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Lessons from the Sampoong department store collapse

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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. It 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.

Keywords: Punching; Shear; Collapse; Prediction; Codes

1. Introduction

At 17:55 June 29th, 1995 the north wing of the Sampoong department store collapsed 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 first, 1989. Fig. 1 shows the north wing after the collapse with the north shear wall structure (left of photograph) and the central service core intact. Fig. 2 shows a plan view of the north wing identifying the various vertical structural elements.

According to witnesses the collapse initiated from the fifth floor. The committee of inquiry [1] concluded that the collapse initiated at column 5E on the fifth, 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 a 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 to 360 mm, column diameters were only 600 mm for the columns supporting the fifth floor and the roof instead of the 800 mm used in the calculations. The change in use of the fifth 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

In the original design, the fifth floor was designated as a skating rink with a dead load of 8.0 kPa and 2.4 kPa live load (p. 277). [1] The fifth floor use was changed from a roller skating area to a restaurant area with no changes to the structural system (pp. 248, 249) even though the dead load was increased by the installation of walls and false floors (p. 292).

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[1] Page numbers refer to [1].

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Nomenclature

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 (N m)
v_c	nominal shear stress (MPa)
V_u	Factored ultimate shear force N

ACI

u	$\pi(c+d)$ (circular columns)
u	$\Sigma b + 4d$ (rectangular columns)
β_c	ratio of longer to shorter dimension of the loaded area.
α_s	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
ρ	$(\rho_x+\rho_y)/2\rho$ 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)
ρ	ratio of flexural tensile reinforcement calculated over a width $c+6d$

Fig. 1. Photograph of collapsed store.

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 (p. 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 (pp. 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 (p. 172).

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

The investigators determined the effective depth of the drop panels, in the negative moment regions of the slabs

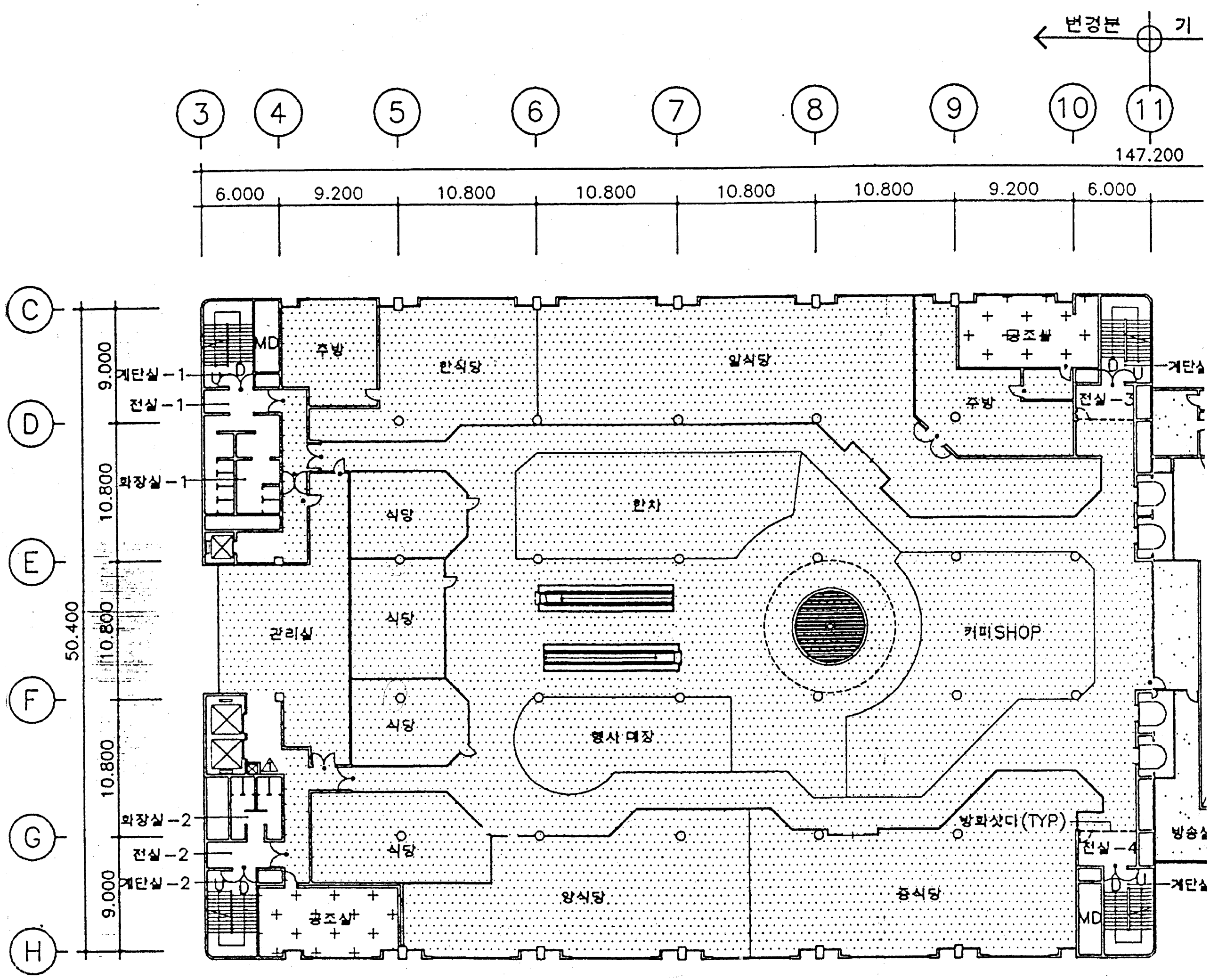

Fig. 2. Plan of north wing.

over the supports, was reduced due to unsuitable placing of the high chairs. Consequently the slabs did not have the predicted flexural and shear resistance (p. 284).

The column size was 600 mm diameter on the fourth and fifth floors (pp. 256, 522, 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 (p. 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 (pp. 270–271).

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 v_c shall be the smallest of:

$$v_c = 0.083\left(2 + \frac{4}{\beta_c}\right)\sqrt{f_{cm}}, \tag{1a}$$

$$v_c = 0.083\left(\alpha_s \frac{d}{u} + 2\right)\sqrt{f_{cm}}, \tag{1b}$$

$$v_c = 0.33\sqrt{f_{cm}}, \tag{1c}$$

where b is the side dimension of rectangular column, c the diameter of circular column, d the slab average effective depth, $u = \Pi(c+d)$ (circular columns), $u = \Sigma b + 4d$ (rectangular columns), f_{ck} is the specified concrete cylinder strength (MPa), v_c the nominal shear stress (MPa), β_c is the ratio of longer to shorter dimension of the loaded area, α_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

$$\gamma_{vx} = 1 - \frac{1}{1 + (2/3)\sqrt{b_1/b_2}}, \tag{2}$$

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

$$v_u = \frac{V_u}{A_c}\left[1 + \frac{A_c \gamma_{vx} M_{ux}}{J_{cx} V_u} y + \frac{A_c \gamma_{vy} M_{uy}}{J_{cy} V_u} x\right], \tag{3}$$

where V_u, M_{ux} and M_{uy} 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_{ux} and M_{uy} are not easily determined for continuous flat slab systems. The quantities J_{cx} and J_{cy} used in Eq. (3) are calculated 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 1.5d from the loaded area for both circular and rectangular loaded areas.

$$\frac{V_u}{ud} < v_c = 0.79(100\rho)^{1/3}(400/d)^{1/4}, \tag{4}$$

where 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), $\rho = (\rho_x + \rho_y)/2$, ρ 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

$$v_u = \frac{V_u}{A_c}\left[1 + \frac{1.5 A_c M_{ux}}{V_u x}\right]. \tag{5}$$

where V_u and M_u 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 increasing the nominal shear forces by 15% accommodates unbalanced moments at an interior column. 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 nonsquare 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 ρf_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.

$$\begin{aligned} v_u &= \frac{V_u}{ud} < v_c \\ &= 0.62(1 + (200/d)^{0.5})(\rho f_y)^{1/3}(f_{cm})^{1/3}(d/4c)^{0.5}, \end{aligned} \tag{6}$$

where f_y is the yield strength of flexural steel (MPa), u the perimeter of loaded area (mm), ρ is the 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 multiplied by a factor to obtain an effective shear force which is used in Eq. (6).

For edge connections subjected to moments perpendicular to the slab edge

$$V_{\text{eff}} = 1.5V_u. \tag{7}$$

For corner column slab connections

$$V_{\text{eff}} = 2.0V_u. \tag{8}$$

3.4. Recalculation of code coefficients

Direct comparisons with code expressions are complicated 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, BS 8110-85 and Gardner 96 the revised equations are:

ACI 318-89

$$v_c = 0.45\sqrt{f_{cm}}. \tag{9}$$

BS 8110

$$v_c = 1.39(\rho f_{cm})^{1/3}(400/d)^{1/4}. \tag{10}$$

Gardner 96

$$v_c = 0.79(1 + (200/d)^{0.5})(\rho f_y)^{1/3}(f_{cm})^{1/3}(d/u)^{0.5}. \tag{11}$$

Table 1 summarises the comparison of the two code expressions and Eq. (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. Figs. 3 and 4 show the variation of experimental punching shear capacity to capacity calculated using ACI 318 with the one third power of steel ratio, steel factor, 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.

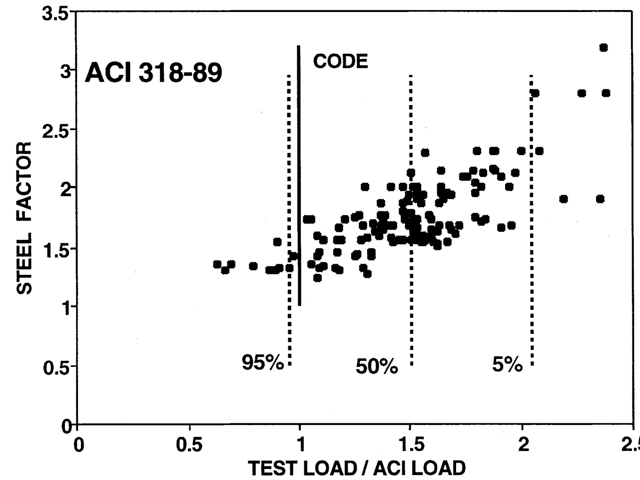

Fig. 3. Effect of steel factor on ACI 318 predictions.

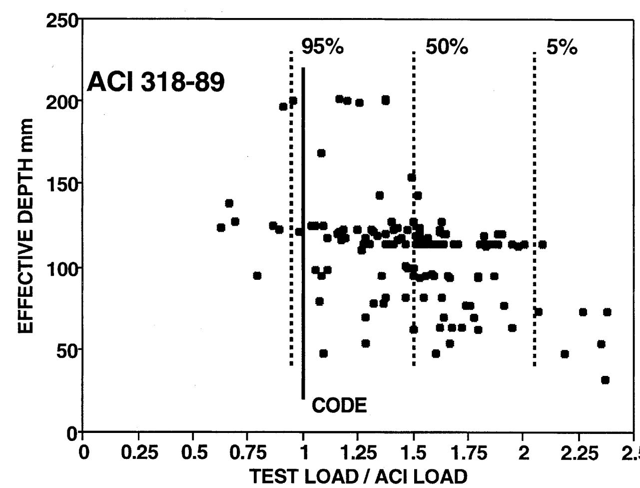

Fig. 4. Effect of slab depth on ACI predictions.

Eq. (6) was developed from the data bank and has the smallest coefficient of variation. The difference between BS 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 Eq. (6), respectively. Code equations should calculate characteristic resistances that should be exceeded by 95% of test results. The 95% confidence level coefficients

Table 1
Analysis of prediction methods with results on isolated punching shear results

	ACI	BS8110	Gardner
Mean coefficient using f_{cm}	0.45	1.39	0.79
Coef. of variation of mean coefficient	22.2%	13.5%	13.3%
95% coefficient using f_{ck}	0.32	0.68 f_{cu}	0.62
Code coefficient using f_{ck}	0.33	0.79 f_{cu}	0.62

Table 2
Comparison of prediction provisions and calculated probabilities of failure

Case	Type of analysis	f_c (Mpa)	Column diameter (mm)	Drop panel thickness (mm)	Steel ratio (%)	Drop panel dead load (KPa)	Slab dead load (KPa)	Live load (KPa)	ACI 318		BS 8110		Gardner	
									v_u/v_c	Prob. of failure (%)	v_u/v_c	Prob. of failure (%)	v_u/v_c	Prob. of failure (%)
1	Design	21	800	410	0.47	12	8.4	2.4	0.72		0.89			
2	Design	21	600	410	0.47	12	8.4	2.4	0.87		1.00			
3	Safety	18.4	600	410	0.47	12	8.4	2.4		0.4		1.4		10
4	Safety	18.4	600	410	0.47	14.95	11.35	2.4		1.5		18		56
5	Safety	18.4	600	410	0.47	14.95	11.35	2.4 + 7[a]		6.4		75		99
6	Safety	18.4	600	360	0.53	14.95	11.35	2.4		4.8		53		88
7	Safety	18.4	600	360	0.53	14.95	11.35	2.4 + 7[a]		19		98		100

[a] 2.4 kPa uniformly distributed plus 7 kPa over part of the area.

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 $3d$ 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 (φ_m or γ) 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 fifth, restaurant, floor. However it is also possible that the roof slab column 5E connection failed. Only the calculations of connection 5E of the fifth floor are presented here.

The design of the connection was checked using the design loads, design concrete strength and member dimensions for both ACI 318-89 and BS 8110-85 using the appropriate code specified load factors and understrength provisions. 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 does not meet the code requirements. The ratio v_u (applied)/v_c (resistance) was calculated using the loads assumed by the design engineer with the original 800 mm diameter column, Case 1, and the reduced, as built, 600 mm diameter column, Case 2. The results are identified as Design Analyses in Table 2. The original design of both the roof slab and the fifth floor slab had adequate safety against punching shear 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, calculated the probabilities of failure using the concrete strength, loads, and member dimensions determined by the investigation committee and the ACI 318, BS 8110 and Gardner 96 prediction equations. The probabilities of failure were calculated assuming the loads are known and a normal distribution for the strengths with the mean coefficients and the coefficients of variation from Table 1. All safety analyses used the measured concrete strength of 18.4 MPa and a column diameter of 600 mm. The probabilities of failure shown in Table 2 were calculated for the following combinations of load and slab depth.

- *Case* 3 – same loads as Case 2 without load factors and partial safety factors.
- *Case* 4 – slab properties 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, concrete strength 18.4 MPa and 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 observation 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, reduced concrete strength, increased load and reduced slab depth, the ACI method only predicts a probability of failure of 19%. BS 8110-85 predict high probabilities of failure for Cases 5–7, unbalanced load and reduced slab thickness, and Gardner 96 predicts high probabilities of failure for Cases 4–7, increased slab dead load, unbalanced load and reduced slab thickness. As noted previously the punching shear provisions of ACI 318 are poor for slabs with reinforcement ratios less that 0.5% and for thick slabs – both factors were present in the slabs in the Sampoong department store. However the major contributions to the collapse were the reduced slab depth and the excessive loads due to change of use of the fifth 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 fifth 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 5–7, unbalanced load and reduced slab depth, and 4–7, increased load due to change of use, unbalanced load and reduced slab depth, respectively, whereas ACI 318-89 does not. The punching shear provisions of ACI 318 are shown 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 causes of this building collapse were the reduced slab depth and the excessive loads applied to the building due to the change of use 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.

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