Transactions of the **14th International Conference on Structural Mechanics in Reactor Technology** (SMiRT 14), Lyon, France, August 17-22, 1997

K06/5

Nonlinear strength analysis of a brick wall in a low seismic region

Henkel F.O.[1], Liersch G.[2], Breuer W.[1]
(1) Wölfel Beratende Ingenieure GmbH + Co, Germany
(2) Bayernwerk AG, Germany

ABSTRACT

A quasi-static out-of-plane analysis of a brick wall in a low seismic region results in a complete damage of the wall, the wall will turn over. On the other hand, the MSK-scale describes as damage that only hair-line cracks in few walls and fall of small pieces of plaster will occur. This large discrepancy between experience observed and static analysis is overpassed through a nonlinear dynamic analysis of the brick wall.

INTRODUCTION

In the event of an earthquake non-supporting brick walls in the environment of safety-related parts of nuclear power plants must be stable. Otherwise the safety-related function of the parts of the plants could no longer be guaranteed in case of a collapse of the brick walls.

With the design of new nuclear power plants the relevant brick walls are constructed in a way (reinforced brickwork) that their stability for the earthquake defined can be proved by means of simple static analyses. By use of the same procedure for review of brick walls in existing nuclear power plants, these walls would fail already for low vibration levels, although the MSK-scale states only hair-line cracks. Thus there is a large discrepancy between the damage stated after an earthquake and the damage determined by analysis.

Purpose of this paper is to demonstrate the procedure how to judge the stability of non-supporting brick walls in a real earthquake case.

LOCAL SITUATION

The walls to be considered are brick walls in a concrete frame construction. They are built with 24 cm high perforated bricks and are not systematically connected with the reinforced concrete construction. The joint of approx. 15 mm each between wall and reinforced concrete beams respectively girders is bridged only by mortar spots and by the 20 mm thick lime cement plaster on both sides over the whole height/width. The walls are spread over the whole building and have a height of 2.90 m until 4.40 m.

EARTHQUAKE LOADING

The seismic values of the earthquake to be used for re-assessment are based on an earthquake intensity for the location of I = VI on the MSK-scale. The belonging free field acceleration of the ground is 0.5 m/s^2 per horizontal direction. For the re-assessment it was increased conservatively to 0.7 m/s^2.

For the re-assessment of mechanical parts of the plant or constructional sub-structures such as brick walls floor response spectra were determined for the relevant buildings. Figure 1 shows the floor response spectra to be applied in the relevant direction of the building for damping of D = 0.07 depending on the floor level of the building.

VIBRATION BEHAVIOUR

The real vibration behaviour of a brick wall depends above all on the boundary conditions. Since there is no systematic structural connection between brickwork and reinforced concrete construction the boundary conditions for linear analyses are not clear. With smaller vibrations one can start out from clamped conditions (displacements and rotations) at bottom and top of the wall. This was also confirmed through a test with the hammer impact method. However, for larger vibrations such as an earthquake analyses, opening of the joints cannot be excluded. As most unfavourable extreme case this results in a freestanding cantilever wall.

REAL DAMAGE

Information about the real damage of a brick wall at earthquake loading is given by the MSK-scale. Here the effects of an earthquake are described in dependence of the earthquake intensity. For the intensity VI defined for the case under consideration the damage is described as follows:

„In many buildings hair-line cracks in very few walls; fall of small pieces of plaster only, in a few buildings cracks in many walls, fall of fairly large pieces of plaster“.

Of course, the MSK-scale only supplies rough criteria for the effects of earthquakes, without consideration of the exact local conditions and of the construction in detail. However, it can be presumed that in nuclear plants the structural design is better than in common industrial and domestic buildings. Thus the damage to be expected will certainly not exceed that of the one here described.

ANALYZED DAMAGE

The damage analyzed for a definitive earthquake by use of quasi-static inertia forces greatly depends on the actual boundary conditions during the earthquake. Figure 4 shows the influence of two different boundary conditions on the vibration behaviour, load supporting behaviour and bending moment for an acceleration $a_h = 1\ m/s^2$ at the example of a strip of 1 m width and 4.4 m height. The resistible horizontal acceleration is obtained by comparing the active bending moment from earthquake with the passive bending moment from dead load.

For the conservative assumption of a freestanding wall, the analyses would show an overturning of the high walls already at an acceleration of $a_h = 0.55$ m/s².

PROCEDURE

Structural improvements as e. g. the arrangement of anchors at the top of the wall or of steel supports in front of the wall are often used as easiest way for the proof of the stability of existing brick walls.

However, this solution is not acceptable, especially if there is extreme discrepancy between the damage expected and that analyzed and if there is a large number of brick walls under concern. The discrepancy between analysis and reality can only be explained by the fact that the real behaviour of a brick wall during earthquake cannot be appropriately determined with a quasi-static calculation.

Therefore the real behaviour of a brick wall is investigated by an exact nonlinear dynamic analysis. As an enveloping case the analysis considers the highest wall with h = 4.40 m and with the highest earthquake excitation at level +12.0 m.

METHOD OF ANALYSIS

The analyses were done with the DYMES program for the dynamic analysis of multibody mechanical systems. Here the systems consist of a number of rigid bodies connected via elements describing constrained couplings (bearings, joints, guides) or force couplings (nonlinear springs, beams, dampers). The systems can execute large spatial motions and fly, slide or roll free. Furthermore, time-independent forces and displacements can be applied as boundary conditions.

For the dynamic analysis of the wall with its nonlinear behaviour at the bottom joint (gaping) the acceleration time histories of the building are necessary. They are generated for each direction from the enveloping floor response spectra. Figure 2 shows the acceleration time history analyzed for the y-direction at level +12.0 m. For control purposes the corresponding response spectrum has been plotted in figure 1. The maximum acceleration of the wall is $a_{max} = 5.1$ m/s².

For the nonlinear analysis the excitation is applied in form of displacement time history (see figure 3).

VIBRATION MODEL

The vibration model used for the dynamic analysis is shown in figure 5. Along the height the brick wall model consists of 6 bodies connected by elastic beam elements. For linear vibration behaviour the lowest bending natural frequency would result at f = 4.3 Hz and would be loaded by maximum excitation according to the spectra to be applied.

The nonlinear behaviour in the bottom joint is modelled by means of 4 vertical and 2 horizontal gap elements transferring compressive forces only.

RESULTS OF THE NONLINEAR ANALYSIS

- The relative displacements from the top to the bottom of the wall are shown in figure 6. The largest relative motions occur immediately following the strong motion phase, with maximum deflection of the top of the wall at ±20 mm.
- Figure 7 shows the maximum size of gap at the bottom of the wall, resulting in 1.0 mm. With the presumption of a rigid wall a displacement at the top of the wall of $1.0 \cdot 4.2 / 0.24 = 17.5$ mm < 20 mm would result. The contribution of the elastic wall to the deflection of the wall is thus approx. 2 - 3 mm.
- Figure 8 shows the forces in the nonlinear vertical springs (gap) at the edge. The appertaining portion of the dead load results at times $T = 0$ and $T > 15$ s. During and shortly after the strong motion phase ($T < 12$ s) the wall swings from one side towards the other so that the respective spring of the edge is then loaded with the whole weight of the wall. On the gaping side the forces remain limited to zero (no tension forces).
- For a realistic width of the contact surface of 20 mm during maximum overturning of the wall, the pressure stress due to dead load of the wall is $\sigma = 0.74$ N/mm^2.
  This stress is clearly less than the allowable stress of $\sigma_{all} = 1.2$ N/mm^2. Thus the stress reserves at the pressure side against failure are large enough.

SUMMARY

Under consideration of the nonlinear behaviour during the earthquake the exact dynamic analyses show that the most unfavourable wall considered remains stable during this small vibration level without any improvements. Since smaller loads and stresses result for lower wall and floor levels during an earthquake (smaller overturning moment) the present investigations can be considered to be enveloping.

REFERENCES

1. DIN 1053, Teil 1 — Rezeptmauerwerk, Berechnung und Ausführung, Februar 1990

2. DYMES User's Manual — Engineering Mechanics Research Corporation, Version 93.1, November 1992

3. KTA 2201.3 — Auslegung von Kernkraftwerken gegen seismische Einwirkungen, Teil 3: Auslegung der baulichen Anlagen

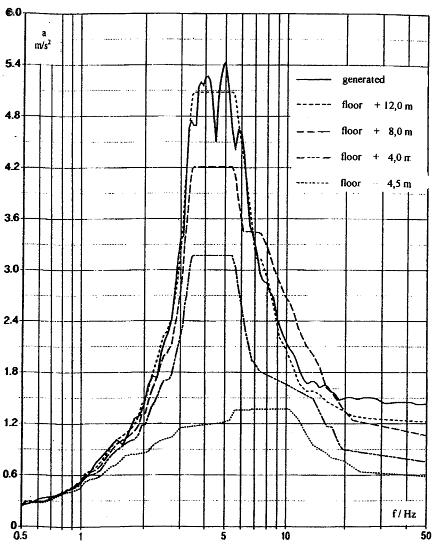

Figure 1: horizontal floor spectra, D = 0.07, various floors

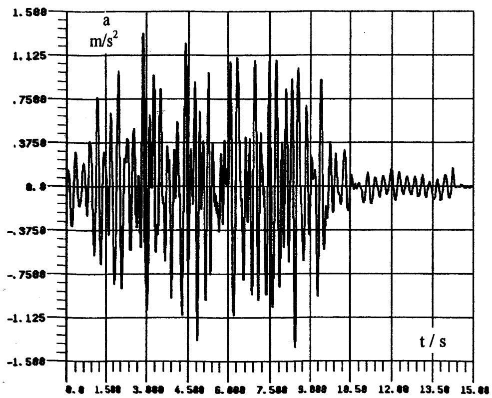

Figure 2: generated acceleration history, according to spectrum floor +12.0 m

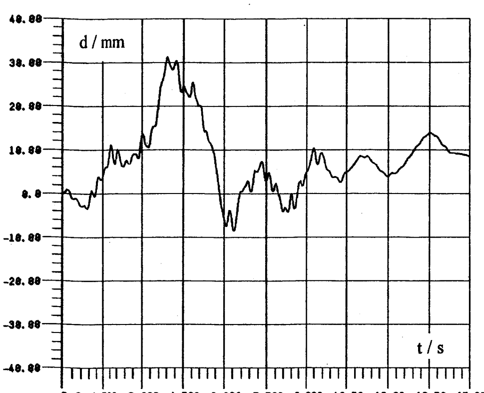

Figure 3: generated displacement time history, according to spectrum floor +12.0 m

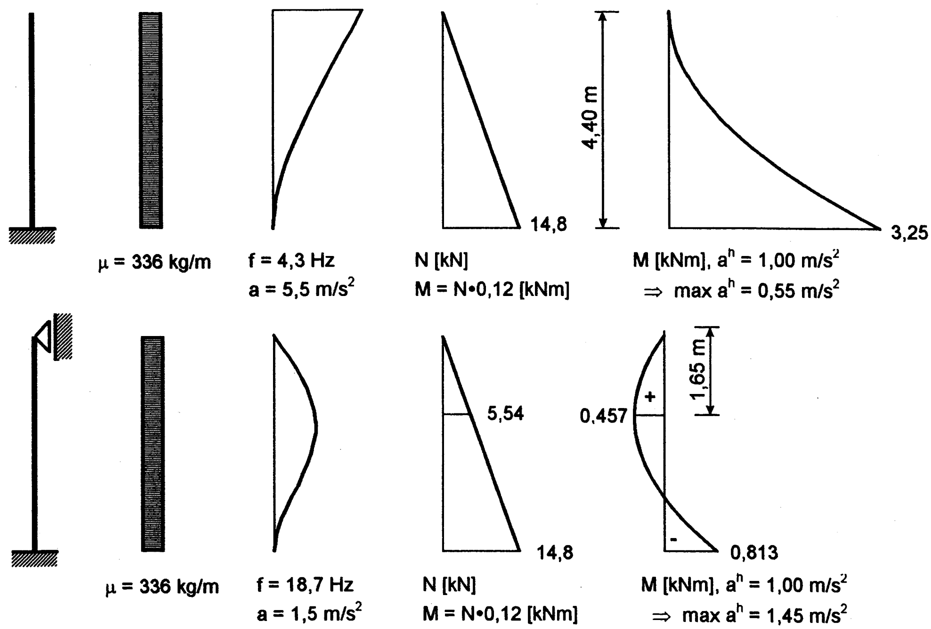

Figure 4: quasi-static solution

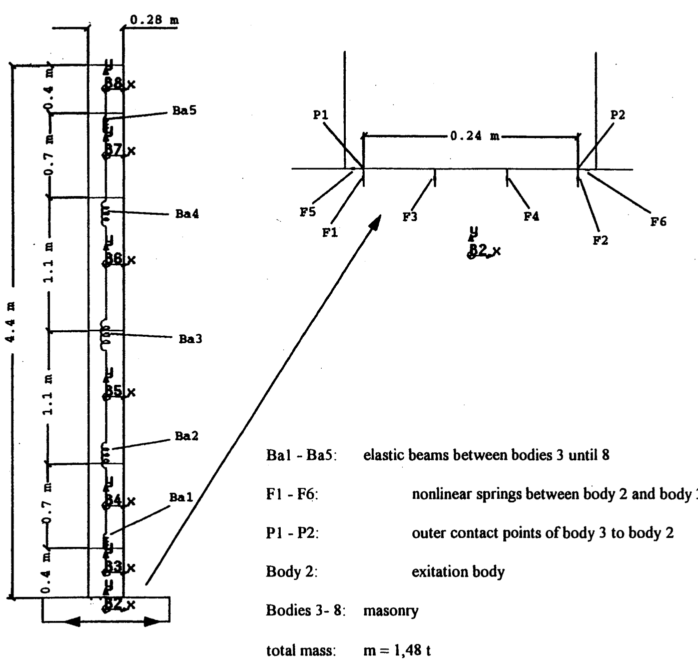

Figure 5: model

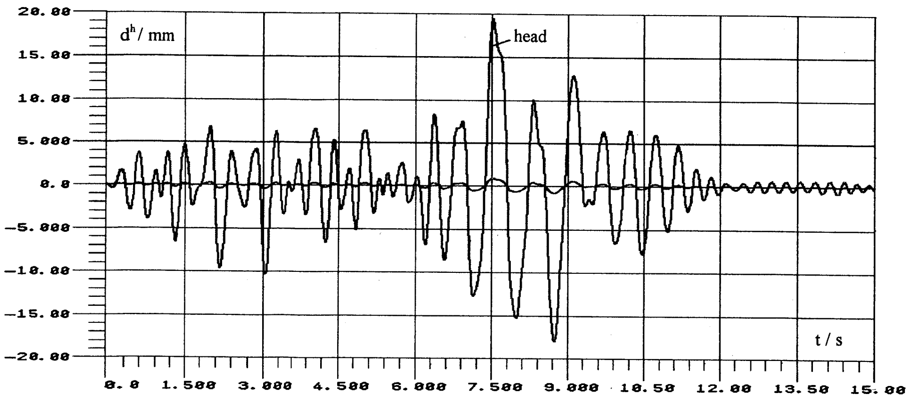

Figure 6: relative displacement base/head, horizontal

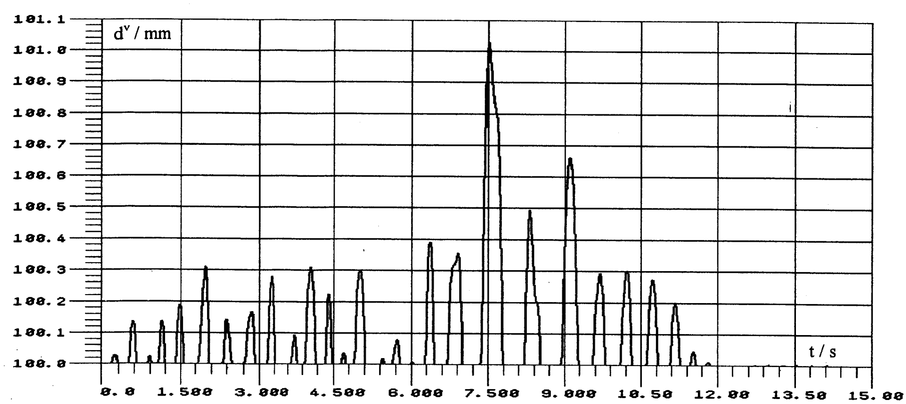

Figure 7: vertical lift-off displacement, edge node

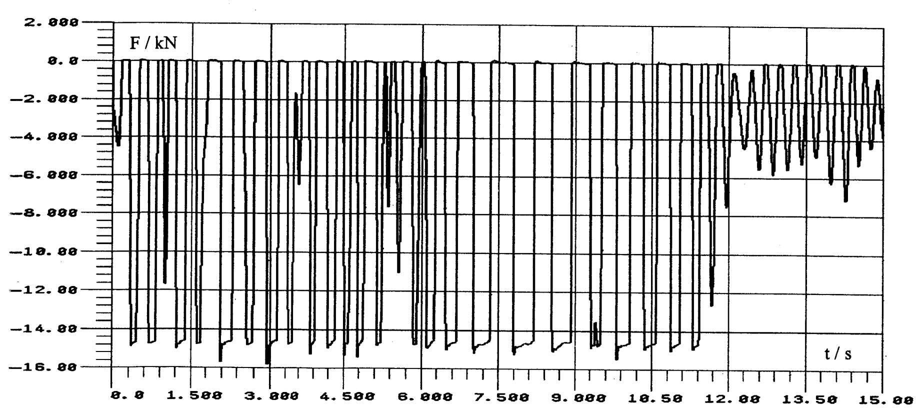

Figure 8: vertical spring forces, edge node