Brick Cladding Distress in Reinforced Concrete Structures: Four Case Studies

G.T. Suter
Professor

M. M. Seddeik
Former Graduate Student

H. Keller
Research Engineer

Department of Civil Engineering, Carleton University, Ottawa, Canada.

Abstract

The computer program outlined in a first paper at this conference entitled "Brick Cladding Distress on Reinforced Concrete Structures: Deformations and Stresses" was applied to four case studies involving cladding distress in Ottawa, Canada. The paper outlines the essential structural details for each building and briefly discusses the nature of the distress. It was found that the brick cladding may be overstressed but not to an extent to cause distress as observed on the actual structures. The paper reviews the computed stresses in the light of allowable and ultimate masonry stresses. It then discusses other contributing factors which, taken together with the level of overstressing, may lead to the distress found in the brick cladding. The paper concludes that while the determination of differential deformations is of great importance for the cladding to function properly, of equal or even greater importance are the shelf angle detailing as well as the construction and maintenance of the veneer.

1. INTRODUCTION

The procedure proposed to calculate the magnitude of deformations of reinforced concrete, clay brick and concrete block [1]* was applied to four case studies involving clay brick cladding distress in Ottawa, Canada. Particulars of the four case studies are summarized in Table 1. Note that in all four case study structures no movement joints were provided in the brick veneer and in the concrete block backup if the latter was present.

Table 1 Summary of Case Study Building Particulars

Case Study	Age at Time of Distress (Years)	No. of Storeys	Height per Storey (m)	Cladding Wall Section	Typical Gravity Loads Dead kN/m^2	Live kN/m^2	Material Properties
1	2	18	2.64	102 mm Brick 152 mm Block 25 mm Insulation 13 mm Drywall	5.5	1.9	R.C.* 27.6 MPa Brick 69 MPa Block 17.2 MPa
2	2	19	2.59	152 mm Brick 38 mm Insulation 13 mm Drywall	4.3	1.9	R.C. 34.5 MPa 1-7th Storey R.C. 27.6 MPa 8-19th Storey Brick 69 MPa
3	1.5	24	2.65	102 mm Brick 51 mm Insulation 152 mm Block	6.0	1.9	R.C. 34.5 MPa 1-9th Storey R.C. 27.6 MPa 10-24th Storey Brick 69 MPa Block 17.2 MPa
4	1	8	3.15	102 mm Brick 38 mm Insulation 13 mm Drywall	7.2	2.4	R.C. 34.5 MPa 1-5th Storey R.C. 27.6 MPa 6-8th Storey Brick 69 MPa

* R.C. denotes reinforced concrete

In order to assess if the cladding distress could have been caused solely by the differential deformation between the structure and its cladding, each case study was examined under two conditions: firstly, severe conditions, where all material properties and the surrounding environmental conditions were chosen to result in the maximum possible differential deformations and secondly, under moderate conditions. The values for these severe and moderate conditions for the Ottawa area [2] are presented in Table 2.

Table 2 Summary of Severe and Moderate Conditions

Variable	Severe Conditions	Moderate Conditions
Concrete:		
Ultimate shrinkage strain	0.0008	0.0004
Brick masonry:		
Maximum moisture expansion strain	0.0004	0.0003
Kiln to wall time lapse (days)	7	30
Block masonry:		
Ultimate shrinkage strain	0.0006	0.0004
Temperature (°C):		
January	-22.1	-16.2
February	-24.3	-15.7
March	-4.7	-4.7
April	+3.5	+3.5
May	+32.6	+21.1
June	+29.5	+23.0
July	+33.4	+26.5
August	+32.4	+25.2
September	+12.5	+12.5
October	+7.3	+7.3
November	-14.9	-4.5
December	-20.2	-7.6

* Numbers in parentheses refer to references at the end of this paper.

For each case study, the free deformations and the stresses resulting from the restrained movements between concrete frame and clay brick cladding were determined. The resulting cladding stresses were calculated as a function of two parameters: horizontal joint width and support stiffness. A computer program was developed [1] to perform the iterative steps required to arrive at deformations and possible stresses in the brick cladding when free movement is restrained due to the shelf angle detail. The computer program covers various time intervals up to a maximum of twenty years and handles buildings of up to 30 storeys in height.

Results from the four case studies and other computer simulation runs indicate that the worst possible cladding stresses would occur under the following assumptions:

- absence of movement joints in brick cladding;
- 100 percent cladding support stiffness;
- severe conditions as outlined in Table 2;
- time period of 20 years which is the maximum time period included in the computer program;
- first storey in a structure.

While the effect of varying these assumptions is treated elsewhere [2], this paper will concentrate on the worst assumptions listed above. The detailed results presented for case study 1 in Table 3 are typical of all four case studies.

Table 3 Case Study 1 Stresses (MPa)

Storey Number	Reinforced Concrete Movement Joint Width (mm)			Brick Cladding Movement Joint Width (mm)			Block Backup Movement Joint Width (mm)		
	0	3.2	6.4	0	3.2	6.4	0	3.2	6.4
1	-3.09	-6.13	-7.92	-6.52	-2.30	0	1.41	-1.01	-2.19
2	-2.68	-5.72	-7.72	-6.52	-2.46	0	1.41	-0.94	-2.21
3	-2.37	-5.44	-7.39	-6.43	-2.41	0	1.50	-0.86	-2.17
4	-2.03	-4.88	-6.78	-6.32	-2.34	0	1.56	-1.06	-2.37
5	-1.64	-4.18	-7.39	-6.26	-2.50	0	1.63	-1.06	-1.40
6	-1.46	-4.08	-6.26	-6.09	-2.06	0	1.85	-1.16	-1.88
7	-1.27	-3.97	-5.37	-5.83	-1.81	0	1.93	-0.99	-2.12
8	-1.08	-3.41	-4.77	-5.66	-1.66	0	2.14	-1.12	-2.10
9	-0.92	-2.93	-4.27	-5.49	-1.48	0	2.36	-1.22	-1.96
10	-0.66	-3.07	-4.39	-5.41	-1.46	0	2.50	-0.99	-1.95
11	-0.49	-2.63	-5.08	-5.23	-1.67	0	2.74	-0.43	-0.77
12	-0.23	-2.63	-4.71	-5.02	-1.83	0	2.87	-0.40	-0.43
13	-0.02	-1.97	-3.08	-4.81	-0.96	0	3.04	-0.70	-1.14
14	0.25	-1.50	-2.32	-4.66	-0.72	0	3.24	-0.79	-1.12
15	0.54	-1.26	-2.08	-4.52	-0.72	0	3.43	-0.36	-0.69
16	0.79	-1.05	-1.61	-4.34	-0.49	0	3.61	-0.21	-0.44
17	1.08	-0.54	-0.54	-4.19	0	0	3.80	-0.67	-0.67
18	1.42	-0.77	-0.77	-4.04	0	0	3.98	0.17	0.17

Assumptions:
- severe conditions as outlined in Table 2
- 100% cladding support stiffness
- time period of 20 years

Negative values denote compression

2. CASE STUDIES

2.1. Case Study 1

Case Study 1 involves an eighteen storey apartment building of reinforced concrete flat plate and column construction. Concrete shear walls are used to resist lateral forces. The building's cladding consists of 100 mm clay brick backed up by 150 mm concrete block. The rigid 25 mm insulation is placed on the inside face of the block masonry wall. The brick cladding is typically supported at each floor level on a steel shelf angle and at ground level it rests on a shprt concrete cantilever. Fig. 1 shows a typical section of the floor plan with the cladding panel and cladding support details for the whole structure. Material properties and other building particulars are summarized in Table 1.

The cladding distress in this case consisted of severe vertical cracking in the lower storeys, especially at corners of the building, and of spalling of some of the cladding units between the second and third floor levels. At the time of the distress problem, the building was approximately two years old.

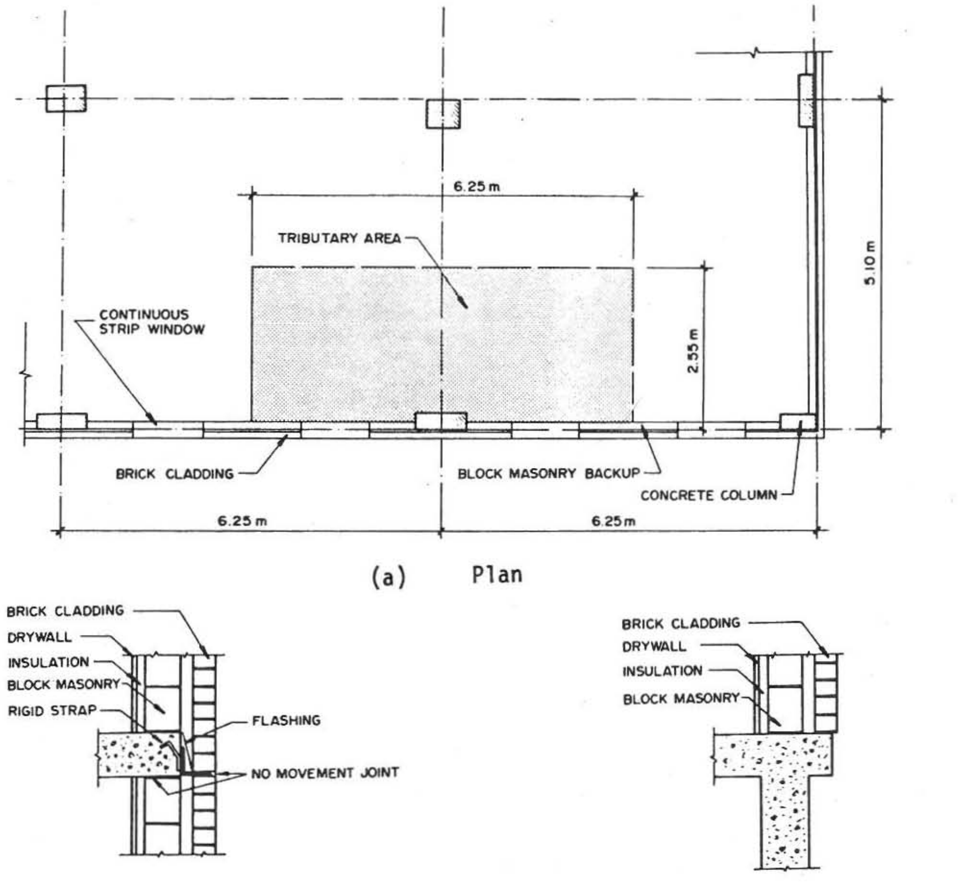
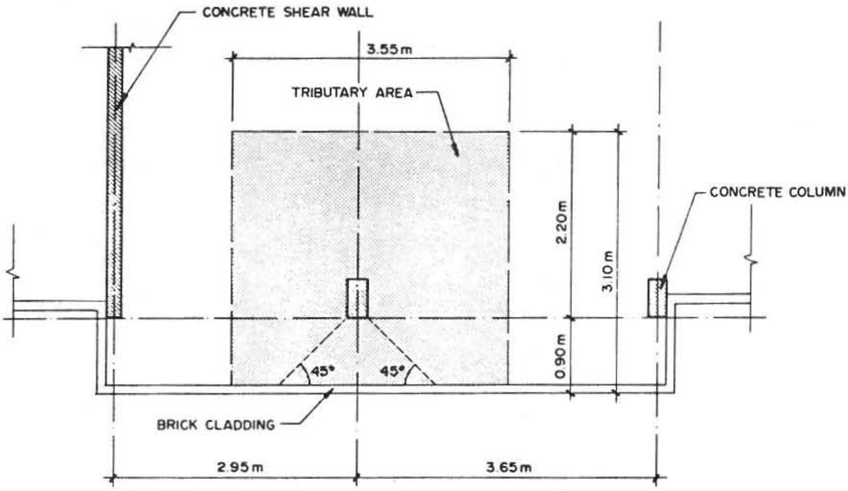


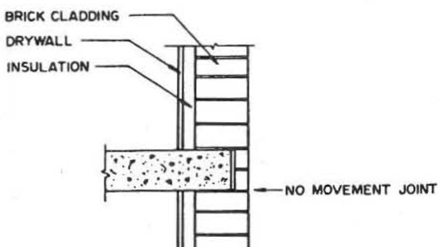
Fig. 1 Case Study 1 Floor Plan and Cladding Detail

2.2 Case Study 2

Case Study 2 refers to a nineteen storey apartment building of reinforced concrete flat plate and column construction with concrete shear walls designed to resist lateral forces. The cladding consists of 150 mm clay brick which rests directly on each concrete floor slab. A smaller 50 mm wide brick is used to continue the cladding at columns and at the front edge of each slab. Fig. 2 shows a typical section of the floor plan which was investigated and the cladding support detail. Note, that the concrete columns are typically set back by about 0.9 m from the building's outer surface. For the purpose of analysis it was assumed that deformations of the concrete columns affect only that part of the concrete slab which is bounded by a triangle forming 45^{o} angles as indicated in Fig. 2. Hence only the cladding for this part of the slab is considered. Material properties and other building particulars are included in Table 1. The distress was noted as vertical cracking between the second and third floor especially at the south-east and north-west corners, and at the west elevation. Some diagonal cracks and some spalling or crushing of mortar joints at the second floor level was also reported. At the time of the distress the building was approximately 2 years old.



(a) Plan of a Typical Floor Panel



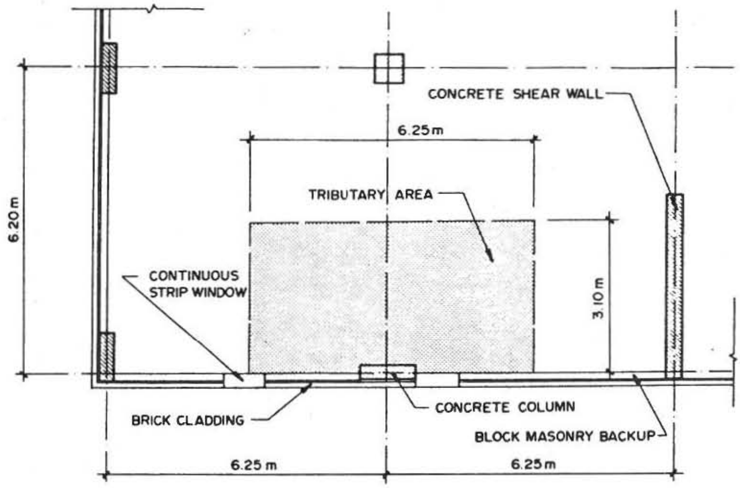
(b) Cladding Support Detail

Fig. 2 Case Study 2 Floor Plan and Cladding Detail

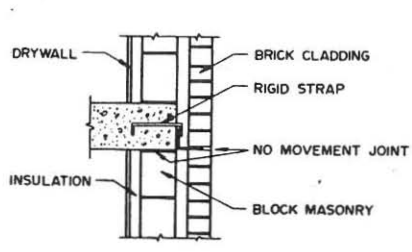
2.3. Case Study 3

Case Study 3 pertains to a twenty-four storey apartment building of reinforced concrete flat plate and columns with concrete shear walls used to resist lateral forces. The building's envelope consists of 100 mm clay brick backed up by 150 mm concrete block. The 50 mm rigid insulation is placed between the brick cladding and its backing. Shelf angle supports for the cladding are provided at each floor level. Fig. 3 shows the typical floor plan section under investigation and the cladding support detail. Material properties and other building particulars are listed in Table 1.

The distress of the cladding was reported as vertical cracking at the corners of the building and near some window openings at the west elevation. Crushing of the cladding was noted at its outer face above the shelf angles and at its inner face below the shelf angles at many levels. Spalling of some bricks at the north-east and south-west corners was also observed. At the time of distress the building was approximately 1.5 years old.



(a) Plan of a Typical Floor Panel

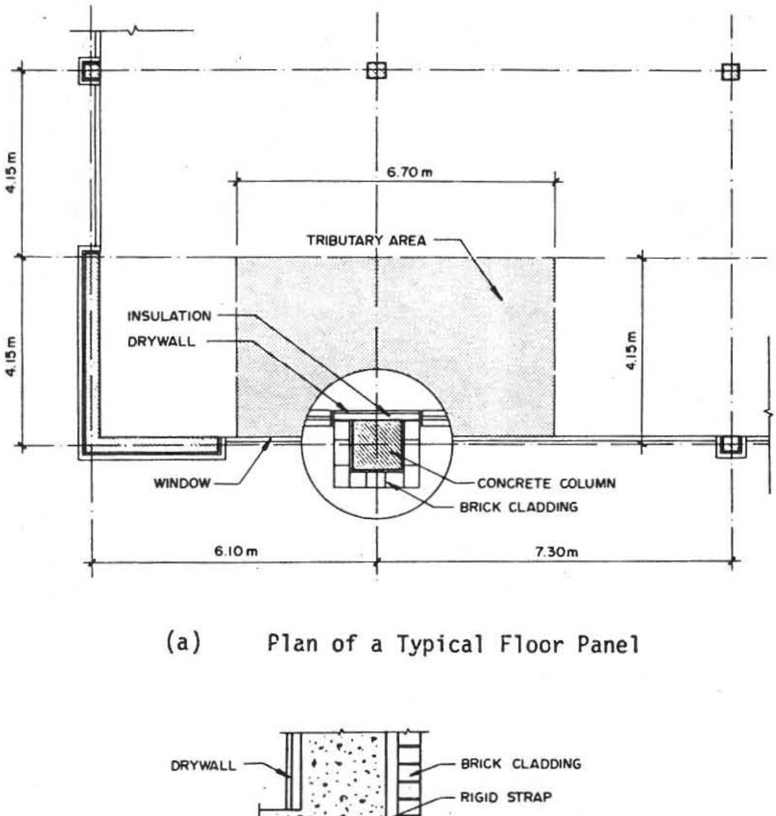


(b) Shelf Angle Detail

Fig. 3 Case Study 3 Floor Plan and Cladding Detail

2.4 Case Study 4

Case Study 4 involves an eight storey office building of reinforced concrete flat plate and column construction with concrete shear walls at corners designed to resist lateral forces. The building's cladding consists of 100 mm clay brick supported on shelf angles at each storey. The 38 mm rigid insulation is placed at the inside surface of the concrete columns or shear walls. Fig. 4 shows a typical section of the floor plan and the shelf angle support detail. Material properties and other building particulars are summarized in Table 1. The cladding distress consisted of vertical cracking in the first storey at the corners of some columns at the south elevation. Some cracks at the upper storeys near the corners of the west elevation were also noted. At the time of the distress the building was approximately 1 year old.



(a) Plan of a Typical Floor Panel

(b) Shelf Angle Detail

Fig. 4 Case Study 4 Floor Plan and Cladding Detail

3. TYPICAL STRESS RESULTS

Typical stress results from the computer program for case study 1 are listed in Table 3 and illustrated in Fig. 5. It is seen that as the movement joint width in the brick cladding decreases, more and more compressive loading is shifted from the reinforced concrete columns and the block backup (known to be built tight to the underside of the slab) to the brick cladding.

Since for all four case studies no cladding movement joints were provided and the block backup, where present, was built in tight to the reinforced concrete slabs, the condition of zero joint width is of greatest interest to this paper. Table 4 summarizes the resultant stresses at the first storey for all four case studies. Two important points can be noted: Firstly, the brick cladding stress in three out of four cases exceeds the allowable stress even to the extent of reaching twice the allowable value. Secondly, the maximum brick cladding stress of 10.15 MPa for case study 4 still represents only about half of the ultimate cladding strength. Could then the cladding stress levels indicated in Table 4 have led to the extent of vertical cracking, crushing and spalling observed on the structures ? Not likely for the following reasons:

- The extent of cracking would be associated with stress levels of at least 75% of ultimate strength, not 50% or lower.
- The cladding stresses in Table 4 apply to extreme deformational conditions which in practice have a very low probability of occurring simultaneously.
- Building distress was observed after 1 to 2 years while the computed stresses in Table 4 refer to 20 years. Cladding stresses at 1 to 2 years would obviously be reduced and hence even for case study 4 a margin of safety against ultimate failure greater than 2 would exist.

In summary it can be stated that although the differential deformations can lead to overstressing, they could not by themselves cause the cracking and failure of the cladding observed on the four structures.

Table 4 Summary of Case Study Stresses at First Storey (MPa)

Case Study	Reinforced Concrete	Brick Cladding	Block Backup	Brick Cladding: Allowable Stress*	Brick Cladding: Ultimate Strength*
1	-3.09	-6.52	1.41	-5.0	-20.0
2	-1.12	-4.48		-5.0	-20.0
3	-4.61	-7.12	-0.10	-5.0	-20.0
4	-7.86	-10.15		-5.0	-20.0

Assumptions:
- no movement joint
- severe conditions as given in Table 2
- 100% cladding support stiffness except for case study 2 (75% due to the given slab flexibility.)
- time period of 20 years

* Stresses based on Canadian code for type S (typically 1: ½ : 4½) mortar [3].

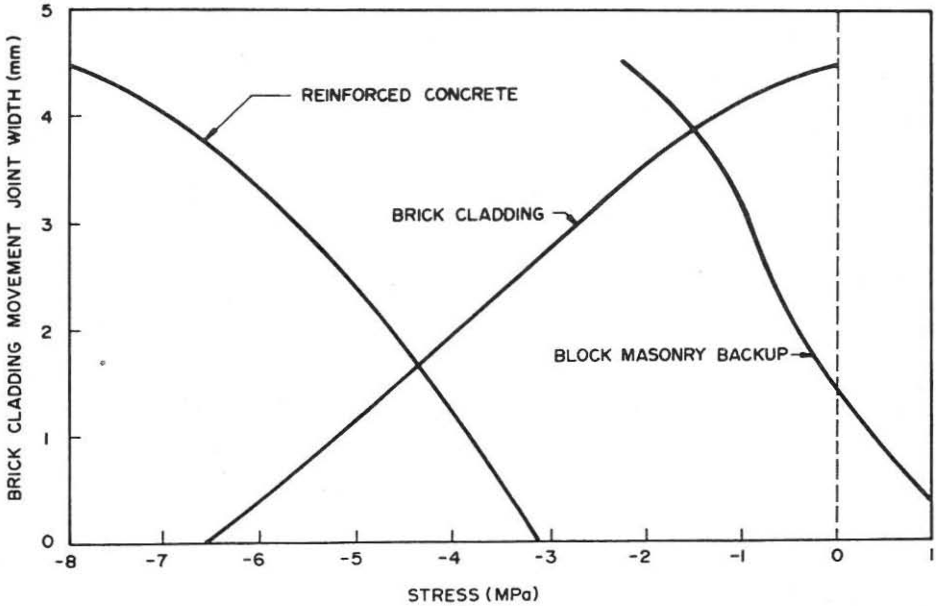


Fig. 5 Case Study 1 Stresses Versus Movement Joint Width at First Storey for 20 years and 100% Support Stiffness

4. CONTRIBUTING DISTRESS FACTORS

A review of the four case studies shows that cracking or failure of the cladding could not solely be due to axial stresses resulting from differential deformations. For failure to occur, other factors must be involved which could lead either to an overall reduction in compressive load capacity or to stress concentrations. Such factors could be grouped into the two main categories of workmanship and detailing of shelf angle connections.

4.1. Workmanship

The influence of various workmanship defects on the compressive strength of masonry has been summarized by Hendry [3] and he has shown that they can lead to overall strength reductions of up to 61%.

Other deficiencies which may further aggravate cladding distress as a result of poor workmanship include:

- Improper installation or absence of tie-backs.
- Variations in the width of seating of the cladding on the shelf angle or slab.

Inadequate lateral tie installation may cause cladding to act as a very slender element of greatly reduced compressive capacity. In the absence of an adequate movement joint, this condition may lead to a buckling type of failure which is particularly dangerous due to its unpredictability.

With regard to the width of seating of the veneer on the shelf angle, the Canadian masonry design code [4] specifies that the brick overhang shall not exceed 30 mm. In practice the amount of overhang is highly variable and has been reported to vary between 13 mm and 51 mm in a single structure [5]. Note that the variable overhang in the absence of an adequate movement joint will produce stress concentrations in the cladding which can triple and quadruple [5] the axial load stresses listed in Table 4 for the four case studies. Suter and Hall [5] have classified three failure mechanisms associated with little, normal and large overhang of brick at the shelf angle. While little or normal overhang under overstressed conditions may lead to mortar crushing and faceshell spalling, both cases represent relatively stable conditions which do not normally lead to serious problems. Excessive overhang on the other hand may result in a sudden buckling failure of the cladding.

Clearly, proper site supervision and control procedures during cladding construction would considerably reduce cladding distress caused by workmanship factors.

4.2 Shelf Angle Connection

The shelf angle connection detail has a significant effect on the possible stress transfer to the cladding. There are basically two methods of fastening shelf angles to concrete slabs. One is by using metal straps embedded in the concrete slab and welded to the shelf angle as seen in Figs. 1, 3 and 4. This method constitutes a fixed connection which is undesirable because it may lead to large variations both in thickness of a horizontal movement joint and width of seating of the brick on the shelf angle. The second method employs a bolted connection between shelf angle and a cast-in-place insert; it allows for adjustment of the shelf angle as the cladding is being constructed and hence is recommended.

5. CONCLUSIONS

Based on the results obtained from the four cladding distress case studies, the following conclusions are drawn:

- Absence of a differential movement joint in the cladding typically leads to overstressing of the cladding. The level of overstressing in each of the four case studies, however, is not severe enough to be the sole cause of cladding distress observed on the structures.

- Contributing distress factors are due to workmanship defects and the shelf angle connection. The contributing distress factors can lead to a significant reduction in compressive load capacity or to stress concentrations in the cladding. It is likely that the combined effects from the level of overstressing and the contributing distress factors caused the cladding distress in the four case studies.

- Provision of adequate movement joints in the design, construction and maintenance of brick-clad structures is required to prevent the costly, and often dangerous, cladding distress encountered in practice.

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REFERENCES

[1] Suter, G.T., Seddeik, M.M. and Keller, H., "Brick Cladding on Reinforced Concrete Structures: Deformations and Stresses", Proceedings of the Sixth International Brick Masonry Conference, Rome, May 1982.

[2] Seddeik, M.M., "Masonry Cladding Distress in Reinforced Concrete Structures", M. Eng. Thesis, Carleton University, Ottawa, June 1980.

[3] Hendry, A.W., "The Effect of Site Factors on Masonry Performance", "Proceedings of the First Canadian Masonry Symposium ", Calgary, June 1976, pp. 182-198.

[4] Canadian Standards Association, "Masonry Design and Construction for Buildings", CSA Standard CAN3-S304-M78, 1978.

[5] Suter, G.T., and Hall, J.S., "How Safe are Our Cladding Connections", Proceedings of the First Canadian Masonry Symposium, Calgary, June 1976, pp. 95-109.