STRUCTURAL DESIGN AND ANALYSIS OF CONCRETE SANDWICH PANELS AND THEIR PRACTICAL APPLICATIONS

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By

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ABSTRACT

The concept of a concrete sandwich panel originated in the United States in 1906 where a building was constructed with sandwich tilt-up wall panels. Since then sandwich panels have developed gradually and have continued to improve.

There are two types of concrete sandwich wall panels:

(1) Architectural wall panels (non-composite type), and

(2) Load bearing wall panels (composite and non-composite type).

In general, the non-composite type of load bearing panel is used more commonly because there are less limitations compared with the composite types. This thesis deals with the structural design and analysis of the non-composite type of load bearing wall panel.

In the past, certain problems besetted architects, engineers, and contractors working with concrete sandwich panels. These problems are associated with:

(1) Bonding between concrete and insulation

(2) Joints in panel

(3) Panel connections

(4) Control of cracking.

This study has been undertaken with the aim of solving some of these problems. Three experiments were conducted accordingly to determine the behavior of concrete sandwich panels. The first experiment was to investigate the effects

of heating and cooling on sandwich panels and to compare the experimental results with theoretical calculations. The principal results of this experiment demonstrated the effects on concrete sandwich panels under varying thermal conditions. These results should serve as a guideline for the design of joints in sandwich panels. The second and third experiments were initiated to investigate and compare the bending stresses, deflections and shear strength in concrete sandwich panels either with or without shear connectors. The panels were subjected to a simulated uniformly distributed load under simply supported conditions. It can be concluded from the experimental results that a well designed anchor system joining the concrete sandwich panel faces through the core insulation is a fundamental requirement in the future production of sandwich wall panels in order to obtain full shear transfer between the faces. Without these shear connectors, a concrete structural sandwich panel will usually fail by shear of the bond between the core and faces or by shear failure of the core itself.

The deflection on a sandwich panel is the sum of ordinary bending deflection and an additional deflection associated with shear deformation of the core. Experimental results showed that deflections were mainly associated with shear deformation of the core for panels without shear connectors.

Deflections measured were less for panels containing shear connectors. For both panels, from theoretical calculations, it was found that the deflections were mainly associated with shear deformation, with less than 1% due to bending.

A proper shear connector system transfers all lateral loads including wind forces from the exterior face to the interior structural face of the panel. Hence the sleeve anchor, torsion anchors and connector pins play a very important part in the performance of concrete sandwich panels. Prestressing techniques can also improve the design of such panels and have been widely used in recent developments of sandwich panels.

For this study, a typical non-composite type of concrete sandwich panel was selected as a design example. For the design of such concrete sandwich panels, the following factors were carefully considered:

(1) The thickness of both faces of the panel

(2) The loading conditions

(3) The fire resistance of the sandwich panel

(4) The temperature gradient between the faces, and

(5) The panel anchor system.

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CHAPTER I

INTRODUCTION

By definition, a concrete sandwich panel consists of two faces of high strength, high density concrete bonded to a core of relatively low density insulation material.

The primary objective of this thesis is to provide background information and assess current procedures in the structural design and analysis of concrete sandwich panels. In addition, the thesis is prepared to provide information which was requested by a local concrete supplier. To achieve this, an extensive literature search and experimental investigations was performed.

The presentation of the thesis is outlined as follows:

Chapter I	is an introduction.
Chapter II	describes the history of sandwich panel concepts, their gradual development and improvements up to today.
Chapter III	describes the types of concrete sandwich wall panels, their characteristics and functions.
Chapter IV	describes some of the problems encountered by the engineer, architect or contractor during fabrication, construction and performance stages.
Chapter V	describes the effects of thermal, shrinkage and creep on concrete sandwich panels. An experiment is carried out on heating and cooling of a sandwich panel.

Chapter VI outlines the theory of ordinary sandwich beam behavior that is applicable for the analysis of the concrete sandwich panels. An experiment is carried out on bending, deflection and shear to determine the behavior of the two panels, with and without shear connectors.

Chapter VII presents basic comments on an effective way of curing concrete sandwich panels.

Chapter VIII describes in detail the design considerations for concrete sandwich panels and a design example of a non-composite sandwich panel for experimental testing.

Chapter IX describes some of the practical applications of concrete sandwich panels and the cost of fabricating such panels.

Chapter X contains the concluding remarks based on the experimental results and research findings for concrete sandwich panels.

CHAPTER II

HISTORICAL DEVELOPMENT OF CONCRETE SANDWICH PANELS

The principle of the structural sandwich panel is not a new invention but simply a re-application of concepts used in plywood. The ancient Egyptians, around 1500 B.C., used the idea of slicing wood into thin sheets and re-constituting it with glue to give it superior strength.

Although concrete sandwich panel design is considered relatively new to the construction industry, there is evidence that the sandwich principle was used as early as 1849. Investigations into the history of sandwich panels have revealed that Mr. William Fairburn was the first person to use the principle in 1849 in experimenting with bridge design using laminated wood decking and concrete as a composite beam.

Besides concrete sandwich panels, there are many other types of sandwich panels utilizing different core and face materials. The three main types are:

(1) Solid core: using materials such as balsa wood, fibre board, foamed glass and expanded plastics.

(2) Honeycomb core: using three main materials such as metals, plastics and resin-impregnated paper. Usually this honeycomb effect takes the form of partitioning at right angles to the faces.

(3) Corrugated core: the materials used in this form are similar to the honeycomb cores. The corrugations create

the form of partitioning at an inclined angle to the panel faces, and run along the full length of the panel.

In 1906, the possibilities of constructing concrete sandwich panels were of great interest to engineers and contractors in the United States. In that year, a building was constructed with sandwich tilt-up panels. Some of the walls were hollow, made by casting a 50 mm layer of concrete, then a 50 mm layer of sand with a final 50 mm layer of concrete. The two outer layers of concrete were tied together with reinforcing ties. The sand between the two outer faces of concrete was washed out with water as the wall was lifted.

In 1930, the sandwich panel used by the building industry was the cement asbestos board, which had cement asbestos faces bonded to a fibre board.

In 1933, Swedish builders made a sandwich wall of 125 mm lightweight concrete block, which was cast and bonded to dense in-site concrete. A 15 mm thick lime cement mortar was then plastered on the outside of the lightweight concrete block.

In 1952, the designers in the United States developed a mineralized wood chip sandwich panel. Each cladding panel was composed of two outer faces of normal concrete 45 mm thick reinforced with wire mesh. Lightweight precast insulation was 40 mm thick using chemically mineralized wood chips as an aggregate. The two outer faces were connected through the insulation by shear ties in order to transfer horizontal shear from one face to the other. Since then concrete sandwich panels have been developed using different insulating materials. Ideally, such insulating materials should possess qualities such as: low density, relatively high compressive strength, high shear strength, good bonding characteristics, high insulation values, and low cost. Available materials that meet the above requirements are: cellular glass, expanded polystyrene, compressed wood fibres in cement, foam concrete, styrofoam and polyurethane.

In the late 1970's, there were many precast concrete sandwich panels fabricated throughout the United States and Canada. The most commonly used type of sandwich panel consists of an inner face (i.e. the load bearing panel), a layer of rigid thermal insulation and an architectural outer face.

In the United States, the Burke Company from San Meteo, California has developed concrete sandwich panels with a special anchor system. This Burke panel anchor system consists of a panel anchor, torsion anchor, and horizontal ties. This system connects the outer face and the load bearing face together to resist the vertical, horizontal and eccentric loads.

In Central Canada, Superior Concrete of Rexdale, Ontario (originating from Houston, Texas) has also developed similar concrete sandwich panels with a sleeve anchor system.

In Atlantic Canada, Strescon Ltd. of Saint John, New Brunswick, has developed a new type of concrete sandwich panel. This panel, which is composed of two prestressed concrete faces with a continuous layer of insulation in between them, is named Corewall.

In addition, Atlantic Concrete Ltd. of St. John's, Newfoundland is considering developing some new types of concrete sandwich panels. It is partly for these reasons that experiments are being conducted for this project.

Due to the more advanced techniques in designing concrete sandwich panels, production costs have been substantially reduced. Therefore, concrete sandwich panels are now more widely used than before.

CHAPTER III

TYPES OF CONCRETE SANDWICH WALL PANELS

A precast concrete sandwich panel consists of two faces of relatively thin, high density and high strength concrete bonded to a core of relatively rigid low density material. The function of the core material is to stabilize the relatively thin faces of high strength concrete, and to provide a high stiffness factor for the combination of materials by separating the faces. This combination produces a lighter, stronger wall and if the core material is a good insulator, it produces a better insulated wall.

From the usage and application points of view, there are two types of concrete sandwich panels, namely:

(1) architectural panels, and

(2) load bearing panels.

3.1 Architectural panels

Architectural sandwich wall panels are non-composite panels in which both faces participate in transferring the lateral loads to the horizontal structural steel or reinforced concrete framework. For the building framework with precast concrete sandwich panels, the precast units are intended for building enclosure only, and other design loads are totally supported by the structural frame to form a complex structural system. The outer face of the panel is connected to the other face by relatively flexible ties, allowing differential movements of the faces, with changing temperatures and humidity conditions. This type of sandwich panel is not commonly used because the structural properties of the concrete are not fully utilized, and therefore considered to be not economically viable.

3.2 Load bearing panels

Load bearing sandwich wall panels can be composite or non-composite.

(a) Composite panels are panels which have stiffer connections between the inner and outer faces so that the two faces will act jointly in resisting loads. This permits a net reduction in the overall wall thickness. Also, depending on the rigidity of the connector system, interaction between the faces may be total or partial. The composite panel will often be subjected to bowing during service. The large temperature changes between faces will tend to bow the panel inwards. The ideas of prestressing both faces of a composite panel are to help counteract thermal bowing, and to improve the behavior of the members by creating uniform compression in both faces.

(b). Non-composite panels are panels which have the inner face (structural face) transfer the lateral loads to the horizontal structural framework and support the weight of the outer face (non-structural face). Non-composite panels can be designed in a way that one face can move without affecting the other. The minimum thickness of the faces is dependent upon structural requirements, reinforcement cover, and handling considerations. Non-composite panels are more commonly used, due to those potential performance problems associated with composite panels, such as the bowing effects due to temperature variations between the concrete faces, and the distribution effects of the eccentric loads on the composite panels.

CHAPTER IV

PROBLEMS AND PERFORMANCE OF CONCRETE SANDWICH PANELS

The problems which architects, engineers and contractors generally encounter with sandwich panels during fabrication, construction and performance stages are described as follows:

4.1 Bonding between concrete and insulation

A concrete sandwich panel when loaded, develops shear stresses at both interfaces of the core and the faces. If the panel is not connected with a proper shear connector, there will be a bond shear failure between the core and the faces. From the shear test on the concrete sandwich panel specimens conducted by G. Singh and J. Clayton (1970), the study indicated that the shear failure in all the specimens was at the interface between the core and the first layer of concrete casted during fabrication. In the same experiment, in order to achieve bond strength between the core and both faces, some cells from both sides of the core surface were cut out to form microholes and porespaces. The objective of this was to permit fresh concrete flowing into the microholes and porespaces in order to produce reliable bonds of equal strength at both interfaces. To achieve this, the core was laid in a horizontal position to receive the fresh concrete on its surface to form the first face. This face was vibrated to consolidate the concrete and forced the finer particles into the microholes and porespaces at the surface of the core. After curing, the panel was inverted to receive the next concrete facing in the same manner. Although the above process of fabricating sandwich panels could develop reliable bonds between the core and the faces, it was not considered practical and economical, because it was time consuming and labour intensive. Hence, new anchor systems have been developed for joining the panel faces through several millimeters of insulation, which will minimize bond failure between the core and the concrete faces.

4.2 Joints in panel

Most concrete forms tend to produce larger elements when they are re-used. This is due to the concrete pressures created by the continuous vibrations of the form bed at each pour. For this reason, it is very difficult to produce the concrete panels within its small allowable deviations. Hence the dimensions, stability and the corner squareness of the formwork should be properly controlled, and re-checked in every concrete pour, so as to minimize the possible close-in problems during erection.

The weathertightness of a horizontal, vertical or corner joint is very important. If the joint is not designed or constructed properly, there will be leakage due to wind pressure, driving rain and water running down the wall. Caulking with a joint sealing compound and a low absorption backing rope is the most common practice of treating a joint between concrete panels. To cope with the moisture problems between sandwich layers, stainless steel anchor systems and shear ties have been developed, and proper construction techniques will ventilate the insulating layers.

To avoid excessive cracking, control joints should be provided in large panels to divide the outer face into units. The pattern for such control joints becomes an important architectural feature and aligning such joints with adjacent panels remains a major design consideration for successful performance.

4.3 Panel connections

The connections for the transmission of shear, axial load, moments and torsions from member to member or member to supporting structure are very important in sandwich panels. The connections in load bearing panels directly affect the structural integrity of the building structure and their design must be adequate for the functions intended.

Cracking of welded connections on precast concrete units is creating problems for the precast industry. These connections may consist of a welded joint between steel angles, channels or plates. Since there are restraints developed on the connections against thermal contractions and other deformations occuring throughout a structure, which have appreciable effects on the damage to connections, or to the panel itself. Bolted connections with oversized slots on angles and plates have been commonly used to allow flexibility on differential movements of connections.

4.4 Control of cracking

Control of cracking becomes as important as control of deflection in reinforced concrete structures as well as sandwich panels. Most cracks are a result of the following actions to which concrete can be subjected to:

(a) Volumetric change

(i) Drying shrinkage

(ii) Creep under sustained load

(iii) Thermal stresses including elevated temperatures.

The above (i), (ii), and (iii) factors will be described more thoroughly in Chapter V of this thesis.

(b) Direct stress due to applied loads or reactions, or internal stress due to continuity, long term deflection or environmental effects including differential movement in structural systems.

(c) Flexural stress due to bending.

While the net result of these three actions is the formation of cracks, the mechanisms of their development cannot be considered identical. American Concrete Institute Committee 224, specially formed to investigate the problems of concrete cracking, has the responsibility of correlating all the available information for the purpose of establishing accurate recommended practices on crack control in any concrete structures in a form satisfactory for use by the designers, engineers and contractors. Such comprehensive recommendations are being formulated by the Committee and should be used to eliminate the drawbacks in the performance of the precast concrete systems.

CHAPTER V

THERMAL, CREEP AND SHRINKAGE EFFECTS ON CONCRETE SANDWICH WALL PANELS

5.1 General

The evolution of concrete sandwich panels is gaining popularity in the building industry. The most troublesome problems encounterd with sandwich panels are summarized in Section 5.1.1 and 5.1.2.

A concrete sandwich wall panel may be subjected simultaneously to:

(a) drying (shrinkage) and cooling (contraction),

(b) drying (shrinkage) and heating (expansion),

(c) wetting (swelling) and cooling (contraction), or

(d) wetting (swelling) and heating (expansion).

The basic cause of a large majority of failures is due to inadequate provision for the effects of creep, shrinkage and temperature changes.

A combination of changes in temperature and humidity can create more severe conditions than the normal design requirements. Excessive stresses and deflections will also be caused by the combined effects of creep, shrinkage and temperature changes.

5.1.1 Maintaining a design factor of heat transmission U(SI) value

The U(SI) value of any wall or roof material is important in the design of buildings. It indicates how much heat in watts passes through a square metre of any given material in 1 hour for each degree difference in temperature (Centigrade), between the exterior and interior faces.

The effectiveness of a building assembly such as a wall or roof in resisting the flow of heat under static conditions is defined as its thermal resistance or R(SI) values. The heat loss U(SI) is inversely proportional to thermal resistance, it is therefore possible to indicate the relative heat loss potentials of these walls by the reciprocals of their R(SI) values.

Figure 5.1 gives the relative U(SI) values of the different types of solid normal, lightweight and foamed concrete panels together with sandwich panels for several panel thicknesses. Insulated sandwich panels are usually designed with a U(SI) ranging from 0.852 to 1.136 $W/m^2.hr.^oC$. The effect of ribs and solid sections in the panel is seldom considered. This effect, however, is great enough to increase the U(SI) values of the panel as much as 50 percent.

Leabu, V.F. (1959) indicated that ribs and solid concrete sections of a sandwich panel using a highly efficient insulation core reduce their insulation qualities considerably, not only at the ribs, but also for some distance beyond the solid concrete sections. To illustrate this effect, some typical precast concrete sandwich panels designed for a U(SI) value of 0.852 are shown in Figure 5.2. The calculation of this value is based on the insulated portion only. The actual value for the over-all panel (including ribs, etc.) show a factor from 1.48 to 1.91. This represents an increase of over 50 percent. The ratio of rib area to the insulated area of panel has a considerable effect on the U(SI) value. A higher percentage of rib area will increase the heat loss considerably. Where possible, ribs and solid concrete stiffeners should be kept to a minimum or eliminated if structural design conditions permit.

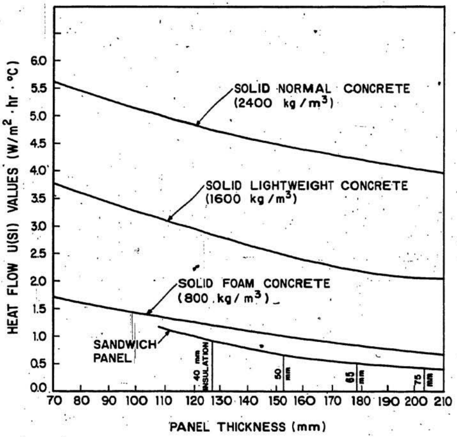


Fig. 5.1 - Thermal properties ——— heat flow U(SI) values for various thickness of solid panels made with normal, lightweight, foamed Concrete and sandwich panels. (Leabu, 1959)

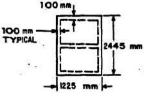
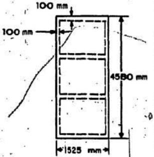
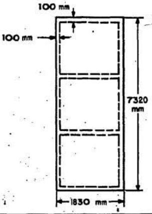
INSULATED SANDWICH PANELS	AREA (m^2)			U(SI) VALUE ($W/m^2.hr.°C$)		
	% RIBS	% INSULATION	TOTAL	THROUGH SOLID RIBS	THROUGH INSULATION	AVERAGE
	27 %	73 %	3.0 m^2	4.77	0.852	1.91
	21 %	79 %	7.0 m^2	4.77	0.852	1.67
	16 %	84 %	13.4 m^2	4.77	0.852	1.48

Fig. 5.2 - Typical sandwich panels designed for a heat flow U(SI) value of 0.852, actual factors as affected by ribs. (Leabu, 1959).

5.1.2 Bulging

Bulging is a very common phenomenon in many precast concrete panels and especially in the sandwich type of panels.

Bulging and warping of precast sandwich panels can result in unsightly cracks at the ceiling, floor and partitions, unless proper provision is made in the design of panel and connecting components. Bulging may be caused by temperature, moisture, curing and shrinkage differences between the interior and exterior faces. Their effects on sandwich panels are described in Section 5.2.

Tension/compression ties, known as the shear connectors, are commonly used to provide enough resistance to temperature and shrinkage stress. Simple tie rods or bent bars are also used as tension/compression ties to resist wind loads and individual layer separation.

A sleeve anchor may be used to connect the concrete faces. When it is installed together with the anchor rods, it will be able to carry all the applied shear load from any direction.

More information and details on shear connector are presented at Section 5.6.

5.2 Effects of creep, shrinkage and temperature changes on concrete sandwich panels

The design for thermal and shrinkage stresses is the most neglected part of today's design practice. An analytical study on a few concrete structures showed that severe stresses were set up by thermal changes. Based on such conditions, all rotations, movements and forces including those anticipated as a consequence of creep, shrinkage and temperature changes should be considered in the design of precast concrete sandwich panels.

5.2.1 Creep

Many materials continue to deform over considerable lengths of time at a constant stress or load. This property is known as creep.

When concrete is subjected to a sustained load, the deformation may be divided into two parts:

(1) An instantaneous deformation e_{inst} which occurs immediately, and

(2) a time-dependent deformation which begins immediately and continues for a longer term.

The latter long-term deformation is called creep.

Creep deformation for a given concrete is proportional to the magnitude of the applied stresses. As can be seen from Figure 5.3, the curve shows concrete was loaded to 4 MPa at age 28 days with resulting instantaneous strain e_{inst}. The load was then maintained for 230 days, during which time, creep deformation e_{creep} had been increased to almost 3 times its instantaneous value. If the load was maintained, the deformation of concrete would follow the solid curve. Creep would proceed at a decreasing rate and ceased at a final value after 2 to 5 years. This value would depend on the concrete strength and other environmental factors. If the load was removed after 230 days, the deformation of concrete would follow the unloading dashed curve.. When the concrete was reloaded at some later date, instantaneous and creep deformations would develop again as shown in the reloading dashed curve.

At any given stress, high-strength concrete shows less creep than low-strength concrete as shown in Table 5.1. If e'_{creep} is the final value of the creep strain, and e_{inst} is the initial, instantaneous strain; the creep coefficient $C_c = e'_{creep} / e_{inst}$, and the specific creep is e'_{creep} per MPa.

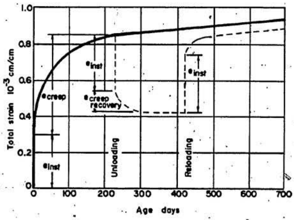


Figure 5.3 - Typical creep curve for concrete loaded to 4MPa at 28 days (G. Winter, 1972)

Table 5.1 Creep parameters (G. Winter, 1972)

Compressive strength MPa	Specific creep 10^{-6} per MPa	Creep coefficient C_c
20	145	3.1
27.5	116	2.9
40	80	2.4
55	58	2.0

Based on Table 5.1, a prediction of concrete creep can be made with the effects of applied stress. Assume the specimen is a concrete panel, with compressive strength at 27.5 MPa, is subjected to a longtime load which causes sustained stress of 8.5 MPa. Then after several years, the final value of the creep strain will be of the order of

$$8.5 \times 116 \times 10^{-6} = 0.00096 \text{ cm/cm.}$$

5.2.2 Shrinkage effects

Shrinkage of concrete while curing presents a problem not only during fabrication but also after erection. Warping of panels due to differential shrinkage, especially in a sandwich type of panel, is difficult to control. The rate of curing and evaporation of moisture of the two faces are not uniform and usually result in warped surfaces. Accelerated curing, removal from forms as soon as practical, and storing panels, with equal exposure on both faces until erected, prevents excessive warping. Concrete with a low slump, the use of curing compounds, water saturated covering and steam curing processes are considered to be important for controlling shrinkage.

Since curing and measurable shrinkage of concrete can continue for as long as 3 years, movement of concrete panels can be expected long after erection due to non-uniform exposure of the two faces.

ACI Publication SP-27 (1971) describes that shrinkage of concrete affects the internal stresses in sandwich panels. In particular, the relative shrinkage of the sandwich panel tends to be more noticeable on the exterior side where it is exposed to climatic conditions.

The shrinkage coefficient is defined as the shortening per unit length, and varies from about 0.0002 to 0.0006, depending on ambient conditions and other causes. Shrinkage is increased in an environment of low humidity and wind exposure. A value close to 0.0002 is commonly used in shrinkage computations.

5.2.3 Temperature effects

Differential temperatures appear to have the greatest effects on precast concrete wall panels, and especially on the sandwich type. Figure 5.4 shows the temperature gradient through four different types of precast concrete panels for

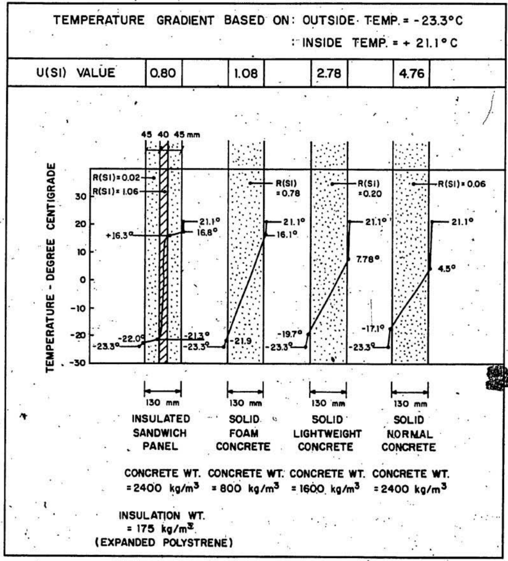


Fig. 5.4 - Temperature gradient through four types of precast wall panel. (Leabu, 1959)

a temperature range of -23.3° C outside to 21.1° C inside. Note that the sandwich panel with its highly efficient insulation, has the greatest temperature differential, and the typical solid concrete panel the least. In the sandwich panel, a temperature variation of approximately 38° C is possible between the exterior and interior layers of concrete.

By the laws of thermodynamics, the total resistance to the heat flow through the wall shown in Figure 5.5 is equal numerically to the sum of the resistance in series. The resistances are the outer surface film of the wall, the material in the wall itself, the insulation, and the surface on the other side of the wall. The drop in thermal gradient can be represented by the wavy line in Figure 5.5. At the first resistance, the air surface film, there is a drop in the gradient necessary to overcome the resistance R_i of the film. The same is true for the material of the wall and the resistance R_o of the air surface film on the other side. The sum of these resistances is the total resistance. The heat transmission U(SI) is the reciprocal of the sum of the resistances. The temperature at various points within the assembly can be calculated by apportioning the overall inside to outside temperature drop, (which is determined by the interior and exterior conditions) in proportional to the thermal resistance of the various components of the assembly.

$$\text{Heat Flow } U(SI) = 1/R_T(SI) \qquad \text{Equation 5.1}$$

$$1/R_T(SI) = 1/(R_i + R_2 + R_3 + R_4 + R_o) \qquad \text{Equation 5.2}$$

Where $U(SI)$ is the heat transmission in (watts/m^2.hr.^oC)

$R_T(SI)$ is the total thermal resistance (m^2.hr.^oC/watt)

X is the thickness of layer (mm)

R_i is the thermal resistance of the air surface film on the interior face of the wall (m^2.hr.^oC/watt)

R_2 is the thermal resistance of the interior concrete layer (m^2.hr.^oC/watt)

R_3 is the thermal resistance of the insulation (m^2.hr.^oC/watt)

R_4 is the thermal resistance of the exterior concrete layer (m^2.hr.^oC/watt)

R_o is the thermal resistance of the air surface film on the exterior face of the wall (m^2.hr.^oC/watt)

For example, we calculate the U(SI) value of the sandwich panel shown in Figure 5.4.

Assume the sandwich panel is made of normal concrete and polystyrene insulation where:

$X_2 = 45$ mm ; $X_3 = 40$ mm ; $X_4 = 45$ mm

$R_i = 0.12$; $R_o = 0.03$

$R_2 = R_4 = 0.02$; $R_3 = 1.06$

From Equations 5.1 and 5.2:

$$U(SI) = 1/R_T(SI) = 1/(R_i + R_2 + R_3 + R_4 + R_o)$$

$$= \frac{1}{(0.12 + 0.02 + 1.06 + 0.02 + 0.03)}$$

$$= \frac{1}{1.25} \text{ watts/hr.m}^2 . {}^\circ C$$

$$= \underline{\underline{0.80 \text{ watts/hr.m}^2 . {}^\circ C}}$$

The drop in gradient is dependent on the resistance on each layer and the temperature range between the exterior and interior of the concrete layer.

With regards to the temperature gradient of a typical sandwich panel shown in Figure 5.5:

$$T_n = T_{n-1} - (R_n/R_T)(T_i - T_o) \qquad \text{Equation 5.3}$$

Where T_n is the temperature on the surface of the n^{th} component of the wall for n = 1,2,3 and 4 (^{o}C)

T_{n-1} is the temperature on the surface of the $(n-1)^{th}$ component of the wall (^{o}C)

T_i is the interior temperature inside the wall (^{o}C)

T_o is the exterior temperature outside the wall (^{o}C)

For the sandwich panel shown in Figure 5.4 and from Equation 5.3:

$$\begin{aligned}
T_1 &= T_i - (R_i/R_T)(T_i - T_o) \\
&= 21.1 - (o.12/1.25)(21.1 - (-23.3)) \\
&= +16.8^{o}\ C \\
T_2 &= T_1 - (R_2/R_T)(T_i - T_o) \\
&= 17 - (0.02/1.25)(21.1 - (-23.3)) \\
&= +16.3^{o}\ C \\
T_3 &= -21.3^{o}\ C \\
T_4 &= -22.0^{o}\ C \\
T_o &= -23.3^{o}\ C
\end{aligned}$$

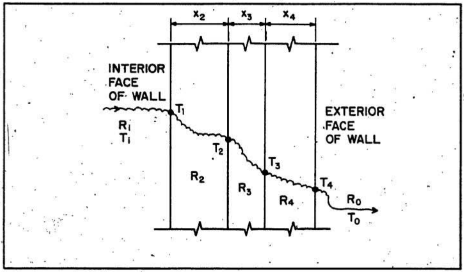


Fig. 5.5- Heat flow U(SI) through typical wall section. (T. Collins, 1954)

In order to design a sandwich wall panel from the insulation standpoint, a U(SI) value can be specified. Since the resistances of each material of the panel and the thickness of the concrete faces are known, the insulation thickness could be increased or decreased until the specified U(SI) value has been achieved.

The horizontal displacement due to temperature differential
1 for normal (2400 kg/m³) and lightweight (1600 kg/m³) concrete under 2 support conditions as shown in Figure 5.6 can be computed as follows:

Condition I - The panel acts as a beam on two pinned end connections. This panel has moment induced at midspan as a result of change in panel length due to temperature differential.

Condition II - The panel acts as a beam on two fixed end supports. This panel has equal moment at each end as a result of change in panel length due to temperature differential.

Assumptions: No change in length at neutral axis
Exterior face elongated = +e
Interior face shortened = -e

<u>For Support Condition I</u>

$$e = C \times \frac{\delta T}{2} \times L$$

$$e = \frac{PL}{E} \times \frac{1}{A/2}$$

$$M = Pd$$

<u>For Support Condition II</u>

$$e = C \times \frac{\delta T}{2} \times \ell \quad \left(\ell = \frac{L}{2}\right)$$

$$e = \frac{P\ell}{E} \times \frac{1}{A/2}$$

$$M = Pd$$

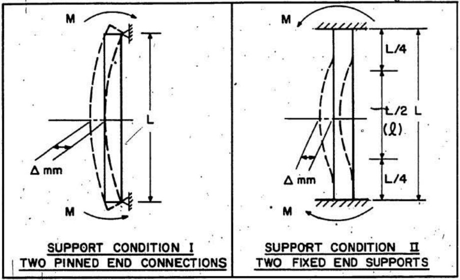


Fig. 5.6 - Horizontal displacement due to temperature differential. (Leabu, 1959)

<u>For Support Condition I</u>

$$P = \tfrac{1}{4}C \times \delta T \times AE$$

$$M = \tfrac{1}{4}Cd \times \delta T \times AE$$

$$I = \frac{Ad^2}{4}$$

$$\Delta = \frac{ML^2}{8EI}$$

$$= \frac{Cd \times \delta T \times AL^2}{32I}$$

$$= \frac{C \times \delta T \times L^2}{8d}$$

Equation 5.4

<u>For Support Condition II</u>

$$P = \tfrac{1}{4}C \times \delta T \times AE$$

$$M = \tfrac{1}{4}Cd \times \delta T \times AE$$

$$I = \frac{Ad^2}{4}$$

$$\Delta = \frac{M\ell^2}{8EI} = \frac{ML^2}{32EI} \quad \left(\ell = \frac{L}{2}\right)$$

$$= \frac{Cd \times \delta T \times AL^2}{128I}$$

$$= \frac{C \times \delta T \times L^2}{32d}$$

Equation 5.5

Where δT = Difference of temperature between outside and inside, degrees Centigrade.

C = Coefficient of expansion, mm/mm.°C normal concrete, $C = 0.000012$ per degrees Centigrade

L = Length of panel (or length between supports depending on the restraint of connections).

d = Effective depth of panel, distance between center of gravity of exterior and interior faces of sandwich panel, mm.

P = Axial load due to temperature differential

e = Elongation or shortening, along the longitudinal axis, mm.

Δ = Horizontal displacement, mm.

A = Total cross-sectional area of inner and outer concrete faces per unit length, mm^2.

I = Moment of Inertia, mm^4.

E = Modulus of elasticity of concrete, MPa.

ℓ = Effective length of panel between fixed end supports.

From the above equations, the lateral deflection for a specified sandwich panel span can be determined for a range of temperature differential.

The lateral deflection versus span length for a typical 130 mm sandwich panel for a 27° C. temperature differential is shown in Figure 5.7. These curves show the effect of panel support and type of concrete on bulging. From the curves, it is evident that the use of lightweight concrete and continuity of panels at points of support reduced the curvature of panels due to temperature differential. In actual practice, panel support is seldom fully fixed or simply supported, but more likely between these two extremes.

A recognized problem with precast concrete sandwich panels is the tendency of longer panels to bow outwards under prolonged exposure to the hot weather conditions. The tendency of panels to bow is mainly influenced by the panel size, the rigidity of the connection between the concrete faces, and the daily temperature variations on the exterior face of the panel. Through good design and detailing practice, the effect of thermal bowing can be controlled. For example, computation of lateral deflection due to temperature gradient can be utilized to control bowing of concrete sandwich panels.

From Figure 5.7, for an allowable lateral deflection of 3.2 mm due to a temperature differential of 27°C for a 130 mm precast sandwich panel, the following dimensions are suggested as approximate maximums for the panel span lengths due to the effects of panel support and type of concrete used.

(a) Normal concrete and Support Condition I - 2200 mm.

(b) Normal concrete and Support Condition II - 4600 mm.

(c) Lightweight concrete and Support Conditon I - 3450 mm.

(d) Lightweight concrete and Support Condition II - 6500 mm.

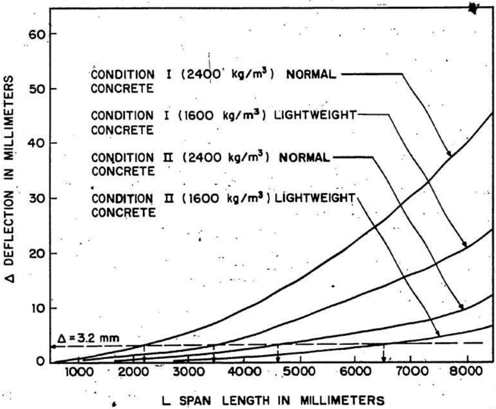


Fig. 5.7 - Lateral deflection versus span length for a typical 130 mm sandwich panel.

Support condition I - simply supported.
II - fixed support.
(Leabu, 1959)

5.3 Experiment on heating and cooling of sandwich panel

5.3.1 Testing material

The first sandwich panel made for the experiment was supplied by Atlantic Concrete Ltd. located in St. John's, Newfoundland. The panel consisted of two layers of 75 mm concrete, sandwiched by a solid core of styrofoam insulation of thickness 50 mm, with a total thickness of 200 mm.

The size of the panel was 1220 mm x 1070 mm x 200 mm. To make the panel, a 75 mm layer of concrete was first cast into the form bed, 2 sheets of 25 mm insulation were then placed on top of the concrete. Two pairs of 150 mm long 10M (#3) rebars were pushed diagonally through the insulation and embedded into the first layer of fresh concrete. A top wire mesh reinforcement was then installed, followed by a final layer of concrete placement. As soon as the concrete gained its initial strength, the sandwich panel was removed from forms and stored in such a way with equal exposure on both faces. After the curing period, the panel was transported to the Structures Laboratory of Memorial University of Newfoundland for experimental analysis.

5.3.2 Testing of heating effects on sandwich panel

The general arrangement of the test apparatus is shown in Figure 5.8. The sandwich panel was rested on a steel frame with its bottom surface exposed to room temperature. The top surface of the panel was covered by an insulated box and exposed to an enclosed temperature ranging from 20^{o} C to 33.3^{o} C. Heating elements were installed on top of the panel (Plate 5.1), and being hooked up to an automatic temperature control for providing a maximum cut-out temperature.

The purpose of this experiment is to test the effects of temperature difference for a normal sandwich wall panel. Dial gages were installed on the top, the bottom, and the sides of the sandwich panel as shown in Figures 5.8 and 5.9. These gages were for measuring the expansion of the panel faces at different temperatures. A series of deflection readings were taken from the exposed surface of the concrete during heating of the panel. A temperature indicator was used for checking the temperature on the surface of the top layer of concrete at different times during the operation of the experiment (Plate 5.2), and the heat was supplied by a heating transformer. Experimental results were recorded and to be analyzed in Section 5.4 of the report.

5.3.3 Testing of cooling effects on sandwich panel

The general arrangement of the test apparatus is the same as the heating part except replaced by a cooling source. A set of copper pipings were installed on top of the panel (Plate 5.3), and hooked up to an air compressor unit as shown in Plate 5.4. The air compressor unit could cool the enclosed surroundings up to a maximum of -13^{o} C with an automatic pressure control for setting the cut-in and cut-out temperatures.

Dial gages were installed in the same way as shown in Figures 5.10 and 5.11 for measuring the contraction of the panel. Intake pipes from the air compressor unit to the panel enclosure were insulated to maintain cold air temperature. All joints around the panel enclosure were caulked and remained air tight during the experiment.

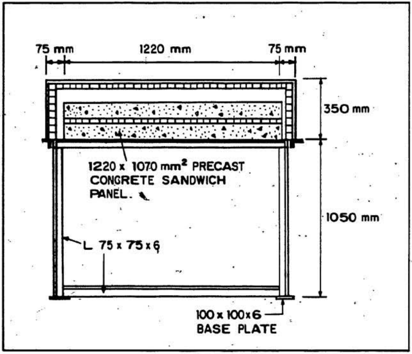


Fig. 5.8 – Elevation showing experimental set - up.
(Not to scale)

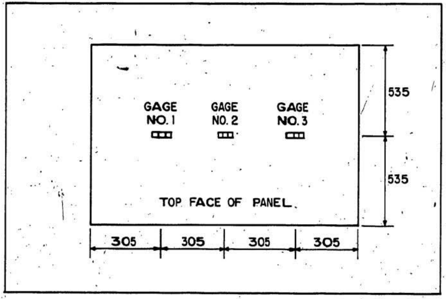


FIG. 5.9 - Plan for layout of dial gages. (N.T.S.)

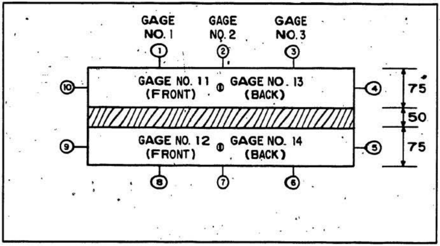


FIG. 5.10 - Elevation showing dial gages location. (N.T.S.)

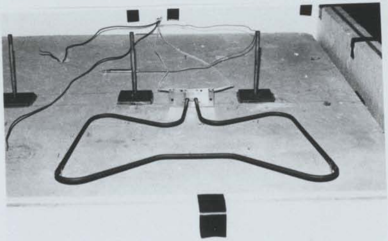


PLATE 5.1 - Layout of heating coil on top of panel

PLATE 5.2 - Temperature indicator for checking temperature on the surface of top layer of concrete

5.4 Experimental results analysis

For the experiment of heating and cooling effects on a sandwich panel, the deflections on exposed surfaces of the concrete panel were measured by dial gages. The temperatures and deflections were recorded on an hourly interval until the temperature reached the maximum output and stabilized for a period of 72 hours. Gages 1, 2 and 3 were used to measure the central deflection profile on the top surface of the panel. Gages 6, 7 and 8 were used to measure the central deflection profile on the bottom surface of the panel. Gages 4, 10, 11 and 13 were used to measure the deflections on the four sides of the top layer of concrete. Gages 5, 9, 12 and 14 were used to measure the deflections on the four sides of the bottom layer of concrete. (Figures 5.8 and 5.9) The top layer of the sandwich panel was assumed to be the outside surface of the structure and was exposed to the warm and cold temperatures changes.

For the heating part of the experiment, the temperature was heated from a room temperature of 20° C to 33.3° C which was the highest temperature the experiment could undergo in order to avoid fire hazards. The temperature differential for the heating part was 13.3° C.

For the cooling part of the experiment, the temperature was cooled from 20° C to -13° C which was the coldest temperature the refrigeration unit could undergo. The temperature differential for the cooling part was 33° C which was about 2.5 times more than the heating part.

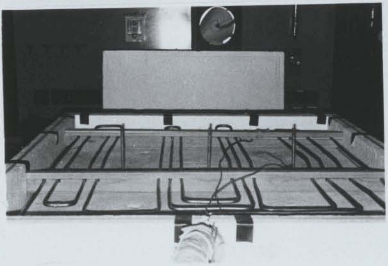


PLATE 5.3 - Layout of cooling coils for providing cold temperature up to a maximum of 10° F (-13° C)

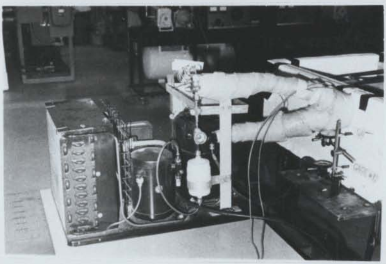


PLATE 5.4 - Air compressor cooling assembly for providing cold temperature to a maximum of 10° F (-13° C)

Tables 5.2 and 5.3 show the maximum deflections measured at different locations of the panel after heating or cooling. Figures 5.11 and 5.12 show the deformed shape of the panel after the surfaces were subjected to warm or cold temperature changes.

For the panel tested, it was found that the shape of the panel warped in a convex profile during high temperature periods and in a concave profile during low temperatures. No observations could be made at the interface of the core and the concrete faces due to the enclosed experimental condition.

From Table 5.2, take Gage 2 for instance, the maximum deflection due to expansion is 0.2 mm, at a temperature differential of 13.3° C. From Table 5.3, take the same Gage 2, in this case the maximum deflection due to contraction is 0.52 mm, at a temperature differential of 33° C. Hence, the amount of contraction is approximately 2.5 times more than the expansion. Since the cooling temperature differential is approximately 2.5 times more than the heating temperature differential, it can be concluded that the rate of expansion of the panel is approximately the same as the rate of contraction.

From Figure 5.11, the results show that the panel expands more at the sides of the top layer than the bottom layer during the heating process. From Figure 5.12, the results show that the panel contracts more at the sides of the top layer than the bottom layer during the cooling process.

TABLE 5.2 Maximum deflections due to expansion from temperature change of 20° C to 33.3° C (13.3° C temperature differential)

Gage #	Maximum deflections (mm)	Location
1	+ 0.190	Top of slab
2	+ 0.200	Top of slab
3	+ 0.225	Top of slab
4	+ 0.055	Side, top layer
5	+ 0.030	Side, bottom layer
6	- 0.160	Underneath slab
7	- 0.210	Underneath slab
8	- 0.145	Underneath slab
9	+ 0.028	Side, bottom layer
10	+ 0.040	Side, top layer
11	+ 0.045	Side, top layer
12	+ 0.030	Side, bottom layer
13	+ 0.060	Side, top layer
14	+ 0.040	Side, bottom layer

TABLE 5.3 Maximum deflections due to contraction from temperature change of 20° C to -13° C (33° C temperature differential)

Gage #	Maximum deflection (mm)	Location
1	- 0.448	Top of slab
2	- 0.521	Top of slab
3	- 0.430	Top of slab
4	- 0.210	Side, top layer
5	- 0.080	Side, bottom layer
6	+ 0.275	Underneath slab
7	+ 0.380	Underneath slab
8	+ 0.240	Underneath slab
9	- 0.080	Side, bottom layer
10	- 0.225	Side, top layer
11	- 0.275	Side, top layer
12	- 0.080	Side, bottom layer
13	- 0.285	Side, top layer
14	- 0.085	Side, bottom layer

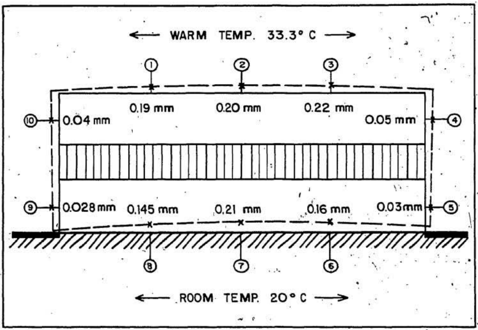


Fig. 5.11 - Deformed shape of sandwich panel after warm temperature differential of 13.3 °C. (Not to scale)

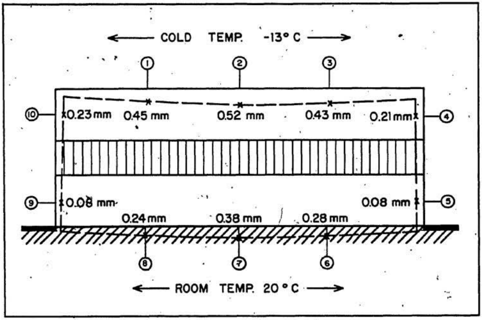


Fig. 5.12- Deformed shape of sandwich panel after cold temperature differential of 33 ° C.
(Not to scale)

5.5 Comparison between experimental and theoretical results

Leabu (1959) stated that if the boundary condition of the panel is to be simply supported, the derived horizontal displacement due to temperature differential will be

$$\Delta = \frac{C \times \delta T \times L^2}{8d} \quad \text{(From Equation 5.4)}$$

From the experiment, under a similar support condition,

Length of panel L = 1220 mm

Effective depth of panel d = 130 mm

For the expansion of the panel, the temperature differential $\delta T = 13.3^\circ$ C

The theoretical center deflection due to expansion is:

$$\Delta_1 = \frac{C \times \delta T \times L^2}{8d}$$

$$= 0.25 \text{ mm}$$

For the contraction of the panel, the temperature differential $\delta T = 33^\circ$ C

The theoretical center deflection due to contraction is:

$$\Delta = 0.66 \text{ mm}$$

The following Table 5.4 shows the comparison between experimental and theoretical results.

TABLE 5.4 Comparison between theoretical and experimental deflection at the center of the top concrete layer

Temperature differential	Theoretical	Experimental
Due to expansion : 13.3^{o} C	0.25 mm	0.20 mm
Due to contraction : 33^{o} C	0.66 mm	0.52 mm

The experimental results of the deflection at the center of the panel, indicated to be correlated with the theoretical computations. The minor discrepancy is possibly due to the non-uniform concrete surface.

5.6 Thermal expansion on connecting pins and heat loss at corner panels : methods of improvement

Joints between wall, roof and corner panels should be detailed so that the passage of flames or hot gases is prevented, and transmission of heat does not exceed the limits specified. These requirements present a challenge to the architects and engineers, particularly for joints that are designed to be weathertight, while permitting thermal expansion and contraction and other movements.

For concrete sandwich panel construction, the exterior face will tend to move in response to the temperature change and concrete shrinkage. To actomodate this movement without causing any damage to the panel, the shear connectors should be sufficiently flexible so that undue restraint will not develop. A metal sleeve anchor, acting as a shear connector, may be used to connect the two concrete faces together. The metal sleeve is perforated rigid sleeve, when it is installed together with anchor rods, carries all applied shear loads from any directions. Since movements due to temperature differentials radiate away from the panel center, the sleeve will not cause any undue restraint to the concrete faces and allows certain free movements. To avoid torsional effects, the sleeve anchor should be installed in a position that coincides with the center of gravity of the sandwich panel. But it may be moved up or down the vertical axis, if panel rotation will be resisted by additional connecting pins.

In order to resist the wind pressure, peeling of the concrete layer, and other torsional effects, additional connecting pins should be provided at the perimeter of the sandwich panel. These connecting pins should not contribute any substantial resistance to the thermal movements of the individual concrete layers. If these pins are not flexible, cracking of the concrete sandwich panel could result. The arrangement of the connecting pins should be installed in the direction of the panel movement.

Heat transmission for sandwich panels has been described in detail in Section 5.2.3. Several building codes require that where non-combustible construction is specified, combustible elements in walls shall be limited to thermal and sound insulation having a flame spread classification of not more than 75 when the insulation is sandwiched between two layers of non-combustible material such as concrete. This flame spread classification means an index indicating the extent of spread-of-flame on the surface of a material or an assembly of materials. The unit 75 is a rating as determined in a standard fire test as prescribed in the National Building Code of Canada. Data on flame spread classification is available from insulation manufacturers. The conventional sandwich wall panels with continuous insulation, proper joint width and suitable caulking materials, can provide the necessary resistance to heat transmission. Joints at the corner of

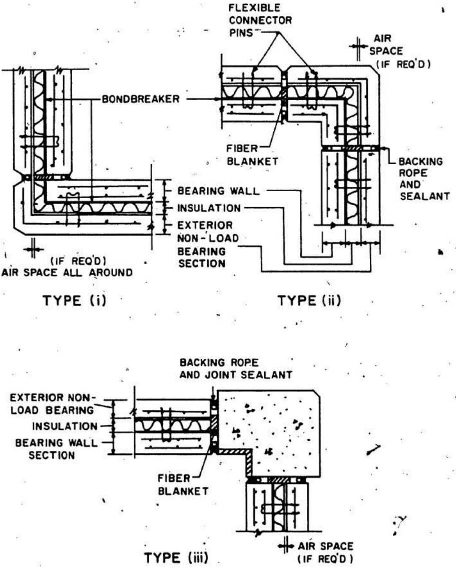


FIG. 5.13 Sandwich panel corner details

sandwich panels are the major problem areas that need to be treated properly. Hence, good corner details are essential to meet the heat loss prevention requirements. Panels with returns are not easy to weather seal as the bowing effects will occur in different planes. Three types of corner details are suggested and as shown in Figure 5.13. The (i) and (ii) type show a method of constructing the corner of a sandwich panel with a continuous insulation for preventing heat losses. Appropriate flexible inserts must be used to ensure that movement is not obstructed at these corner joints. A small air space is shown between the insulation and the exterior concrete face. This is an architectural detail and used only if required by the architect. Conventional jointing materials are used between the panel edges. The (iii) type is a corner piece with concrete only, and insulation to be installed inside after the erection of the panel. This special corner unit is not necessary to be flushed with the adjacent panels, and can be effectively used to minimize bowing in the two different planes.

5.7 Discussion

The following are some of the remaining factors to be considered by the engineer in controlling the effects of thermal, shrinkage and creep on concrete sandwich panels.

(a) The size of panels should be fabricated in short lengths within the allowable limits because of the thermal bowing and warping effects.
(b) The drying shrinkage should be determined from the particular type of concrete used.
(c) The degree of exposure to the weather, and particularly to the range of temperature change, and to the relative humidity likely to be encountered.
(d) The quality control to be accomplished over the mixing, placing, and curing procedures.

It is also necessary for the engineer to consider the various differential movements, rotations and forces including those anticipated as a consequence of creep, shrinkage and temperature changes in the design of the sandwich panels. All temperature stresses should be taken up internally. That is, tension and compression, stresses induced by restrained expansion, and/or contraction are added to the stresses due to dead, live and wind loadings. It is also very important to develop acceptable structural, as well as architectural details, for the satisfactory performance of the sandwich panels.

CHAPTER VI

STRUCTURAL ANALYSIS OF CONCRETE SANDWICH PANELS FOR DIFFERENT CORE MATERIALS AND FACE THICKNESS

6.1 Theory

For the concrete sandwich panel, the analysis may be simplified by assuming the sandwich beam behavior. The theory serves to illustrate how the core and faces of the panel act as a single unit. The panel consists of two faces each of thickness t separated by a core thickness c. The overall depth of the panel is h and the unit width is b as shown in Figure 6.1. The stresses and deflections are calculated to the first approximation using ordinary theory of bending. In order to do so, two assumptions are made. Firstly, it is assumed that cross-sections which are plane and perpendicular to the longitudinal axis of the unloaded beam remain so after bending. Secondly, it is assumed that the materials making the core and faces of the beam are isotropic. For the analysis of sandwich beam, H. G. Allen (1969) developed the following equations:

The relationship between bending moment (M) and curvature (1/R) is: $\frac{M}{EI} = -\frac{1}{R}$ Equation 6.1

EI is the flexural rigidity which is the product of modulus of elasticity E and the second moment of area I. The negative sign in the equation complies with the sign convention as shown in Figure 6.2.

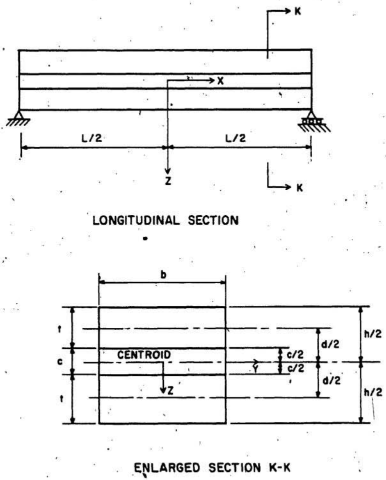
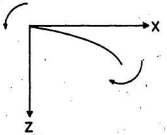
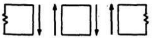


FIG. 6.1 Dimensions of the sandwich beam of equal face thickness :



POSITIVE DEFLECTION SLOPE AND CURVATURE, NEGATIVE BENDING MOMENT.



POSITIVE SHEAR FORCE, SHEAR STRESS, AND SHEAR STRAIN.

FIG. 6.2 Sign conventions.

Let D denote the flexural rigidity. The sandwich beam is a composite beam and hence its flexural rigidity is the sum of the flexural rigidities of the two separate panels, faces and core measured about the centroidal axis C-C of the cross-section.

Therefore, $D = E_f\frac{bt^3}{6} + E_f\frac{btd^2}{2} + E_c\frac{bc^3}{12}$ Equation 6.2

E_f and E_c are the moduli of elasticity of the faces and core. The first term of the right hand side of Equation 6.2 represents the local stiffness of the faces about their own centroidal axis. The second and third term represents the stiffness of the faces and core respectively bending about the centroidal axis. In practice, the first term may be neglected if

$$\frac{d}{t} > 5.77$$ Equation 6.3

If this condition is fulfilled, the local bending stiffness of each face about its own separate centroidal axis makes a negligible contribution to the flexural rigidity of the sandwich.

The second term is always the dominant one and the third term may be neglected if

$$6\,\frac{E_f}{E_c}\,\frac{t}{c}\left(\frac{d}{c}\right)^2 > 100$$ Equation 6.4

If this condition is fulfilled, the bending stiffness of the core is negligible.

For this particular project, where the faces are thick and rigid and the core is too weak to provide a significant contribution to the flexural rigidity of the sandwich, the shear stress is therefore assumed to be constant over the depth of the core.

As a result, the condition for Equation 6.4 is satisfied and the third term is neglected leaving

$$D = E_f\frac{bt^3}{6} + E_f\frac{btd^2}{2} \qquad \text{Equation 6.5}$$

This equation only applies to a sandwich beam with faces of equal thickness. For sandwich beam with faces of unequal thickness as shown in Figure 6.3, the flexural rigidity D is expressed as follows:

$$D = E_f\frac{b}{12}(t_1{}^3 + t_2{}^3) + \frac{E_f bd^2 t_1 t_2}{t_1 + t_2} \qquad \text{Equation 6.6}$$

6.1.1 Bending and shear stresses

The bending and shear stresses for the faces and core can be determined by simple bending theory adapted to composite beams. The strain at a distance z below the centroidal axis C-C is:

$$\varepsilon = \frac{M\mathfrak{z}}{D} \qquad \text{Equation 6.7}$$

The stress at point $\mathfrak{z}$ varies according to the type of beam analyzed and its applied load condition.

(a) For sandwich beam with equal face thickness,

$$I = \frac{bt^3}{6} + \frac{btd^2}{2} \quad \text{and} \quad I_f = \frac{bt^3}{6} \qquad \text{Equation 6.8}$$

Where I is the second moment of area of the faces about the centroid of the sandwich.

I_f is the sum of the second moments of area of the faces about their own centroid.

If the beam is simply supported and subjected to an uniformly distributed load q,

The maximum bending stress is:

$$\sigma_{MAX} = \frac{qL^2}{8}\left(\frac{c+2t}{2I} + \frac{t}{2I_f}\right) \qquad \text{Equation 6.9}$$

The maximum shear stress is:

$$\tau_{MAX} = -\frac{qL}{2bd}\left(1 - \frac{I_f}{I}\right) \qquad \text{Equation 6.10}$$

(b) For sandwich beam with unequal face thickness

$$I = \frac{bd^2 t_1 t_2}{t_1 + t_2} + \frac{b}{12}\left(t_1^3 + t_2^3\right)$$

and $I_f = \frac{b}{12}\left(t_1^3 + t_2^3\right)$ Equation 6.11

The direct stress σ in the faces varies according to the level at which it is measured.

For example, the four critical levels marked a, b, i, j, in Figure 6.3, the bending stresses are given as follows:

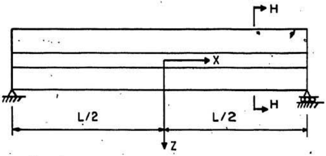
$$\sigma_a = -\frac{qL^2}{8}\left(-\frac{1}{I}\left(\frac{dt_2}{t_1+t_2} + \frac{t_1}{2}\right) - \frac{1}{I_f}\frac{t_1}{2}\right) \quad \text{Equation 6.12}$$

$$\sigma_b = -\frac{qL^2}{8}\left(-\frac{1}{I}\left(\frac{dt_2}{t_1+t_2} - \frac{t_1}{2}\right) + \frac{1}{I_f}\frac{t_1}{2}\right) \quad \text{Equation 6.13}$$

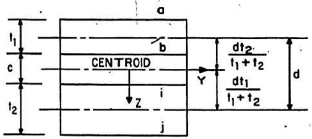
$$\sigma_i = -\frac{qL^2}{8}\left(+\frac{1}{I}\left(\frac{dt_1}{t_1+t_2} - \frac{t_2}{2}\right) - \frac{1}{I_f}\frac{t_2}{2}\right) \quad \text{Equation 6.14}$$

$$\sigma_j = -\frac{qL^2}{8}\left(+\frac{1}{I}\left(\frac{dt_1}{t_1+t_2} + \frac{t_2}{2}\right) + \frac{1}{I_f}\frac{t_2}{2}\right) \quad \text{Equation 6.15}$$

The maximum shear stress τ_{MAX} for unequal face thickness is determined from Equation 6.10 with d as a variable.



LONGITUDINAL SECTION



ENLARGED SECTION H-H

FIG. 6.3 Dimensions of the sandwich beam of unequal face thickness.

6.1.2 Deflection of a sandwich beam

The maximum deflection of a sandwich beam under an uniformly distributed load is the sum of an ordinary bending deflection Δ_1 and an additional deflection Δ_2 associated with the shear deformation of the core for both equal and unequal face thickness.

$$\Delta_{MAX} = \Delta_1 + \Delta_2$$

$$= \frac{5qL^4}{384D} + \frac{qL^2}{8AG} \qquad \text{Equation 6.16}$$

where $A = \frac{bd^2}{c}$ (d is the distance between the centerlines of the upper and lower faces)

G is the core shear modulus.

6.2 Testing materials and apparatus

There were two types of sandwich panels made for bending and deflection tests, using a 30 MPa normal concrete as panel faces and two different types of insulation as core material.

6.2.1 Small panel without shear connector

The small panel consisted of two equal layers of concrete, each of 75 mm thickness, acting as the interior and exterior faces, and 2 sheets of 25 mm thick styrofoam insulation acting as the core. This panel was used previously for conducting the heating and cooling experiment.

6.2.2 Large panel with shear connector

The large panel consisted of a 62.5 mm concrete layer as the exterior face, a 50 mm thick rigid polyurethane sheet as the core, and a 100 mm concrete layer as the interior structural face. This panel was also supplied by Atlantic Concrete Ltd. of St. John's, Newfoundland.

The overall dimensions of the large panel was 2400 mm x 1200 mm. The panel was designed with a sleeve anchor, which transferred the weight of the exterior face through the insulation into the interior load bearing face. A series of type L connector pins were installed to provide additional flexible connections for the two layers as shown in Figure 6.4. Refer to Appendix I for a full description of connector pins and sleeve anchor supplied by Superior Concrete Accessories (Canada) Ltd.

Before concrete casting, a bottom layer of wire mesh and the lifting inserts were placed, then the sleeve anchor was installed by means of anchor rods tied to the bottom wire mesh as shown in Figure 6.5. The bottom layer of concrete was poured and vibrated, followed by the placement of a 50 mm thick polyurethane insulation, with a sheet of plastic laid on top of the insulation. A top layer of wire mesh reinforcement was then placed, and tied to the sleeve anchor's top layer of anchor rods. Then all the type S and type L connector pins were installed by pushing across the reinforcement and through the insulation, and reaching into the still fresh concrete layer as shown in Figure 6.5. Finally, the last layer of concrete was placed and vibrated. Conventional heat and moisture curing were provided for the panel fabricated.

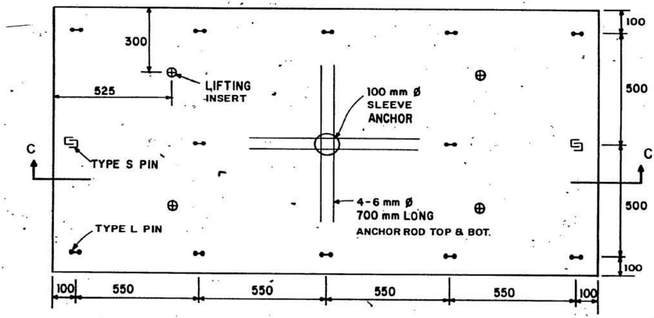


FIG. 6.4 Plan for layout of shear connector pins and details for sandwich panel anchor system.

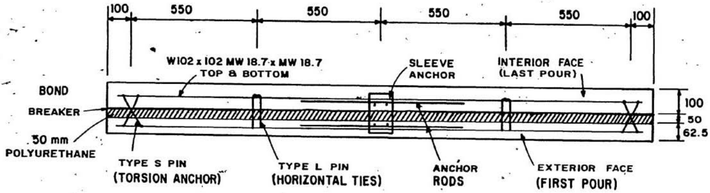


FIG. 6.5 Sandwich panel anchoring system. Section C-C.

6.2.3 Testing apparatus

The testing rig was situated in the Structures Laboratory with the frame rested on four concrete piers as shown in Plate 6.1.

The supporting frame was rigid enough so that all the measured deflections and strains were due to the applied load, and not to the vibrations and movements of the supporting frame.

(a) Load application

In order to simulate real conditions of loading in the test, a uniformly distributed load was required. However this type of loading condition would present many problems, if used over the whole area of the panel, such as interference with the installation of the top dial gages, and also in any method used of increasing the load as the panel deflected. Thus from a practical point of view, it was impossible to produce a true uniformly distributed load. Hence, to simulate a uniformly distributed load, a sixteen point load system was developed for testing the small panel, (Figure 6.6) and a twenty point load system was developed for testing the large panel. (Figure 6.7)

The point load, applied from a lever system, was distributed to the points by means of a system of brackets and I-section beams. The brackets were made of angle section in the shape of an "H". The cross piece at both sides was fixed to the centre pieces by means of a high strength loose

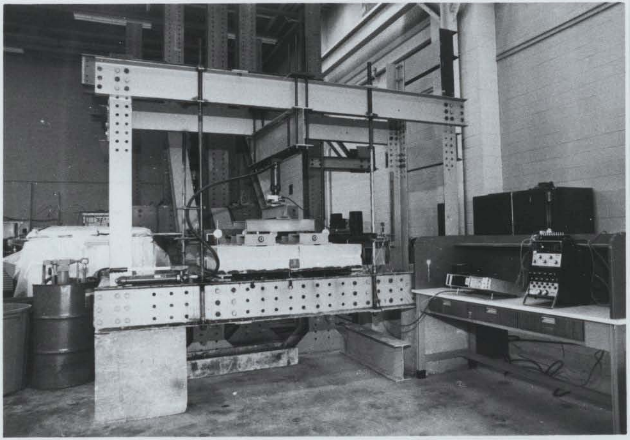


PLATE 6.1 - General view of small panel set up for bending and deflection test

bolt, allowing the brackets to tilt with the panel as it deflected. Small pieces of wooden hardboards, were placed on the surface of the panel at the points of application of load in order to eliminate the concentration of stresses due to local imperfections. The brackets were loaded by means of 2 I-sections. The 2 I-sections were loaded by means of another I-section, which received the point load from the lever. All points of contact between the sections had either a roller or a ball bearing in order to provide a degree of movement for the system during loading. The layout of the load distribution system, dial and strain gages, for both the small and large panels are shown in Figures 6.6, 6.7, 6.8, 6.9, 6.10, 6.11 and Plates 6.1, 6.2, and 6.3. The combined weight of the I-beams and H frames for the small panel was 110 kilograms and for the large panel was 177 kilograms. These dead loads were considered to be small compared with the weight of the panels and would not significantly affect the deflection of the panel. They would in fact straighten out any initial uneveness between the panel and the supports. Initially, the contacts between the panel and supports were not perfectly smooth and these inconsistencies were overcome after a certain amount of load had been applied.

The load was applied to the lever by means of a 18 metric ton hydraulic pull ram.

For the large panel, the distribution of loads (Figure 6.9) may not be uniform along the unsupported edge of the panel. For this reason, the panel stiffness along the edge would be affected due to the simulated load distribution system. However, emphasis for this test was placed on the measurement of deflections and strains along the centerline of the panel between supports, the arrangement of using the twenty point load system was considered to be adequate.

(b) Deflection and strain measurement

Dial gages were used to measure deflections both at the top and at the bottom of the panel. The gages layout is shown in Figures 6.8 and 6.11. All the dial gages read between the scale of 0.01 mm and 50 mm. Strains encountered at the bottom face of the panel were measured by utilizing M-M precision strain gages. The strain gages were installed at the bottom of the panel prior to testing. The strains were recorded on a V/E-20 digital strain recorder manufactured by Vishay Instruments Inc.

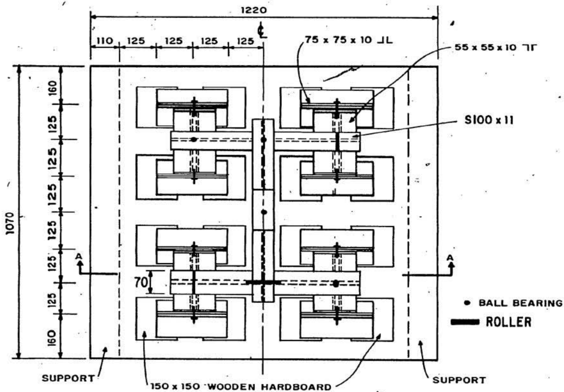


FIG. 6.6 Plan for layout of loading frame for small panel without shear connector.

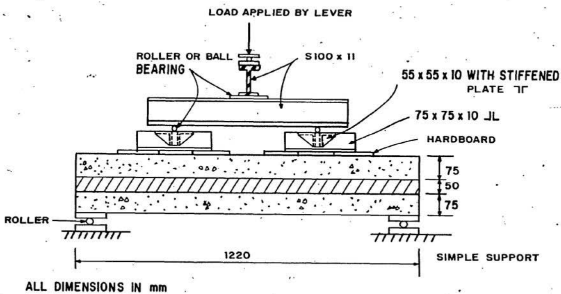


FIG. 6.7 Load distribution system for small panel. Section A-A.

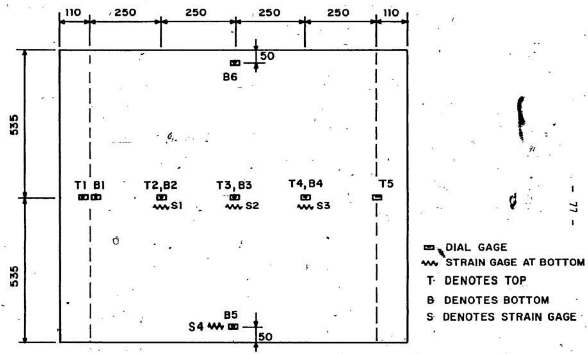


FIG. 6.8 Dial and strain gage layout for small panel .

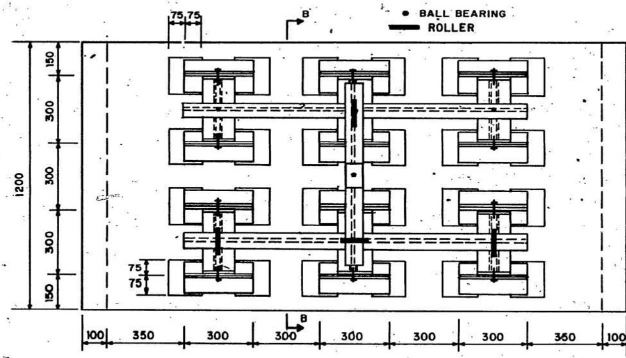


FIG. 6.9 Plan for layout of loading frame for large panel with shear connector

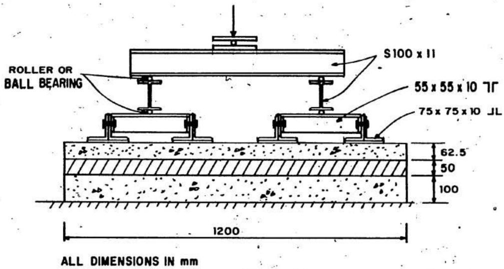


FIG. 6.10 Load distribution system for large panel. Section B-B

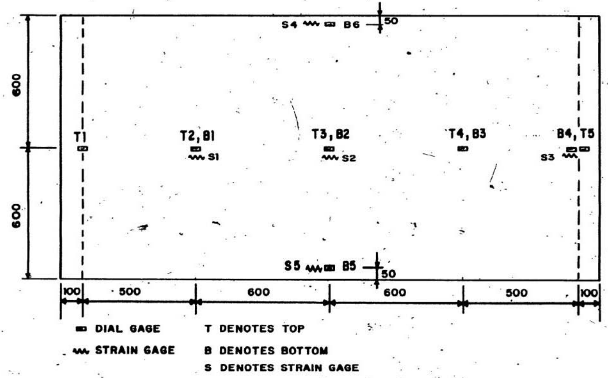


FIG. 6.11 Dial and strain gage layout for large panel.

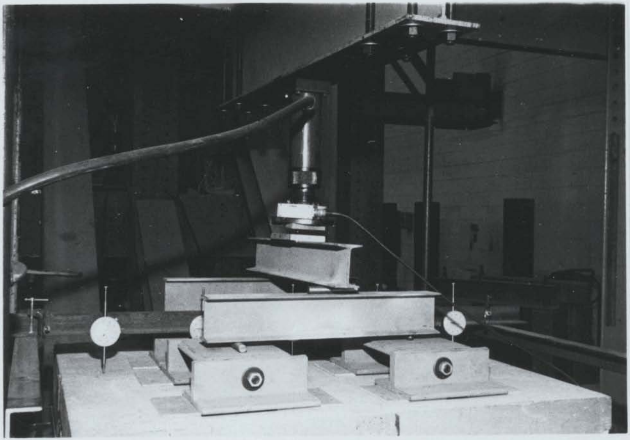


PLATE 6.2 - Close-up of small panel showing top dial gages, load distribution system, rollers and ball bearings

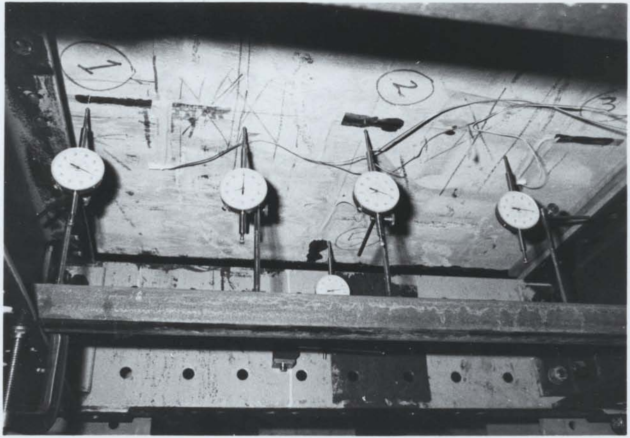


PLATE 6.3 - View of underside of small panel showing layout of bottom dial gages and strain gages

6.3 Experiment for bending and deflection tests on sandwich panel with and without shear connector

6.3.1 For small panel without shear connector

The general arrangement of the test is shown in Plate 6.1. Initially the dial gages were checked, and ensure that they were not sticking. All the strain gages were tested, and the recorder calibrated. The strain reading channels and all dial gages were then zeroed at an initial datum load of 25 kilograms. Readings were taken for a series of load increments as illustrated in Table 6.1. After application of each load increment, a full minute was allowed to pass before the next reading was taken in order to allow the panel to settle under the load. At all times, every dial gages was checked to see if the plunger was sticking, and if so, it was tapped gently.

During the experiment, the behavior of the panel under each applied load was monitored closely, and the deflections and strains were recorded. When the load approached 3,000 kg, a hairline crack started to initiate at the bottom face located about 150 mm to the right side of centerline of panel. Up to this point, the applied load was reduced back to the datum load, and the experiment was repeated twice at different times in order to obtain average readings between each load increment up to 3,000 kg. When the load was further increased, it was visualized that tension cracks developed across the bottom

TABLE 6.1 Average dial gage (in millimeters) and strain gage (1×10^{-6}) measurements for small panel

Load (kgs) \ Dial Gage	T_1 (-)	T_2 (-)	T_3 (-)	T_4 (-)	T_5 (-)	B_1 (+)	B_2 (+)	B_3 (+)	B_4 (+)	B_5 (+)	B_6 (+)
200	0.05	0.07	0.08	0.08	0.06	0.02	0.06	0.06	0.05	0.01	0.01
500	0.21	0.23	0.26	0.25	0.24	0.16	0.17	0.18	0.16	0.16	0.18
1000	0.38	0.44	0.45	0.43	0.40	0.32	0.39	0.47	0.38	0.33	0.40
1500	0.57	0.69	0.77	0.62	0.55	0.50	0.59	0.62	0.50	0.53	0.68
2000	0.69	0.75	0.92	0.77	0.65	0.64	0.70	0.77	0.71	0.65	0.77
2500	0.77	0.88	1.15	0.90	0.79	0.81	0.89	0.95	0.82	0.88	0.92
3000	1.18	1.25	1.32	1.30	1.03	1.30	1.23	1.08	0.99	1.10	1.20
3500	1.28	1.64	1.73	1.90	1.97	1.75	1.70	1.54	1.17	1.45	1.50
4000	1.70	2.05	2.25	2.30	2.40	2.75	2.40	2.15	1.29	2.06	2.30
4500	2.05	2.92	3.04	3.40	3.35	3.70	3.50	2.70	1.54	2.35	3.05

Load (kgs) \ Strain Gage	S_1	S_2	S_3	S_4
200	4	5	2	0
500	7	8	5	4
1000	14	15	10	4
1500	19	18	15	5
2000	28	23	19	5
2500	47	29	27	5
3000	55	34	33	10
3500	72	42	41	17
4000	79	59	62	20

face of the panel. The bonding between the two layers of insulation failed and started to open up at the left side of the centerline of the panel. When the load reached 4,300 kg, the core was crushed at the right side and lifted up at the left side, the cracking of concrete also occurred as shown in Plate 6.4. The experiment was completed at an applied load of 4,500 kg, and final readings were recorded.

6.3.2 For large panel with shear connector

The general arrangement of the test is shown in Plate 6.5. The experiment was conducted in a similar procedure as the small panel. Readings were recorded and illustrated in Table 6.2. During the experiment, the behavior of the panel under each applied load was monitored closely. When the load approached 2,500 kg, a hairline crack started to initiate at the bottom face located at the centerline of the panel. Up to this point, the applied load was reduced back to the datum load, and the experiment was repeated twice at different times in order to obtain average readings between each load increment up to 2,500 kg. When the load was further increased, the crack continued to develop and spread across the bottom face of the panel. There was a slight bond failure observed between the core and the bottom concrete face as shown in Plate 6.6. The experiment was completed at an applied load of 3,000 kg, and the final readings were recorded.

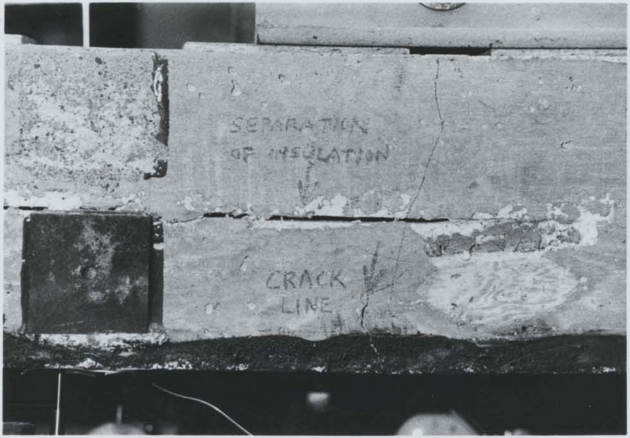


PLATE 6.4 - View of cracking at lower concrete face and bond shear failure at core for small panel

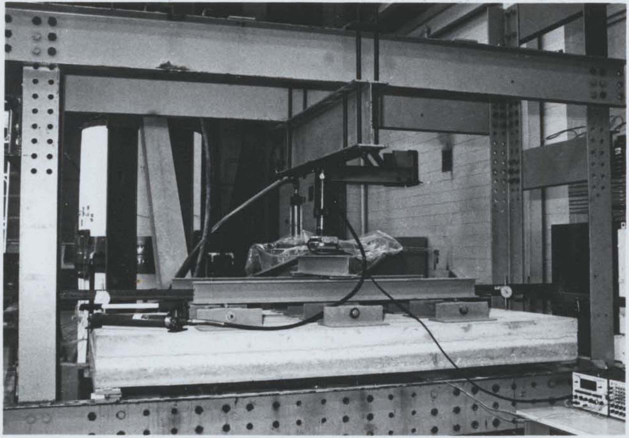


PLATE 6.5 - General view of large panel set up for bending and deflection test

PLATE 6.6 - View of cracking at lower concrete face and lower bond failure at core for large panel

TABLE 6.2 Average dial gage (in millimeters) and strain gage (1×10^{-6}) measurements for large panel

Load (kgs) \ Dial Gage	T_1 (-)	T_2 (-)	T_3 (-)	T_4 (-)	T_5 (-)	B_1 (+)	B_2 (+)	B_3 (+)	B_4 (+)	B_5 (+)	B_6 (+)
200	0.04	0.11	0.15	0.09	0.03	0.09	0.13	0.11	0.03	0.10	0.12
500	0.09	0.22	0.27	0.19	0.07	0.23	0.28	0.22	0.09	0.23	0.30
1000	0.24	0.53	0.72	0.52	0.17	0.54	0.73	0.52	0.21	0.67	0.78
1500	0.38	0.93	1.25	0.95	0.26	0.99	1.22	0.85	0.32	1.09	1.34
2000	0.64	1.49	1.91	1.38	0.42	1.70	2.00	1.36	0.48	1.82	2.15
2500	0.86	2.13	2.95	2.00	0.55	2.39	2.92	1.92	0.65	2.68	3.02
3000	1.45	2.93	3.51	2.90	1.12	3.40	3.74	2.81	1.15	3.35	4.04

Load (kgs) \ Strain Gage	S_1	S_2	S_3	S_4
200	6	5	4	6
500	12	24	9	25
1000	17	30	12	32
1500	23	38	17	40
2000	30	45	23	49
2500	44	56	30	60

6.4 Shear test experiment on sandwich panel with and without shear connector

6.4.1 For small panel without shear connector

The general arrangement of the test is shown in Plate 6.7. The panel when loaded, developed shear stresses at the interface of the core and the faces. As the applied load increased, the diagonal reinforcing ties started to yield between the steel and concrete. It was observed that at an applied load of approximately 5,600 kg, the panel failed and relative displacement took place at the interface of the two layers of insulation as shown in Plate 6.8. This failure was due to a combination of the bond stress limit of the diagonal rebars and the weak bond between the insulation.

6.4.2 For large panel with shear connector

The general arrangement of the test is shown in Plate 6.9. For the core, there was only one 50 mm layer of insulation used. The concrete faces were connected by a sleeve anchor at the center, four lifting inserts, and a series of connector pins around the perimeter of the panel. There was no failure observed between the core and the faces when the load reached a value of 12,000 kg. There was also no relative displacement encountered between the core and the concrete faces.

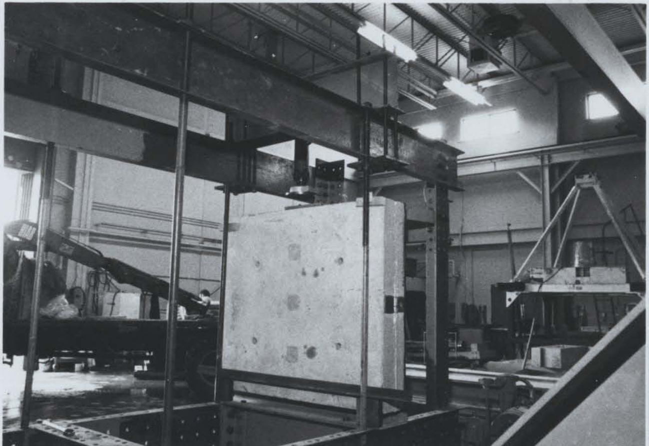


PLATE 6.7 - General view of small panel set up for shear test

PLATE 6.8 - View of core failure for small panel after shear test

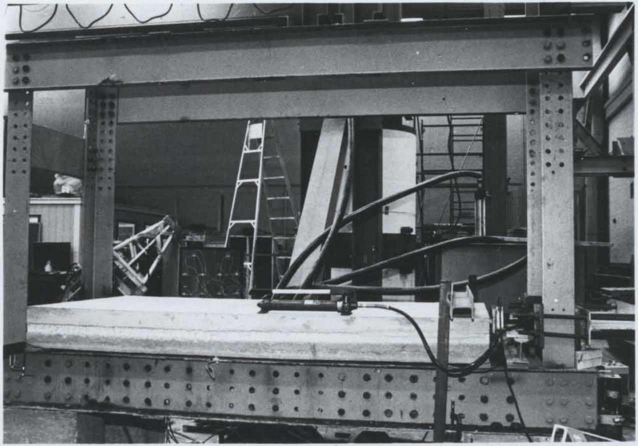


PLATE 6.9 - General view of large panel set up for shear test

6.5 Results analysis

Experimental tests for bending, deflection and shear were carried out for the two sandwich panels. The average dial gage and strain gage readings are tabulated in Table 6.1 for the small panel; and the Table 6.2 for the large panel. The maximum central deflection measurements under different applied loads were obtained by assuming the average of the readings on the top gage T_3 and bottom gage B_3 for the small panel; and on the top gage T_3 and bottom gage B_2 for the large panel. Comparison of the theoretical and the experimental maximum central deflections under different applied loads are shown in Tables 6.3 and 6.4, and their relations are plotted in Figures 6.12 and 6.13.

6.5.1 For the small panel

From the experimental results, at loads under 3,000 kg, the recorded deflections increased linearly with the applied loads as shown in Figure 6.12. At loads above 3,000 kg, the core started to deform, and the readings were no longer in agreement with the theoretical values as computed from Equation 6.16. Hence, beyond an applied load of 4,300 kg, it appeared that the deflection was mainly associated with the core crushing at the right side, and bond failure at the left side of the panel as shown in Plate 6.4. The shear test for the small panel indicated that failure occurred at the interfacial bond between the two layers of insulation, which is an undesirable factor for sandwich construction.

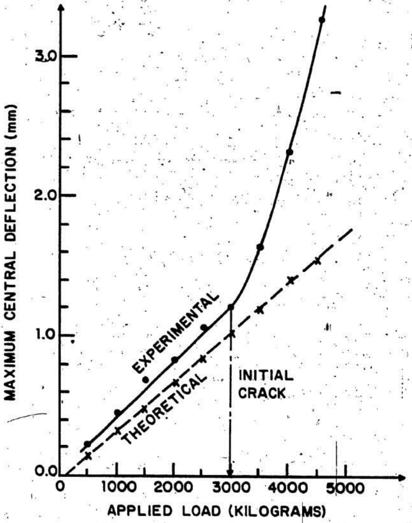


FIG. 6.12 Maximum central deflection for small panel.

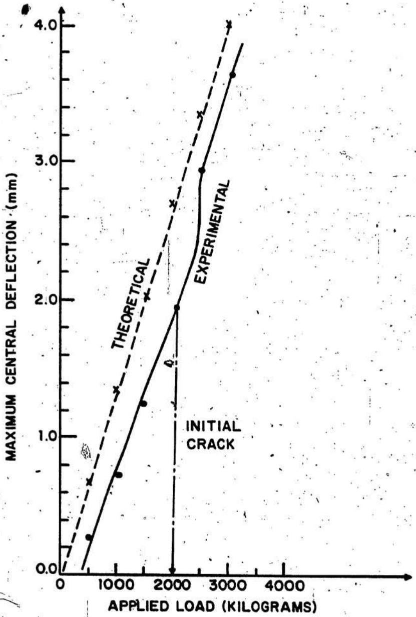


FIG. 6.13 Maximum central deflection for large panel.

TABLE 6.3 Comparison of theoretical and experimental results for maximum central deflection (millimeters) for small panel

Load (kgs)	500	1000	1500	2000	2500	3000	3500	4000	4500
Theo.	0.17	0.34	0.52	0.69	0.86	1.02	1.19	1.38	1.55
Exp.	0.22	0.46	0.69	0.83	1.05	1.20	1.64	2.32	3.27

TABLE 6.4 Comparison of theoretical and experimental results for maximum central deflection (millimeters) for large panel

Load (kgs)	500	1000	1500	2000	2500	3000
Theo.	0.68	1.34	2.01	2.68	3.35	4.02
Exp.	0.27	0.72	1.23	1.95	2.93	3.62

6.5.2 For the large panel

Due to the installation of the shear connector and connector pins between faces, the local bending stiffness of the faces had a definite effect on the shear deformation of the core. Considering the graph as illustrated in Figure 6.13, the experimental results for the deflection are generally less than the theoretical values. The shear test for the large panel indicated that no shear failure occurred at the interface between the core and the concrete faces. Thus, the flexible connector pins were contributing to the stiffness of the panel by joining the two concrete layers and prevented them from peeling off the core.

For both panels, from theoretical calculations, it was found that the deflections were mainly associated with shear deformation, with less than 1% due to bending. The two panels behaved differently because of the span length, the panel thickness and the core materials. The styrofoam has a higher core shear modulus G than polyurethane.

From Equations 6.9 and 6.10, for the small panel of equal face thickness under an applied load of 2,000 kg:

The maximum bending stress σ_{MAX} = 131 kPa (19 psi)

The maximum strain ε_{MAX} = $\frac{\sigma_{MAX}}{E_f}$ = 6.6×10^{-6}

The maximum shear stress τ_{MAX} = 5.5 kPa (0.8 psi)

The experimental measured strain = 23×10^{-6}

From Equations 6.15 and 6.10, for the large panel of unequal face thickness under an applied load of 2,000 kg:

The maximum bending stress σ_{MAX} = 558 kPa (81 psi)

The maximum strain ε_{MAX} = 22.4×10^{-6}

The maximum shear stress τ_{MAX} = 11 kPa (1.6 psi)

The experimental measured strain = 45×10^{-6}

For the strain measurements, the above results show a small difference between the experimental and the calculated strain which is considered to be acceptable.

Based on the analysis of the experimental and theoretical results, the following conclusions can be made:

1. Due to the low shear strength of the core, and where there was no shear connector system holding the two concrete faces together, the flexural strength of the small panel was limited. With the shear connector system installed, the two concrete faces of the large panel were connected to provide full shear transfer between faces and to prevent them from peeling off the core.
2. The crushing of the core would cause more deflection on the top of the panel than on the bottom of the panel where the load was applied. This was noticed in the test results for the small panel where there was no shear connector.
3. The measured deflections were found to be mainly associated with shear deformation of the core for both the small and large panels.
4. The installation of a shear connector, torsion anchor and connector pins between the faces effectively reduced the shear deflection for the large panel.
5. The theoretical bending and shear stresses for the large panel were much higher than the small panel.

CHAPTER VII

CURING OF CONCRETE SANDWICH PANEL

Fresh concrete gains strength most rapidly during the first few days, and the normal weight Portland Cement concrete should have about seventy percent of the specified strength at the end of the first week after placing. The final concrete strength depends greatly on the conditions of moisture and temperature during this initial period. Thirty percent of the strength or more can be lost by premature drying out of the concrete, and similar amount may be lost by permitting the temperature to drop to 4° C or less during the first few days, if no proper curing is provided. To prevent such damage, concrete should be protected from loss of moisture for at least seven days. When high early strength cements are used, curing periods can be cut in half, but accelerated heat curing techniques are required.

The conventional way for curing of precast concrete or sandwich panels takes place in the form. The exposed surface of concrete panels should be covered to minimize water loss during curing. For the precast bed that has a curing system which is oil heated, only the bottom of the sandwich panel is heated from underneath. Hence, curing of the bottom layer (below the insulation) is normally much better than curing of the top layer, because heat from underneath the bed is retained by the insulation and water cannot evaporate. Consequently, the bottom layer will have earlier high strength compared with the top layer. In order to provide the top layer with the same treatment, additional external heat source will be required. The top exposed surface should also be covered and kept moist to minimize the water loss, or the cover to be insulated to retain the normal hydration heat. For early removal of sandwich panels from forms, high early strength cement may be used, but proper heat and moist-cure system are definitely required. Heat curing will accelerate the early age compressive and tensile strength of the concrete which is needed at the time when panels are removed from forms. Following stripping, further curing is required to ensure both faces of the sandwich panel will be exposed to similar conditions. The panels should be protected from direct sunshine during final curing. In general, uneven curing will be minimized if the maximum possible strength is achieved prior to stripping.

CHAPTER VIII

DESIGN OF TYPICAL CONCRETE SANDWICH PANELS

8.1 Design considerations

The following factors should be taken into consideration when designing concrete sandwich wall panels.

(1) The thickness of both faces of panel

The minimum thickness of the panel faces depends upon structural requirement, architectural finish, reinforcement cover, handling considerations and past performance experience. The thickness of the outside face should be kept as thin as possible. However, for production reasons, a minimum thickness of 60 to 70 mm. for the exterior face is usually recommended.

To assure the lateral stability of the sandwich panel, the thickness of the load bearing face should be at least 100 mm if the exterior face is 60 mm thick, and it varies according to the application. The thickness of the panel faces will be reduced if prestressing techniques are utilized. The thickness of a load bearing wall should also be sufficient at all points to ensure that the stresses due to the worst conditions of loading for which the structure is designed, are within the limits prescribed in the building code.

(2) The loading conditions

The forces which must be considered in the design of concrete sandwich panels can be classified as follows:

(a) Gravity loads:

For non-composite panels, the dead weight of the outer face is supported by the load bearing face, which assumes the total structural function of the panel, and is analyzed assuming no interaction with the other parts of the panel. The stresses in a totally non-composite panel may be calculated using only the properties of the load bearing face. For a composite panel, the degree of interaction between connected faces must be known and the stresses may be calculated using the composite properties.

(b) Earthquake forces:

To design an earthquake resistant structure, the design engineer should be very familiar with the minimum requirements specified by the National Building Code 1980 for seismic design. For structural or non-structural elements, such as wall panels, which do not participate in the lateral resistance of the building, special load factors are applied to the connections to assure that the element will remain in place.

(c) Wind load:

Sandwich wall panels are subject to loads applied in both vertical and horizontal direction. Lateral loads applied normal to the wall are the result of wind pressure. Local applicable codes specify the distribution of wind pressure for which a building is to be designed. The magnitude and distribution of these lateral loads, and the means for resisting these loads using shear walls and floor diaphrams, should be considered.

(d) Snow and floor loads:

Both of these loads must also be considered as a separate gravity loading condition for load bearing walls only, since these loads are transmitted by connections between the panels. As the load accumulates from each floor downwards, the transfer of the load from panel to panel becomes a much more significant factor which can determine the minimum panel dimension.

(e) Loading from manufacturing to erection process:

The in-service loads were generally not as critical as those loads imposed on the wall panels during the production and erection stages. Thus the forces imposed during the stripping, handling, transportation, and erection stages are considered as part of the design calculations.

(f) Eccentricity effects:

Eccentricities of the compressive load on the composite sandwich panels can be a major design consideration. This is particularly the case when the composite panels are to be used in multi-storey buildings. At present, provisions in the design for eccentricity of load is largely dependent on the assumption of the behavior of concrete columns under eccentric loads. Unless some publicized information is obtained on the actual eccentricities of loads that can apply to sandwich panels in buildings, it is unlikely that the assumption can be made much less conservative.

(3) The fire resistance of sandwich panels

Behavior of precast concrete sandwich panels under fire conditions is also a governing factor in the design.

Based on fire tests, the PCI Committee on fire protection presents design data for calculating the thickness of many types of walls that will provide fire endurances of 1,2,3 and 4 hours. In particular, tables and design charts are included for determining the thickness of sandwich panels. Suggestions are offered for the treatment of joints between wall panels, the protection of connections, and the fire stopping between floors and wall panels.

(4) The temperature gradient between faces

The thickness of insulation will be determined by the thermal characteristics of the material and the design temperature of the structure. Composite sandwich panels tend to bow to a greater degree than non-composite panels. Hence the maximum height of a composite panel is recommended not to exceed 4500 mm (15 feet) unless the panel is prestressed on both faces. For non-composite panels, the installation of bond breakers between the insulation and the load bearing face is necessary so as to allow relatively free movement between both layers for the dissipation of temperature and other volume change stresses. An air space may also consider to be necessary for ventilation between the outside face and the insulation to avoid moisture built up.

(5) The sandwich panel anchor system

The sandwich panel anchor system is intended to interconnect the outside face and the load bearing face of the sandwich panel, and to transmit to the latter the stresses acting on the outside face. The panel anchor system can be supplied by Superior Concrete Accessories, Inc. or by the Burke Company. The sandwich panel anchor system is basically composed of three elements: -

(a) The sandwich panel anchor, acting as a shear connector, is designed for transferring the weight of the non-loadbearing section into the structural bearing section. The panel anchor can be either a perforated sleeve anchor, or a pair of tension and compression struts.

(b) The torsion anchor is intended for taking the eccentric loads. It is in principle a sandwich panel anchor, but it consists of just one tension and compression pin or strut, because of the fact that it is subjected to less loading. The torsion anchor is installed horizontally so that it does not participate in the vertical load. The anchor design provides extremely high torsional stiffness, thereby possessing resistance to small eccentricities. The torsion anchors are important for panels containing windows and doors, because the main panel anchor is difficult to be situated at the center of gravity of the panel. Thus torsion anchors are required to prevent the panel from rotation during thermal movements.

(c) The connector pins are horizontal ties for taking the horizontal force, such as wind pressures. They also offer resistance to the change in length of the outside face in every direction, and yet be able to take the tension and compression forces without buckling or tearing out of the concrete.

8.2 Design example of a non-composite sandwich panel

Based on the above design considerations, the theory of sandwich panels, the design procedures suggested by the PCI manual, and the manufacturer's brochures, a design example is illustrated below with reference to Figures 6.4, 6.5 and 8.1.

Compressive strength of concrete at 28 days = 30 MPa

Density of normal weight concrete = 2400 kg/m^3

Density of insulation = 32 kg/m^3

Size of panel = 2400 mm x 1200 mm

(1) Panel section properties

The location of the centroidal axis is as shown in Figure 8.1.

Y_t = 100.5 mm from the top of load bearing face

Y_b = 112.0 mm from the bottom of exterior face

From Figure 8.1 :

The moment of inertia about the centroidal axis

$I_{c-c} = 24.5 \times 10^6 \ mm^4$ (from Equation 6.11)

The section modulus S of the load bearing face =

$$\frac{I}{Y_t} = \frac{24.5 \times 10^6}{100.5} = 244 \times 10^3 \ mm^3$$

Panel weight : Concrete $= 3.90\ kN/m^2$

Insulation $= 0.01\ kN/m^2$

Total weight $w = 3.91\ kN/m^2$

(2) Stripping design (Figure 8.1)

Multiplier $= 1.2$

$$-M = \frac{1}{2} w l_1^2 \times 1.2 = \frac{1}{2} \times 3.91 \times 0.525^2 \times 1.2$$

$$= 0.65\ kN\text{-}m/m$$

$$+M = (\frac{1}{8} w\, l^2 - \frac{1}{2} w l l_1) \times 1.2$$

$$= (\frac{1}{8} \times 3.91 \times 2.4^2 - \frac{1}{2} \times 3.91 \times 2.4 \times 0.525) \times 1.2$$

$$= 0.43\ kN\text{-}m/m$$

$$f_t = \frac{M}{S} = \frac{0.65 \times 10^6}{244 \times 10^3}$$

$$= 2.66\ MPa \angle 0.95\sqrt{30}\ MPa \qquad O.K.$$

Therefore, panel will not crack.

Provide minimum reinforcing steel in each face of the panel.

Use wire mesh W102 x 102 MW 18.7 x MW 18.7

(3) Stripping insert (Two point pick up)

$$R = 3.91 \times 1.2 \times 1.2 \times \frac{1}{2} = 2.81\ kN$$

Safety factor for insert = 4 (PCI Manual)

Therefore, provide 4 stripping insert, each of carrying capacity of 11.24 kN

(4) Handling insert (Two point pick up)

$R = 3.91 \times 2.4 \times 1.2 \times \frac{1}{2} = 5.63$ kN

Safety factor for insert = 4 (PCI Manual)

Therefore, provide 2 handling inserts, each of carrying capacity of 22.52 kN

(5) Wind design (Figure 8.1)

Wind pressure $q = 0.96$ kN/m^2

Assume panel connections are located at 450 mm from the bottom of the panel, and 150 mm from the top of the panel.

Distance between connections is 1800 mm

$$-M = \frac{ql_1^2}{2} = \frac{0.96 \times 0.45^2}{2} = 0.1 \text{ kN-m/m}$$

$$+M = \frac{ql_1^2}{8} - \frac{0.1}{2} = \frac{0.96 \times 1.8^2}{8} - 0.05$$

$$= 0.34 \text{ kN-m/m}$$

$$f = \frac{0.34 \times 10^6}{244 \times 10^3} = 1.4 \text{ MPa} < 2.66 \text{ MPa} \quad \text{O.K.}$$

Therefore, stripping stress governs the design.

(6) Panel anchor system (manufactured by Superior Concrete Inc.)

a. Shear connector : Weight of the non-structural face is 432 kgs. Provide one stainless steel sleeve anchor of minimum carrying capacity 1000 kgs, and install at the center of the panel to transfer the weight of the non-structural face to the load bearing face.

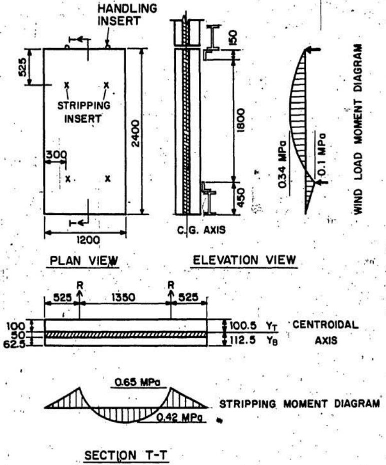


FIGURE 8.1 Design example of a non-composite sandwich panel .

b. Torsion anchor : Provide two sets of Type S pins in locations as shown in Figure 6.4.

c. Horizontal ties : Provide Type L connector pins around the perimeter of the panel as shown in Figure 6.4.

(7) <u>Fire resistance of the sandwich panel</u>

It is possible to calculate the fire endurance of the sandwich panel by using the following formula from PCI Manual (1977).

$$R = (R_1^{0.59} + R_2^{0.59} + R_3^{0.59})^{1.7}$$

where R is the total fire endurance of the sandwich panel, R_1, R_2 and R_3 are the fire endurances of each of the layers in minutes.

For the 100 mm thick load bearing face,

R_1 = 90 minutes, $R_1^{0.59}$ = 14.2

For the 50 mm thick polyurethane insulation,

R_2 = 5 minutes, $R_2^{0.59}$ = 2.58

For the 62.5 mm thick exterior face,

R_3 = 40 minutes, $R_3^{0.59}$ = 8.8

Total fire endurance of the sandwich panel is

$R = (14.2 + 2.58 + 8.8)^{1.7}$

$= 247$ minutes

$= \underline{\underline{4.1 \text{ hours}}}.$

(8) Temperature gradient between faces

For this particular design example, the temperature gradient is calculated in accordance with the formulas as described in Section 5.2.3.

$R_i = 0.12, \quad R_o = 0.03\ m^2.hr.^oC/watt$

$X_2 = 100\ mm, \quad R_2 = 0.06\ m^2.hr.^oC/watt$

$X_3 = 50\ mm, \quad R_3 = 2.1\ m^2.hr.^oC/watt$

$X_4 = 62.5\ mm, \quad R_4 = 0.04\ m^2.hr.^oC/watt$

Total thermal resistance $R(SI) = 2.2\ m^2.hr.^oC/watt.$

Total heat loss $U(SI) = 0.45\ watts/hr.m^2.^oC$

From Figure 5.5, the temperature gradient at the surface of each layer of the panel is:

Temperature $T_i = +21.1^o\ C$

Temperature $T_1 = +18.8^o\ C$

Temperature $T_2 = +17.5^o\ C$

Temperature $T_3 = -21.5^o\ C$

Temperature $T_4 = -22.3^o\ C$

Temperature $T_o = -23.3^o\ C$

(9) Volume change loads

All concrete is subject to volume change resulting from temperature change, shrinkage and creep effect. For the sandwich panel, a shrinkage coefficient of 0.0002 mm/mm is commonly used in shrinkage effect computations.

Forces induced by restrained differential movements between the panels and the supporting structure, are best avoided by allowing sufficient movement in the design of panel connections.

In common practice, a joint width of 12 mm is provided for the volumetric changes encountered between panels.

(10) Other load conditions

Other design components such as snow and floor loads will depend on the location of the structure, its occupancy, and the local building code requirements.

A typical non-composite panel installation details is shown in Figure 8.2. The sandwich panel is connected to the concrete structural frame with details shown at the base, floor and roof slab.

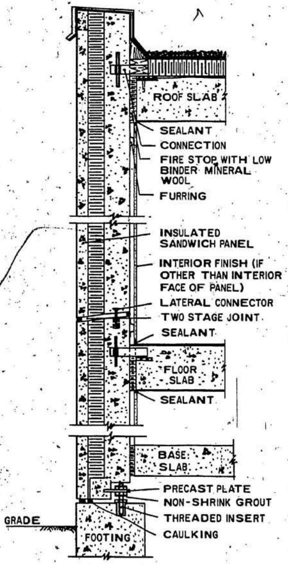


FIG. 8.2 Typical non-composite sandwich panel installation details.

CHAPTER IX

PRACTICAL APPLICATION AND COST EVALUATION OF CONCRETE SANDWICH PANELS

Recent development of well designed anchor systems for connecting the concrete faces through several millimeters of insulation have made the sandwich panel more commonly used in construction industries. Many factors have contributed to the trend towards greater use of the wall panel. These include the growth of using building systems, the availability of efficient production techniques and the realistic cost of sandwich materials.

The sandwich wall panels similar to precast panels have many practical advantages against conventional brick and block buildings. The advantages of utilizing sandwich panels can be summarized as follows: -

(1) offers a wide range of good surface finish.

(2) provides an outstanding rigidity.

(3) provides good thermal, acoustical, and aesthetical properties.

(4) provides standard mass production.

(5) requires no operation and maintenance after erection.

In addition, two concrete mixes can be arranged to cast the sandwich panels, concrete for the exterior face, chosen for color and texture to suit the clients' requirements. The interior face is poured on top, providing a smooth finish for inside attachment of building system.

In the United States and parts of Canada, some schools, hospitals, apartment buildings and institutions have used sandwich wall panels to good advantage, particularly where a high degree of repetition has made the labour and equipment cost more economical.

In general, the cost of fabricating concrete sandwich panel would include: -

(1) preparing precast bed and forming panels.

(2) installing anchoring systems, connector pins, and inserts.

(3) installing rigid insulation and bond breaker.

(4) placing and finishing concrete to the specified texture and color.

(5) curing, stripping, handling and erection.

From discussions with a local supplier, Atlantic Concrete Ltd., it was indicated that the fabrication of sandwich panels would be more time consuming, and required additional labour and facilities for both pouring and curing. The unit cost for fabricating such sandwich panels also depends on the thickness, occupancy, exposure finish and how complex the system is. The cost will be in the approximate range of \$ 270 to 320 per square meter (\$ 25-30 per square feet). This approximate range applies to building size of less than 1,800 square meters (20,000 square feet). The unit cost of production will be reduced to an extent for a larger size of building, because of mass production and high degree of repetition.

CHAPTER X

CONCLUSIONS

Experimental investigations on concrete sandwich panels have been carried out, and based on the analysis of the data obtained, the following conclusions can be made:

10.1 For the heating and cooling of the sandwich panel

(1) The rate of expansion was approximately the same as the rate of contraction, and as a result, allowance could be made for the design of joints between sandwich panels.

(2) For the panel tested, the shape of the panel warped in a convex profile during high temperature periods, in relation to the face subjected to the temperature change; and in a concave profile during low temperature periods.

(3) The sides of the exterior layer of the panel expanded or contracted more than the sides of the interior layer.

(4) The maximum measured deflection at the top of the concrete face was in close agreement with the theoretical calculation under heating and cooling conditions.

10.2 For the bending, deflections and shear test on the sandwich panel

(1) The measured deflections for both panels were mainly due to shear deformation of the core, and the measurements were not symmetrical at a given load.

(2) The measured bending deflections and strains were reasonably close to the results obtained by theoretical analysis at loads below 2,000 kg for both panels.

(3) For the large panel, the bond at the interface of the concrete and insulation was found to be sufficient. No shear failure occurred when the load reached a value of 12,000 kg.

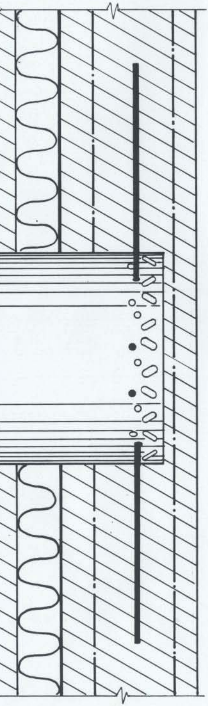
(4) For the small panel, the bond at the interface of the concrete and insulation was found to be insufficient. Shear failure occurred when the load reached a value of 5,600 kg.

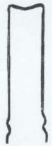
(5) The shear capacity of the small panel was found to be small, unless the two faces were held together so that they could not move or rotate in relation to one another.

It is important to note that the results obtained were under experimental conditions only, and if applying them to design, due allowances must be made for practical conditions and uncertainties.

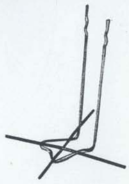
SUPERIOR CONCRETE ACCESSORIES, INC.

SLEEVE ANCHOR
FOR
SANDWICH PANELS





Type S
Connector Pin



Type L
Connector Pin

The following factors should be examined and considered for the future design and construction of concrete sandwich panels: -

(1) The temperature gradient between faces.

(2) The thickness of both faces of panel.

(3) The fire resistance of sandwich panels.

(4) The loading conditions.

(5) The sandwich panel anchor system.

Until now, the principal use of sandwich panels by the building industry has been mostly non-composite panels, because of the unavailability of publications on the actual behavior of composite panels under eccentric loads. Hence further research is required to determine the effects of eccentricities of the compressive load on the composite panel.

REFERENCES

1. ACI Publication 1971. Designing for effects of creep, shrinkage and temperature in concrete structure. ACI Publication, SP-27, American Concrete Institute, Detroit, Michigan.

2. Allen, H.G. 1969. Analysis and design of structural sandwich panels. Pergamon.

3. Borton, S.J. 1975. An experimental investigation of the behavior of four different types of sandwich panels. B.Eng. Thesis, Dept. of Civil Engineering, University of Bradford.

4. Christian, A.J. 1977. Engineering design of sandwich panels made with fibre reinforced materials. Mechanical Engineering Publishers, London and New York.

5. Christian, A.J. 1979. Assessment of strength of sandwich panels used as cladding in the building industry. Proceedings of the Seventh Canadian Congress of Applied Mechanics, Sherbrooke, May 27-June 1, 1979.

6. Collins, F.T. 1954. Precast concrete sandwich panels for tilt-up construction. Journal of American Concrete Institute, 26, no. 2, October.

7. Gerfen, W.H. and Anderson, J.R. 1962. Joinery of precast concrete. ACI, Proceedings, 59.

8. Gustaferro, A.H. 1974. Fire resistance of architectural precast concrete. Prestressed Concrete Institute Journal, 19, no. 5, October.

9. Leabu, V.F. 1959. Problems and performance of precast concrete wall panels. American Concrete Institute Proceedings, 56, October.

10. PCI Manual 1977. Sandwich panels. Prestressed Concrete Institute Manual for Structural Design of Architectural Precast Concrete. PCI Publication number MNL-121-1977.

11. Seddon, A.E. 1956. The strength of concrete wall under axial and eccentric loads. Symposium on the strength of concrete structures, London.

12. Singh, G. and Clayton, J.T. 1970. An inherent bond between concrete and expanded plastics. Agricultural Engineering, 51, no. 10, October.

13. Superior Concrete Accessories Inc. 1979. Sleeve anchor for sandwich panels. Superior Concrete Accessories Inc., Rexdale, Ontario.

14. The Burke Company 1979. Sandwich panel anchor system. The Burke Company, San Mateo, California, 1979.

15. Toprac, A.A. and Thompson, J.N. 1963. Welding between precast concrete units. Prestressed Concrete Institute Journal, 8, no. 3, June.

APPENDIX I

List of Suppliers

LIST OF SUPPLIERS FOR CONCRETE SANDWICH PANEL ANCHOR SYSTEM

(1) Canada

Superior Concrete Accessories (Canada) Ltd.
230 Belfield Road,
Rexdale, Ontario.

Supplier of the following items (refer to picture)

(a) Sleeve anchor - The sleeve anchor is a metal sleeve made of stainless steel material, acting as a shear connector.

(b) Type S connector pin - A pair of connector pins inserted at 45 degrees into the panel, acting as a torsion anchor to prevent the panel from rotation.

(c) Type L connector pin - The pins are horizontal ties installed to resist "peeling" of the panel. They are used when the bottom concrete layer is allowed to harden before the top layer is poured.

(2) United States

The Burke Concrete Accessories Ltd.

2655 Campus Drive,

San Mateo, California 94403.

Alternate Supplier of the following items (refer to picture)

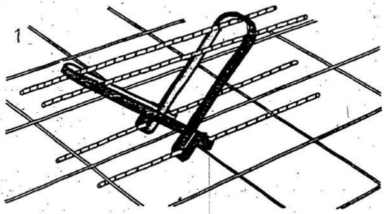
(a) Panel anchor - The anchor is a pair of tension and compression struts, serving the same purpose as a sleeve anchor.

(b) Torsion anchor - The torsion anchor is just one tension and compression strut, serving the same purpose as a pair of Type S connector pins.

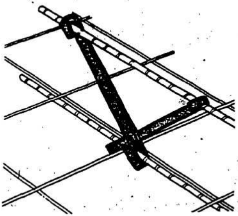
(c) Ties - The horizontal ties are made from steel rods, serving the same purpose as Type L connector pins.

THE BURKE COMPANY

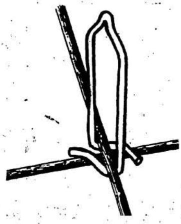
SANDWICH PANEL ANCHOR SYSTEM



Panel Anchor



Torsion Anchor



Ties