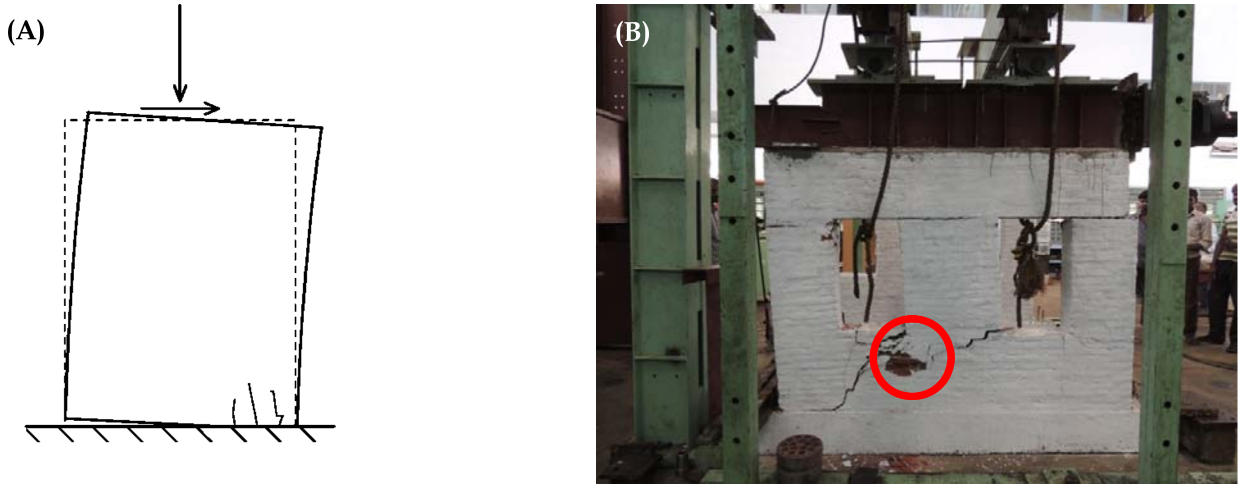


Figure 21: (A) Sketch showing the crushing of the compressed toe at the base of the masonry wall due to in-plane rocking mechanism in the wall pier; (B) Crushing observed in the wall pier in a wall subjected to in-plane deformation

4. Damage to arches

4.1 Dislodged keystone or wide crown cracks

Load-bearing arches in masonry structures are either built out of stone units with a key stone at the crown or with brick units and binding lime mortar. Arches are very sensitive to relative lateral movement of the supporting piers or walls of the arch, leading to a loss of compression action in the arch, which is the load carrying mechanism in an arch. Dislocation or collapse of the keystone at the crown of an arch in stone masonry or wide cracks running to the full depth in a brick masonry arch at the crown, render the arch ineffective against the gravity loads they are meant to carry. This also leads to loss of box action of the masonry structure, due to the dilation of the walls and opening of the arches.

Figure 22: Dislocation and collapse of the keystone (at the crown) and voisseurs (other units) of the arch

5. Damage to columns

5.1 Crushed or out-of-plumb column

Splitting cracks or crushing at the base of masonry load-bearing columns, and/or loss of alignment of the columnsresults in eccentric loading of the columns and potential for collapse, rendering the supported elements susceptible to collapse.

Figure 23: Splitting crack in the stone column and crushing at the base

6. Out-of-plane damage mechanisms

6.1 Parapet collapse

Non-structural damage and collapse of elements such as chimneys at the uppermost level of buildings is a recurrent feature. They pose a serious falling hazard that can cause death or injury. A structure can be made accessible after such falling hazards have been secured or removed. Hence, the building is YELLOW-tagged when such damage is noticed.

6.2 Chimney collapse

Non-structural damage and collapse of elements such as parapet walls at the uppermost level of buildings is a recurrent feature. They pose a serious falling hazard that can cause death or injury. A structure can be made accessible after such falling hazards have been secured or removed. Hence, the building is YELLOW-tagged when such damage is noticed.

Figure 24: Damage to the chimney (a non-structural component) of the building, posing a serious falling hazard

7. Damage to arches

7.1 Crown cracking

When cracks at the crown of an arch are not wide and do not run to the full depth of the arch, the structure can be accessed after necessary interventions. Hence the building is yellow-tagged.

8. Material quality

8.1 Disintegration of masonry constituents: Unit, mortar or assembly

Masonry walls could show localised disintegration of load-bearing walls due to poor quality of the units (brick units or stone blocks) and the binding mortar (typically mud or poor lime mortar). Interventions are required before accessing the structure. Hence the building is yellow-tagged.

Figure 25: Local disintegration of masonry due to poor quality of units (brick or stone blocks) and mortar (mud or poor lime mortar)

7.None of the above

If none of the life-threatening parameters listed in points 1-5 (for RED-tagging) and 6-8 (for YELLOW-tagging are observed, then the building should be GREEN-tagged.

B. Description of Life Threatening Parameters for Reinforced Concrete Structures

If any of the items in points 1 to 3 are observed in the building, then the RC building being surveyed is RED-tagged.

1. Site-Level Issues:

1.1 Ground failure: a. Landslide or Fissures

Features such as formation of fissures on the ground due to surface manifestation of fault rupture or due to ground displacements, or landslides and failure of hill slopes, in the vicinity of the structure, render the structure susceptible to further damage or collapse, hence potentially unsafe.

Figure 26: (A) Buildings precariously placed in landslide affected areas; (B) Ground fissures due to loss of slope stability in hill slopes or surface rupture due to fault movement

b. Liquefaction

When the phenomenon of liquefaction is observed at the site of a building, wherein saturated or partially saturated loose or cohesionless soils significantly lose their bearing capacity under earthquake groundmotion, then the building situated on such soils should be tagged red.

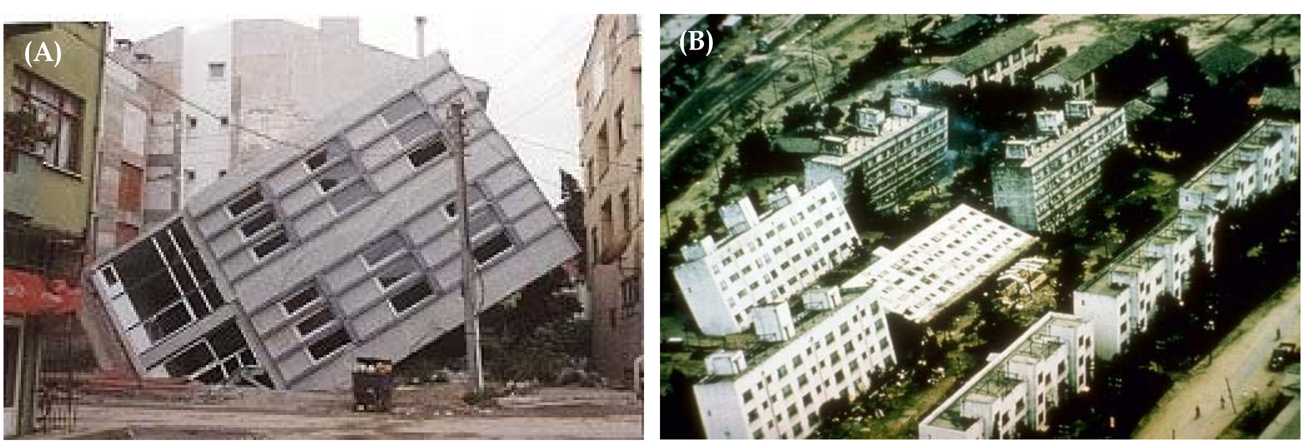


Figure 27: (A) Tilting of building due to liquefaction in the Kocaeli earthquake of August 1999; (B) Tilting of buildings due to liquefaction in the Nigata earthquakeJapan, 1964

c. Tilt
A building which has perceptible tilt due to localised foundation settlement or due to structural damage is considered to be unusable.

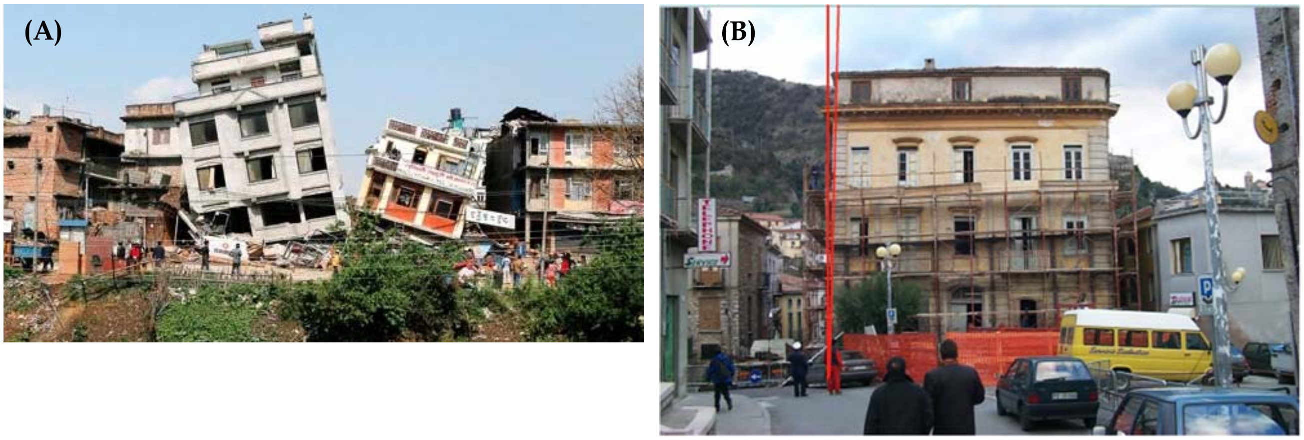


Figure 28: (A) Tilting of building due to structural damage at lower stories; (B) Tilting of building due to soil settlement

1.2 Unsafe adjoining or uphill building
Any building that is sitting either adjoining one that has collapsed or heavily damaged or below one that is uphill and has collapsed or heavily damaged is vulnerable to structural damage in the event of any further movement or collapse of the adjoining or uphill building.

Figure 29: (A) Life safety hazard to building due to collapsed adjoining structure; (B) Life safety hazard to buildings downslope due potential sliding or collapse of heavily damaged uphill building

2. Form-related damage

2.1Open ground storey frame with shear cracks in columns or beam-column joints
Presence of an open ground storey in an RC frame building results in irregular distribution of strength and stiffness along the height of the structure, and a ground storey with lower strength and stiffness relative to the rest of the structure. The open ground storey is then highly susceptible to structural failure, bringing the entire structure down. If the building being surveyed is an open ground storey building with shear failure (as discussed in point 3.1, and shown in Figure 26), then the building must be RED-tagged.

Figure 30: Failure of ground storey columns in an open ground storey building

2.2Frame with floating columns, with cracked supporting beams

Presence of floating columns in RC frame buildings(i.e. columns that are terminated at upper storeys and not taken down to the foundation, creating discontinuity in the vertical load path, or columns that are offset at lower storeys) results in discontinuities in the vertical load path for gravity loads, and large spans for supporting beams. If the building being surveyed has floating columns with structural cracks in the adjoining beams then the building must be RED-tagged.

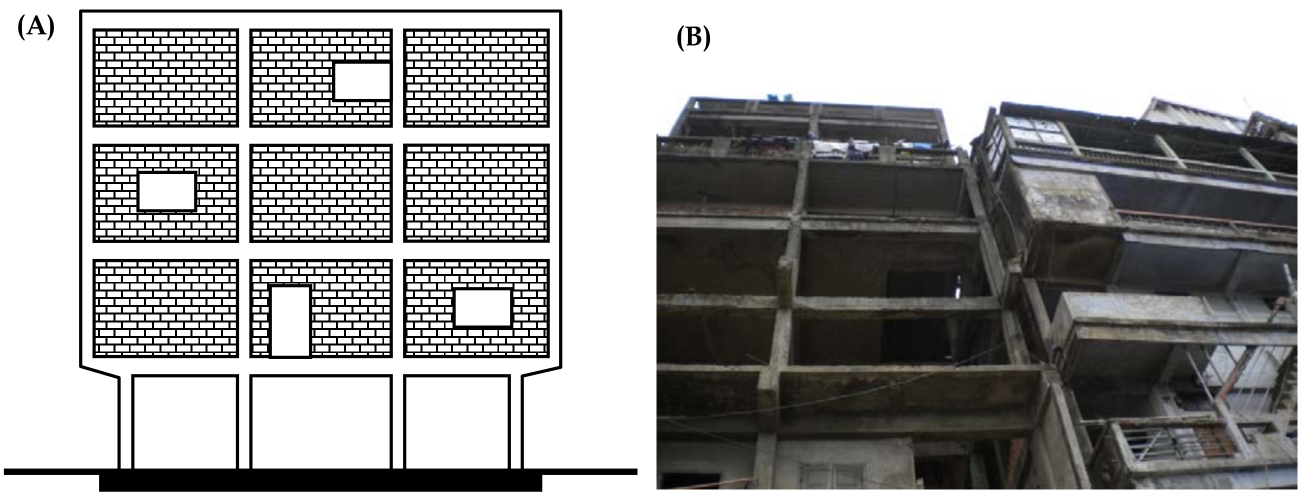


Figure 31: (A) A sketch showing the columns offset at the lower storey for architectural and functional reasons; (B) An example of a building with floating columns

2.3Short columns with shear cracks in columns

When RC frames are provided with partial height infill masonry panels, then the abutting partial height panel reduces the effective length of the columns and leads to a short-column effect with shear damage or failure in the column, unless the column is designed for the increased shear demand.If the building being surveyed has short columns with shear failure (as discussed in point 3.1, and shown in Figure 26), then the building must be RED-tagged.

Figure 32: Shear failure in RC short columns, created due to the partial height infill panels in the RC frame

2.4Extensive cracking or out-of-plane collapse of infills

Due to RC frame and infill masonry interaction during an earthquake, there is propensity for separation at the junction of the frame and infill. In case the infill panel is heavily damaged and/or collapses out of its plane, then the building should be RED-tagged.

Figure 33: Extensive cracking in the infill masonry panels and out-of-plane collapse of some infill panels in an RC structure

2.5 Collapse or damage to staircase or any blockade of staircase

The staircase core of an RC frame is typically the stiffest location and element of the structure, attracting significant share of the shear demand during an earthquake. Collapse of the staircase or damage in the structural elements of the staircase, or any blockade of the staircase due to fallen debris renders the building inaccessible, and hence must be RED-tagged.

Figure 34: Collapse of the staircase of an RC frame structure with shear damage to the columns

3. Strength-related damage

3.1 Shear cracks in columns, beam-column joints or shear walls with and without spalling of cover concrete

Shear failure of RC columns is the most undesirable behaviour in an RC frame structure. Shear cracking and failure is associated with inclined structural cracks in the concrete, spalling of cover concrete, snapping of steel ties (which may have only 90° hooks) and bucking of longitudinal steel. Shear failure of columns jeopardises the gravity load carrying capacity of the frame, and hence the building must be red-tagged.

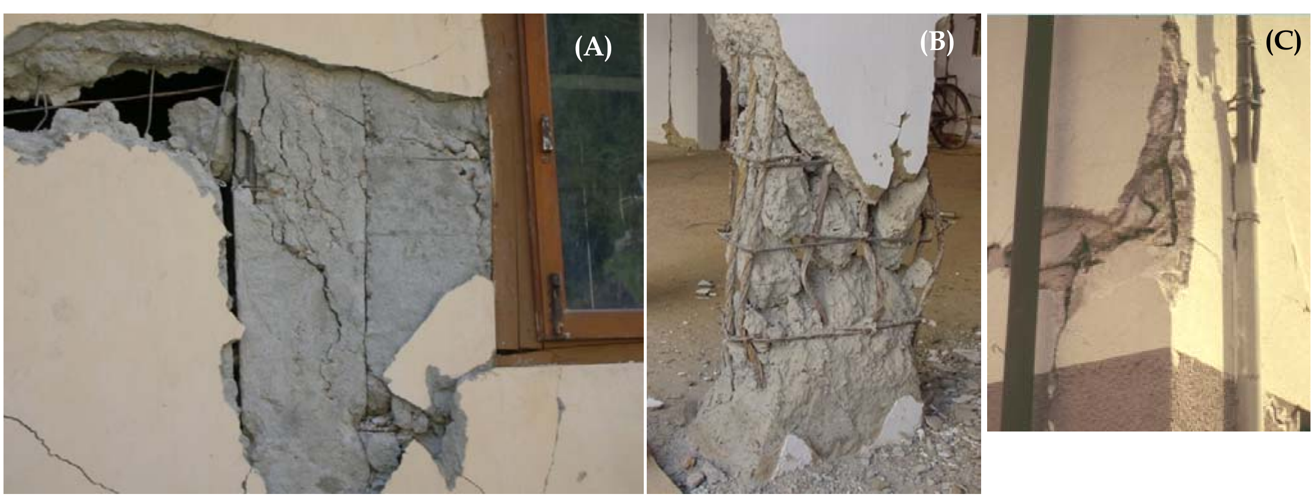


Figure 35: (A) Inclined cracks and spalling of cover concrete; (B) Inclined failure plane with spalling of concrete, crushing of core concrete, snapping of ties(note 90° hooks in stirrups)and buckling of longitudinal bars, (C) Inclined failure plane with spalling of concrete, crushing of core concrete and buckling of longitudinal bars

3.2Flat slab with punching shear failure

RC structures with flat slab construction are susceptible to punching shear failure, where the column punches through the RC slab, if the slab is not designed adequately to account for additional shear forces.

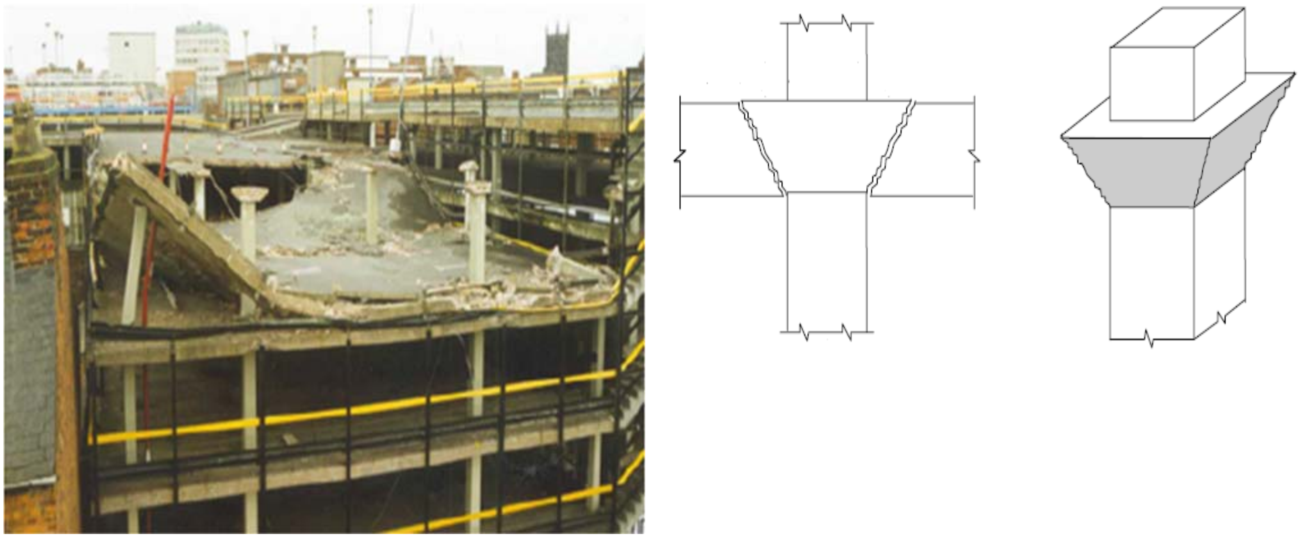


Figure 36: (A) Tilting of building due to structural damage at lower stories; (B) Tilting of building due to soil settlement

4. Strength-related damage

4.1Shear cracks in beams, with and without spalling of cover concrete, and infill cracks

If inclined shear cracks are noticed in the beams with and without spalling in the cover concrete and damage to infill panels, the structure should be tagged yellow.

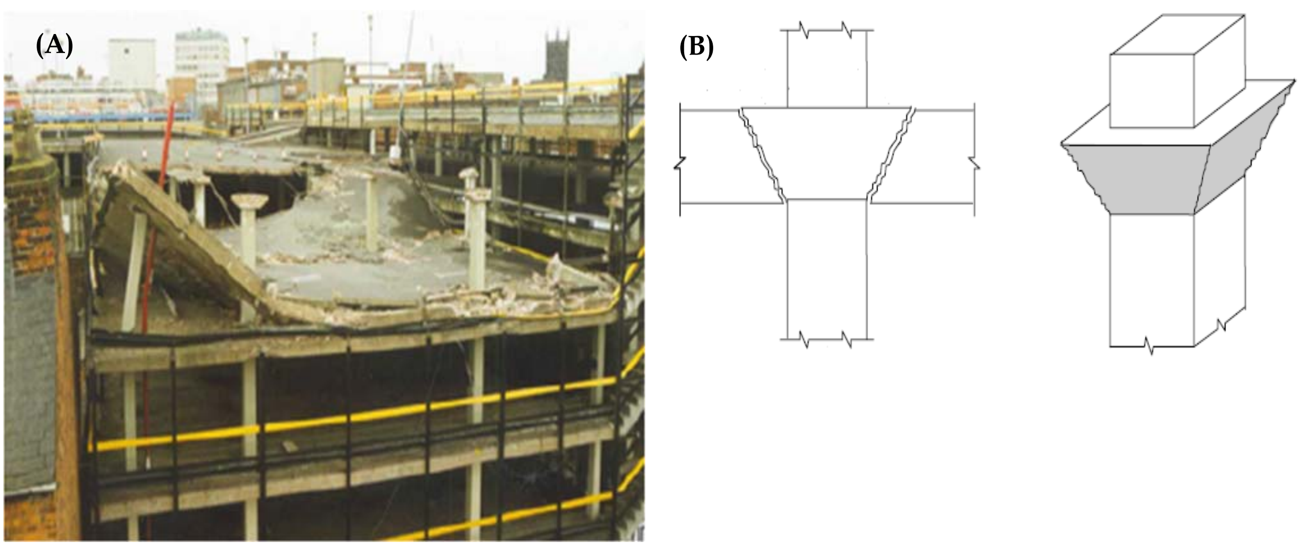


Figure 37: (A) Building showing punching shear failure in the flat slab; (B) Formation of punching shear damage at the interface of column and flat slab

5. Form-related damage

5.1Infill-frame separation cracks with no damage to columns

Due to RC frame and infill masonry interaction during an earthquake, there is propensity for separation at the junction of the frame and infill. In case the infill panel is separated from the frame and there are no structural cracks in the frame, then the building can be yellow-tagged and accessible after necessary interventions are carried out to address the free-standing infills.

Figure 38: (A) Infill-frame separation in an RC frame; (B) Infill damage alone

5.2Dislodging/sliding of rooftop water tanks, lift machine rooms

Non-structural damage and collapse of elements such as water tanksand structural damage involving dislodging or sliding of machine room, at the uppermost level of buildings, is a recurrent feature. They pose a serious falling hazard that can cause death or injury. A structure can be made accessible after such falling hazards have been secured or removed. Hence, the building is YELLOW-tagged when such damage is noticed.

Figure 39: Overturning or dislodging of overhead water tank

5.3Parapet wall collapse

Non-structural damage and collapse of elements such as parapet walls at the uppermost level of buildings is a recurrent feature. They pose a serious falling hazard that can cause death or injury. A structure can be made accessible after such falling hazards have been secured or removed. Hence, the building is YELLOW-tagged when such damage is noticed.

6. Material deterioration

6.1 Longitudinal cracks and/or spalling in members with reinforcement corrosion

Often corroded steel reinforcement in RC structures could aggravate earthquake-related damage. Longitudinal splitting cracks in beams or columns and spalling of cover concrete could be induced by corrosion, and aggravated by earthquake shaking.

7. None of the above

If none of the life-threatening parameters listed in points 1-3 (for RED-tagging) and 4-6 (for YELLOW-tagging are observed, then the building should be GREEN-tagged.