Abstract

This paper summarises the report of the committee of inquiry into the collapse, by punching shear, of the Sampoong Department Store in Seoul, Korea in 1995. Examines the adequacy of the design and calculates, using ACI 318-89, BS 8110-85 and Gardner 96, the contribution of the various deficiencies to the probability of failure. ACI 318 does not predict the collapse but BS 8110 and Gardner 96 predict a large probability of collapse. Both BS 8110 and Gardner 96 include size effect and reinforcement ratio.

Key words: Punching, shear, collapse, prediction, codes.

1. Introduction

At 17:55 June 29th, 1995 the north wing of the Sampoong department store collapsed catastrophically killing some 500 persons. The large number of fatalities was due in part to a distinct lack of concern by the building owners/occupiers in failing to take note of signs of serious structural distress before the collapse and evacuating the occupants. The five storey building was a flat plate structure with elevator shafts and services located in rigid shear wall structures between the two wings and at the building extremities. There was no extreme weather conditions or seismic activity. The building was relatively new having opened as a department store on December 1st 1989. Figure 1 shows the north wing after the collapse with the north shear wall structure (left of photograph) and the central service core intact. Figure 2 shows a plan view of the north wing identifying the various vertical structural elements.
According to witnesses the collapse initiated from the 5th floor. The committee of inquiry concluded that the collapse initiated at column 5E on the 5th, restaurant, floor. The investigating committee noted design errors, many construction faults, poor construction quality control, reduction in the cross section of the columns supporting the fifth floor and roof and change of use of the fifth floor from an roller skating rink to a restaurant area. However the reasons for the serious structural distress should be investigated to determine if the deficiencies identified were sufficient to eliminate the code specified load factors and partial safety factors.

The Korean building regulations are essentially identical to the provisions of ACI 318-83 [2].

The construction deficiencies identified included concrete strength of 18 MPa rather than the specified 21 MPa, effective slab depth in the negative moment areas reduced from the specified 410 mm to 360 mm, column diameters only 600 mm for the columns supporting the 5th floor and the roof instead of the 800 mm used in the calculations. The change in use of the 5th floor from a roller skating rink to a restaurant area increased the dead loads of this floor by 35%.

2. **Summary of findings of the Committee of Inquiry**

   * page numbers refer to reference [1]

   In the original design, the 5th floor was designated as a skating rink with a dead load of 8.0 kPa and 2.4 kPa live load (*page 277). The 5th floor use was changed from a roller skating area to a restaurant area with no changes to the structural system (pages 248, and 249) even though the dead load was increased by the installation of walls and false floors (page 292).

   The design calculations assumed 1.9 kPa for the self weight of the lightweight concrete topping specified for the roof but the investigators determined that the actual dead load of the installed topping was 4 kPa (page 274).

   The air conditioning cooling tower was moved to the west side of the building because of complaints from the east-side residents. It was not separated into smaller units for the move because of difficulties in reassembly, so it caused damage to the roof slab during the move. The structural calculations considered 1 kPa for live load, but the self-weight of the cooling tower was 4 kPa which caused unbalanced moments on the slab-column connections (pages 289 and 291).
The average measured concrete compression strength was 18.4 MPa from the collapsed area (north wing), 20.3 MPa in the area of the north elevator of the north wing, and 19.3 MPa in the slabs of the south wing (page 172).

Only the self weight of the 300 mm slab was considered in the calculations; the self weight of the drop panels was neglected (page 231).

The investigators also found that in the negative moment areas of the slabs, over the supports, the effective depth of the slab was reduced due to unsuitable placing of the high chairs so that the slabs did not have the predicted flexural and shear resistance (page 284).

The column size was 600 mm diameter on the 4th and 5th floors (pages 256, 522 and 523) instead of the 800 mm diameter used in the design calculations.

In the case of the punching shear design of the slab, moment transfer to the exterior columns was considered, but moment transfer to the columns in the interior area was not considered (page 233).

The investigation discovered that the drop panel was not constructed at the top area of column line 4 and E, so the slab thickness was 300 mm instead of 450 mm (page 270 to 271).

3. Prediction Methods

Design recommendations are written in terms of nominal shear stresses calculated with reference to a control perimeter around the column. Suitable adjustments of the shear strength parameter means that recommendations using very differently defined perimeters give similar predictions of punching capacity for typical test specimen dimensions.

3.1 ACI 318-89

The Korean Building Design Code is identical, as are many other national codes, to ACI 318 [2]. The nominal shear stress for nonprestressed slabs and footings \( \nu_c \) shall be the smallest of:

\[
\nu_c = 0.83(2 + \frac{4}{\beta_c}) \sqrt{f_{cm}} \tag{1a}
\]

\[
\nu_c = 0.83(\alpha_s \frac{d}{u} + 2) \sqrt{f_{cm}} \tag{1b}
\]

\[
\nu_c = 0.33 \sqrt{f_{cm}} \tag{1c}
\]
b = side dimension of rectangular column

c = diameter of circular column

d = slab average effective depth

\[ u = \pi (c + d) \] (circular columns)

\[ u = 2b + 4d \] (rectangular columns)

\( f_{ck} \) is the specified concrete cylinder strength, MPa.

\( v_c \) = nominal shear stress, MPa.

\( \beta_c \) is the ratio of longer to shorter dimension of the loaded area.

\( \alpha_s \) is 40 for interior columns, 30 for edge columns, 20 for corner columns.

When gravity load, wind, earthquake, or other lateral forces cause a transfer of moment \( M_u \) between a slab and column, a fraction of the unbalanced moment \( \gamma_v M_u \) is considered to be transferred by eccentricity of shear, which is assumed to vary linearly about the centroid of the critical section.

The fraction of the unbalanced moment is given by:

\[
\gamma_v = 1 - \frac{1}{1 + (2/3) \sqrt{b_1/b_2}}
\]  

(2)

where \( b_1 \) and \( b_2 \) are the sides of the control perimeter of a rectangular column, with the side \( b_1 \) being parallel to the moment vector.

The nominal factored shear stress \( v_u \) can be calculated by:

\[
v_u = \frac{V_u}{A_c} \left[ 1 + \frac{A_c \gamma_v M_{ax}}{J_{cx} V_u} y + \frac{A_c \gamma_v M_{ay}}{J_{cy} V_u} x \right]
\]  

(3)

\( V_u, M_{ax} \) and \( M_{ay} \) are the factored shear force and unbalanced moments determined at the centroidal axis of the critical section; \( A_c \) is the concrete area of the assumed critical section and \( x \) and \( y \) are the coordinates of any point on the critical section from the centroidal axis. The shear force \( V_u \) and the moments \( M_{ax} \) and \( M_{ay} \) are not easily determined for continuous flat slab systems. The quantities \( J_{cx} \) and \( J_{cy} \) used in Equation (3) are properties of the assumed critical section analogous to the polar moment of inertia.
3.2 BS 8110-85

The British code BS 8110-85 [3] uses a rectangular control perimeter 1.5 \( d \) from the loaded area for both circular and rectangular loaded areas.

\[
\frac{V_u}{u d} < v_c = 0.79 (100 \rho )^{1/3} (400/d)^{1/4}
\]

(4)

\( f_{cu} \) is the characteristic concrete cube strength, MPa.

\( u = 4 \ (c + 3d) \) for circular loaded areas, mm

\( u = 4 \ (b + 3d) \) for square loaded areas, mm

\[ \rho = (\rho_x + \rho_y)/2 \]

\( \rho \) is calculated for a width equal to \((c + 3d)\) or \((b + 3d)\)

For characteristic concrete cube strengths greater than 25 N/mm\(^2\), \( v_c \) may be multiplied by \((f_{cu}/25)^{1/3}\). The value of \( f_{cu} \) should not be taken as greater than 40 MPa.

The British code offers two options to calculate the effect of combined shear and unbalanced moments; a variation of the eccentric shear expression or simple multipliers. The nominal factored shear stress \( v_u \) at interior column can be calculated by:

\[
v_u = \frac{V_u}{A_c} \left[ 1 + \frac{1.5 A_c M_{ax}}{V_u x} \right]
\]

(5)

\( V_u \) and \( M_{ax} \) are the factored shear force and unbalanced moments determined at the centroidal axis of the critical section; \( A_c \) is the concrete area of the assumed critical section and \( x \) is the length of the side of the control perimeter parallel to the axis of bending.

Alternatively unbalanced moments at an interior column are accommodated by increasing the nominal shear forces by 15%. Corner column connections and edge connections subjected to moments perpendicular to the slab edge are treated by a single expression independent of the eccentricity of the load: \( v_{max} = 1.25 \ v_{avg} = 1.25 \ V_u/ud \)

3.3 Gardner 96

Gardner [4] proposed a prediction equation for the punching shear strength of interior slab column connections of reinforced and prestressed concrete flat slabs, by extending the work of
Shehata and Regan [5] and Shehata [6]. Gardner examined the dependence of the punching shear strength to the concrete strength and tie strength, for reinforced and prestressed concrete slabs, using a control perimeter at the periphery of the loaded area and a Shehata and Shehata [7] type strength enhancement expression. All non square cross section columns were considered as square columns of the same cross sectional area. Using a control perimeter at the periphery of the load area, taking the depth of the compression zone to be a function of the tension tie strength $\rho f_y$, and the CEB size effect expression, the following lower 5% coefficient equation was derived. A sensitivity analysis, using the coefficient of variation of the equation coefficient as the criterion of goodness, confirmed that the one third power of concrete strength and steel force were close to optimal.

\[ v_u = \frac{V_u}{u \cdot d} < v_c = 0.62 \left( 1 + \left( \frac{200}{d} \right)^{0.5} \right) \left( \rho f_y \right)^{1/3} \left( f_{cm} \right)^{1/3} \left( d/u \right)^{0.5} \]  

\(f_y\) = yield strength of flexural steel, MPa.
\(u\) = perimeter of loaded area, mm
\(\rho\) = ratio of flexural tensile reinforcement calculated over a width \(c + 6d\)

For combined shear and moment transfer two alternative methods were suggested; using the ACI linear interaction formula with a control perimeter around the loaded area or a simple BS 8110 type multiplier. For edge and corner column slab connections of slabs subjected to gravity loads the nominal shear force is multiplied by a factor to obtain an effective shear force which is used in equation (6).

For edge connections subjected to moments perpendicular to the slab edge
\[ V_{\text{eff}} = 1.5 V_u \]  

For corner column slab connections
\[ V_{\text{eff}} = 2.0 V_u \]  

3.4 Recalculation of code coefficients

Comparison with code expressions is not straight forward because the code expressions use specified concrete strength, not mean strength, and the expressions were designed to be
conservative. To obtain mean coefficients for the code equations the measured punching shear loads, for connections without moment transfer, were divided by the code equation, using the appropriate control perimeter, to obtain a new coefficient. These revised coefficients were then averaged to obtain an unbiased mean coefficient for the equation. For ACI 318-89 and BS 8110-85 the revised equations are:

**ACI 318-89**

\[ v_c = 0.45 \sqrt{f_{cm}} \]  

(9)

**BS 8110**

\[ v_c = 1.39 (\rho f_{cm})^{1/3} (400/d)^{1/4} \]  

(10)

**Gardner 96**

\[ v_c = 0.79 (1 + (200/d)^{0.5} ) \left( \rho f_y \right)^{1/3} (f_{cm})^{1/4} (d/u)^{0.5} \]  

(11)

Table 1 summarises the comparison of the two code expressions and equation (6) with published experimental results. The coefficient of variation of the ACI 318-89 expression is considerably larger than the other two methods. The ACI method does not consider reinforcement ratio or a slab size effect. Figures 3 and 4 show the variation of experimental punching shear capacity to capacity calculated using ACI 318 with steel factor, one third power of steel ratio, and slab thickness. The predictions are poor for slabs with a low steel ratio or a large effective depth.

The use of a control perimeter \(1.5d\) from the loaded area for BS8110-85 gives consistent results for interior slab column connections without moment transfer but is difficult to interpret for connections with moment transfer or to reconcile with observed, experimental failure surfaces.

Equation (6) was developed from the data bank and has the smallest coefficient of variation. The difference between CEB 8110-85 and Gardner 96 is the choice of shear perimeter.

From Table 1 it can be seen that the variation of the coefficients in the shear stress equations are 22.2%, 13.5%, and 13.3% for the ACI 318-89, BS 8110-85 codes and equation (6).
respectively. Code equations should calculate characteristic resistances which should be exceeded by 95% of test results. The 95% confidence level coefficients were calculated assuming an infinite population and the coefficients of variation from Table 1 giving reduction factors of 0.64, 0.78 and 0.78 respectively and that $f_{cm} = 1.25 f_{ck}$.

Steel ratio should be calculated over the width of the column strip, and should be concentrated for moment transfer at edge and corner columns. The minimum steel ratio should be 0.005 calculated over a strip extending 3$d$ each side of the column.

4. **Nominal safety factors**

The uniform use of characteristic resistances does not in itself ensure a fair comparison between the levels of safety actually achieved by different codes as there can be differences between their partial safety factors for resistances ($\varphi_m$ or $\gamma$) and load factors. The nominal safety factors implied in the various codes, which are the combined effect of load factors and material resistance factors, are not identical.

*ACI 318-89*

Load factor on dead load is 1.4 and the behaviour factor is 0.85 giving a combined effect of $1.4/0.85 = 1.65$.

*BS 8110-85*

Load factor on dead load is 1.4 and the partial safety factor for shear is 1.25 giving a combined effect of $1.4 \times 1.25 = 1.75$.

5. **Calculations**

The investigating committee concluded that the collapse initiated at column 5E on the 5th, restaurant, floor. However it is also possible that the roof slab column 5E connection failed. Only the calculations of connection 5E of the 5th floor are presented here. The design of the connection was checked using the design loads, design concrete strength and member dimensions using the code specified load factors and understrength provisions of both ACI 318-89 and BS 8110-95. The unbalanced moments applied to the 5E slab column connections were determined using the student version of SAP 90. If the calculated ratio $v_u$(applied)/ $v_c$ (resistance) is greater than one the slab column connection would not meet the code requirements. The ratio $v_u$(applied)/ $v_c$ (resistance) was calculated for two cases, loads as
assumed by the design engineer with column diameters 800 mm and 600 mm respectively, Case 1 and Case 2, Table 2. The original design of both roof and the 5th floor had adequate safety using either ACI 318 or BS 8110. The calculation using ACI 318 indicates more conservatism than the calculation using BS 8110-85.  

A second set of calculations, identified as safety analyses, were performed using the loads, concrete strength and member dimensions determined by the investigation committee and the prediction equations with the mean value coefficients from Table 1. The probabilities of failure were calculated assuming the loads are known and a Gaussian distribution for the strengths with the coefficients of variation from Table 1. The probabilities of failure were calculated for the following combinations of load and member dimensions. 

Case 3 - same loads as Case 2 without load factors and partial safety factors  
Case 4 - same as Case 3 but with loads as determined by the investigation  
Case 5 - same as Case 4 but with an extra load over part of the area  
Case 6 - same as Case 4 but with reduced slab thickness of 360 mm  
Case 7 - same as Case 5 but with reduced slab thickness of 360 mm  

The calculations for Case 3, column diameter 600 mm, indicate low probabilities of failure, less than 2% for ACI 318 and BS 8110-85 and 10% for Gardner 96; confirming the conclusion that had the slabs been built as designed, and subjected to the design assumed loads they would have satisfied the code requirements. As expected the calculated probabilities of failure increase as more deficiencies are included in the calculations. However even including all the deficiencies determined by the investigating committee the ACI method only predicts a probability of failure of 19%. BS 8110-85 predict high probabilities of failure for Cases 5, 6 and 7 and Gardner 96 predicts high probabilities of failure for Cases 4, 5, 6 and 7. As shown above the punching shear provisions of ACI 318 are known to be poor for slabs with reinforcement ratios less that 0.5% and for thick slabs - both of these factors were present in the slabs in the Sampoong Department Store. However the major contribution to the collapse was the excessive loads due to change of use of the 5th floor. Analogous calculations for the roof show similar high probabilities of failure when the air conditioning units were being moved from the west side to the east side.
6. Conclusions

According to the summary of the results, the original design of both roof and the 5th floor had adequate safety using either ACI 318 or BS 8110.

Noting the dubiousness of the statistics, BS 8110-85 and Gardner predict significant probabilities of collapse under load cases 5, 6 and 7 and 4, 5, 6 and 7 respectively whereas ACI 318-89 does not. The punching shear provisions of ACI 318 are known to be poor for slabs with reinforcement ratios less that 0.5% and for thick slabs - both of these factors were present in the slabs in the Sampoong Department Store. In addition the loads were significantly larger and the slab effective depth less than assumed by the design engineer, the concrete strength was low and the diameters of the supporting columns were undersized. Each one of these factors would reduce the code implied safety margin.

ACI 318-89, ACI 318-99 and CSA A23.3-94 [8] require some of the positive moment steel to be continuous through the columns. Integrity steel required in the Canadian code CSA A23.3-94 capable of carrying twice the dead load of the slab tributary area would have minimised the extent of collapse.

The most important cause of this building collapse was excessive loads applied to the building due to the change of purpose of the space. Further the building was poorly constructed. Finally, even though symptoms of structural distress were evident in several locations before the collapse of the building the people in positions of knowledge and authority took no action with the consequence hundreds of people were killed.

References

2. ACI 318-89 "Building Code Requirements for Reinforced Concrete" American Concrete Institute, Detroit, 1989, 353 pp.


Notation

The three methods define the shear and steel ratio differently

\( b \) side dimension of rectangular column

\( c \) diameter of circular column

\( d \) slab average effective depth

\( f_{ck} \) specified concrete cylinder strength, MPa.

\( M_u \) Factored ultimate moment Nm

\( v_c \) nominal shear stress, MPa.

\( V_u \) Factored ultimate shear force N

ACI

\( u \) \( \pi (c + d) \) (circular columns)

\( u \) \( 2b + 4d \) (rectangular columns)

\( \beta_c \) ratio of longer to shorter dimension of the loaded area.

\( \alpha_c \) 40 for interior columns, 30 for edge columns, 20 for corner columns.

BS 8110-85

\( u \) \( 4 (c + 3d) \) for circular loaded areas, mm

\( u \) \( 4 (b + 3d) \) for square loaded areas, mm

\( \rho \) \( (\rho_x + \rho_y)/2 \)

\( \rho \) steel ratio calculated for a width equal to \( (c + 3d) \) or \( (b + 3d) \)

Gardner 96

\( f_y \) yield strength of flexural steel, MPa.

\( u \) perimeter of loaded area, mm

\( \rho \) ratio of flexural tensile reinforcement calculated over a width \( c + 6d \)
Table 1  Analysis of Prediction Methods with Results on Isolated Punching Shear Results

<table>
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<th>ACI</th>
<th>BS8110</th>
<th>Gardner</th>
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<tr>
<td>mean coefficient using $f_{cm}$</td>
<td>0.45</td>
<td>1.39</td>
<td>0.79</td>
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<td>coef. of variation of mean coefficient</td>
<td>22.2%</td>
<td>13.5%</td>
<td>13.3%</td>
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<td>95% coefficient using $f_{ck}$</td>
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<td>$0.79 f_{cube}$</td>
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* 2.4 kPa uniformly distributed plus 7 kPa over part of the area
Figure 1 - Photograph of Collapsed Store

Figure 2 - Plan of north wing

Figure 3 - Effect of steel factor on ACI 318 predictions

Figure 4 - Effect of slab depth on ACI predictions