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Structural failure and assessment of a department store

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INTRODUCTION

The load bearing structure of a recently built department store in Prague consists of the flat (double ribbed) reinforced concrete slabs supported directly on columns located within span distances 12×12 m. Slabs above the first and second storey cantilever out by 3 m beyond the edge columns. After few years in service serious performance defects of cladding, interior partitions, and other secondary elements had been observed [1,2].

Incidentally at the same time another department store of the same user collapsed. This was perhaps partly the reason why all the performance defects of the new building had been carefully recorded and publicly reported, although the collapsed department store was a steel structure and its failure occurred due to well-recognised causes. Unfavourable engineering climate (psychological aspects) seems to have played a significant role in the subsequent assessment of structural damage. The observed defects were often exaggerated and interpreted as structural condition indicating insufficient safety against collapse, not just serviceability defects. Consequently, after less than 10 years in service, the second storey of the two-floor building was completely closed and the damaged non-bearing components were reconstructed.

A detailed analysis has shown that the serviceability failure of the second storey was primarily caused in design by lack of consideration of deflections from permanent load and shrinkage [3,4]. Presented theoretical model of public perception, which is a substantial extension of the previous study [4], takes into account uncertainty and vagueness in perceiving observed defects as well as effects of possible consequences. It appears that the developed theoretical model can well explain the wide differences between the public perception and the expert assessment of its structural condition.

DESCRIPTION OF THE STRUCTURE

The plan view dimensions of the building are 78×53 m. The load-bearing structure consists of reinforced concrete double ribbed slabs of the total thickness 0.45 m supported directly by columns of the cross section 0.5×0.5 m or 0.7×0.7 m located within span distances 12×12 m (see Figure1). The slabs are provided by in-depth heads, 1.65×1.65 m (at the edge columns) and 3.35×3.35 m (at the interior columns), where the coffer ceiling is replaced by solid slab. In the remaining part of the slab, the ribs of the cross section 0.18×0.38 m support a thin plate (of the thickness 0.07 m and interior spans 0.8×0.8 m). Equivalent thickness of a solid slab having the same rigidity would be 0.34 m only, which indicates that the stiffness of the slab is very low. Moreover, slabs above the first and second storey cantilever out beyond edge columns by 3 m.

Design loads of the slab above the ground floor considered in the original analysis consist of a permanent part 7.0 kN/m^2 and a variable part 4.0 kN/m^2 , corresponding values for the slab above the first floor are 7.2 kN/m^2 and 1.5 kN/m^2 . However, the actual permanent load of this slab is due to actual roof and ceiling greater and could be as high as 10 kN/m^2 . Nevertheless, the ultimate strength of the slab is sufficient, and no strengthening of the load bearing structure was needed.



Figure 1. Internal view of the structure

Non-bearing elements of the second storey, which were affected by deformations of bearing structures consist of façade cladding, interior partition walls and interior built-in components like glass walls and shelf stands. Cladding of the building consists of large glass windows, brick walls and window pillars, located within regular distances of 2.4 m. The window pillars, reinforced by steel ties anchored into floor and ceiling, were built in the bearing slabs without any movement joints. Similarly all interior masonry partition walls, reinforced by rolled steel elements of I- and U-section were constructed without any separation from roof slab. Expansion joints were not used in any interior built-in components. All of these non-bearing structures were evidently designed without desirable consideration of differential deflection of both floor and ceiling slabs due to their different stiffness and loading.

PERFORMANCE DEFECTS

After few years in service serious performance problems concerning cladding as well as interior non-bearing structures have been observed and analysed [3,4]. The most alarming were perhaps the defects appearing in the non-bearing structures: cracks of partition walls (see Figure 2 and 3).



Figure 2. Partition wall

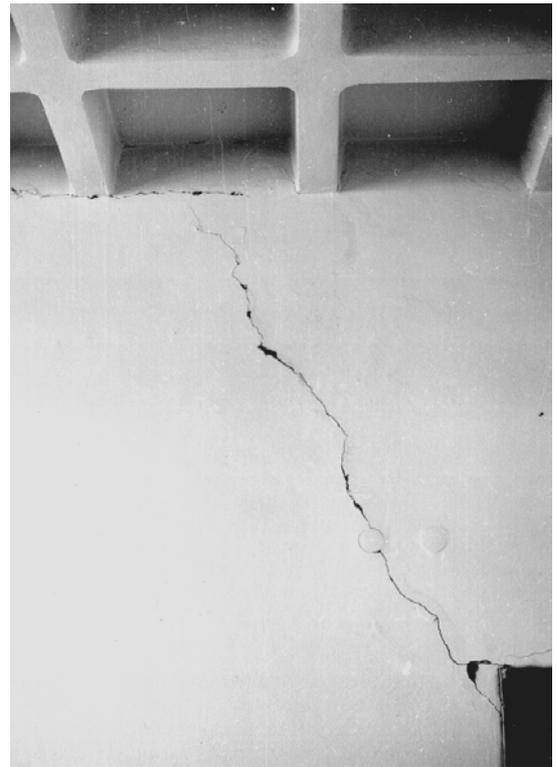


Figure 3. Partition wall



Figure 4. Façade corner

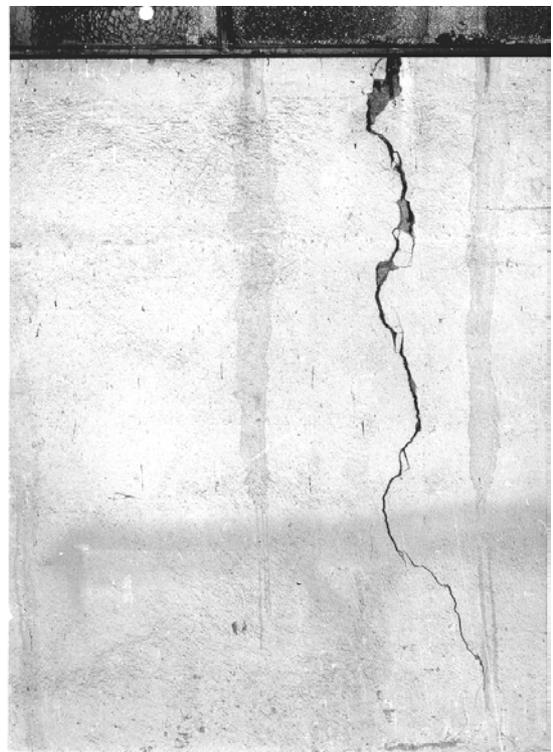


Figure 5. Façade wall

Tensile cracks of the window pillars and cladding elements were particularly noticeable near the exterior corners of the building (see Figure 4 and 5). Also, the deformed doorframes and a buckled shelf stand were visible and impressive. There were also sporadic shear cracks in the slab ribs close to the hidden column heads, however, they were mostly near construction joints. These cracks, which were of smaller importance for the overall damage assessment of the building, were the only detected defects of the load bearing structure.

ASSESSMENT

All of the defects of non-bearing structures in the first floor are caused by differential deflections of the floor slab (above the ground floor) and of the roof slab, primarily due to permanent load and shrinkage. As mentioned above the permanent load of the more flexible roof is by 3 kN/m^2 greater than that of the slightly less flexible floor slab. This could lead to considerable differences in the midspan deflection (shortening up to 30 mm) as well as cantilever deflection (extension up to 5 mm, at exterior corner up to 10 mm). Also shrinkage may lead to similar mutual differences in slab deflections; midspan shortening and cantilever extension may reach 5 mm (extension at exterior corners may be 10 mm).

Deformations due to temporary load and temperature are of smaller importance and may cause a maximum midspan extension 6 mm and shortening 1 mm, cantilever extension or shortening less than 1 mm. The observed deformation effects correlate well with theoretical results. Unfortunately, no calculation of structural deformations had been made in the original design. This was partly due to inadequate provisions of the contemporary standards.

Another construction fault mentioned above concerns the permanent load of the roof slab. Due to the actual self-weight of the ceiling and roof (sloped concrete layers) the permanent load of the slab is by 30 % greater than that assumed in design calculation. However, this discrepancy arises from the neglect of some load components in the design calculation. Also the actual permanent load of the slab above the ground floor may be slightly greater than that considered in design. Even though the deformation effects would be considerably lower if the actual load equalled the assumed load, many of the serviceability defects would have appeared anyway. This concerns, above all, the cracks in the cladding elements, particularly at the exterior corners of two-way cantilevered slabs. Most of the observed defects were primarily due to lack of consideration of deflection in the design and due to inaccurate determination of the design load. Construction errors, however, considerably added to deformation effects.

REPAIR

Although there were almost no defects apparent on the load bearing structure (except sporadic shear cracks in ribs), it was decided to close the first floor and to reconstruct damaged interior partitions and other non-bearing structural components as well as the cladding components. Interior partitions including steel elements have been separated from the ceiling and new movement joints have been covered by panel strip. New ties located near window pillars have mutually tied up the cantilevered slabs. These measures were proposed by the designer, even though an additional analysis shows that the expected deformations due to temporary load and temperature (as mentioned above) are limited, and in some cases could be admitted without any modification of the non-load-bearing structures.

PUBLIC PERCEPTION

Public perception (recently discussed in [5]) played an important role in the assessment and final decision concerning the building. The new department store became soon a building

closely watched by a large number of users and local authorities. For more, at the same time another department store suffered from construction faults and this was partly the reason why all the performance deficiencies have been carefully recorded (similar experience is described in [5,6]). This unfavourable engineering climate seems to enhance the intensity of public perception. The observed defects were often exaggerated and regarded as indicators of insufficient structural safety. Widespread public perception of defects and discrepancies in expert assessments was reported in newspapers and finally resulted in a strong public demand for strengthening of the building.

THEORETICAL MODEL OF PUBLIC PERCEPTION

Evaluation of public as well as expert assessments has indicated that there is no distinct point in any commonly used performance indicator x (e.g. deflection, crack width) that would uniquely separate acceptable and unacceptable structures. Rather there seemed to be a transition region $\langle a, b \rangle$ in which the structure gradually becomes unserviceable and the degree of caused damage $v(x)$ increases [4,7]. A conceivable model for $v(x)$ is indicated in Figure 6 as a trilinear function. Note that the values of $v(x)$ are within the conventional interval from 0 to 1. There is no damage below a certain lower limit value a , and full damage above the upper limit b .

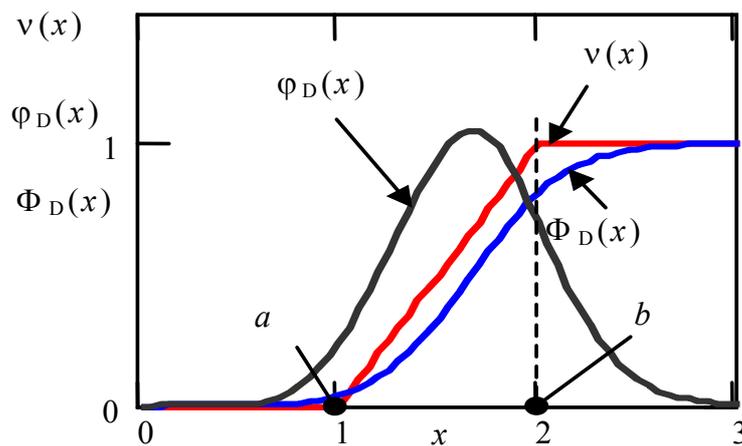


Figure 6. Perception model

Obviously, at any damage level $v(x)$ there may be a perception scatter expressed by the distribution function $\Phi_P(x, \mu_P, \sigma_P)$, for which lognormal distribution having the mean $\mu_P = x$ and standard deviation $\sigma_P = s \times a$, where s is a relative measure of scatter related to the lower bound a , is accepted here. Taking into account all levels $v(x)$, the cumulative damage $\Phi_D(x)$ is [4,7]

$$\Phi_D(x) = K \int_a^b v(\xi) \Phi_P(x, \xi, \sigma_P) d\xi \quad (1)$$

where K is the normalising factor to normalise $\Phi_D(x)$ into the interval from 0 to 1 and ξ is the integration variable. The cumulative damage $\Phi_D(x)$ and a corresponding density function $\phi_D(x)$, shown in Figure 6 for $a = 1$, $b = 2$ and $s = 0.3$, can be considered as generalised probabilistic models (involving economic aspects). The cumulative damage $\Phi_D(x)$ can be used to specify in a rational way the required serviceability constrains [2,3].

Further, considering an appropriate load effect E (e.g. deflection, crack width) having the mean μ_E and an assumed coefficient of variation $w_E (=0.2)$ the expected perception level $\pi(\mu_E)$ can be defined as

$$\pi(\Delta, \sigma_P, \mu_E) = \int_{-\infty}^{+\infty} \varphi_E(x) \Phi_D(x) dx \quad (2)$$

Here $\varphi_E(x)$ is the probability density function of E . Gamma distribution having the mean μ_E is assumed. The mean μ_E equal to the lower limit of the transition region a , $\mu_E = a$, and the standard deviation $\sigma_E = 0.2\mu_E$ (the coefficient of variation 0.2) is assumed in the example shown in Figure 7, where the expected perception level $\pi(\Delta, \sigma_P, \mu_E)$ is indicated as a function of the ratio $\Delta = (b-a)/a$ for selected σ_P . Figure 8 shows $\pi(\Delta, \sigma_P, \mu_E)$ versus μ_E .

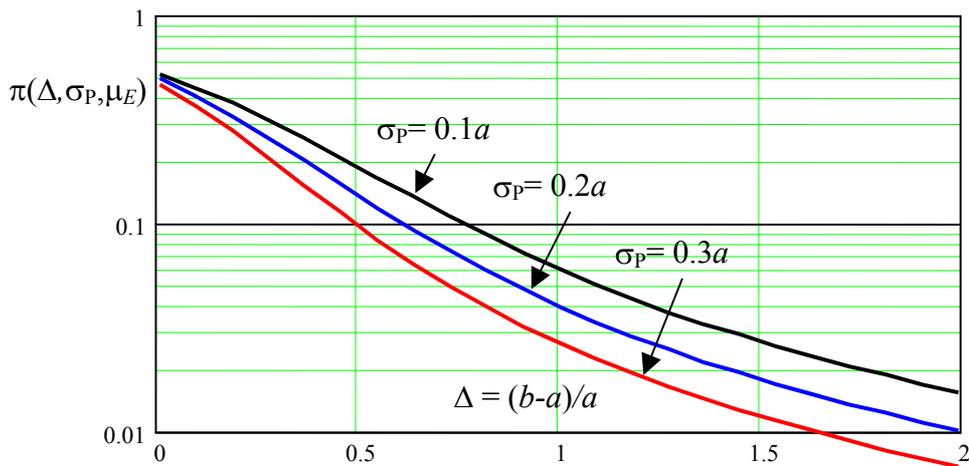


Figure 7. Perception level $\pi(\Delta, \sigma_P, \mu_E)$ for $\mu_E = a$.

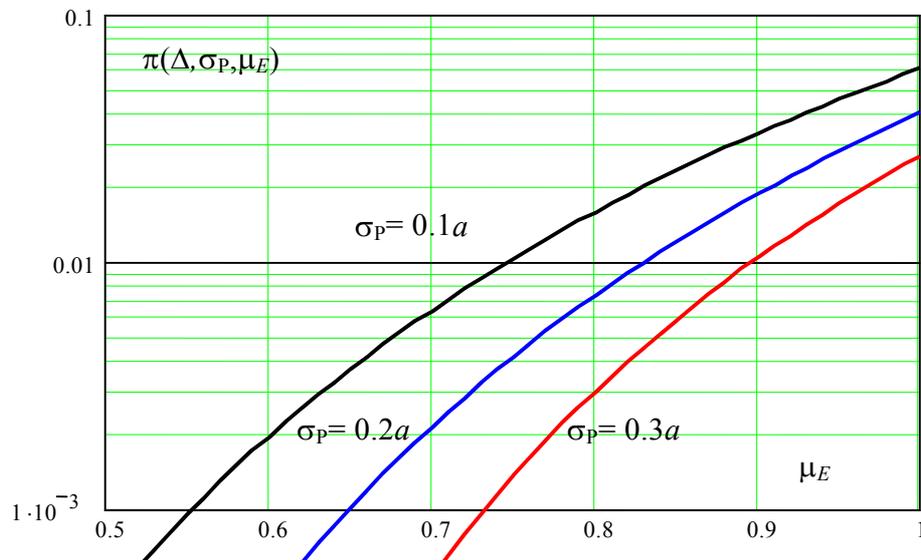


Figure 8. Perception level $\pi(\Delta, \sigma_P, \mu_E)$ for $\Delta = 1$.

It follows from Figure 7 and 8 that the expected damage (perception) level $\pi(\Delta, \sigma_P, \mu_E)$ is strongly dependent on both the ratio $\Delta = (b-a)/a$ and the mean μ_E , which may further depend

on the sensitivity or experience of the observer. This finding explains the observed differences in public perception and discrepancies in expert assessments.

EFFECTS OF EXPECTED CONSEQUENCES

In addition to psychological aspects an assessment of a structure may be affected also by the expected economic (and other) consequences. Both these factors may be considered explicitly or implicitly only. On the one hand consideration of expected consequences may lead to a rational specification of structural constraints, on the other hand it may significantly affect the assessment of the structure. Moreover, the expert assessor may use uncertain data and information concerning the malfunctioning cost. The following analysis investigates the possible effects of expected consequences.

Assume that the optimum mean μ_E of the load effect E would correspond to the minimum of the total cost C_{tot} . Assuming a given coefficient of variation w_E , the optimum mean μ_E of the load effect E (the most suitable value assessed by an expert) may be obtained from the minimum of the total expected cost of a structure. Consider the total cost C_{tot} as a sum

$$C_{tot} = C_0 + C_m/\mu_E + \pi(\Delta, \sigma_P, \mu_E) C_f \quad (3)$$

where C_0 denotes the initial cost independent of μ_E , and C_m/μ_E is an additional cost affected by modification of the mean μ_E , where C_m is a relevant marginal cost. Finally the multiple $\pi(\Delta, \sigma_P, \mu_E) C_f$ describes the expected cost due to structural failure. Obviously the minimum C_{tot} will occur when the partial cost

$$C_{part} = 1/\mu_E + \pi(\Delta, \sigma_P, \mu_E) C_f/C_m \quad (4)$$

is the minimum, where $C_{part} = (C_{tot} - C_0)/C_f$. Figure 9 shows the partial cost C_{part} for the following parameters: $(b-a)/a=1$, $\sigma_P=0.3a$, $\sigma_E=0.2\mu_E$.

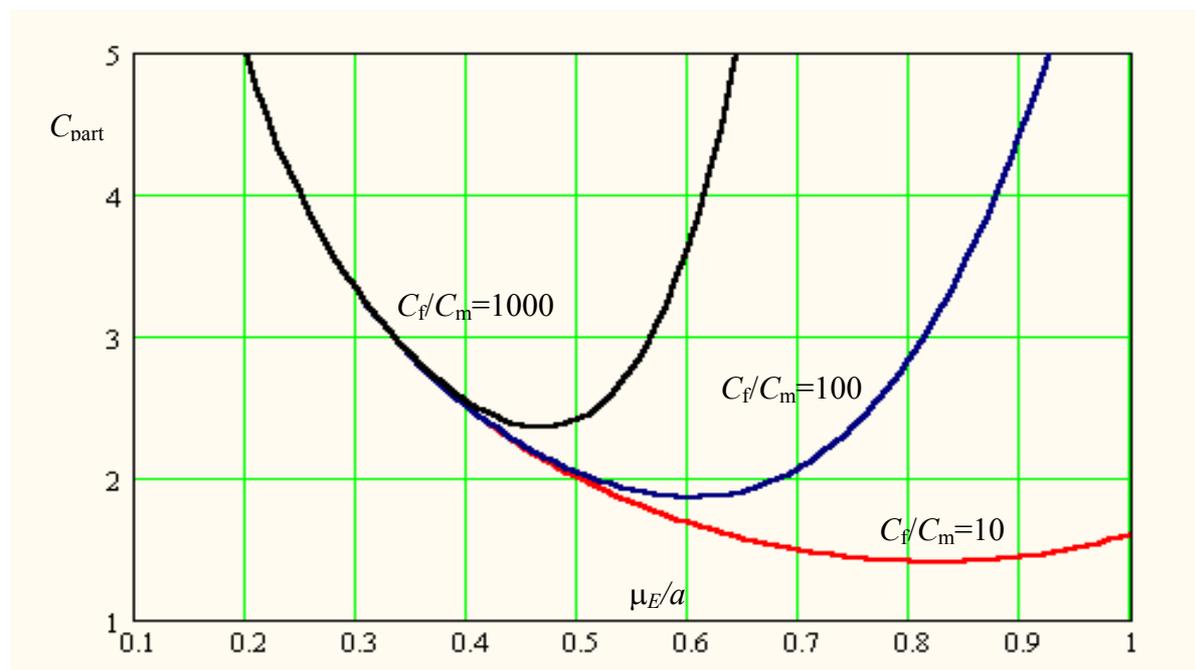


Figure 9. The partial cost C_{part} versus μ_E/a for $\Delta = (b-a)/a=1$, $\sigma_P=0.3a$, $\sigma_E=0.2\mu_E$

It follows from Figure 9 that the optimum mean μ_E may be expected from $0.5a$ to $0.8a$. However, for extremely high cost of failure C_f greater than $1000 C_m$ the optimum mean will be lower than $0.5a$. Obviously, depending on the assumed cost ratio C_f/C_m the optimum (the most suitable) mean μ_E of the load effect E may considerably vary. A further analysis is certainly needed in order to take into account various parameters and expected consequences, including life safety.

CONCLUSIONS

1. The serviceability failure of the department store was primarily caused by lack of consideration in design of deflection due to permanent load, shrinkage, variable loads including thermal actions.
2. The construction faults (additional permanent load and imperfect anchoring of ties) increased the unfavourable deformations of the structure.
3. The current engineering climate (psychological aspects) and the expected consequences seem to play an important role in the public perception of performance deficiencies, in the subsequent structural assessment and in the decisions made by the public authorities.
4. There is no distinct value that would uniquely distinguish acceptable and unacceptable structural conditions.
5. The disturbing variance in public perception and in expert assessment of the observed defects may be well explained using proposed theoretical model for public perception.
6. There seems to be an optimum value of the performance indicator that would lead to the minimum total cost and may be considered by an expert as most suitable.
7. The effects of vagueness in structural requirements and of the possible unfavourable consequences, including jeopardy of life, on the judgement of experts requires a more detailed investigation..

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LIST OF SYMBOLS

a	lower bound of the transition region
b	upper bound of the transition region
s	relative measure of the perception scatter σ_p/a
x	performance indicator
w_E	coefficient of variation of the load effect E
C_0	initial cost
C_f	malfunction cost
C_m	marginal cost
C_{part}	partial cost
C_{tot}	total cost
E	load effect
K	normalising factor
μ_E	mean of the load effect E
μ_p	mean of the perception level
σ_p	standard deviation of the perception level
$v(x)$	damage level
$\varphi_D(x)$	probability density of the damage
$\varphi_E(x)$	probability density function of E
$\pi(\Delta, \sigma_p, \mu_E)$	perception level
ξ	generic point of the performance indicator x
Δ	transition region ratio $(b-a)/a$
$\Phi_p(x, \mu_p, \sigma_p)$	distribution function of the perception level
$\Phi_D(x)$	distribution function of the damage level