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LA THÈSE A ÉTÉ MICROFILMÉE TELLE QUE NOUS L'AVONS RÉCEUE
Tests and Strength Evaluation of Composite Fibre Reinforced Panels (Bumaco Laminated Panels)

M. A. Rohadi

A Thesis in the Department of Civil Engineering

Presented in Partial Fulfillment of the Requirements for the Degree of Master of Engineering at Concordia University Montréal, Québec, Canada

March 1986

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ABSTRACT

Tests and Strength Evaluation of Composite Fibre Reinforced Panels

M. A. Rohadi

Tests and evaluations of the flexural strength and compressive bearing capacity of new composite panels designed for cladding walls and for the construction of terraces are presented in this thesis. These new composite panels are the product of lamination of marble conglomerate or solid granite and asbestos - cement denseite. Asbestos - cement serves as the main reinforcing material.

Full scale composite panels were tested as simply supported beams and slabs, and as bearing walls subjected to axial compression. Additional tests were performed for racking and impact loads.

The analysis method for evaluating the flexural capacity of these composite panels was developed based on the non - linear stress distribution at cracking. Calculated results were then compared with test values. The application of the modified empirical ACI formula (section 14.2.3 of ACI 318 - 83) for concrete walls was verified for walls made of these new panels.
ACKNOWLEDGEMENTS

This research was carried out based on a contract agreement with Atlas Turner Inc. and Bumaco (Building Materials Corp.), and special thanks go to them for producing and preparing the test panels used in this research project.

The author wishes to express his sincere gratitude to Dr. Z. A. Zielinski, project director, for his guidance, cooperation and encouragement throughout all stages of this research program and the preparation of this thesis.

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Last, but not least, the author would like to thank his mother, Mrs. A. R. Roekjat for her infinite patience and understanding.
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NOMENCLATURE

- $A_s$: Area of tension reinforcement
- $A'_s$: Area of compression reinforcement
- $\Delta A_c$: Area of asbestos-cement in tension in excess of basic rectangular section (bh)
- $\Delta A'_c$: Area of asbestos-cement in compression in excess of basic rectangular section (bh)
- $b$: Width of basic rectangular section, as defined in Figure 4.3 (a)
- $E_s$: Modulus of elasticity of steel
- $\varepsilon_{cc}$: Strain at the extreme compression fibre of asbestos-cement section
- $\varepsilon_{ct}$: Maximum strain at the extreme tension fibre of asbestos-cement section
- $f_{ci}$: Compressive stress in asbestos-cement section corresponding to $\Delta A'_{cc}$
- $f_{cc}$: Compressive stress at the extreme compression fibre of asbestos-cement section, as defined in Figure 4.3 (c)
- $f'_s$: Stress in the compression reinforcement, as defined in Figure 4.3 (c)
- $f_s$: 29.0 ksi, stress in tension reinforcement at cracking stage assuming that strain in tensioned asbestos-cement is 0.001
- $f_{to}$: Flexural tensile strength of asbestos-cement
- $f_{tm}$: Flexural tensile strength of marble conglomerate
- $f_{tol}$: Flexural tensile strength of asbestos-cement in the longitudinal direction
- $f_{tot}$: Flexural tensile strength of asbestos-cement in the transverse direction
- $\sigma_{Lult}$: Ultimate direct tensile strength of asbestos-cement in the longitudinal direction
- $d''$: Compression steel cover
- $d_s$: Tension steel cover
- $d_\Delta$: Distance from the extreme tension fibre to the centre of gravity of $\Delta A_c$
- $d_0$: Distance from the extreme tension fibre to the centre of gravity of the basic rectangular section below the neutral axis
Distance between compression steel and tension steel

Distance from neutral axis to the extreme compressive fibre

Steel and asbestos - cement modular ratio $\left(\frac{E_s}{E_c}\right)$

Overall depth of basic rectangular section, as defined in Figure 4.3 (a)

Thickness of asbestos - cement layer in veneer specimen, as defined in Figure 4.5 (a)

Thickness of marble slab in veneer panels, as defined in Figure 4.5 (a)

Thickness of granite slab in granite pavers, as defined in Figure 4.6 (a)

Thickness of the first asbestos - cement layer in granite pavers, as defined in Figure 4.6 (a)

Thickness of the second asbestos - cement layer in granite pavers, as defined in Figure 4.6 (a)

Distance from the extreme tension fibre to the centroid of asbestos - cement area in veneer panels, as defined in Figure 4.5 (d)

Distance from the extreme tension fibre to the centroid of the upper asbestos - cement layer in veneer panels, in granite layers, as defined in Figure 4.5 (d)

The tensile force contribution of asbestos - cement layer in veneer panels

The tensile force contribution of the upper (first) asbestos - cement layer in granite panels

Strain in the extreme compression fibre of marble, during elastic stage

Strain in the extreme compression fibre of asbestos - cement, during elastic stage

Compressive stress in the extreme compression fibre of marble, during elastic stage

Compressive stress in the extreme compression fibre of asbestos - cement, during elastic stage

Tensile stress in the extreme tension fibre of asbestos - cement, during elastic stage
$M_{cr}$  The ultimate (cracking) moment

$\phi$  Capacity reduction factor

$\psi_c$  Coefficient of strengthening of section in compression in excess of basic rectangular section

$\psi_t$  Coefficient of strengthening of section in tension in excess of basic rectangular section

$\sigma_i \sum \psi_i \delta_i$  Terms defined by equation (4.8)

$w_1$  Uniform linear failure load

$w$  Failure load per unit area

$w_{uv}$  Ultimate failure load on veneer panels in bending

$w_{avd}$  Allowable load on veneer panels under dry conditions

$w_{avw}$  Allowable load on veneer panels under wet conditions

$w_{up}$  Ultimate failure load on granite pavers in bending.

$w_{apd}$  Allowable load on granite pavers under dry conditions

$w_{apw}$  Allowable load on granite pavers under wet conditions

$P_u$  Ultimate bearing capacity of wall

$P_n$  Nominal bearing capacity of wall

$f_c$  Specified compressive strength of concrete

$f_{cm}$  Specified compressive strength of marble

$f_{ca}$  Specified compressive strength of asbestos - cement

$k$  Effective length factor

$l_e$  Critical length between supports

$A_g$  Gross area of section

$A_m$  Gross area of marble layer

$A_{asb}$  Gross area of asbestos - cement layer

$m$  Ratio of $f_{ca}$ to $f_{cm}$ ($f_{ca}/f_{cm}$)

$\rho$  Ratio of $A_{asb}$ to $A_m$ ($A_{asb}/A_m$)
$f_t$  Direct tensile strength of concrete

$f_a$  Average tensile stress of concrete
CHAPTER I
INTRODUCTION

1.1 General

Stone products are used mainly for architectural cladding and paving (granite, marble and travertine). However, regular stone products are generally too heavy, brittle and expensive. Due to these drawbacks, architects/designers hesitate to use natural stone products as the main components for establishing the intended aesthetic appearance of a building.

Natural stone products which are commonly used in building construction have certain setbacks which can be summarized as follows:

1. the conventional natural stone products are generally heavy and require delicate handling. In addition, they can only be delivered in small segments; and
2. connection details are difficult to standardize due to varying structural conditions as this will result in high cost of erection, and therefore be uneconomical.

In an attempt to overcome the above disadvantages, Atlas Turner Inc. and Bumaco Inc., both of Montreal, introduced a new laminated composite stone panel. This new product is a sandwich of a thin stone slab and asbestos-cement which has several distinct advantages including:

1. the architects/designers can use natural stone products as large size panels for wall claddings and pavings, floors and plaza surfacing due to the increased flexural strengths;
2. the composite stone panels are good for energy conservation;
3. the cost savings on installation and transportation can be achieved due to the large size and smaller thicknesses of the composite panels; and
4. the composite stone panels are durable and maintenance free.
1.2 Scope of the Study

The development of the Bumaco composite panels was undertaken based on the strength advantages of asbestos - cement. The main purpose of this research program is to define the strength properties of the Bumaco composite panels and to provide the required design guidelines. The research program is based on both an experimental and theoretical investigation.

The experimental program includes full - scale tests on composite panels made of marble and granite to define strength in service condition, and small - scale samples to define the strength characteristics of composite materials. The test results are meant to provide for a basis for verification of the analytical techniques.

The theoretical study program is directed towards the development of design formulae and recommendations for applications for composite panels which may be subjected to flexural and compressive loadings.
CHAPTER II
DESCRIPTION OF BUMACO SYSTEM

2.1 Bumaco Veneer Panels

Veneer panels are the product of lamination of two materials, namely a stone slab of solid granite or marble or marble conglomerate and asbestos-cement (densite). The facing material (1/2" stone slab) and a 3/8" densite sheet are glued together under pressure with epoxy resin produced by the Sika Corporation.

Two marble conglomerates, one made of a dark green marble named 'Verde Polcevera' (designated as Type I marble) and the other of a white marble (designated as type II marble), were tested. These marble types were produced as a conglomerate of marble aggregate and a special synthetic resins by Adige Marmoresina of Italy.

Bumaco veneer panels may have various applications in the construction of a building, which include:

1. Veneer panels for lobby walls in old or new construction. They can be installed directly to stud walls replacing gypsum boards which have been used traditionally. If walls of brick, block or concrete are existing, veneer panels can be fastened by drilling through the face of the panel and installing threaded masonry screws of high strength alloy steel. Screw heads can then be concealed with the use of coloured epoxy to match the veneer (see Figure 2.1).

2. Veneer panels may be used as one of the components for the "s" panels shown in Figure 2.2. The "s" panel is fabricated by coating the apex of each corrugation on the asbestos-cement (densite) core with a gel type epoxy. A 3/8" flat asbestos-cement sheet (densite) is then placed on top of the epoxied core and the two sheets are riveted together. The panel is then reversed and the process is repeated to
complete the base panel. The preassembled panel is then brought to a laminating
station where the epoxy and finish facing (marble or other stones) are applied and
subjected to pressure on hydraulically activated tables. Following a curing period,
the laminated panels can be sawed and modified to special shapes as required by
design. The "s" panel cores can then be injected with polyurethane foam
insulation (approximate "R" value of 15). Various anchor assemblies may be
adapted to suit suspension systems.

2.2 Burnaco Granite Pavers

Granite pavers are the product of lamination of solid granite slabs and two layers
of asbestos - cement denseite sheets. The 3/8" denseite sheets are glued together with epoxy
resins with their fibre directions oriented perpendicular to each other. This combined sheet
and 1" granite slab are then laminated under pressure with the same epoxy resin. The
tested pavers were produced using grey granite from St. Gerard, Quebec.

The granite pavers are used mainly as an architectural cover for the construction of
terraces and plazas which are supported at four corners. The laminated panels may be used
as square slabs or modified to special shapes and geometric patterns which will suit the
span length between supports.

2.3 Material Properties

The mechanical properties of asbestos - cement sheet (denseite), granite and marble
are given in Table 2.1 in accordance with the specifications of Atlas Turner and Burnaco.

The stress-strain curve for asbestos - cement 'denseite' was obtained from the
direct tensile test done by Perinayegon [7], and which is shown in Figure 2.3. The
modulus of elasticity of 'denseite' was also found to be \( E = 1.75 \times 10^6 \) psi and the ultimate
direct tensile strength 1900 psi. Samples were tested with their fibre orientation being
parallel to the tension direction.

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<td>(lb./sq. in)</td>
<td>Across fibers</td>
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<td></td>
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<tr>
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<td>Across fibers</td>
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<tr>
<td>Compressive strength</td>
<td>(lb./sq. in)</td>
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</table>

Table 2.1: Mechanical properties of materials used for producing Bumaco composite panels as specified by Atlas Turner Inc. and Bumaco.

Notes:
- modulus of rupture, flexural strength and compressive strength of natural building stone were obtained based on ASTM Specifications number C99, C880 and C170 respectively; and
- mechanical properties of asbestos-cement densite were defined according to the ASTM specifications number C220-75.
- the uniaxial tensile strength of marble conglomerate was defined by dividing the modulus of rupture (obtained in accordance to ASTM Standard C99) by a factor of 1.75. The factor of 1.75 is used based on literatures [10, 11].
Figure 2.1  Typical application of veneer panels in lobby walls

Figure 2.2  Veneer panels on A/C 'S' panel
Figure 2.3 Stress-strain curve for asbestos-cement in tension along the fibres
CHAPTER III

EXPERIMENTAL PROGRAM AND RESULTS

3.1 Introduction

Testing of full-scale Bumaco laminated panels is of major interest to the present study arising from the fact that full-scale testing is the most reliable means of obtaining real performance data by which the importance of the different analytical techniques can be judged.

Standard ASTM test methods [1, 2, 3] were used in this study whenever they were applicable. Procedures and results of each of the tests performed are described in the following sections.

3.2 Test Specimens

A total of five veneer panels of 23" x 114" x 7/8" size were used for flexural tests. Two of the above panels were composed of the dark green 'Verde polcevera' marble conglomerate and three of the white 'Carrare veine c' marble conglomerate. Additional tests were performed on three 37" x 39" x 2" granite paver specimens. Detailed descriptions of these test series are given in Tables 3.1 and 3.2 respectively.

To determine the compressive strength of veneer panels, two 23" x 114" x 7/8" veneer specimens containing white 'carrare veine c' marble were tested. Racking as well as impact loading tests were performed on the veneer specimens. The detailed dimensions are shown in Table 3.3.
3.3 Flexural Tests on Veneer and Granite Paver Specimens

3.3.1 Testing Procedures

Flexural tests on veneer panels consisted of two phases. The first phase was to test veneer panels as a simply supported beam under a uniformly distributed load and with densite sheet located at the bottom (tension) face (test series BMC1-BMC2/TP1 and BMC1-BMC3/TP2). Panel was rested on roller supports consisting of 1/4" steel plates and steel pipes of 1" diameter. The testing arrangements are shown in Figures 3.1 and 3.2. Once the panel was installed and ready for testing, dial gauges were placed and adjusted. A uniformly distributed load was gradually applied by means of 10 lb sand bags placed on top of the panel in a prescribed order. The deflection and load readings were taken in increments of 50 lbs. The panel was allowed to stabilize for 2 minutes before any readings were recorded. This loading was continued until the ultimate capacity was reached. At failure, crack patterns as well as failure load were marked on the panel and photographs were taken.

The second phase involved the testing of cut-out portions of veneer panels used in the first phase. These portions were cut from areas where the material was not under maximum stress during first phase testing nor injured. Their span length varied from 38 inches to 57 inches. The same roller supports were used as in the first phase, but with different loading arrangements. In test series BMCN1-BMCN3 and BMCR1-BMCR3 of both types I and II marble respectively, loads were applied at a distance of 0.25L from both supports by means of cross beams attached to two hydraulic jacks, and with load increments of 47.5 lbs. In the second set-up test series BMSN1-BMSN3 and BMSR1 of type II marble, loads were applied at midspan through a single cross beam connected to a hydraulic jack at the center. Deflection and load readings were taken at an increment of 47.5 lbs. In both cases, samples were tested with densite sheets located at the top surface of the panel (BMCR and BMSR series) and bottom surface (BMCN and BMCR series). Both loading arrangements are shown in Figures 3.3 and 3.4 respectively.
Granite paver specimens were tested as two way four point loaded slabs, placed on four corners on supporting pads with densite sheet on the bottom tension side (test series 1-GP1 and 1-GP3). Each supporting pad was made of two 4" x 4" x 3/4" neoprene. The loads were applied by means of two cross beams attached to a main loading beam which in turn was connected to a single hydraulic jack at the center (Figure 3.5). This loading arrangement was intended to simulate a uniform load. Dial gauges were used to measure deflections. Deflection readings were recorded at a load increment of 2,375 lbs. Failure modes and crack patterns were noted and photographed.

3.3.2 Test Results

For the test series BMC1-BMC2 of type I marble and BMC1-BMC3 of type II marble, all the specimens failed suddenly (broke into two pieces) when a tension crack appeared in the vicinity of the midspan (see Figures 3.6 and 3.7). It was also observed that veneer panels lost their flexural resistance as soon as the asbestos-cement sheet ruptured. However, no crushing of marble stone was found at the compression zone. The ultimate loads for panels BMC1 and BMC2 type I were 300 lbs and 320 lbs respectively, while panels BMC1-BMC3 type II failed under the loads of 520 lbs, 450 lbs and 390 lbs respectively. Corresponding equivalent uniformly distributed loads were calculated as shown in Table 3.4. The load deflection curves for these test series are shown in Figures 3.8 and 3.9.

In test series BMCN1-BMCN3 of type I marble, the three specimens failed by a sudden tension crack in the area between the two loading beams (Figure 3.10). The same failure mode was observed in test series BMCR1-BMCR3 of type I and type II marble (Figures 3.11 and 3.12), but the failure loads were much lower than the ones obtained in test series BMCN. This proved that asbestos-cement increased the flexural strength of ordinary marble stone panels. A summary of experimental results was shown in Tables 3.5 and 3.6 load deflection curves for the panels were shown in Figures 3.13 and 3.14.
All panels in test series BMSN1-BMSN3 and BMSR1-BMSR3 of type II marble failed directly under the point of load application, by tearing apart in the tension zone. It was again noticed that failure loads obtained from test series BMSR were much lower than results obtained from BMSN. Test results were summarized in Tables 3.7 and 3.8 while Figures 3.15 and 3.16 show the load deflection curves of the tested panels.

Test results on granite pavers are illustrated in Figures 3.17 and 3.18. Failure due to a sudden tension crack in the region between the two cross-beams was observed in all granite paver specimens (see Figure 3.17) except for specimen 1-GP3 where cracks were radiated from P1 to L2 and to the adjacent edges (Figure 3.18). The test results are given in Table 3.9.

Additional tests were performed on broken pieces saved from test series 1-GP. In these tests, samples were tested as one way beams, and placed on rolling supports, with granite on the bottom tension side. Two loads were applied by means of two cross-beams and main beam which was attached to the Tinus-Olsen loading machine as shown in Figure 3.19. All specimens failed when crack appeared between the two loading beams. Test results are shown in Table 3.10.

3.4 Compression Tests on Veneer Panels

3.4.1 Testing Procedures

Compression tests were performed on two veneer panels (BMW1 and BMW2 of type II marble) in a special frame shown in Figure 3.20. The set-up consisted of an overhead mounted hydraulic jack connected to a rigid steel beam. Panels were braced at front and back faces by 2" x 4" pieces of wood. Bracing locations were shown in Figure 3.20. Dial gages were used to measure panel displacements and were placed near the bracing woods and at the center line between two braces (Figure 3.20). Loads were then exerted gradually in equal increments of 1,180 lbs until the ultimate capacity was reached.
3.4.2 **Tests Results**

All panels showed a similar mode of behaviour. Failure occurred suddenly in the region between two braces. The average failure load of the two panels tested was 20.58 kips. The experimental results were tabulated in Table 3.11. The horizontal displacement of center points of panels BMW1 and BMW2 (nodes CT and CB) are illustrated in Figure 3.21. The mode of displacement, prior to failure, is shown in Figure 3.22.

3.5 **Racking Tests on Veneer Panels**

3.5.1 **Testing Procedures**

The apparatus for the simulated racking tests were assembled as shown in Figure 3.23 and 3.24. The test panel was positioned in such a way that its diagonal was perpendicular to the base of the testing machine. In the case of test panel BMR2, wood bracings were provided at a distance of 30.5 inches from the base (Figure 3.24). Load was applied to the specimen through steel saddles which were specially designed to accommodate the test panel. The steel saddles were securely bolted to the base and top of the testing machine. Electric strain gauges (type EA-06-20 CBW-120, manufactured by Micro Measurement) were provided at the locations, as shown in Figure 3.23 and 3.24, to measure the compressive strains along the diagonal and the tensile strains across the diagonal. The load was applied continuously throughout test at a uniform rate until failure. Displacement readings were recorded at load increments of 1,000 lbs. The behavior of the panel during the test and at failure was carefully observed. Any visible failure from loading was marked and photographed.

3.5.2 **Test Results**

For the unbraced veneer specimens, failure was caused by buckling across the vertical diagonal as shown in Figure 3.25. The maximum load at which buckling occurred was 19.975 kips. For braced veneer specimens, failure was caused by splitting along the vertical diagonal as shown in Figure 3.26. The maximum load at which splitting took
place was 49.4 kips.

3.6 Impact Tests on Veneer Panels

3.6.1 Testing Procedures

Standard ASTM test methods [2, 3] are employed in impact tests. The purpose of this test was to provide data that can be used to evaluate the relative performance of wall and floor constructions under conditions representative of those sustained in actual service when subjected to impact by a heavy blunt object.

The specimen was tested as shown in Figure 3.27, as a simple beam on a span of 48 inches. The two roller supports were designed to prevent longitudinal restraint and provide bearing for the entire width of the specimen. The ends of the panel were secured by hold-downs to minimize specimen bounce. Special care was taken to assure that the hold-downs did not affect deflection of the specimen. An impact load was then applied to the upper face of the specimen by dropping the 50 lb sand bag beginning with a height of 6 inches (152 mm) and increasing the height by 6 inch increments. Instantaneous deflection measurements were recorded for each drop by means of two displacement transducers attached to the midspan edges of the panel. For the first drop, the height of the bag was measured from the upper face of the specimen at a point directly beneath the bag, and for subsequent drops, from a taut cord in contact with the upper face vertically above the supports. The impact test was continued until the specimen could no longer support the 50 pound load.

3.6.2 Test Results

The recorded maximum drop height at which the veneer specimens broke in half was 66 inches. No significant damage prior to this maximum drop height was observed on the specimen. Instantaneous deflections were plotted against the drop height and is shown in Figure 3.28.
Figure 3.1  View of testing arrangement for uniform load (before loading)

Figure 3.2  View of testing arrangement for uniform load (during loading)
Figure 3.5  Testing arrangement for granite pavers: (a) side view; (b) plan view
Figure 3.6  Typical view of BMC type I panel at failure

Figure 3.7  Typical view of BMC type II panel at failure
Figure 3.8  Load-deflection curves for BMC type I panels

Figure 3.9  Load-deflection curves for BMC type II panels
Figure 3.10  Typical view of BMCN type I panel at failure.

Figure 3.11  Typical view of BMCR type I panel at failure.
Figure 3.12  Typical view of BMCR type II panel at failure.
Figure 3.13 Load-deflection curves for BMCN type I panels.

Figure 3.14 Load-deflection curves for BMCR type I panels.
Figure 3.15  Load-deflection curves for BMSN type II panels

Figure 3.16  Load-deflection curves for BMSR type II panels
Figure 3.17  View of granite specimen (1-GP1) at failure

Figure 3.18  View of granite specimen (1-GP3) at failure
Figure 3.19  Testing arrangement for GPR panels
Figure 3.20  Experimental set-up for compression tests.
Figure 3.21  Horizontal displacements of nodes CT and CB
Figure 3.22  Mode of displacement of veneer specimen under compression prior to failure.
Figure 3.24  Racking test arrangement for braced veneer specimen

Figure 3.23  Racking test arrangement for unbraced veneer specimen
Figure 3.25  Typical view of unbraced veneer specimen under racking load (at failure)

Figure 3.26  Typical view of braced veneer specimen under racking load (at failure)
Figure 3.27  Testing arrangement for impact loading
Figure 3.28  Drop-height vs. deflection curve for veneer specimen under
<table>
<thead>
<tr>
<th>TEST SERIES</th>
<th>SAMPLE NO.</th>
<th>L (in)</th>
<th>W (in)</th>
<th>( t_1 ) (in)</th>
<th>( t_2 ) (in)</th>
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<tbody>
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<td>0.375</td>
<td>0.500</td>
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<td>23</td>
<td>0.375</td>
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<td>0.375</td>
<td>0.500</td>
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Table 3.1  Detailed descriptions of test series B1 - B5 for veneer specimens
### Table 3.2  Detailed descriptions of test series GI for granite pavers

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<th>W (in)</th>
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<th>t₂ (in)</th>
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### Table 3.3  Detailed descriptions of veneer panels used in compressive, racking and impact tests

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<th>L (in)</th>
<th>W (in)</th>
<th>t₁ (in)</th>
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Table 3.4  Results for test series BMC

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Table 3.5  Results for test series BMCN
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Table 3.6 Results for test series BMCR

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<th>Span Length (in)</th>
<th>Total Failure Load $.P$ (kips)</th>
<th>Equivalent Ultimate Uniform Load (lbs/sq.ft)</th>
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<tr>
<td>BMSN1/TPII</td>
<td>48</td>
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<td>241.3</td>
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Table 3.7 Results for test series BMSN
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<th>Sample No.</th>
<th>Span Length (in)</th>
<th>Total Failure Load P (kips)</th>
<th>Equivalent Ultimate Uniform Load (lbs/sq.ft)</th>
<th>SCHEMATIC DIAGRAM</th>
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Table 3.8 Results for test series BMSR

<table>
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<th>Span Length (in)</th>
<th>Total Failure Load (kips)</th>
<th>Equivalent Ultimate Uniform Load lbs/sq.ft</th>
<th>SCHEMATIC DIAGRAM</th>
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<tr>
<td>I- GPI</td>
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<tr>
<td>I- GP2</td>
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<td>14.012</td>
<td>1853</td>
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<td>I- GP3</td>
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Table 3.9 Results for test series GP
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<th>Average Width (in)</th>
<th>Failure Load (kips)</th>
<th>Equivalent Ultimate Uniform Load (lbs/sq.ft)</th>
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<td>23.750</td>
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<td>2-GPR2</td>
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<td>2-GPR3</td>
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Table 3.10: Results for test series GPR

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<th>Sample No.</th>
<th>Critical Height (in)</th>
<th>Failure Load (kips)</th>
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<td>BMLI/TPII</td>
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<td>BMW2/TPII</td>
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Table 3.11 Results for compressive bearing capacity veneer panels
CHAPTER IV

STRENGTH EVALUATION AND PROPOSED DESIGN PROCEDURE FOR
COMPOSITE FIBRE REINFORCED PANELS

4.1 Cracking Strength of Asbestos-Cement Members Loaded in Bending

Practice introduced by De Mahieu [4] for calculating the flexural strength of asbestos-cement beams is based on an assumption that the concentrated compressive force acts at the top of the beam cross section while the tensile stress follows a triangular distribution as shown in Figure 4.1. The section fails when the maximum tensile strength is reached at the bottom fibre:

Calculations based on the above theory gives, for rectangular sections, results comparable with tests, but rather deviating results for non-symmetric sections. This theory does not confirm the true behavior of fibre reinforced cement materials such as asbestos-cement, which under high tensile stress close to failure, demonstrates an elasto-plastic mode of behavior [12, 13].

Based on research work on flexural strength of ferrocement [15, 16, 17] and reinforced and plain concrete [5, 6, 18], it appears justified to expect that flexural strength of asbestos-cement can be best predicted assuming a stress distribution shown in Figure 4.2 and in simplified form for design practice as shown in Figure 4.3 (c). Based on tests, it can also be assumed that the maximum strain at failure, close to the bottom tension zone, is equal to:

\[ \varepsilon_{ct} = 0.001 \]

Corresponding to this strain, the stress in reinforcing steel, if present, will be:

\[ f_s = \varepsilon_{ct} \cdot E_s = 29.0 \text{ ksi (200.0 Mpa)} \]
Figure 4.3 shows a generalized I-shaped section at failure (cracking), a strain diagram and a simplified stress diagram. The I-shaped section is obtained by transforming the non-symmetric section into an equivalent rectangular section, with the flanges and reinforcement representing the required additional areas.

The following relationships can be noted, based on Figure 4.3 (c):

\[ f_{cc} = 2f_{to} \left( \frac{x}{x-h} \right) \]  
\[ f_s = 2nf_{to} \left( \frac{x-d''}{h-x} \right) \]  

In the above equations, \( n \) denotes the modular ratio \( (E_s/E_c) \) and \( f_{to} \) is the flexural tensile strength of asbestos-cement. From the equilibrium of forces \( \sum P = 0 \),

\[ nA_s f_s + \Delta A_s f_s + 1/2(bz f_{cc}) - b(h-x) f_{to} - \Delta A_c f_{to} - A_s f_s = 0 \]  

After substituting values from (4.1) and (4.2) into equation (4.3), \( x \) is obtained as follows:

\[ x = \frac{1 + \left[ \frac{\Delta A_c f_{to} + A_s f_s}{bhf_{to}} \right]}{2 + 2 \left[ \frac{\Delta A_c f_{to} + A_s f_s}{bh} \right] + \left[ \frac{\Delta A_c f_{to} + A_s f_s}{bhf_{to}} \right]} \]  

Equation (4.4) can be rewritten as:

\[ \frac{x}{h} = \frac{1 + \psi_t}{2 + 2 \psi_c + \psi_t} \]  

where \( \psi_t \) and \( \psi_c \) are the shape factors describing the strengthening in the tension and compression zone respectively of basic rectangular section.
Figure 4.1  Stress distribution at failure based on De-Mahieu hypothesis:
(a) section; (b) stresses

Figure 4.2  Stresses in concrete beam immediately before cracking

Figure 4.3  Generalized I-shaped section at cracking: (a) cross-section;
(b) strain diagram; (c) simplified stress diagram
\[
\psi_t = \frac{\Delta A_c f_{to} + A_s f_s}{bhf_{to}} \\
\psi_c = \frac{\Delta A_c' + nA_s'}{bh}
\]  

(4.6)

If there are no flanges (rectangular section) \(\Delta A_c\) and \(\Delta A_c'\) are equal to zero and:

\[
\psi_t = \frac{A_s f_s}{bhf_{to}}; \quad \psi_c = \frac{nA_s'}{bh}
\]

If there is no tension reinforcement \(A_s\), then \(\psi_t = 0\), and if there is no compression reinforcement \(A_s\)'s, then \(\psi_c = 0\).

The flexural cracking strength can be now calculated from the equilibrium of moments, \(\Sigma M = 0\), as follows:

\[
M_{cr} = f_{to} bh^2 (A + B\psi_t + \Sigma \psi_t \delta_i)
\]

(4.7)

where,

\[
A = \frac{1}{2} \cdot \frac{\gamma \psi_c}{1 + 2\psi_c} - \frac{1 - 2\psi_c}{6(1 + 2\psi_c)} - \frac{1}{6} \left[ \frac{1 + 2\psi_c}{2 + 2\psi_c} \right]^2
\]

\[
B = 1 - \frac{\gamma \psi_c}{1 - 2\psi_c} - \frac{1}{3(1 + 2\psi_c)}
\]

(4.8)

\[
\psi = \frac{t}{h}
\]

\[
\Sigma \psi_t \delta_i = \frac{\Delta A_c dA}{bh^2} + \frac{A_s f_s d_s}{bhf_{to}}
\]

Table 4.1 gives the values of \(A\) and \(B\) for different \(\psi_t, \psi_c\) and \(\psi\) values.
From the equations shown, it is possible to calculate the flexural cracking moment of any geometrical section. The procedure of calculation consists of the following steps:

(a) transform the actual cross-section into an equivalent rectangular section;
(b) find all the necessary coefficients ($\psi_t$, $\psi_c$, $A$, $B$, $\gamma$ and $\sum \psi_t \delta_t$);
(c) consider a value of $f_{to}$ based on recorded test results or obtained from the rupture strength (by using a shape factor $\psi$) which varies from 1.75 for rectangular to 1.25 for I-sections [10, 11]; and
(d) apply equation (4.7) to find the cracking moment $M_{cr}$

These steps have been used successfully in predicting the flexural cracking strength of asbestos-cement panels made of 'v' and 'u' shaped sections tested at Concordia structural laboratories earlier [7, 8].

The present research work is carried out to verify the applicability of equation (4.7) for thin composite sections made of natural stone and one or two layers of asbestos-cement dense sheet.

4.1.1. Flexural Cracking Strength of Veneer Panels

At low loads, 'veneer' panels, as tested, demonstrated an elastic behavior (Figure 4.4) and the neutral axis is located in the asbestos-cement part of the cross-section. With increasing loads, the asbestos-cement part of the panel (under tension) reaches the plastic stage. The neutral axis rises and part of the marble is now under tension. This behavior compares with results obtained by Allen [9] for thin flexural specimens of composites. Failure occurs when the ultimate tensile strain of asbestos-cement is reached at the bottom. Based on this observation, the following assumptions can be made for the flexural strength analysis:
(1) the stresses in the tensile zone can be represented by a constant stress $f_{tm}$ across the tension zone (Figures 4.5 (c) and (d)); and

(2) the tensile contribution of asbestos-cement layer (dense) is represented by a force $T_{asb} = (f_{tol} - f_{tm})A_{asb}$, and where $(f_{tol} - f_{tm})$ is the difference between the tensile strength of asbestos-cement and marble.

![Diagram](image)

Figure 4.4. The State of Strain and Stress for Veneer Section at Elastic Stage:
(a) cross-section, (b) strain diagram, (c) stress diagram.

![Diagram](image)

Figure 4.5. Assumptions for the Analysis Method (veneer section):
(a) cross-section; (b) strain diagram; (c) assumed stress distribution;
(d) simplified representation.
The location of the neutral axis of the cross-section at cracking can be defined using equation (4.4) or (4.5) which in this case may be written as follows:

\[
\frac{x}{h} = \frac{1 + \frac{h_d}{h} \left[ \frac{f_{tol} - f_{tm}}{f_{tm}} \right]}{2 + \frac{h_d}{h} \left[ \frac{f_{tol} - f_{tm}}{f_{tm}} \right]} \tag{4.9}
\]

In this case, the coefficients of strengthening of section in tension and compression are given as follows:

\[
\psi_t = \frac{h_d}{h} \left[ \frac{f_{tol} - f_{tm}}{f_{tm}} \right] \tag{4.10}
\]

\[\psi_c = 0\]

The coefficients A, B and \(\gamma\) for calculating the flexural cracking moment of the veneer panel can then be established as follows:

\[A = 0.292 \quad B = 0.667 \quad \gamma = 0\]

Further, it can be calculated:

\[\sum \psi_i \delta_i = \frac{h_d}{h} \left[ \frac{f_{tol} - f_{tm}}{f_{tm}} \right] \frac{d_{asb}}{h} \tag{4.11}\]

The flexural cracking moment, based on equation 4.7, can then be expressed as:

\[M_{cr} = f_{tm} bh^2 \left[ 0.292 + \frac{0.667h_d}{h} \left[ \frac{f_{tol} - f_{tm}}{f_{tm}} \right] - \frac{h_d d_{asb}}{h^2} \left[ \frac{f_{tol} - f_{tm}}{f_{tm}} \right] \right] \tag{4.12}\]
Comparison of calculated and test recorded capacities are shown in Table 4.2. As seen from this Table, the computed values are very close to the tested-values. Therefore, the foregoing method of analysis can be recommended for design practice. Detailed calculations of composite veneer panels tested are shown in Appendix A.

4.1.2 **Flexural Cracking Strength of Granite Paver**

The presence of two layers of asbestos-cement sheet with their fiber directions oriented perpendicular to each other modifies the stress distribution assumed in section 4.1.1. Noting that at failure, the bottom layer breaks along the longitudinal fiber direction (loaded perpendicular to its fibers) and which has a lower strength than the upper asbestos-cement layer (loaded in the direction of the fibers). The neutral axis lies in the vicinity of the granite-densite interface, the following assumptions can be made for the analysis methods:

(a) the stresses in the tensile zone can be represented by a constant stress $f_{tot}$ across the bottom layer and a constant stress $f_{tot}$ across the upper layer (Figure 4.6 (c)); and

(b) the tensile contribution of the upper asbestos-cement layer can be represented by a force:

$$T_{asb} = (f_{tot} - f_{tot})A_{asb}$$

where $(f_{tot} - f_{tot})$ is the difference between the flexural tensile strength of asbestos-cement (densite) loaded in the longitudinal and transverse directions (Figure 4.6 (d)).
The location of the neutral axis at the cracking stage in this case is given as follows:

\[ \frac{x}{h} = \frac{1 + \frac{h \cdot d_1}{h} \left[ \frac{f_{tol}}{f_{tot}} - \frac{f_{tot}}{f_{tot}} \right]}{2 + \frac{h \cdot d_1}{h} \left[ \frac{f_{tol}}{f_{tot}} - \frac{f_{tot}}{f_{tot}} \right]} \]  

(4.13)

Figure 4.6. Assumptions for the Analysis Method (granite pavers):
(a) cross-section; (b) strain diagram; (c) assumed stress distribution;
(d) simplified representation.

The coefficients for the strengthening of a section in tension and compression are given as:

\[ \psi_t = \frac{h \cdot d_1}{h} \left[ \frac{f_{tol}}{f_{tot}} - \frac{f_{tot}}{f_{tot}} \right] \]  

(4.14)

\[ \psi_c = 0 \]
The coefficients for calculating the flexural cracking moment of granite pavers can be established as follows:

\[ A = 0.292 \quad ; \quad B = 0.667 \quad ; \quad \gamma = 0. \]  

\[ \sum \psi_i \delta_i = \frac{h}{d_i \cdot d_{asb}} \left[ \frac{f_{tol} - f_{tot}}{f_{tot}} \right] \]  

(4.15)

and the flexural cracking moment can then be expressed as:

\[ M_{cr} = f_{tot} \cdot bh^2 \left[ 0.292 + \frac{0.667h}{d_i} \left[ \frac{f_{tol} - f_{tot}}{f_{tot}} \right] - \frac{h}{h^2} \left[ \frac{f_{tol} - f_{tot}}{f_{tot}} \right] \right] \]  

(4.16)

Calculated results and tested results are compared in Table 4.3. Detailed calculations are given in Appendix B.

4.2 Determination of Safe Service Load (in Flexure)

Service loads are defined normally based on prescribed load factors. Building codes normally prescribe different load factors for dead and live loads together with various load combinations. For reinforced concrete structures, the 1983 ACI Code [10] recommends overload factors of 1.4 and 1.7 for dead load and live load respectively. The products of lamination of asbestos - cement and natural stones normally have thin thickness, and hence, the dead weight is insignificant as compared to the imposed live load. Therefore, single value of load factor of \( S_i = 1.7 \) can be used for all loads (dead and live). However, introduction of an additional capacity reduction factor is required to reflect the degree of ductility of the member under the load effects being considered.

While asbestos - cement elements collapse under flexure in a semi-brittle manner at the moment of cracking, reinforced concrete members will not collapse when concrete cracks in the tension zone and even later when the tension steel yields. Normally,
reinforced concrete structures collapse after the rupture strength of steel has been reached, which is 70% higher than the yield strength as used in the ultimate strength design. In order to put the composite products of asbestos - cement and natural stones at par with reinforced concrete in terms of safe and failure loads, it is proposed to consider an additional load factor of $S_2 = 1.35$. The overall proposed factor of safety becomes:

$$S = S_1 \times S_2 = 1.7 \times 1.35 = 2.295 \approx 2.3.$$ 

or more conservatively, a factor of safety of $S = 2.5$ should be used for structures erected on site where adequate technical supervision is not warranted.

Consequently, the following design procedure can be adopted for composite products of asbestos - cement and natural stones:

1. establish the service loads based on the intended use of the products;

2. multiply the service loads by a factor of 2.3 for products produced in a controlled type condition or by a factor of 2.5 for products erected on site where adequate technical supervisions are not available to obtain the ultimate loads; and

3. design the products using the given formulae for ultimate capacity.

For products which are exposed to humidity and increased water content, a 20% strength reduction should be considered.

4.2.1 Safe Loading Capacity for Veneer Panels

The uniform linear failure load $w_1$ can be calculated as follows:

$$w_1 = \frac{8M_{cr}}{L^2} \quad (4.17)$$
The failure load per unit area can be derived by dividing equation (4.17) by the width b. Equation (4.17) can be written as:

\[
W = \frac{8M_{cr}}{L^2 b}
\]

(4.18)

After substituting the expression for \( M_{cr} \) from equation 4.12, the following formula can be established:

\[
W_{uv} = \frac{f_{tm} h^2}{L^2} \left[ 2.336 + \frac{5.336 h_d}{h} \left( \frac{f_{tol}}{f_{tm}} - \frac{f_{tm}}{f_{tm}} \right) + \frac{8 h_d d_{asb}}{h^2} \left( \frac{f_{tol}}{f_{tm}} - \frac{f_{tm}}{f_{tm}} \right) \right]
\]

(4.19)

With a factor of safety of \( S = 2.3 \) for a dry condition, and panel production is under factory controlled conditions, the allowable load is given as:

\[
W_{avd} = \frac{f_{tm} h^2}{L^2} \left[ 1.015 + \frac{2.320 h_d}{h} \left( \frac{f_{tol}}{f_{tm}} \right) - \frac{3.478 h_d d_{asb}}{h^2} \left( \frac{f_{tol}}{f_{tm}} - \frac{f_{tm}}{f_{tm}} \right) \right]
\]

(4.20)

For wet conditions, there is a 20% strength reduction, equation (4.20) becomes:

\[
W_{avd} = \frac{f_{tm} h^2}{L^2} \left[ 0.812 + \frac{1.856 h_d}{h} \left( \frac{f_{tol}}{f_{tm}} \right) - \frac{2.782 h_d d_{asb}}{h^2} \left( \frac{f_{tol}}{f_{tm}} \right) \right]
\]

(4.21)

Figure 4.7 shows the ultimate and allowable strength diagram in terms of load vs. span, and which was developed based on equations (4.19) and (4.20).

4.2.2 Safe Loading Capacity for Granite Pavers

Derivation similar to that of veneer panel for failure load per unit area W can be obtained using equations (4.17) and (4.18). The final expression is given as:
\[ w_{up} = \frac{f_{tot}}{L^2} \left[ \frac{2.336h_{d1}}{h} \left( \frac{f_{tot} - f_{tot}}{f_{tot}} \right) - \frac{5.336h_{d1}}{h^2} \left( \frac{f_{tot} - f_{tot}}{f_{tot}} \right) \right] \]

and the allowable load for dry conditions \((S = 2.3)\) is given as:

\[ w_{apd} = \frac{f_{tot}}{L^2} \left[ \frac{2.320h_{d1}}{h} \left( \frac{f_{tot} - f_{tot}}{f_{tot}} \right) - \frac{3.478h_{d1}d_{asb}}{h^2} \left( \frac{f_{tot} - f_{tot}}{f_{tot}} \right) \right] \]

For wet conditions, equation (4.23) becomes:

\[ w_{apw} = \frac{f_{tot}}{L^2} \left[ \frac{0.812}{h} \left( \frac{f_{tot} - f_{tot}}{f_{tot}} \right) - \frac{2.882h_{d1}d_{asb}}{h^2} \left( \frac{f_{tot} - f_{tot}}{f_{tot}} \right) \right] \]

Figure 4.8 shows the ultimate and allowable strength diagram in terms of load vs. span for granite paver supported at four corners.

4.3 Bearing Capacity of Veneer Panels

Failure loads of veneer panels subjected to compression can be calculated using the empirical equation 14.1 of section 14.2.3 of the American Concrete Institute, ACI 318-83 code of practice [14]. The formula gives the ultimate bearing capacity of a wall, under axial compression as:

\[ P_u = 0.55 \phi f_c A_g \left[ 1 - \left( \frac{k l_c}{32h} \right)^2 \right] \]

Equation (4.17) should be modified if it is to take into account the presence of an asbestos-cement layer in the sandwich panels as follows:
\[ P_u = 0.55 + A_m f_{cm} [1 + m \rho] [1 - \left( \frac{k I_c}{32h} \right)^2] \]  \hspace{1cm} (4.26)

where,

\[ m = \frac{f_{ca}}{f_{cm}} \quad \rho = \frac{A_{asb}}{A_m} \]

Considering the nominal bearing capacity as \( P_n = P_u / \phi \), equation (4.18) can be rewritten as:

\[ P_n = 0.55 A_m f_{cm} [1 + m \rho] [1 - \left( \frac{k I_c}{32h} \right)^2] \]  \hspace{1cm} (4.27)

A comparison of calculated nominal failure loads with test results are shown in Table 4.4.
Figure 4.7 Strength diagram for veneer panels in terms of load vs. span developed based on equations (4.19) and (4.20)

Note: The capacities are given for a uniformly distributed load and panel supported on two edges.
Figure 4.8 Strength diagram for granite pavers in terms of load vs. span developed based on equations (4.22) and (4.23)

- **C1**: 1" Granite thickness and 2 layers of 3/8" densite.
- **C2**: 3/4" Granite thickness and 2 layers of 3/8" densite.

Note: The capacities are given for a uniformly distributed load and slab supported on four corners.
<table>
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<th>1.0</th>
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<th>0</th>
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<td>0.425</td>
<td>0.450</td>
<td>0.490</td>
<td>0.670</td>
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<td>0.760</td>
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Table 4.1  The 'A' and 'B' coefficients for flexural cracking moment calculations
<table>
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<tr>
<th>Test Series</th>
<th>Sample No.</th>
<th>Tested Failure Load (kips)</th>
<th>Equivalent Ultimate Uniform Load (tested) (lbs/sq.ft)</th>
<th>Equivalent Ultimate Uniform Load (calculated) (lbs/sq.ft)</th>
<th>Allowable Uniform Load (calculated for dry condition) (lbs/sq.ft)</th>
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<td></td>
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<td>31.37</td>
<td>13.64</td>
</tr>
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<tr>
<td>B4</td>
<td>BMSN1/TPII</td>
<td>0.997</td>
<td>260.68</td>
<td>199.37</td>
<td>86.68</td>
</tr>
<tr>
<td></td>
<td>BMSN2/TPII</td>
<td>0.922</td>
<td>240.52</td>
<td>199.37</td>
<td>86.68</td>
</tr>
<tr>
<td></td>
<td>BMSN3/TPII</td>
<td>0.925</td>
<td>241.30</td>
<td>199.37</td>
<td>86.68</td>
</tr>
</tbody>
</table>

Table 4.2  Comparison of calculated and test recorded flexural capacities for veneer panels
<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Tested Failure Load (kips)</th>
<th>Equivalent Ultimate Uniform Load (tested) (lbs/sq.ft)</th>
<th>Equivalent Ultimate Uniform Load (calculated) (lbs/sq.ft)</th>
<th>Allowable Uniform Load (calculated for dry condition) (lbs/sq.ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-GP1</td>
<td>17.951</td>
<td>2371</td>
<td>2043</td>
<td>681</td>
</tr>
<tr>
<td>1-GP2</td>
<td>14.012</td>
<td>1853</td>
<td>2043</td>
<td>681</td>
</tr>
<tr>
<td>1-GP3</td>
<td>16.957</td>
<td>2242</td>
<td>2043</td>
<td>681</td>
</tr>
</tbody>
</table>

Table 4.3 Comparison of calculated and test recorded flexural capacities for granite pavers

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Tested Failure Loads ( F' ) (kips)</th>
<th>Nominal Bearing Capacity ( P_n ) (kips)</th>
<th>( \frac{F'}{P_n} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>BMW1/TPII</td>
<td>18.658</td>
<td>20.250</td>
<td>0.92</td>
</tr>
<tr>
<td>BMW2/TPII</td>
<td>22.515</td>
<td>20.250</td>
<td>1.11</td>
</tr>
</tbody>
</table>

Table 4.4 Comparison of calculated nominal failure loads with test results for compressive bearing capacity tests
CHAPTER V
SUMMARY AND CONCLUSIONS

5.1 Summary of the Work

The present work is the first attempt in North America to study and evaluate the strength of sandwich systems with asbestos - cement sheet used to produce increased tensile strength of natural stone products. The study emphasized on developing a simple analysis method for calculating the flexural capacity of natural stone composites such as veneer panels and granite pavers which are manufactured in Canada by Atlas Turner Inc. and Bumaco.

The experimental work was conducted on full scale specimens. A total number of thirty tests were conducted. Mechanical properties of asbestos - cement and building stone materials were specified by Bumaco and Atlas Turner Inc.

A practical analysis method was developed for predicting the flexural cracking strength of veneer panels and granite pavers. The development was based on the knowledge of the stress-strain relationship of the tensioned asbestos - cement and on an assumed stress distribution at the cracking stage. The application of the empirical formula (section 14.2.3 of ACI 318 - 83) for the ultimate compressive bearing capacity of concrete walls was verified for veneer panels.

Guidelines for estimating the flexural capacity of veneer panels and granite pavers together with the bearing capacity of veneer panels have been outlined and illustrated by a practical design example shown in Appendices A, B, and C.
5.2 Conclusions

On the basis of the work described, the following conclusions are drawn:

1. The flexural capacities of veneer panels and granite paver in the reversed direction (marble or granite being at the bottom tension side) are 30% and 31% of the capacities in the normal direction (marble or granite being at the top compression zone) respectively. This observation demonstrates that the introduction of one or two layers of asbestos - cement dense in these sandwich panels is of significant improvement in comparison with solid marble or granite used in conventional settings. This sandwich combination with asbestos - cement on the tension side allows the construction of panels of larger span and loading capacity, and lesser sensitivity to breakages during transportation.

2. The results obtained from the flexural tests indicated that the proposed analysis method which is based on the non-linear stress distribution at cracking can be used to calculate the flexural capacity of the composite panels. A safety factor of 2.3 to 2.50 have been recommended. For panels exposed to high humidity or increased water content, a 20% strength reduction should be considered. Also, the design charts of Figures 4.7 and 4.8 which gives the capacity of the composite panels at different span lengths can be recommended for design practice.

3. The bond strength between each different layer of material was sufficient and did not affect the overall capacity of the composite panels.

4. The strength of veneer panels subjected to compression can be calculated according to the present ACI practice for wall design (ACI 318 - 83). However, the validity of this conclusion is influenced by the critical parameter $k_l c / 32h$. The limit for this parameter is:

$$\frac{k_l c}{32h} < 1.0$$
REFERENCES


14. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318 - 83)", American Concrete Institute, Detroit, 1983.


APPENDIX A

CALCULATION EXAMPLES OF FLEXURAL CAPACITY OF VENEER PANELS

Panels BMC1/TPII - BMC3/TPII (Densite at the bottom)

Geometric Properties of the Cross-Section:
\[ b = 23'' \]
\[ h = \frac{7}{8}'' = 0.875'' \]
\[ A_{asb} = 23 \times \frac{3}{8} = 8.625 \text{in}^2 \]

Material Properties:
\[ f_{tol} = 2100 \text{ psi} \]
\[ f_{rmarb} = 2030 \text{ psi} \]
\[ f_{tm} = 2030 + 1.75 = 1160 \text{ psi (} \gamma = 1.75 \text{ for rectangular section)} \]

The Coefficients of Strengthening of Section:
\[ \psi_t = \frac{A_{asb} (f_{tol} - f_{tm})}{bhf_{tm}} \]
\[ = \frac{8.625 (2100 - 1160)}{(23)(0.875)(1160)} = 0.347 \]
\[ \psi_c = 0; \quad \gamma = \frac{t'}{h^*} = \frac{0}{0.7/8} = 0 \]
Location of Neutral Axis at Cracking Stage:

\[
\frac{x}{h} = \frac{1 + \psi_t}{2 + 2 \psi_c + \psi_t}
\]

\[
x = 0.5''
\]

\[
A = 0.5 \cdot \left[ \frac{0(0)}{1 + 2(0)} \right] \cdot \left[ \frac{1 - 2(0)}{6(1 + 2(0))} \right] \cdot \frac{1}{6} \left[ \frac{1 + 2(0)}{2 + 2(0)} \right]^2
\]

\[
= 0.292
\]

\[
B = t \cdot \left[ \frac{0(0)}{1 + 2(0)} \right] \cdot \left[ \frac{1}{3(1 + 2(0))} \right] = 0.667
\]

\[
\Sigma \psi_{ti} \delta_i = \frac{(23)(0.375)(0.1875)(2100 - 1160)}{(23)(0.875)^2(1160)} = 0.074
\]

\[
M_{cr} = f_{um} bh^2 \left[ A + B \psi_t - \Sigma \psi_{ti} \delta_i \right]
\]

\[
= (1160)(23)(0.875)^2 \left[ 0.292 + 0.667(0.347) - 0.074 \right]
\]

\[
= 9171 \text{ lbs - in.}
\]

and the corresponding tested value was \( M_{cr} = 9055 \text{ lbs-in. (average) } \)

\[
M_{cr} = \frac{w L^2}{8}
\]

\[
w = \frac{8M_{cr}}{L^2} = \frac{8(9171)}{(114)^2} = 5.65 \text{ lbs/in.}
\]

\[
\text{Ultimate load per unit area} = \frac{5.65}{23} = 0.245 \text{ lbs/\text{in}^2} = 35 \text{ psf.}
\]
APPENDIX B

CALCULATION EXAMPLE OF FLEXURAL CAPACITY OF GRANITE PAPER

**Geometric Properties of Cross - Section:**

- \( b = 37'' \)
- \( h = 2'' \)
- \( A_{asb2} = 37 \times \frac{3}{8} = 13.875 \text{ in}^2 \)

**Material Properties:**

- \( f_{tot} = 1450 \text{ psi} \)
- \( f_{tol} = 2220 \text{ psi} \)

**The Coefficients of Strengthening of Section:**

\[
\psi_t = \frac{A_{asb2} \left[ f_{tol} - f_{tot} \right]}{bhf_{tot}} = \frac{(13.875)(2220 - 1450)}{(37)(2)(1450)} = 0.0996
\]

\[
\psi_c = 0; \quad \gamma = \frac{t'}{h} = \frac{0}{2} = 0.
\]
Location of Neutral Axis at Cracking Stage:

\[ \frac{x}{h} = \frac{1 + \Psi_t}{2 + 2 \Psi_c + \Psi_t} \]

\[ \frac{x}{2} = \frac{1 + 0.0996}{2 + 2(0) + 0.0996} \]

\[ x = 1.05'' \]

\[ A = 0.5 \left[ \frac{0(0)}{1 + 2(0)} \right] - \left[ \frac{1 - 2(0)}{6(1 + 2(0))} \right] - \frac{1}{6} \left[ \frac{1 + 2(0)}{2 + 2(0)} \right]^2 \]

\[ = 0.292 \]

\[ B = 1 - \left[ \frac{0(0)}{1 + 2(0)} \right] - \left[ \frac{1}{3(1 + 2(0))} \right] = 0.667 \]

\[ \sum \Psi_{ti} \delta_i = \frac{(37)(0.375)(0.5)(2220 - 1450)}{(37)(2)^2(1450)} = 0.0249 \]

\[ M_{cr} = f_{tot} bh^2 [A + B\Psi_t - \sum \Psi_{ti} \delta_i] \]

\[ = (1450)(37)(2)^2 [0.292 + 0.667(0.0996) - (0.0249)] \]

\[ = 71461.8 \text{ lbs} \cdot \text{in.} \]

and the corresponding tested value was 75386.1 lbs-in. (average).

\[ M_{cr} = \frac{wL^2}{8} \]

\[ w = \frac{8M_{cr}}{L^2} = \frac{8(71461.8)}{(33)^2} = 525 \text{ lbs/in.} \]

\[ \frac{w}{b} = \frac{525}{37} = 14.2 \text{ lbs/\text{in.}^2} = 2043 \text{ psf} \]
APPENDIX C

BEARING CAPACITY CALCULATION FOR VENEER PANELS

Panels BMW1/TPII - BMW2/TPII

Geometric Properties of the Cross - Section:

\[ b = 23' \]
\[ h = 0.875' \]
\[ A_{asb} = \frac{3}{8} \times 23 = 8.625 \text{ in}^2 \]
\[ A_m = \frac{1}{2} \times 23 = 11.5 \text{ in}^2 \]

Material Properties

\[ f_{ca} = 8600 \text{ psi} \]
\[ f_{cm} = 16795 \text{ psi} \]
Coefficients for Calculations:

\[ m = \frac{f_{ca}}{f_{cm}} = \frac{8600}{16795} = 0.512 \]

\[ \frac{A_{asb}}{A_m} = \frac{8.625}{11.5} = 0.75 \]

\[ k = 0.5 \text{ for fixed supports} \]

\[ l_c = 52'' \]

\[ P = 0.55A_m f_{cm} \left[ 1 + m \rho \right] \left[ 1 - \left( \frac{kl_c}{32h} \right)^2 \right] \]

\[ = 0.55(11.5)(16795) \left[ 1 + (0.512)(0.75) \right] \left[ 1 - \left( \frac{0.5(52)}{32(0.875)} \right)^2 \right] \]

\[ = 20.252 \text{ lbs. } \approx 20.25 \text{ kips} \]

Ultimate bearing capacity per linear foot = \[ \frac{20.25}{23} \times 12 = 10.56 \text{ k/ft.} \]

and the corresponding tested value was 20.58 kips (average).