DESIGN FOR PROGRESSIVE COLLAPSE[1]

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This Technical Note outlines the design of column-supported conventionally reinforced or post-tensioned floor systems against progressive collapse.

A.1 PROGRESSIVE COLLAPSE

The primary design objectives of a floor system are:
- ❖ Acceptable "in-service" performance;
- ❖ adequate strength against probable overload, namely "safety;"
- ❖ integrity of the structure against local failure of one of its primary structural members – "robustness;" and
- ❖ Economy.

"Robustness," also referred to as "toughness," is the ability of the structure to avoid disproportionate collapse in part, or its entirely, as a result of failure of one of its primary structural members. The focus of this Technical Note is the practical design for robustness in common residential and commercial buildings.

The probable causes for collapse of a structural member are:

- ❖ Blast or explosion
- ❖ Earthquake
- ❖ Fire
- ❖ Human error in original structural design
- ❖ Overload

Design for damage control against extensive blast forces, such as may be caused by terrorism is limited to specific construction that requires security beyond the common commercial and residential buildings. The design for such events is beyond the scope of this Technical Note

For common residential and commercial buildings, the required robustness is limited to avoiding progressive collapse of the structural frame resulting from failure of a single floor panel, a single column, or a limited length of a load bearing wall.

ACI 318/IBC 's prescriptive design for gravity loads includes provisions for loss of a floor panel. The provision is handled through installation of "integrity" steel along the support lines. The integrity steel is sized to carry the weight of the collapsed panel under the assumption that the collapsed panel is hanging over its otherwise supporting columns. It is emphasized that the scenario used to size the integrity reinforcement is an empirical procedure, deemed to provide adequate safety against progressive collapse.

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A.2 DESIGN ASSUMPTIONS

Statistically, the likelihood of collapse of a load bearing member of a structure is considered to be in the same class as that of a major earthquake – a rare event. Conceptually, from an economical design perspective, excessive damage is accepted, but failure of the structure has to be avoided. The design considers "life safety," to override "damage control."

A.2.1 Extent of Damage
Consider the concrete fame of the building shown in Fig. A.2-1. The framing is assumed to consist primarily of concrete floor slabs and column supports, with or without core walls.
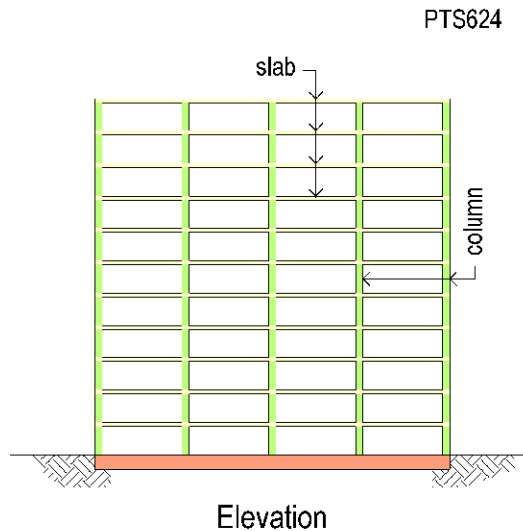

FIGURE A.2-1 Building Frame

Figure A.2-2 shows the concrete frame with one of the interior columns removed. The robustness deemed necessary for design of the frame is to contain the adverse effects of the removed column to the adjoining floor panels, and to the levels above it. The anticipated damage is to be limited to large floor panel displacements. However, the displaced panels should not disintegrate. The panels are to retain their integrity in sustaining the gravity loads likely to be present at the time.
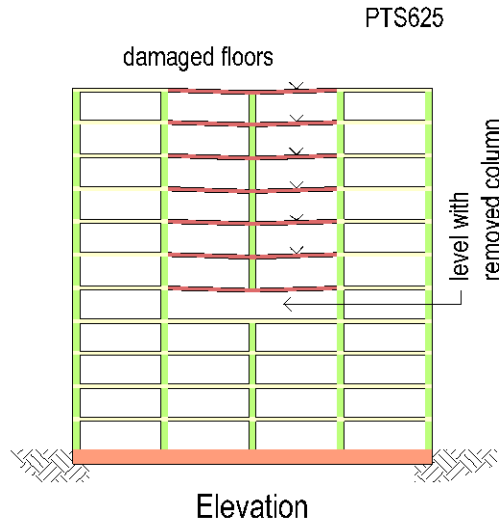
FIGURE A.2-2 Progression of Damage Caused by Removal
of an Interior Column

A.2.2 Design Loads
The design load for progressive collapse is based on the following load combination [UBC].

$$U_1 = 1.00DL + 0.25LL \qquad \text{(Exp A.2.2-1)}$$

Where,

U_1 = design load for progressive collapse;
DL   = actual selfweight of the affected regions; and
LL   = specified design live load over the affected regions.

A.3 - COLUMN DESIGN FOR ROBUSTNESS

Unless a floor slab is a transfer plate, for common commercial and residential construction, the floor slabs do not have the strength to support the weight of a column from above, should the continuity of a column line be interrupted through the removal of one of the columns. Removal of a supporting column results in large displacement of the slab under the load from above. This is followed by the downward movement of the column line from above, coupled with successive damage to the upper level slabs tributary to the load of the removed column (Fig. A2.2). The effective elimination of the load carrying capacity of the interrupted column line leads to the transfer of the load of the disabled columns to the adjacent columns.

Based on ACI 318/IBC, or European code EC2, the design capacity of a column is provided with a margin of safety, adequate to sustain the added load caused from the collapse of an adjacent column. Consequently, no additional design or construction measures are required for the columns to account for the overload resulting from the failure of an adjacent column. The following explains.

Figure A.3-1a shows partial plan of a column supported floor. Part (b) of the figure shows the loss of an interior column and the re-assignment of the tributary of the panel it supported to an adjacent column. The figure also shows the original tributary of an adjacent column. The review of the tributaries marked on the figure leads to the conclusion that the increase in load of a column adjacent to the one failed is 25% for a regular square layout. The increase in load will be of the same order of magnitude for other support arrangements.

(a) Partial plan

(b) Plan - failed central column
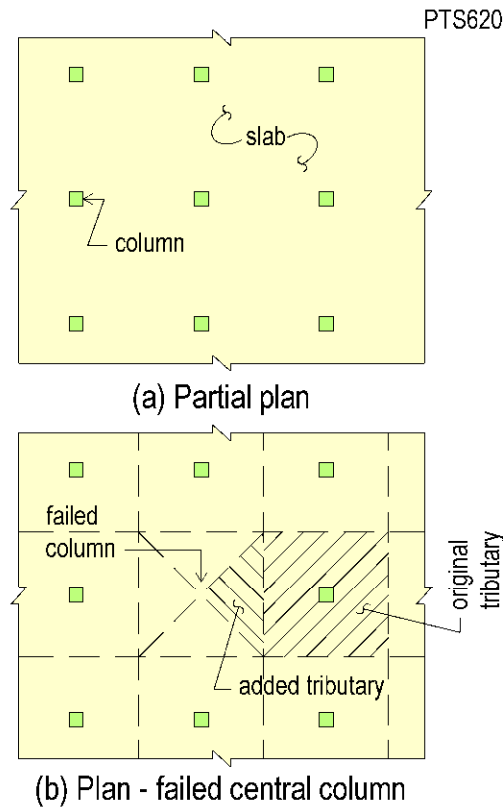
Load Redistribution, Failed Central Column

FIGURE A.3 -1

Using ACI 318/IBC the load combination for gravity design of a column is given by:

$U_2 = 1.2DL + 1.60LL$ (Exp. A.3-1)

Where,

U_2 = design load for column.

Comparison of the design loads U_1 and U_2 leads to the conclusion that the basic design load for a column under gravity design exceeds the design load under progressive collapse by well over 25%. In conclusion, the gravity design of columns provides adequate reserve of strength to share the load from failure of an adjacent column. No additional measure is required, beyond providing adequate ductility around the column/slab connection to accommodate the large rotations that are likely to take place from displacement of the damaged panels.

Figures A.3-2 and 3 illustrate the same concept for an exterior or corner column. The information in the figures leads to the same conclusion as for the removal of an interior column.

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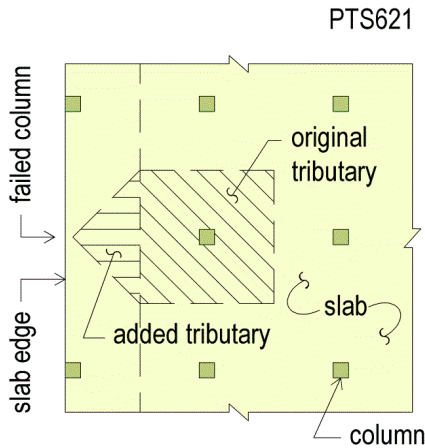
Plan - Redistribution of Load
Due to Failed Edge Column

FIGURE A.3-2
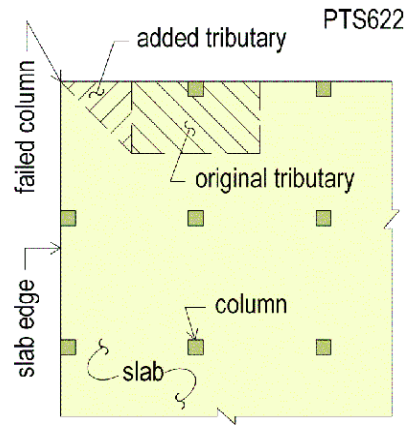
Plan - Redistribution of Load
Caused by Failed Corner Column

FIGURE A.3-3

A.4 TRANSFER OF LOAD FROM DAMAGED SLAB PANELS

A.4.1 Removal of a Central Column: The premise of slab design for robustness is that a floor panel shall not disintegrate and fall over the level below. The fall of a panel over the level below can lead to the failure of the lower slab, followed by progression of panel collapse down the building. The force diagram and the load path for the panels supported by the removed column are shown in the partial elevation of Fig. A.4.1-1. The figure shows that the displacement in the damaged panels will result in a tensile force T in the affected slabs. The capacity of the slab reinforcement to develop the tensile force T, along with a reduction in force P from above, will determine the downward displacement of the damaged slab. In the limit, on the premise that the damaged panels retain their cohesiveness, the displacement at its lowest point will reach the point of removed column. It is important to note that the described scenario will neither overload, nor damage the levels below.
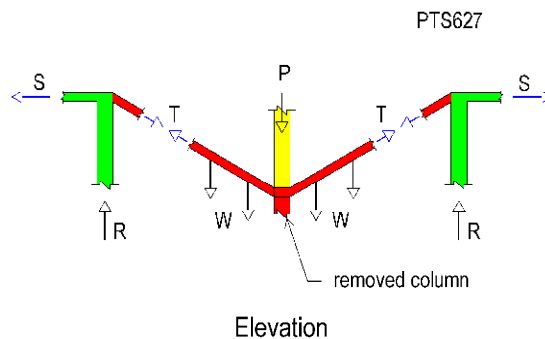
Force Diagram of Panels with
Removed Interior Column

FIGURE A.4.1-1

It is important to note that the first line of resistance by the panel's tributary to the removed column will be development of large flexural stresses, hinge lines and failure in the configurations similar to those shown in Fig. A.4.1-2. Whether the initial failure mode of the slab will be in the form of a circular central

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region or that of triangular shaped sub-panels shown in the figure depends on the disposition of the reinforcement in the slab. It is not economical to design a slab according to the failure modes depicted in Fig. A.4.1-2. The diameter of the failure circle shown in part (b) of the figure, and the final mode shown in par (a) depend on the amount and arrangement of top reinforcement. Top reinforcement will be necessary over the entire panel, since the location of the failure circle will depend on the location and amount of top bars. More importantly, the large downward deformation necessary to relieve the force P from the upper columns may not be readily accommodated by the failure mechanism depicted in the figure.
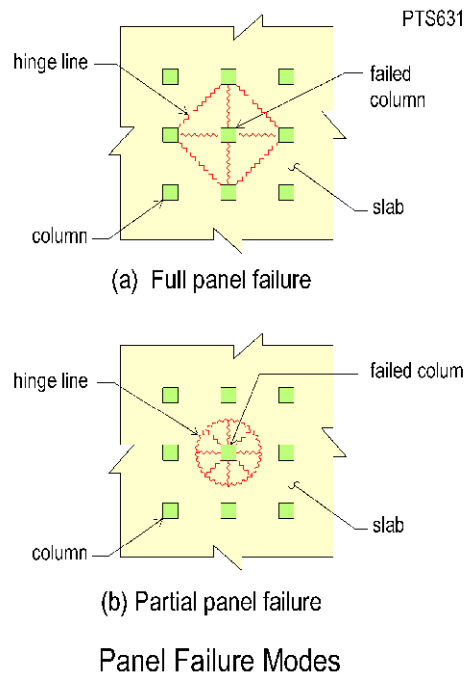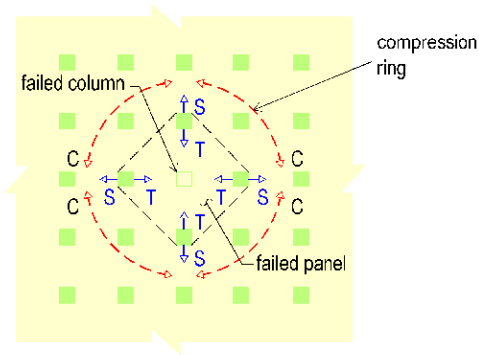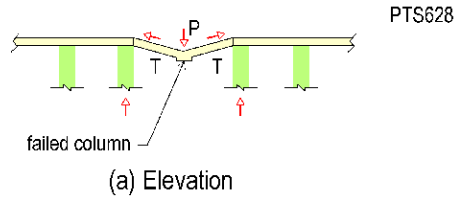
FIGURE A.4.1-2

Figure A.4.1-3 is based on the load path considered feasible and used in design for the removed column. The tensile force S resulting from the failure of the adjacent slab and developed to balance the tension of the integrity steel T will create a compression ring around the panels of collapsed column as shown in Fig. A.4.1-3b. The tensile force T in the slab will also result in the force S in the adjacent panels. The load path will be completed by the compression ring shown in the figure. The inherent resistance of the slab in compression can sustain the forces of the compression ring, but the force S has to be designed for.

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(a) Elevation

(b) Plan-Failed region and vicinity

Force Diagram for Faliure of an Interior Column

FIGURE A.4.1-3

A.4.2 Removal of an Edge Column: Figure A.4.2-1 shows the elevation of a concrete frame showing the progressive damage to upper levels from removal of an edge, or corner column.
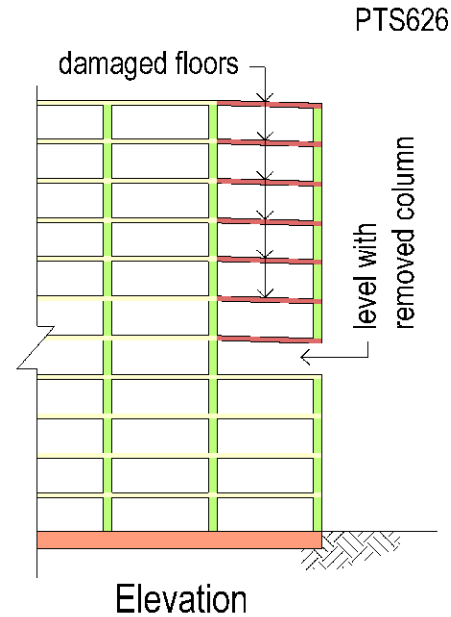

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Elevation

FIGURE A.4.2-1 Progressive Damage from Removal of an Edge
or Corner Column

The elevation of the load path envisaged for the design of the damaged region is analogous to that shown for the failure of a central column in Fig. A.4.1-1. The plan view of the load transfer is shown in Fig. A.4.2-2.
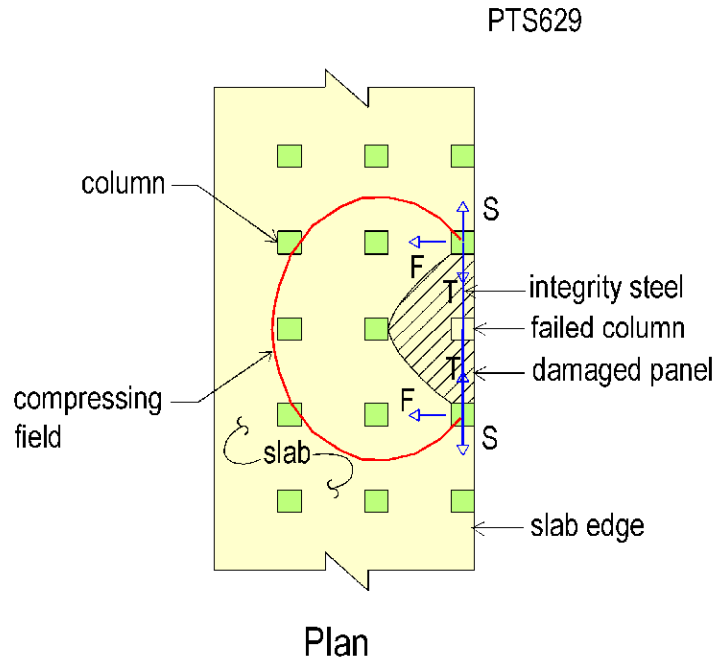
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Plan

FIGURE A.4.2-2 Force Diagram for Removal of an Edge Column

A.4.3 Removal of a Corner Column: The load transfer to avoid progressive collapse in event of a corner column failure is shown in Fig. A.4.3-1. In this case the required robustness is provided by a combination of integrity steel bars extending in the diagonal direction between the penultimate columns, and extending beyond, as shown in part (a) of the figure, and top reinforcement along the diagonal to provide the moment capacity M. The weight of the displaced overhang region, shown with hatched lines has to be resisted by a combination of moment M and shear W along the diagonal shown in part (a) of the figure.

In summary, it will be required to provide adequate top reinforcement over the diagonal, along with integrity steel in an amount necessary to resist the moment generated by the slab hanging over the corner. The shortfall in the resistance required along the diagonal will result in the tip of the displaced slab wedge reaching for support at the location of the removed column. The parameters of the load path shown are:

W = load from the overhang slab, using the load combination U_1;
T = tensile force of the integrity reinforcement;
M = moment along the diagonal hinge line generated resulting from the load U_1;
S = tension generated in the extension of the integrity bars in adjacent panels; and
F = compression force in the slab to balance the forces T and S at the penultimate column.

The forces T and S need to be designed for, using either non-prestressed or prestressed reinforcement. The compression force F will be resisted by the inplane forces in the floor system.
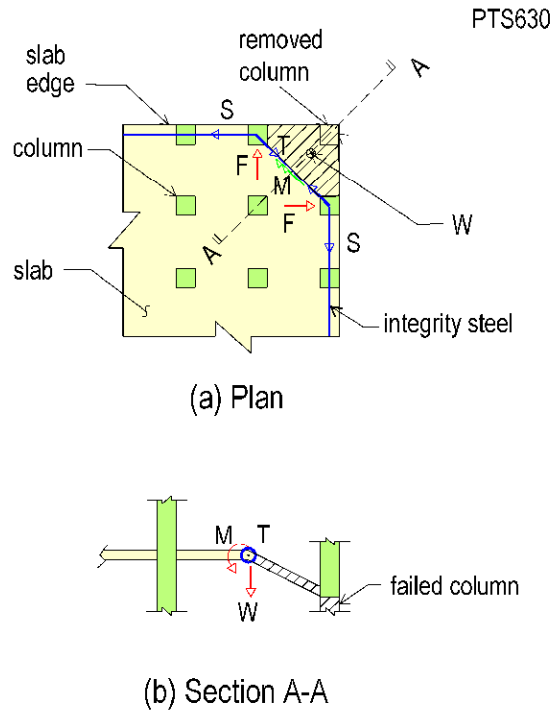
(a) Plan

(b) Section A-A

FIGURE A.4.3-1 Force Diagram for Removal of a Corner Column

A. 5 INTEGRITY REINFORCEMENT

The reinforcement necessary to avoid progressive collapse shall extend from one column to the next and be anchored to develop in full at the face of the support. The reinforcement can be either non-prestressed, or prestressed. Profiled prestressed tendons have the advantage that they can accommodate a larger vertical displacement of the damaged slab on account of the tendons' profiled shape. The large downward displacement of the slab along with excessive cracking can be accommodated through straightening of the otherwise profiled tendon, before resulting in force demand in tendons. If non-prestressed reinforcement is used for robustness, it will develop added strain commensurate with the downward displacement of the damaged slab. The integrity reinforcement shall be placed preferably over the column, but not beyond the effective width of support (Fig. A.5-1)
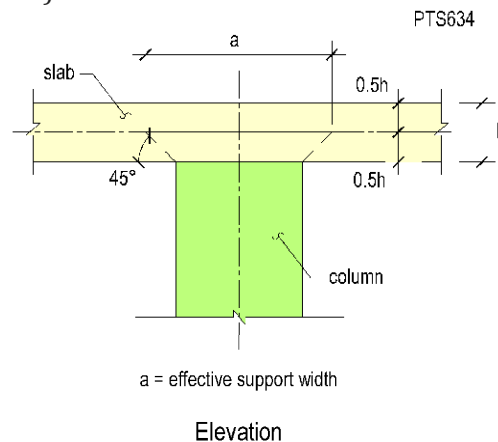

FIGURE A.5-1 Effective Support Width for Integrity Reinforcement

A.6 NUMERICAL EXAMPLE

Given:
Multi-story building constructed with concrete slab and columns. The floor slab is supported on an orthogonal array of uniformly spaced columns with 9 m (29' 6")spans .
Floor to floor height: 3.50 m (11' 6")
Slab thickness: 25 mm (9.5")
Column dimensions: 600x600 mm (23.5"x23.5")

Load on the slab
Selfweight   0.25x24   = 6 kN/m2 (125 psf)
Superimposed dead load   = 1.5 kN/m2 (31.3 psf)
Design live load   = 3.0 kN.m2 (62.6 psf)

Required:
Determine the required robustness reinforcemet to avoid progressive collapse of the structure in event of removal of a central column.

Design:
Figure. A.6-1. Part (a) shows the partial elevation of the building at the level of removed column, and the displaced condition of the damaged panels' tributary to the removed column. The progressive collapse is to be mitigated through provision of integrity reinforcement, sized to support the gravity design load of the damaged panels tributary to the removed column.

Assume the damaged panels are bent and displaced through the height of the floor at the location of the removed column. The force diagram for the displaced panels is shown in Fig. A.6.1b.

From Fig. A.6-2

T = WL/3H

Where,

H   = floor to floor height;
L   = span length; and
W   = weight of each panel.

From part (b) of Fig. A6.1

$W = L^2 * U_1$

Hence

$T = L^3 U_1 / 3H$

$U_1 = 1.00DL + 0.25LL$

$U_1 = 1.00(6 + 1.50) + 0.25*3.0 = 8.25$ kN/m^2 (172.26 psf)

T = 9³*8.25 /3*3.5= 616.84 kN (138.66 k)

Consider providing 13 mm (0.5 in) post-tensioning strands
Strand area 99 mm2 (0.153 in²)
Material strength 1860 MPa (270 ksi)
Material factor γ =1.15 (φ = 0.9)
Force per strand = 99*1860/1.15*1000 = 160.12 kN (36 k)
Number strands to be mobilized as integrity steel =
= 616.84/160 = 3.85 assume 4

The total post-tensioning reinforcement for the 9 m bay of the structure is likely to be of the order of 2,200 kN. With each strand providing approximately 110 kN force when in service, the number of stands per bay is likely to be 20, of which, 4 strands need to be placed over the effective width of the support in each direction. The remainder of strands will be positioned to suit the construction.

The displaced panel will crack extensively, but it should not disintegrate into pieces, and fall over the level below, leading to possible overload and collapse of the lower slab. To avoid disintegration, it is recommended to provide a bottom mesh for connectivity of the extensively cracked parts of the displaced panels. The bar spacing of the bottom mesh is recommended not to exceed 1.5 times the slab thickness.
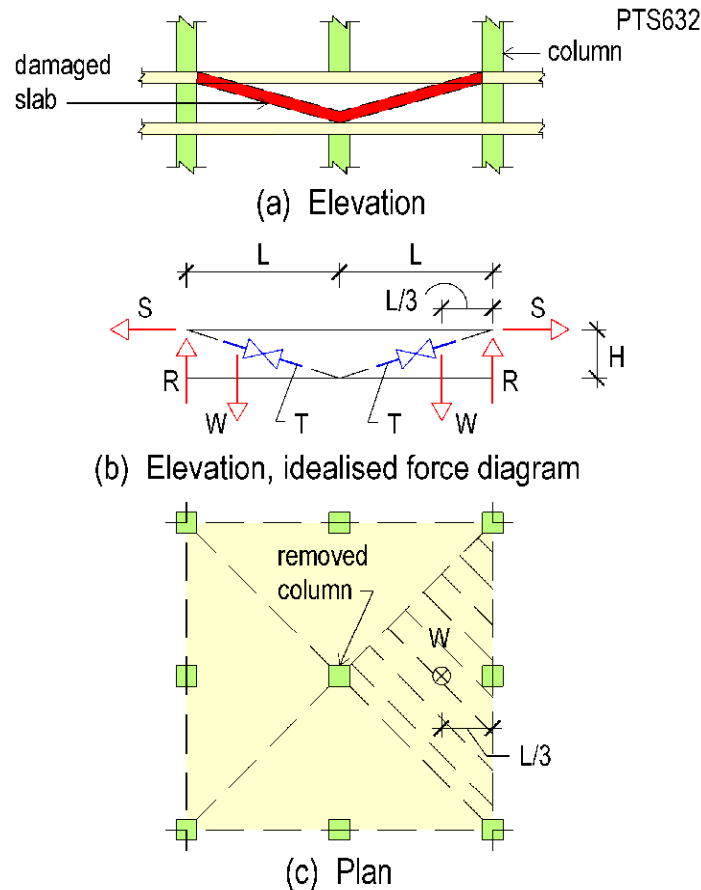
Slab Response to Removed Column
(c) Plan of panels affected by removed column

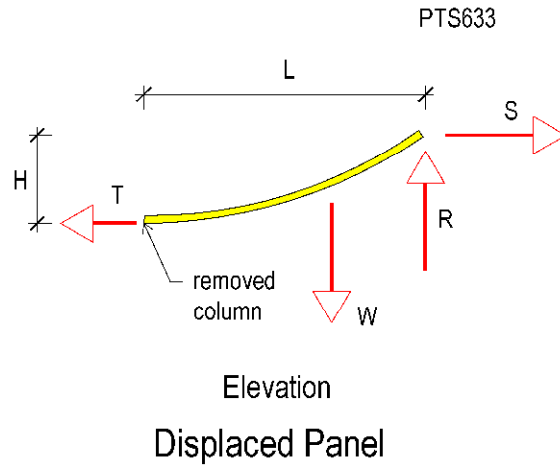FIGURE A.6-1 Partial Elevation and Plan of Framing with Removed Central Column

FIGURE A.6.1-2 Partial Elevation of a Panel Adjacent to Removed Column