Development of Thin Wall Cladding to Reduce Drift in Hi-Rise Buildings

Revêtement de façade à paroi mince réduisant le mouvement horizontal des immeubles multi-étagés

Dünnwandige Fassadenelemente zur Reduktion der Horizontalauslenkung von Hochhäusern

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SUMMARY

One Mellon Bank Center represents the first development of a thin wall metal facade panel (stress skin tube) to provide major stiffness for a hi-rise building. To achieve this results, a substantial analytical, design, detailing and testing program was undertaken. This paper reviews the program and presents the final product that was achieved.

RÉSUMÉ

Le «One Mellon Bank Center» est le premier bâtiment employant la technique des panneaux de façade en acier à paroi mince (structures en tube) qui renforcent la raideur d'un immeuble multi-étagé. Pour réaliser cela, un programme de calcul, d'essais et de conception des assemblages fut conduit. Cet article présente le programme ainsi que l'ouvrage réalisé.

ZUSAMMENFASSUNG

Das «One-Mellon-Bank-Zentrum» zeigt zum ersten Male die Entwicklung einer mitwirkenden dünnwandigen Fassaden-Verkleidung mit dem Ziel, eine grössere Steifigkeit des Hochhauses zu erreichen. Zu diesem Zweck wurde ein umfangreiches Programm für Berechung, Entwurf, konstruktive Details und Erprobung durchgeführt. Der Artikel beschreibt das Programm und die gewählte Lösung.

INTRODUCTION

One Mellon Bank Center, (OMBC), is a 54 story, 222 meters high office building in Pittsburgh, Pennsylvania (Fig. 1).   It represents the first example of a stressed-skin tube using steel plate facade panels to provide major stiffness against building drift. The basic principle which makes the plate facade panels cost-effective is the separation of strength requirements from drift requirements. Since current U.S. building codes define required strength (wind pressure, seismic coefficients) but not wind drift, items provided only for wind drift need not be fireproofed.   Thus, the facade panels have the structural plate face exposed, without the expense of fireproofing or flame shields.

STRUCTURAL FRAME

The building utilizes tube frame construction which consists of closely spaced perimeter   columns   and   stiff   spandrel   beams.   Columns 10 feet apart were architecturally desirable and no problem if kept narrow. However, to provide adequate stiffness in tube frames, spandrel beams are commonly made as deep 3-plate sections and located upset, so that about half the beam depth projects above the floor.   This beam is usually concealed by an extra-deep windowsill, taking away valuable usable floor space.   With plate stiffening this is not required and the spandrel can be placed below the floor, thus adding valuable usable floor space.

Since the panels cannot be counted upon for strength, a two-step analysis was performed.   First,   tower   framing   was   modeled without facade panels. Several trials with condensed, simplified models aided in member selection for the first run.   Approximately 3000 nodes and 6000 members were modeled using the EASE-2 computer program.   The use of rigid diaphragms at each floor reduced the degrees of   freedom   to   be   solved   to   approximately 9,200.   Loading consisted of code-required   wind   forces   in   the   longitudinal, transverse and quartering direction.   Code requirements were found to be more critical than the overall wind pressures determined in wind tunnel tests at Colorado State University.

The   second model included tower framing plus façade panels (Fig. 2). To simulate the   effect   of   the   façade   panels,   equivalent   uniform   membrane elements were inserted in the model. The membrane stiffness required was established by using a separate model of each individual type of façade panel, using fine-mesh finite elements and racked using a 100N shear force. These models were later used for design of individual panels. The overall tower model showed that building drift with façade panels would be H/590, which was acceptable.

Panel membrane stresses were translated directly into panel shear forces on a panel-by-panel basis.   From this a table similar to a column schedule was developed, and panel thicknesses and stiffener patterns were selected to meet the load requirements.

In addition, the change in shear force from panel to panel was determined. This was needed to establish the panel-to-column connection requirements, since only the change in force travels through this connection. The balance of shear travels through panel-to-panel connections.

STRUCTURAL FAÇADE PANEL

Description

The basic façade panel consists of a 300.7 cm. wide by 1107.4 cm. high panel covering three stories. The typical story height is 3.66 meters. There are six

window openings 133.3 cm. wide by 202.2 cm. high in a typical panel (see fig. 3). Stiffeners are required on the inside of the panel to prevent plate buckling and provide out-of-plane strength for wind. These stiffeners are placed on all sides of the windows and are continuous for the height and width of the panel.

Strength Analysis

The classical analysis approach to plates assumes that adequate stiffness would be provided from the panel stiffeners so that each subpanel could be analyzed as an independent plate simply supported at its longitudinal edges and loaded along its perpendicular edges, or width. The basic equation for plate buckling (Johnston, 1976) is:

Where:

$$fc = k \frac{\pi^2 E \left(\frac{t}{b}\right)^2}{12 (1-\nu^2)}$$ (Eq. 1)

fc = Critical Shear Stress

E = Modulus of Elasticity

b = Width of Subpanel

t = Thickness

ν = Poissons Ratio

a = Length of subpanel

α = a/b

k = Constant for each load

The critical stresses are valid in the elastic range. To adjust for the inelastic range, for compressive stresses, the elastic solution was used to determine an "equivalent column" slenderness ratio (KL/r). The equivalent slenderness ratio (Johnston, 1976), for ν =0.3, would be:

$$(KL/r) \text{ equivalent} = \frac{3.3 (b/t)}{\sqrt{k}}$$ (Eq. 2)

This "equivalent column" slenderness ratio was used to determine allowable compressive stresses following AISC allowable compressive stress criteria.

Allowable shear stresses can be determined elastically and also by post-elastic-buckling analysis. Since visual buckling of the plate would not be desireable in the façade, only the elastic buckling solution was considered. The elastic buckling solution to web shear stress allowable has been adjusted by the AISC in their allowable shear stresses for inelastic behavior.

The actual state of stress in the panel is a combination of axial, bending and shear stress. The method of combining these stresses was based on the Column Research Council recommendations (Johnston, 1976). The following interaction equation was thus used:

$$(fa/Fa) + (fb/Fb)^2 + (fv/Fv)^2 \leqslant 1.0$$ (Eq.3)

Where subscripts a,b,v are axial, bending & shear respectively and f = actual stress, F = allowable stress.

The key to this classical analysis was the principle that the stiffeners were fully effective so that each subpanel defined by the stiffener grid could be

checked on an individual basis for buckling. To determine whether the stiffeners were effective, two analytical techniques were used.

First, the panel with its stiffeners is analogous to a cold formed structural steel member as a deep, relatively thin walled section. Therefore, the requirements of the AISI specification for the design of cold-formed steel for laterally unbraced beams were used as design criteria for the stiffeners (AISI, 1977).

As a second approach, the classical methods of analysis were confirmed by a state of the art computer analysis for plate buckling. The method applied was a finite element analysis for buckling, using the NASTRAN program. The goals were to verify that the subpanels would buckle before the stiffeners. In addition, the calculated stress at which the panels and stiffeners buckle was compared to classical solutions. A simulation of the computer buckling analysis used is shown in fig. 4.

Good agreement was found between the finite element buckling analysis and classical buckling analysis. It was therefore considered acceptable to use classical analysis to size the panels.

Stiffness and Stress Analysis

Once the buckling stresses for different thicknesses of panels and stiffener arrangements were determined, it was necessary to determine the state of stress in the panels and the deflection due to an arbitrary "unit" lateral force to be able to select the correct thickness and stiffener configuration for each actual load condition. To accomplish this, a fine mesh finite element analysis was performed (see Fig. 5). Resulting isolines for principal stress are shown in fig. 7.

The stresses in each subpanel were then broken up into axial, bending, and shear stresses for gradually increasing increments of force. This was then checked against the capacity of the panel using classical analysis.

The finite element analysis also provided the means of linking panel analysis to building frame analysis so that the force in the panels at each location of the building could be determined. This linkage was made by developing a solid panel "membrane" equivalent in shear stiffness to the actual perforated panel. The equivalent solid panel was then incorporated in the structural analysis of the overall building frame as an infill shear membrane. The effect of this equivalent infill membrane on the overall structure was then determined, the force induced in the infill panels was determined, and panel types were selected.

DESIGN AND DETAILING

The detailing of the panel to the structure was critical to the execution of the structural system. Each panel must act primarily as a shear diaphragm with respect to the structural frame and be relatively isolated against axial shortening of the frame. To accomplish this, two distinct groups of connections were necessary.

1. Panel-to-panel connections

2. Panel-to-frame connections

The top and bottom horizontal connections have three functions:
1. To pass horizontal shear between upper and lower panels.

2. To distribute a share of the horizontal wind shear from the frame into the

panels.

3. To relieve any vertical stress between panels due to frame shortening, thermal effects, etc.

These functions are achieved by continuous wide horizontal plates at the top and bottom edges of the panels, which are high-strength friction bolted to the top flange of the spandrel beam about 30.5 cm. away from the face. By having the shear plates span the 30.5 cm. distance they become flexible and act to form a stress relieving joint, (see fig. 6).

At the vertical edges, extensive analysis showed that the most effective connection between panels occurred adjacent to the window openings. This location tends to minimize the weakening effect of the window openings.

Since the panels in any one wall face act as a unit, there is little vertical shear transfer to or from the structural frame at vertical connections except at the ends of wall faces where the panels end and must unload their shear back into the structure frame. Therefore, at interior vertical joints it is necessary to connect to the structural frame only at approximately midheight of each panel, using small connection fins. (See fig. 8).

At the end panels a larger edge stiffener is used as a collector of the shear force so that, although a larger force has to be transferred to the frame, bolts can still be bunched together at a mid-height location. This mid-height connection greatly limits the zone in the panel affected by the frames axial shortening.

The connections for vertical shear consist of high-strength friction bolts in holes or slots punched in outstanding legs of angle edge stiffeners. At panel-to-frame locations (mid-height) a vertical shear plate or fin projecting from the column fits in the gap between two panels. Bolts clamp panel edge angles to the fin and to each other (See fig. 9). At panel-to-panel locations, the gap between panel edges is filled with finger shims. Bolts clamp panel-to-panel.

In addition to shear-transfer connections at panel edges, tiebacks are required to resist wind pressure and brace the stiffener grid against buckling. They must be stiff perpendicular to the facade, but not produce substantial restraint to frame shortening. The solution was to provide horizontal flexible tieback plates from the structural frame to the ends of horizontal stiffeners of the panels at each window head and sill.

TESTING

Testing of a full scale prototype was done at Lehigh University's Fritz Engineering Laboratory. American Bridge Division of U.S. Steel fabricated a 6mm thick test panel and a special test frame. To test for shear the panel and frame assembly was loaded diagonally. The heavy test frame surrounding the panel acted to break this force into components so the frame took axial load and the panel carried shear load.

The test panel was designed for a lateral load of 2.7×10^5 N which would correspond to a diagonal test load of 10^6 N. The panel showed non-visible signs of incipient buckling at about 1.89 times the design load. Visible signs of buckling occurred at 2.44 times the design load. At this point, the panel was exhibiting post-buckling strength. At about 3.33 times the design load the welds failed, but not the panel itself.

In general, the test showed that panel strength was in close agreement with

analytical predictions. It also indicated that there was substantial reserve post
buckling strength in the panel. It further indicated that deflections were in the
range anticipated and were acceptable both visually and structurally.

CONCLUSION

The successful testing of the prototype panel completed the development of an
innovative structural system. The use of this panel allows for many positive
structural benefits. The important benefits include:

1. Reducing deflections of the structural frame by half.
2. Allowing the structural frame to be designed for strength rather than
   deflection.
3. Improving the efficiency of the structural tube frame system.
4. Less costly, normal rolled sections are used in conjunction with the facade
   stiffening plate system.
5. Additional useable floor space is gained.

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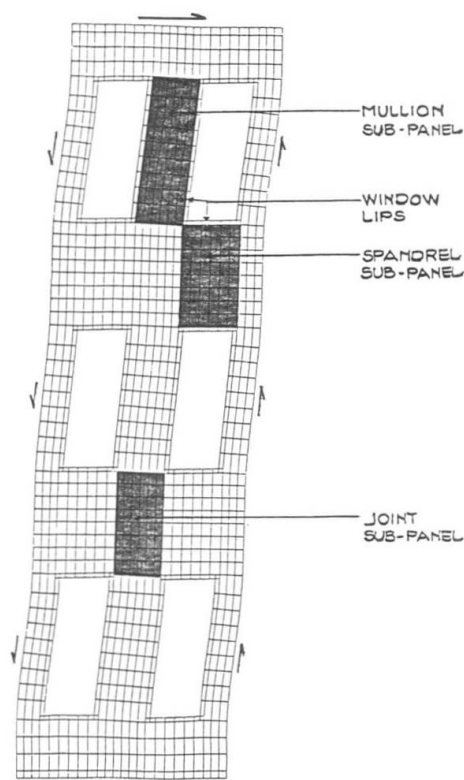
FIG. I
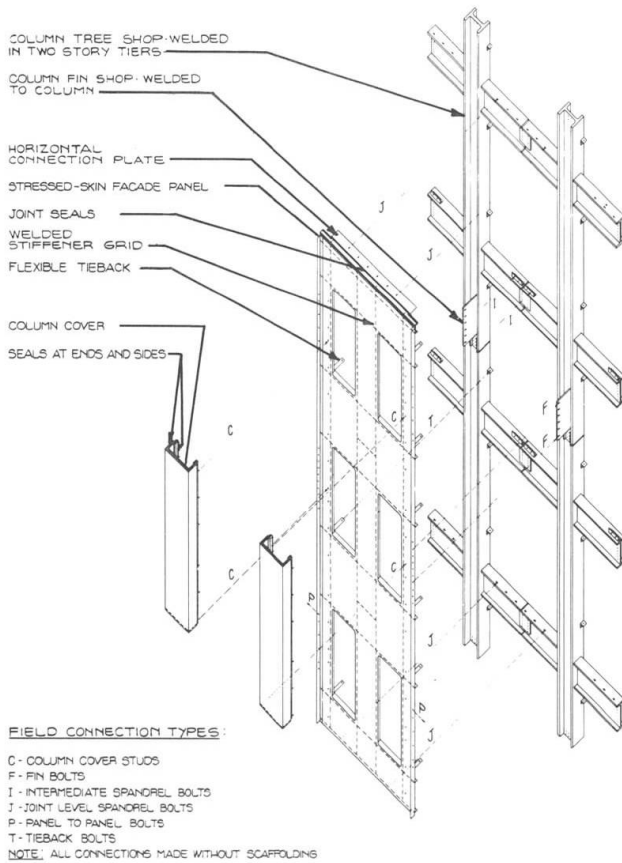
FIG. 2
RACKED PANEL
(EXAGGERATED)

COLUMN TREE SHOP-WELDED IN TWO STORY TIERS

COLUMN FIN SHOP-WELDED TO COLUMN

HORIZONTAL CONNECTION PLATE

STRESSED-SKIN FACADE PANEL

JOINT SEALS

WELDED STIFFENER GRID

FLEXIBLE TIEBACK

COLUMN COVER

SEALS AT ENDS AND SIDES

FIELD CONNECTION TYPES:

C - COLUMN COVER STUDS
F - FIN BOLTS
I - INTERMEDIATE SPANDREL BOLTS
J - JOINT LEVEL SPANDREL BOLTS
P - PANEL TO PANEL BOLTS
T - TIEBACK BOLTS
NOTE: ALL CONNECTIONS MADE WITHOUT SCAFFOLDING

FIG.3   ONE MELLON BANK CENTER:
STRESSED SKIN - TUBE INTERFACE

PANEL SPANDREL BUCKLING MODE
UNDER SHEAR LOAD
FIG. 4

NONSTRUCTURAL (FREE-FLOATING) FACADE PANELS UNSHADED.

STRUCTURAL FACADE PANELS SHOWN SHADED (EXPOSED FACES ONLY - HIDDEN FACES NOT SHADED FOR CLARITY).

THICK INFILL PANELS AT BELT TRUSSES.

STIFF SPANDRELS, NO PANELS ON THREE FACES WHERE "BUSTLE" CONNECTS.

BASEMENT BRACING

FOUNDATION WALLS

FIG. 5
TOWER MODEL
WITH FACADE
PANELS.

STEEL FASCIA PLATE

311
51   38   C12 x 20.7
NEOPRENE
FIN. FL.
57

CONT. NEOPRENE SEAL
MC 8 x 20.0
SPACER PLATE
VERTICAL STIFFENER.

℄ BEAM
COMPRESSIBLE FILLER
HARDENED WASHER
10 PLATE WASHER

19 X 63 ℞
6 ℞
SHIMS

305

FIG.6 HORIZONTAL JOINT

CONTOUR LEVELS
MIN	-6.8819
A	-6.8819
B	-2.0981
C	-2.6857
D	7.4695
E	12.253
F	17.037
G	21.821
H	26.605
I	31.388
J	36.172
MAX	36.172

LOCAL CORNER CONDITION

PLOT OF MAX PRIN STRESS

STIFFENER BOUNDARY

FIG. 7   σ₁ MAX. PRINCIPAL STRESS FOR 100 N
HORIZONTAL SHEAR ON 8 MM PANEL

FIG.8   PANELS-TO-COLUMN VERTICAL CONNECTION

FIG. 9   TYPICAL ELEVATION
WITH COLUMN COVERS REMOVED - DIAGONAL FACE SHOWN