

# Bunker silo wall loads

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Jofriet, J. C., Zhao, Q., Darby, D. E. and Bellman, H. 1989. **Bunker silo wall loads.** *Can. Agric. Eng.* **31**: 187-193. Structural problems in the precast concrete walls of some large bunker silos in Alberta have raised questions about the adequacy of the 1983 CFBC design loads for high (over 3 m) bunker silo walls. This paper examines these design loads in light of design procedures used for earth retaining walls. One of the A-frames with structural problems is analyzed to try to explain the observed cracking. Also, the preliminary results of a full-scale experiment on a bunker silo with a 4.9-m-high wall are provided. Only static wall loads have been considered thus far. The stored material was corn silage with an average moisture content of about 71% at the time of loading. The average bulk density was approximately 870 kg/m<sup>3</sup>. Analysis of the silo frame using equivalent fluid pressures indicated the highly stressed areas. Bending moments and shear forces exceeded the strength at several points. The experiments showed that pressure increased with depth about linearly down to the second row of sensors 1.37 m from the silo floor. The rate of increase was approximately 5 kPa/m. At the bottom row of sensors, 0.71 m from the floor, the pressure was slightly lower than that in the second row.

## INTRODUCTION

Design loads for the structural design of the walls of horizontal silos are specified in the current Canadian Farm Building Code (CFBC 1983). The design load for walls tilted up to 10° from vertical is a uniform pressure of 6.7 kPa normal to the wall from a point 0.6 m below the top down to its base. The pressure in the upper 0.6 m varies linearly from 0 at the top to 6.7 kPa. In addition there is design load of 5 kN located 0.6 m below the silage surface to account for the wheel loads of compaction equipment.

Darby (1986, personal communication) reported serious structural cracking in the wall panels and support buttresses of several concrete 6.1-m-high bunker silo walls in Alberta (Figs. 1, 2 3 and 4). Two of three independent engineering reports (Turnbull et al. 1987; Jofriet 1987 personal communication) concluded that the design loads specified in the present Canadian Farm Building Code are not sufficient for bunker silos with walls over 2.5 m high.

The objectives of this paper are:

- (1) To review available research literature on bunker silo wall loads.
- (2) To review the CFBC (1983) design loads for bunker silo walls in light of conventional design procedures for earth retaining walls.
- (3) To examine the magnitude of the bending moments and shear forces resulting from the various loads.
- (4) To report on preliminary results of a full-scale experiment involving a 5-m-high bunker silo wall.

## LITERATURE REVIEW

Research on bunker (or horizontal) silos can be traced back to the early 1950s. Experiments were conducted on wooden and concrete silos (Esmay and Brooker 1955; Esmay et al. 1956;

Young 1957). The results of these early investigations and others were summarized by Easton (1969). They formed the basis for the design loads that have been used in Canada and the USA for bunker silos for the past 20 yrs.

Early investigations were limited to silos with walls less than 2.4 m (8ft) high. Bunker silos are now constructed with walls up to a height of 6 m (20 ft). Messer and Hawkins (1977) investigated several on-farm bunker silos with walls over 4 m high. The results showed a different load pattern from those reported earlier, i.e. an equivalent liquid pressure as is typically used for earth retaining wall design. These research results were incorporated in the British Code, BSS-5502 (1978), which recommends a lateral pressure,  $L = 3.5 + 3.5z$  kPa. In this expression  $z$  is the depth below the surface of the silage in metres.

Recent measurements of loads on bunker silo walls were conducted in Sweden by Kangro (1986). His experimental pressures were also much larger than those specified in the Canadian Farm Building Code (CFBC 1983). Kangro's results were obtained from silos with walls less than 2 m high. The dynamic loads caused by packing equipment were also measured by Kangro. His recommended design pressures were  $L = 7 + 2.5z$  kPa.

This recent research on the magnitude of loads on the walls of bunker silos and the occurrence of structural difficulties in high bunker silo walls in Alberta indicate that the CFBC specified loads are not adequate for walls over 2.5 m. In the next section, an estimate will be made of the pressures on a bunker silo wall using accepted theories of soil mechanics.

## PRECAST CONCRETE WALL ANALYSIS

The principles of retaining wall design (Craig 1978) were applied to the analysis of a 6.1-m-high bunker silo built in Alberta from Canada Plan Service (CPS) (1982) plan no. 7435. The wall slopes 14° with the vertical. At least one of the A-shaped support frames of this silo is very severely cracked at the top of the rear leg (Darby 1986, personal communication) (Figs. 1 and 2). The analysis was carried out to find the cause of the failure (Jofriet 1987, personal communication).

In most large bunker silos, the walls are sufficiently far apart to be considered independent retaining walls. The silos reported by Darby (1986, personal communication) to suffer from structural distress were all three to four times wider than the height of the wall. The vertical pressure in the silage is then simply  $\rho gz$  except very near the wall where the friction between wall and silage may reduce the vertical stress somewhat.

In the design of retaining walls, it is assumed that there is a constant ratio, the pressure ratio  $k$ , between the lateral pressure,  $L$ , and the vertical stress in the retained material. The magnitude of the pressure ratio,  $k$ , is a function of the material being retained, the relative movement of the retaining wall when subjected to the lateral pressure, the friction between wall and



Figure 1. Break in the top of the rear leg of a bunker silo support frame; crack initiated by a combination of high negative moment and shear.



Figure 2. Close-up of the break illustrated in Figure 1.

retained material and the slope of the surface of the retained material.

When a retaining wall is allowed to move away from the load a small amount, the ratio is smallest. It is then referred to as the active pressure coefficient. If the wall is moved in the opposite direction, a passive pressure condition may result. The passive coefficient,  $k$ , can be as much as 10 times the active one. If no movement is permitted, then the lateral strain in the retained material is zero and an 'at rest' state will result with pressures somewhere between the active and passive pressures.

Bunker silo walls constructed of precast concrete panels supported on A-frames (Canada Plan Service M-7435) are very

rigid in comparison to the highly compressible silage. It is therefore reasonable to assume that a state of 'at rest' exists. The same state of strain exists in tower silos and the recommendations for the pressure ratio made for tower silos are appropriate here. Jofriet and Negi (1988) recommended a value for  $k$  of 0.4 for whole-plant silage. More recent experimental evidence (Quah 1988; Zhao et al. 1988) pointed to a somewhat higher value of 0.5. This more conservative value of 0.5 will be used in the analysis.

The observed average bulk density of the 60% moisture content corn silage in the silo in Fig. 1 was  $670 \text{ kg/m}^3$ . The silage had a crown of 2.5 m at the centre of the silo; an overburden



Figure 3. Positive moment cracks in the front leg of a bunker silo support frame.



Figure 4. Cracks in the bottom of the front leg of a bunker silo support frame; cracks initiated by a combination of high negative moment and shear.

of 1 m was assumed near the wall for wall pressure calculations. Thus, the vertical stress at the bottom of the wall is

$$V_{\max} = (6.1 + 1.0) \times 0.67 \times 9.81 = 46.7 \text{ kPa.}$$

The ratio,  $k'$ , between the vertical stress in the silage and the pressure normal to a wall that has a slope of  $a^\circ$  with the vertical is (Zhao et al. 1988):

$$k' = 0.5 [(1 + k) - (1 - k) \cos(2a)] \quad (1)$$

Assuming  $k = 0.5$  and given the wall slope of  $14^\circ$ ,  $k'$  is 0.53 and the maximum pressure normal to the wall then is  $0.53 \times 46.7 = 24.7 \text{ kPa}$ . The resulting wall pressure diagram is shown in Fig. 5. The CFBC (1983) pressure diagram has been included for comparison.

The A-shaped support frame was analyzed for the pressure diagram in fig. 5 using a linear elastic frame analysis program. The frame model is shown in Fig. 6. Both legs were assumed fixed at the bottom. The shear forces and bending moments along the front leg are plotted versus position in Fig. 7a. Similarly, the shear force and bending moment diagrams for the rear leg are shown in Fig. 8a. The front leg has a tensile force of 96 kN, the rear leg a compressive force of 119 kN. The shear and bending resistances of the legs of the A-frames were analyzed (CSA A23.3 1984) for a silo frame built from CPS plan no. 7435. The factored bending strength of the A-frame legs was calculated to be 65.3 kN.m, the factored shear resistance was 54.3 kN.

The largest bending moment is at node 1 at the bottom of the front leg of the A-frame. At the top of the horizontal cross member,  $M = 81 \text{ kN.m}$ , 24% greater than the calculated factored bending strength. There is also a high shear force at that location. The linear elastic assumption is a gross oversimplification of the actual behavior in the case of reinforced concrete structures. The moment-curvature relationship of reinforced concrete sections is non linear after the concrete in the tension zone cracks. This is estimated to occur at a moment of about

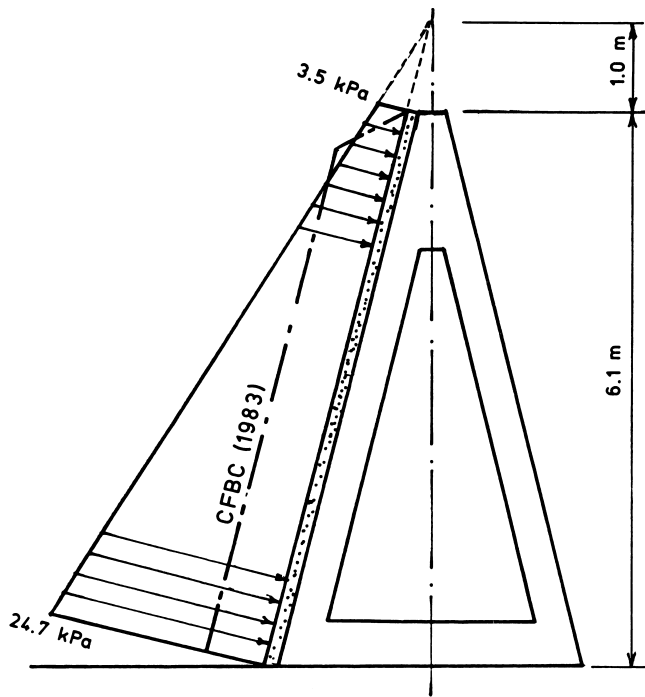


Figure 5. A-frame supported bunker silo wall subjected to a linearly increasing pressure; CFBC (1983) pressure shown for comparison.

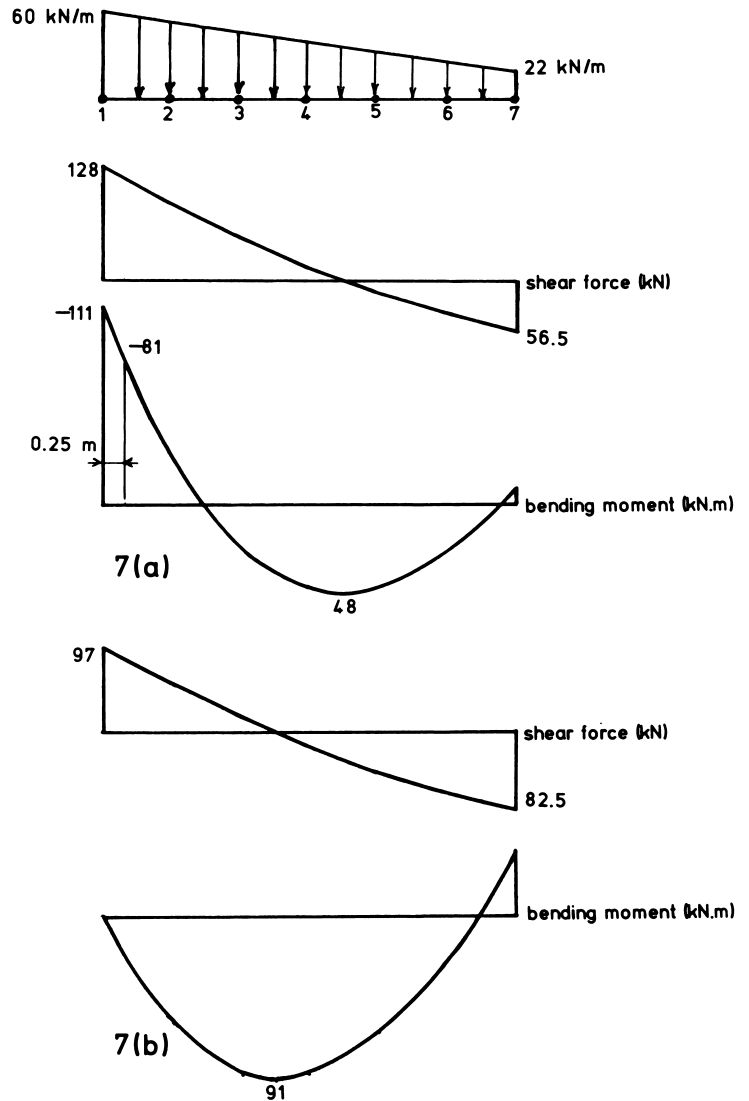


Figure 7. Shear forces and bending moments in the front leg.

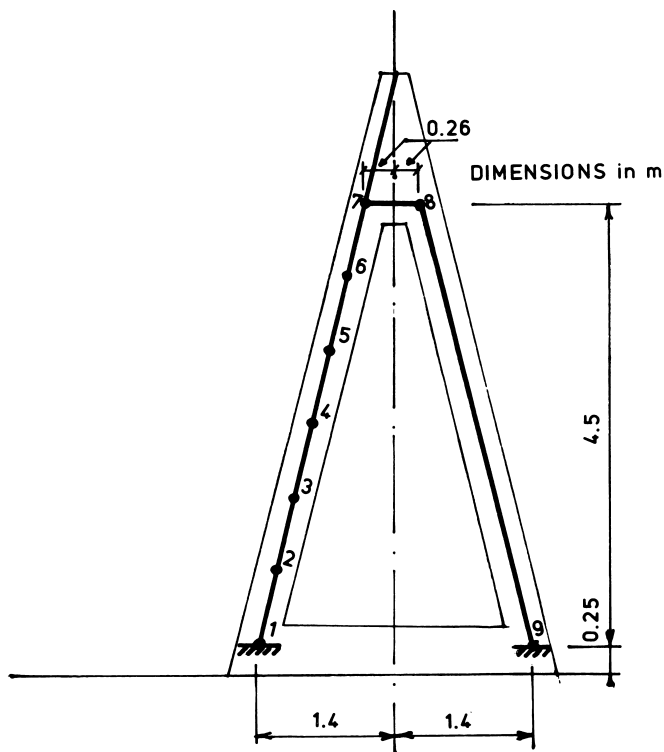


Figure 6. Layout of nodes and members in the model used for structural analysis of A-frame.

17 kN.m for the 235 × 400-mm legs. Beyond this cracking moment of bending rigidity  $EI$  reduces as more of the concrete in the tensile zone of the cross section cracks. As a result bending moments redistribute away from points of high stress.

To approximate the extreme limit of redistribution of bending moments, a second analysis was carried out with a perfect hinge

placed at node 1, the bottom of the front leg and the highest stressed point of the frame. The shear force and bending moment results from this analysis are shown in Fig. 7b for the front and Fig. 8b for the rear leg of the A-frame. The bending moment at node 1 has of course reduced to zero and the largest bending moments are now near the centre of the front leg ( $M = 91$  kN.m) and at the top of the rear leg at node 8 ( $M = 96$  kN.m).

It is obvious from Fig. 8b that both the shear forces and the bending moments along the entire rear legs are increased considerably as a result of severe cracking at the bottom of the front leg. As well, the tension in the front leg increased to 119 kN and the compression in the rear leg to 153 kN.

The actual behavior of the frame probably lies somewhere between the internal forces presented in Figs. 7a and 8a, and those in Figs. 7b and 8b. However, the results of the two analyses clearly indicate where severe cracking is likely to occur. The most highly stressed areas are (1) the bottom of the front leg with tension on the outside face of the frame, and high shear forces (Fig. 4), (2) the center of the front leg with tension on the inside face (Fig. 3), and (3) the top of the rear leg with tension on the outside and again a moderately high shear force (Fig. 1).

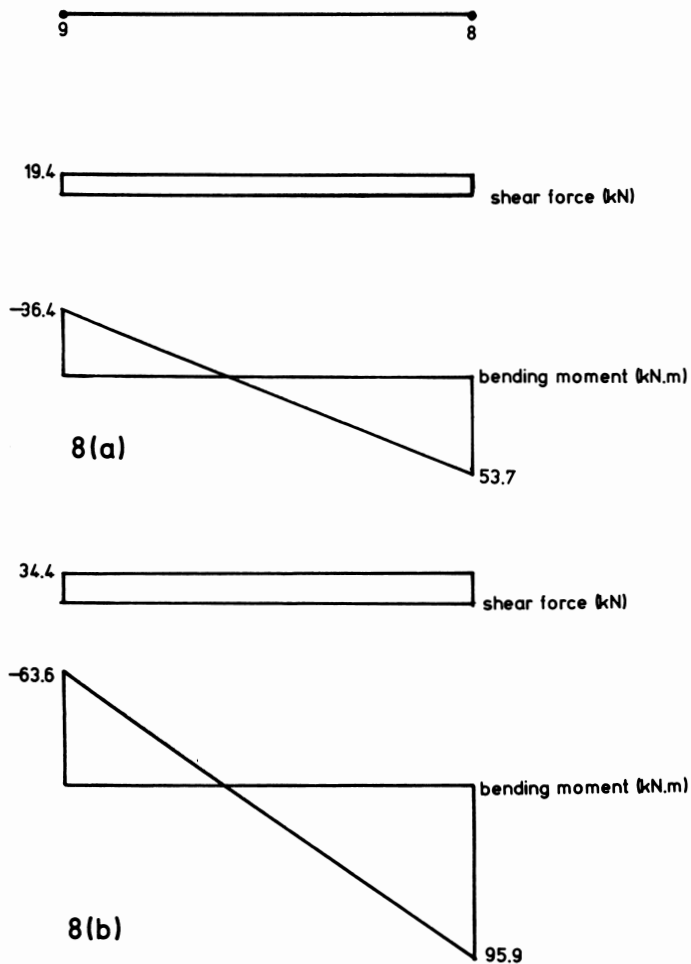


Figure 8. Shear forces and bending moments in the rear leg.

It is clear that, since no stirrups were used, the legs are susceptible to failures where high bending moments and high shear forces act at the same location such as at the bottom of the front leg and at the top of the rear leg. This mode of failure, shown clearly in Fig. 4, occurred in several silo frames (Darby 1986, personal communication).

The pressure diagram that was used for the analysis is probably on the conservative side or many more failures would have resulted with bunker silos constructed to CPS 7435. However, it is likely to be closer to the actual pressures than those specified in CFBC 1983. The latter result in bending moments that are less than 29 kN.m everywhere (Turnbull et al. 1987).

### EXPERIMENTAL WORK

In 1987, an experimental project was started at a farm owned by Jemstar Farms Ltd. The objective of that project was to measure wall pressures on walls of large bunker silos. The project is sponsored jointly by Alberta Agriculture and the Ontario Ministry of Agriculture and Food. Although the first year of testing was plagued by instrumentation problems, some preliminary results are available. A summary of this experimental work follows. A more complete description was reported by Zhao et al. (1988).

The experimental work is being carried out on the farm of Jemstar Farms Ltd., near Walkerton, Ontario in a large bunker silo with 4.9 m (16 ft) high walls sloped  $10^\circ$  from vertical. The

silos are constructed of precast concrete panels (3.7 m  $\times$  4.9 m) supported by precast concrete A-frames placed 3.7 m on centre.

Wall pressure measurements are made by means of strain gauge based force transducers. One of the precast wall panels is equipped with three vertical rows of six transducers providing force measurements over an area of about 0.03 m<sup>2</sup> at spacings of approximately 0.66 m vertically and 0.92 m horizontally. A layout of the force transducers is shown in Fig. 9.

The design of the wall transducers is such as to provide a contact surface with the stored material flush with the face of the silo wall. The contact surface is also designed to have a texture that is similar to that of the silo wall so that meaningful friction force measurements can be made. A 200-mm-diameter concrete filled disk supported firmly by an L-shaped stainless steel tubular frame forms the force transducer. The geometry of the force sensor frame was selected such that the displacement of the face of the sensor is minimal for the expected ratio of normal to downward friction force of about two. A detailed drawing of a force transducer is shown in Fig. 10.

The silo was filled with whole-plant corn silage in the first week of September 1987. The initial moisture contents of the silage on a wet basis (*wb*) are shown in Table 1. The average cut length was 10 mm. The silage mass was piled up and packed frequently by 20-ton bulldozer on tracks. After 1 wk, the top of the silage mass was covered with plastic sheets. Old tires and soil were placed on the sheets to hold them in place.

During unloading of the silo, undisturbed silage samples were taken from the vertical unloading face on three separate occasions to determine the moisture content and the density of the settled silage. The sampling was done at five heights along a vertical line about 1.5 m in from the wall-floor intersection at three locations. Location A (23 Nov.) was about three wall panels west of the test panel, Location B (7 Dec.) two panels west and Location C (21 Dec.) just opposite the test panel.

The moisture content and density measurements are given in Tables II and III, respectively. Each value in the tables is the average of three replicates. The moisture content of the silage mass varied from 68.1 to 64.6% and increased with depth. The moisture content appeared to be on average 5.6% lower than the initial value. Saturation occurred at the floor-silage interface and juice flow was observed.

Table III shows that the average bulk density of the silage was slightly higher at the top and the bottom than at midheight. However, the variation was not significant and it is clear that a fairly uniform compaction was reached. The overall mean bulk density for the silage was 874 kg/m<sup>3</sup>; the mean of the densities at the station nearest the test panel was 932 kg/m<sup>3</sup>.

It may be seen from the bulk density values in Table III that the compaction in the experimental silo was very thorough. It is possible that the use of a tracked vehicle for compaction was a factor. A more typical bulk density in a bunker silo filled with 66% moisture content corn silage would be 700-725 kg/m<sup>3</sup>. In the setting of standards it is well to note that considerably higher densities can be achieved because of their direct effect on lateral wall pressures.

The lateral pressures measured at 11:00 h on each of five days are plotted versus the depth below the top of silage in Fig. 11. Each data point represents the mean of the three lateral pressures obtained from three transducers at the same level. Straight lines were drawn to connect corresponding means at the six levels thus providing five pressure profiles each based on 18 measurements. The design pressure curve of CFBC (1983) is also shown for comparison, as is the pressure diagram of

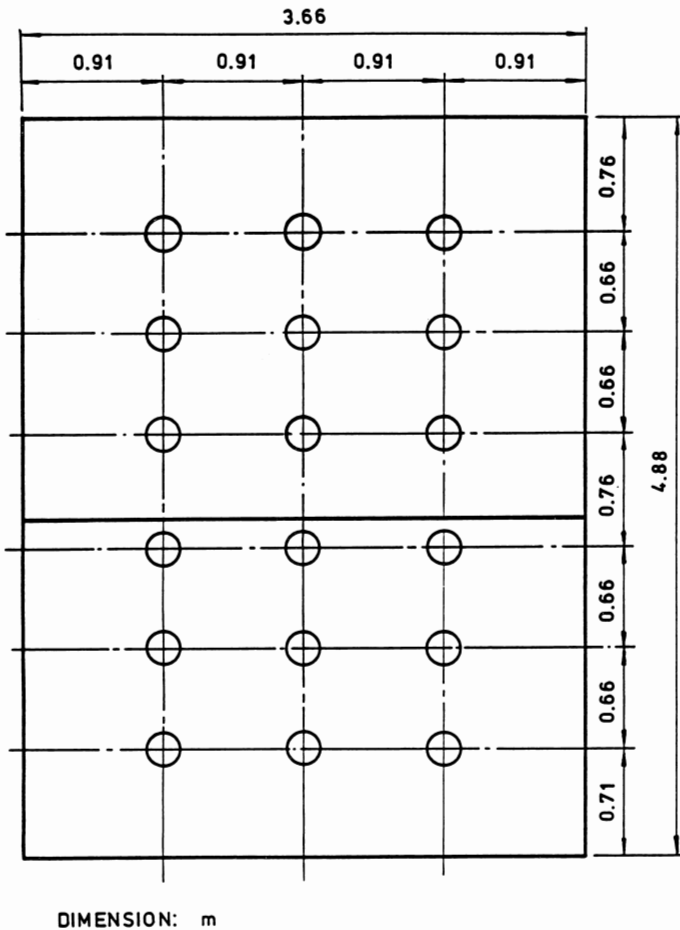


Figure 9. View of the test panel with layout of circular hole for force transducers.

BSS-5502 (1978). The latter has been recommended for incorporation in the 1990 Canadian Farm Building Code.

The results in Fig. 11 indicate that the static silage pressures normal to the wall of the bunker silo increased almost linearly with depth down to the 5th transducer, about 1.4 m above the silo floor. The rate of increase is about 5.1 kPa per metre depth. These findings are consistent with the soil mechanics theories discussed earlier. The observed pressures at the lowest row of transducers, about 0.7 m from the floor, were marginally less than those measured along the 5th row. This is probably due to the effect of friction with the floor of the silo. Similar reductions in pressure were observed near the floor of tower silos (Brunet and Jofriet 1985) and bulk vegetable bins (Anonymous 1987 a,b).

The slope of the experimental pressures (5.1 kPa/m) can be used to estimate the pressure ratio,  $k$ . If the bulk density of the silage is assumed to be  $874 \text{ kg/m}^3$ , the mean of all observations (Table III,  $k$  works out to be 0.58. If the average bulk density at the station nearest the test panel is used ( $932 \text{ kg/m}^3$ ) then  $k$  is closer to 0.54. It is well to bear in mind the limited amount of data available. Any conclusions are at best tentative.

The results in Fig. 11 show that the pressures on most transducers exceeded CFBC (1983) provisions. The maximum value 3.5 m from the surface of the silage was three times that specified by the CFBC. The experimental results agreed fairly well with design pressures specified in BSS-5502 (1978). The major difference is the rate of increase in pressure which is  $3.5z$  in BSS-5502 and  $5.1z$  in the experiment. The difference may well

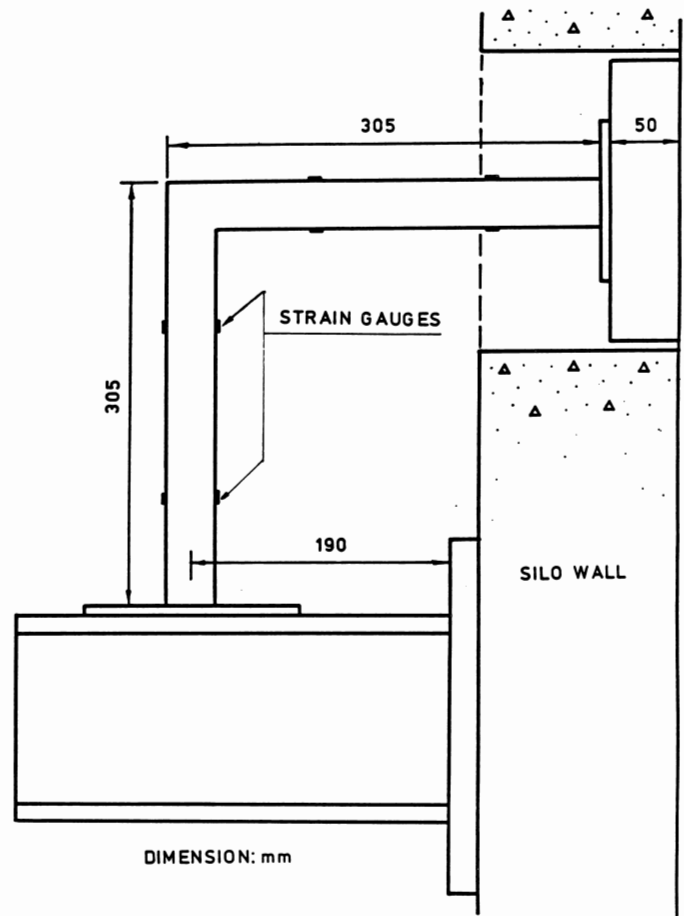


Figure 10. Side view of a force transducer supported on a WF beam bracker.

Table 1. Initial moisture contents of the silage

Date YY/MM/DD	Moisture content (WB) %
87/08/31	72.4
87/09/01	73.5
87/09/02	67.7
87/09/03	70.4
87/09/04	70.1
Mean	70.8

Table II. Moisture content after compaction in % (WB)

Location	Height (m) from bottom					Mean
	0.5	1.0	2.0	3.0	4.0	
A	71.1	69.5	68.2	68.1	63.7	68.1
B	68.9	69.3	69.7	63.4	64.6	67.7
C	67.9	64.8	64.7	62.4	63.0	64.6
Mean	69.3	67.9	67.5	64.6	63.6	

be a result of the high degree of compaction of the silage in the silo used for the experiments.

The recommendations for bunker silo wall pressures in future codes will probably have to become a function of silage density as well as depth below silage surface. The next edition of the CFBC will have provisions that are virtually identical to the British Code, BSS-5502 (1978) (i.e.,  $L = 3.5 + 3.5z \text{ kPa}$ )

**Table III. Densities after compaction ( $\text{kg/m}^3$ )**

Location	Height (m) from bottom					Mean
	0.5	1.0	2.0	3.0	4.0	
A	923.1	844.0	833.8	884.7	873.5	882.4
B	927.1	878.9	749.2	722.6	754.4	806.4
C	980.2	931.2	881.8	948.3	949.2	932.1
Mean	943.5	884.7	821.6	851.9	859.0	873.6†

†Grand mean.

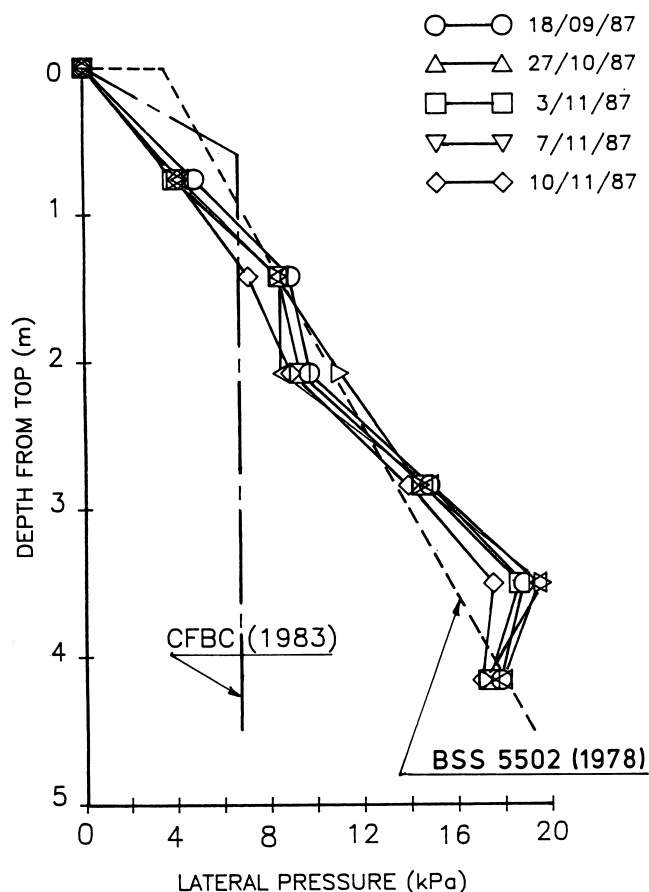


Figure 11. Average pressures at the six force transducer levels.

as an interim change. At the completion of the research project that is ongoing at Guelph (Zhao et al. 1988) further recommendation for code provisions will be made.

### SUMMARY

This paper is a progress report of a project that concerns pressures on walls of large bunker silos. The analysis of the A-frame support of the wall of a bunker silo that exhibited severe cracking provides some indication of the reasons for the failure. It also indicates, as does recent research in Britain and Sweden, that the CFBC (1983) provisions for the design of bunker silo walls are inadequate.

The experimental part of the project was described briefly and preliminary results were reported. These preliminary results

indicate that the maximum lateral pressures on the 5.0-m-high bunker silo wall are about three times those specified in the present (1983) CFBC. The measured wall pressures are in good agreement with the most recent work on the same topic by Messer and Hawkins (1977). Silage pressures were found to increase linearly with depth, which corresponds with soil mechanics theories for retaining walls. A tentative experimental value for the pressure ratio  $k$  is about 0.55

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