SECTION 6

Aggregate Interlock Test System Development

An important mechanism in the performance of PCC pavements is the ability to effectively transfer shear loading across joint systems. In general, this mechanism is accomplished through dowel and aggregate interlock action. In addition to joint performance, aggregate interlock is also very important in the transfer of shear stress across cracks that form in the in the PCC due to shrinkage, durability problems or excessive vehicle loading. An important parameter in the effectiveness of aggregate interlock, especially at larger cracks, is the strength and durability of the aggregate itself. Recent research at the University of Illinois has shown that aggregate interlock is directly related to aggregate strength. Section Five presented the results of the both the static and dynamic strength testing of the aggregate, cement matrix and PCC, where the rate sensitivity of each of these materials was determined. To compare these results with aggregate interlock in PCC, an aggregate interlock test system was designed and constructed for this research. In addition, initial testing was conducted to evaluate the performance of the system.

There were a number of design criteria for the aggregate interlock test system and the PCC tested. First, the system had to be designed such that a test could be set up and conducted in a minimum amount of time. Second, the system had to simulate as close as possible the behavior of a joint and in particular maintaining the crack at a constant crack width during shear loading. Third, the data acquisition and control system needed to be able to control the test and collect the load level and displacements to verify the performance of the testing. Fourth, the PCC had to be as consistent as possible with the only variable being the coarse aggregate.

The research reported in this section is taken from a master's thesis conducted by Richard Ver Strate at Michigan Tech and advised by Dr. Stan Vitton.

1 Introduction and Background

1.1 Introduction

An important aspect of rigid pavement design (concrete pavements) is the design and installation of construction joints, which are used to accommodate initial shrinkage and later expansion and contraction of the pavement due to changes in temperature. Historically, two types of joints have been used in concrete pavements: contraction joints and expansion joints. Contraction joints are placed at regular intervals soon after the concrete has been placed by cutting a partial depth groove in the fresh pavement. As the concrete begins to shrink due to moisture loss, shrinkage cracks will develop at the groove location thus controlling the placement of the cracks at regular intervals. Expansion joints are placed at less frequent intervals in the pavement to handle thermal expansion as the pavement experiences daily and seasonal temperature changes. However, expansion joints are now used less frequently since field experience has indicated that the contraction joints appear sufficient to handle thermal changes. In addition, it has been noted that a possible negative affect of expansion joints is that they may allow the contraction joints, which are located between the expansion joints, to open up resulting in less efficiency in transferring wheel loads across the joint.

The ability of the joint to transfer wheel loads from one slab to the next is an important factor in maintaining the integrity of the roadway. There are at least three important mechanisms involved in the functioning of joints: (1) aggregate interlock, (2) dowel reinforcement, and (3) base support. When the ability of the joint to transfer wheel loads decreases, faulting at that joint occurs resulting in an uneven roadway. When the faulting at the joint becomes excessive the joint must be removed or the entire roadway reconstructed. Consequently, it is important that the design of the joint be considered in pavement design.

The emphasis of this report will be on the development and evaluation of a test system to study aggregate interlock as the mechanism for transferring shear stress at joints in Portland Cement Concrete (PCC) pavements. Aggregate interlock is the interlocking action of the aggregate particles at the joint surfaces, as illustrated in Figure 1.1. Even though most joints are reinforced with dowels to aid in shear transfer, it has been shown by Reinhardt and Walraven (1982) that aggregate interlock still plays a major role in shear transfer. This is especially true for smaller crack widths in which aggregate interlock plays a dominant role. The effectiveness of aggregate interlock is dependent on the composition and surface characteristics of the crack interface. The surface characteristics or roughness of the crack interface may be different for each type of aggregate used in concrete. The greater the roughness, the more protrusions of aggregates, the greater ability a joint within pavements has to withstand the loading of vehicle traffic. It has been found that for similar aggregates of the same hardness, the effectiveness is increased with an increase in particle angularity as well (Colley and Mumphrey, 1967). In the past, research has concentrated on understanding the mechanism of shear transfer across joints in pavements by varying crack width, aggregate size (whether reinforced or not reinforced) and type of loading. However, to better understand how to make a long-lasting pavement, a better understanding of the materials is still needed.

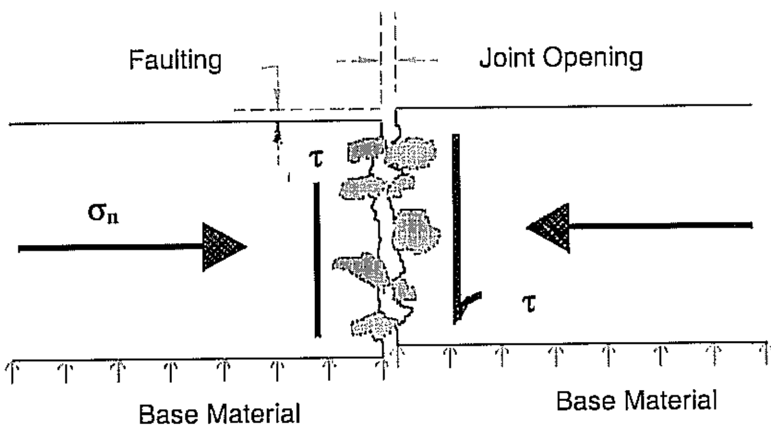


Figure 1.1. Shear transfer mechanism through aggregate interlock.

1.2 Background

In addition to vehicle loading, concrete pavements develop stresses from temperature change, shrinkage, warping, and freeze-thaw conditions. All of these stresses can result in cracks developing in concrete pavements. Once the cracks develop, however, then the performance of the concrete pavement will depend to a large extent on the interaction of the crack surfaces. Common forms of cracks in concrete pavements are transverse cracks, which generally form due to shrinkage (tensile stress) of concrete during curing. As discussed previously, contraction joints are placed in the pavement to relieve the tensile stresses that develop in the concrete. However, cracks will continue to develop throughout the life of the pavement at mid-panel locations due to a combination of wheel loading, temperature changes, and warping and will also affect the performance of the pavement. In both the contraction joints as well as the transverse cracks, aggregate interlock becomes an important factor in the service life of the pavement. For pavements with high traffic levels and traffic loads over time both the contraction joints and transverse cracks will undergo faulting. Faulting of the joint is the break down of the shear mechanisms resulting in relative vertical displacement of pavement sections as illustrated in Figure 1.1.

1.2.1 Previous Research

Due to the importance of aggregate interlock in concrete pavements, research has been conducted to understand the mechanisms involved. The Bureau of Public Roads (now the Federal Highway Administration), Portland Cement Association (PCA), Delft University, and more recently the University of Illinois are some of the institutions that have conducted research in aggregate interlock.

1.2.1.1 Large-scaled research

One of the first organizations to conduct a field study on aggregate interlock was the Bureau of Public Roads during the 1940's and early 1950's (Sutherland, 1956). This field study characterized contraction and expansion joint geometry and performance. Since the publication of this report, researchers have used this information as a basis for the design of aggregate interlock experiments. In the 1960's the Portland Cement Association conducted a large scale laboratory experiment that studied five variables that were considered significant in the performance of joints: (a) joint opening, (b) depth of concrete slab, (c) vehicle loading, (d) base support, and (e) shape of aggregate. Some of the important conclusions reached in this study were as follows (Colley and Humphrey, 1967):

1) as the joint opening increases, the effectiveness of the joint decreases,
2) usually 90% of the joint efficiency was lost during the first 500,000 cycles when the joint opening, test load, slab depth and base material were held constant,
3) joint effectiveness increased with increasing base support,
4) for a given joint design, effectiveness is not influenced by loads less than a critical value,
5) For a given aggregate of the same hardness, effectiveness increases with increase in angularity.

Nowlen (1968) reported that aggregate interlock also improved with increasing aggregate hardness. Additionally, Nowlen reported that early fracture of the joint, which resulted in aggregate pullouts as opposed to aggregate fracture, increased joint efficiency under repeated loads.

1.2.1.2 Small-scaled research

Due to of the complexities of completing a large-scale experiment in the laboratory, research has been conducted on smaller size samples, which were then correlated to field conditions. Delft University and University of Illinois are a few of the institutions that have concentrated their work on the smaller size samples. With a smaller scaled experiment, a larger number of tests can be completed in the lab in addition to effectively controlling test variables.

The study at Delft University used a direct shear test set up to investigate aggregate interlock during shearing (Reinhardt and Walraven, 1982). As shearing develops at the interface the resulting dilation forces the blocks outward. To restrain this motion steel bars were used, resulting in a normal force developing at the interface. Their sample used in the testing was similar to that used by Mattock (1980), who also studied aggregate interlock. The sample size was roughly 60 x 40 x 12 cm, with a shear area of 360 cm^2. The samples were cast in the horizontal position. Two days later the molds were stripped and the samples were placed back in a curing room for the remainder of the time. Before the samples were tested a crack was initiated along the shear plane by splitting forces (knife-edges) at grooves, which were formed on the front and rear faces. The crack width was measured as the sample was being split to control its desired width. Portions of the samples were tested with external restraining bars so that the normal stress could be controlled. The bars were used to vary the external stiffness between different tests, thus controlling the normal stress. The other portion had reinforced bars cast in the samples, which were varied in diameter to study different reinforcement ratios. The samples were loaded with a specified shear displacement rate (monotonic loading) in which loading was applied until a displacement value of 2 mm was achieved. The crack width, shear load, and shear displacement were measured throughout each test.

Walraven (1982) has also done analytical modeling work in studying the shear mechanism of aggregate interlock. The fundamental aspect of this model is that the concrete is represented by a two-phase system, a matrix, which is the hardened cement paste, and the coarse aggregate particles. According to Walraven, the weakest link of the system is the contact area between both the mortar and the coarse aggregate. Therefore, during the curing stages it is at the interface, which fails during shrinkage. An important assumption in this work is that the paste is weaker than the coarse aggregate and breaks down during shearing action. This is illustrated in Figure 1.2, where one surface moves with respect to the other and where the paste that is in direct contact with the aggregate is being crushed and worn down, which allows for the faulting of the joint. In addition, Walraven states that the micro-roughness of the crack, caused by the aggregate particles projecting from the crack plane, dominates the macro-roughness, due to overall undulations of the crack faces.

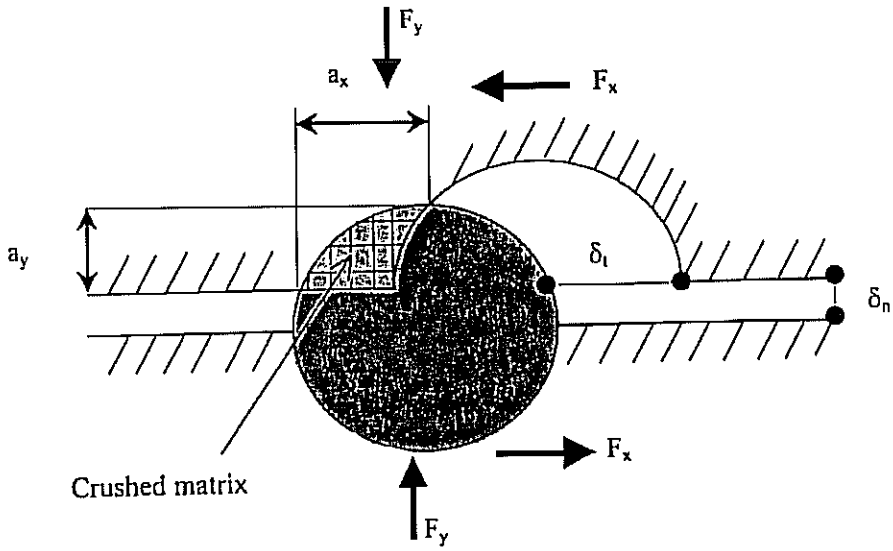


Figure 1.2. Contact stresses on aggregate particle and matrix.

A study at the University of Illinois (Abdel-maksoud, 1997), is one of the latest projects that has been conducted on aggregate interlock. The sample size used in the study was roughly 30.5 x 30.5 x 61 cm, with a shear area of 645 cm^2. The sample is fractured in tension to create the crack interface representing a joint in the field. To produce the crack a groove is made in the sample during casting using 25.4 mm steel strips. The strips are placed at the center of the mold, leaving a reduced area concentrated at midpoint of the sample. The mold has a set of four threaded bars placed at both ends, which were cast in the concrete sample. These threaded bars were used to grip the sample to fracture the sample in tension. At eight hours of cure the sample was then prepared for fracturing. After the eight hours the end plates of the mold were loosened and shims placed between them and the side plates of the mold. During casting the threaded bars were held in place by two nuts on either side of the end plates. The nuts on the outside of the mold were then tightened to place a tension stress in the sample. By tightening one nut at a time, the sample was slowly pulled apart. It was found that when both sides of the mold were tightened at the same time a higher quality fractured surface was obtained as shown in Figure 1.3. The sample was then set aside for the remainder of it's curing time. After the sample has cured for a designated time it was placed in a device that utilized a 100 kip MTS actuator to apply the shear loading. The sample is oriented so that the crack plane is horizontal making it easier for actuator placement. The actuator was then attached to the top half of the sample while the bottom half was securely fixed in place. Using different sized rollers, which are set between the two sample halves, controlled the crack width. Four load cells were attached to the top half of the sample to monitor the normal loads being generated by the shear force. Two types of shear loading were used including fully reversed cyclic shear and monotonic shear loading. The data measured was shear load, normal load, shear displacement, and change in crack width during testing. With the work completed thus far, an important conclusion made was that the aggregate interlock is dependent on the joint opening, joint tortuosity and surface roughness. With large joint openings the resistance is mobilized by dilatancy rather than small joint openings where the resistance is mobilized by trying to shear through the roughness by friction. Therefore, the roughness of the crack interface is dependent upon aggregate type and size.

a) Pulling from one side   b) Pulling from both sides

Figure 1.3. Shape of crack from two methods.

1.3 Research Objective

An important area of research at the Michigan Department of Transportation is the study of aggregate interlock, and in particular the effect that coarse aggregate type has on aggregate interlock. To study aggregate interlock it was proposed that a small-scaled (or bench type) test system be constructed to evaluate the fracture characteristics of different coarse aggregate types under an applied shear stress. The focus of the project is to closely simulate shrinkage cracks in PCC pavements subjected to vehicle loading, investigating coarse aggregate type as the main variable in its behavior. The focus of this research report was to construct and evaluate an aggregate interlock test system for testing concrete samples under a shear load.

The detailed objectives of this study are as follows:

1) Design and construct a structural testing frame that will handle the envisioned loading with its orientation of test sample.
2) Develop and construct a system to restrain the bench type sample used for testing so that it replicates pavement movement in the field.
3) Design and construct a device that would create a shrinkage crack within a concrete sample in the lab.
4) Develop the data acquisition and control system for monitoring and controlling the sample movement during the test.
5) Mix and prepare samples that will be used for running and evaluating the test system.
6) Develop test procedures for operating the test system.
7) Evaluate the test system performance.

2 Experimental Design

2.1 General Design

During the design phase of this research a number of factors had to be considered. One of the major considerations was simulating field conditions in the experimental design. In general the movement of the concrete pavement, after the contraction joints have formed, is due to environmental factors, such as warping from moisture changes, and expansion/contraction and shrinkage conditions from temperature changes. In undoweled pavements, vehicle loading is transferred from one panel to the next across the contraction joint by aggregate interlock, where the opposing joint surface is considered fixed with the exception of movement in the vertical direction. Therefore, in an experimental situation the longitudinal and transverse directions need to remain fixed. These directions are shown in Figure 2.1. In the transverse direction the pavement would see very minimal movement due to temperature changes, which would have little effect on aggregate interlock. Walraven (1982) claims that in this direction all forces are cancelled out due to shear loading, resulting in no transverse movement. Based on this information, the transverse direction was not restrained in this research study, although this motion was monitored. Due to temperature changes, though, there will be movement in the longitudinal direction, which will be greatest between summer and winter. However, for testing purposes it was assumed that relatively little movement occurs during the testing period, thus resulting in a constant crack width during the test. Consequently, the longitudinal direction had to be held constant.

Two 55 kip MTS actuators were available to provide the required loading. One actuator was used to apply the shear force while the other actuator was used to maintain a constant crack width by reacting against any normal force generated from the shearing action of the interface. Unlike the research at the University of Illinois, which had the crack plane placed in the horizontal direction, the interface was placed in the vertical plane so that any debris created from the shearing force could fall downward and be collected. In the Illinois research, the sheared particles remained at the interface.

Vertically orienting the joint better simulates the crack orientation in the field, thus if material worked loose from the shearing action it would work its way through the crack and potentially fall out. To utilize the two actuators in this orientation, one actuator was placed in the vertical direction and the remaining actuator in the horizontal direction. To accomplish this a large structural frame was required to hold both the actuators and concrete sample.

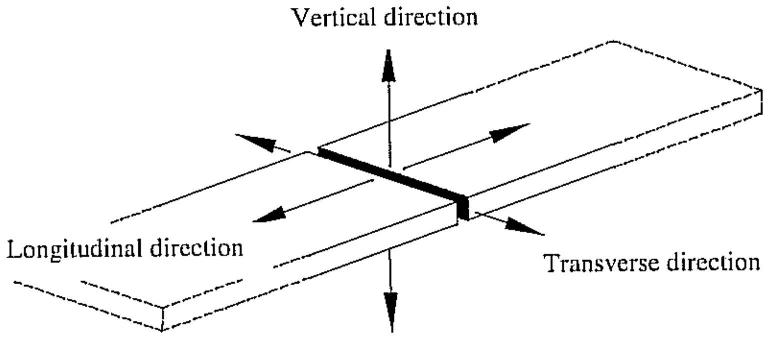


Figure 2.1. Referred pavement directions within text.

2.1.1 Structural Frame

As mentioned previously, the structural frame had to accommodate two MTS hydraulic actuators each with a 55 kip loading capacity. Therefore, the frame design ultimately had to withstand full loading of a single 55 kip actuator in either direction. In addition, the frame had to be designed for other research applications, thus requiring the design to be as versatile as possible. Due to limited funds, it was decided to design the frame so that only a portion had to initially be built, but when additional funds became available, the remaining portions could be constructed. The design of the frame was entirely based on the LRFD steel codes. An overview of the designed frame is illustrated in Figure 2.2.

The frame was designed so that it could be fabricated elsewhere then moved and assembled using bolted connections, which made it ideal for future modifications. With the bolted connections, some parts were also designed with the capability of being adjustable. One of which was a crossbeam on a vertical tower, consisting of two columns, which enabled a single actuator to be placed vertically, as can be seen in Figure 2.2b. It was given roughly a 15-ft travel so that whatever position might be needed in the future additional drilling would not be required. Another consideration was to pre-drill two floor beams, to which the single tower was connected, to handle three additional towers. These two floor beams are connected by three cross beams placed at quarter points. The addition of more towers would then develop a structural cage that could be utilized for many applications in the future. One unique feature of the frame design was that it is self-reacting, meaning that it does not rely on the building floor to supply load reactions, but only the self-weight of the frame. By making it self-reacting, an additional beam was placed along the center points of the three crossbeams, which also aided in mounting the second actuator. This allowed any force that would be generated from the hanging actuator to be transferred into the frame and not against the floor. The second actuator was placed in the horizontal direction to control the crack width of the sample during testing, as illustrated in Figure 2.2a. Two thrust boxes were designed for the horizontal loading, which one thrust box held the horizontal actuator while the second held the fixed-end holder. Discussion on the fixed-end holder is provided in the following section. The structural frame's final length is 30 ft. and the height is 20 ft. The completed structural frame can be seen in Figure 2.3. The dimensions of the frame were designed so that it would fit in the limited space contained in the structural bay area in Dillman Hall at Michigan Tech.

A time consuming task for the project was the drawing and detailing of the frame and sample holders, which were sent to Yalmer Mattila Contracting Inc., of Houghton Michigan, for fabrication and construction. During the final detailing of the drawings, the contractor reviewed the plans and estimated the time and cost of construction. With some exceptions, the frame materials were ordered, cut and welded within four weeks. Installation of the frame took approximately two days. After erection of the frame, it was found that a misalignment occurred on the drilled holes of the two floor beams used for placement of the diagonal bracing to the tower. However, this was easily fixed by re-drilling onsite. As-built drawings for the frame are provided in Appendix 6-A.

2.1.2 Sample Holders

An important design element for the test system was the fixtures or holders for the concrete blocks. The holders needed to be designed so that loading and unloading of the samples was easy and required a minimum amount of installation time. The holders also needed to be designed to restrict movement of the concrete blocks within the holders with respect to the acting force. For simplification of the test, one side of the sample was fixed to the frame in the fixed-end holder while the other side received the shear loading. What allowed the fixed-end holder to be fixed to the frame and still allow the capabilities of monitoring and controlling the normal stress, which is developed from the shear force, the horizontal actuator was to attached to the same side of the sample as the vertical actuator. Consequently, one actuator was attached to the top of the holder providing the shear load and the other actuator was attached to the end of the same holder, which would then react against any normal loading that is developed due to dilation of the interface. By allowing the actuators to pivot at their ends it would eliminate the need for an elaborate roller system to restrain the movement of the sample, but yet maintain an average crack width throughout the testing.

The resulting design of the holders was two steel boxes that had removable lids for the loading and unloading of the concrete samples. The lids acted as a clamping mechanism to restrain the sample, thus preventing sample movement during a test. A projected face on each end of the two holders would give them the ability to bolt up to the actuators and frame. The design was simplified so that each holder was the mirror image of the other, which assisted in fabrication as well. However, the lids had to be designed differently. The fixed-end holder required only a steel plate that is bolted to the top for restraining the sample. The load-bearing holder on the other and had to be designed to

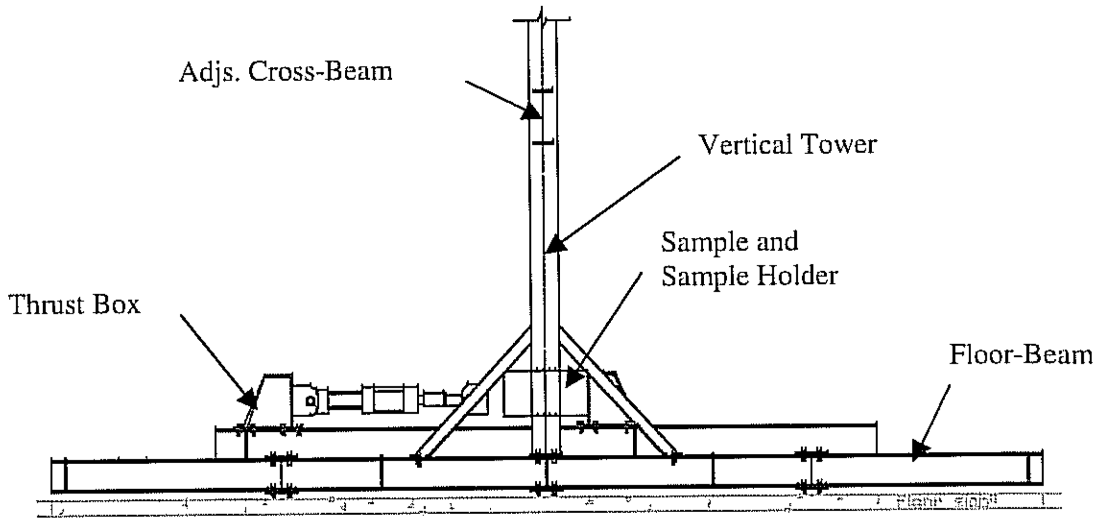


Figure 2.2a. Side view of structural frame.

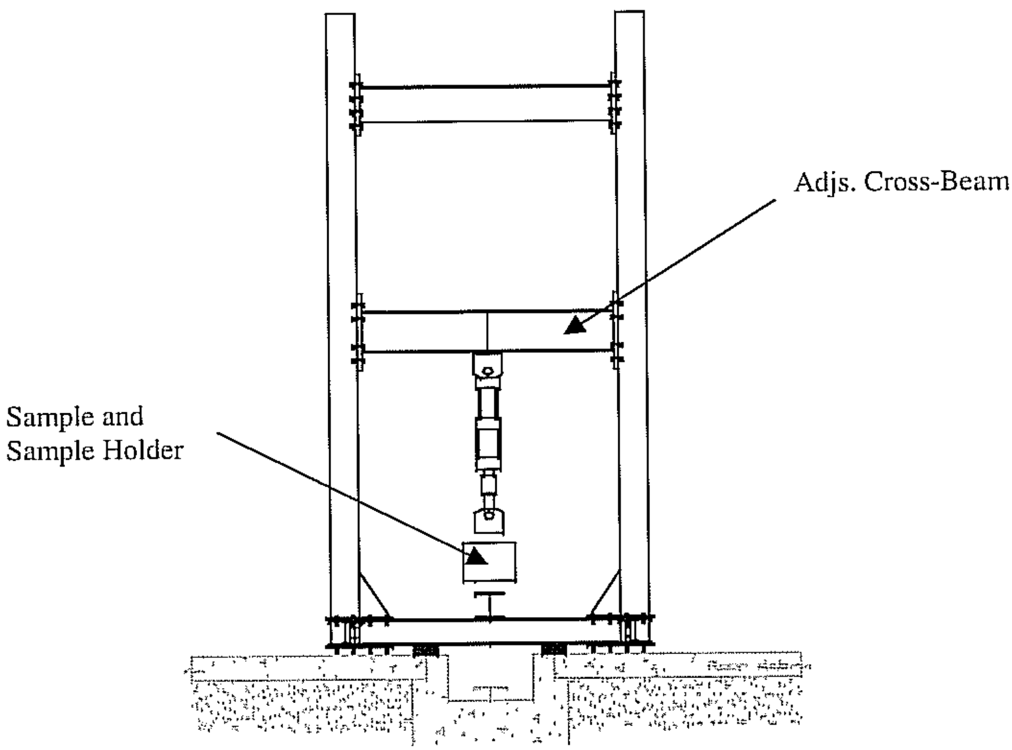


Figure 2.2b. Front view of structural frame.

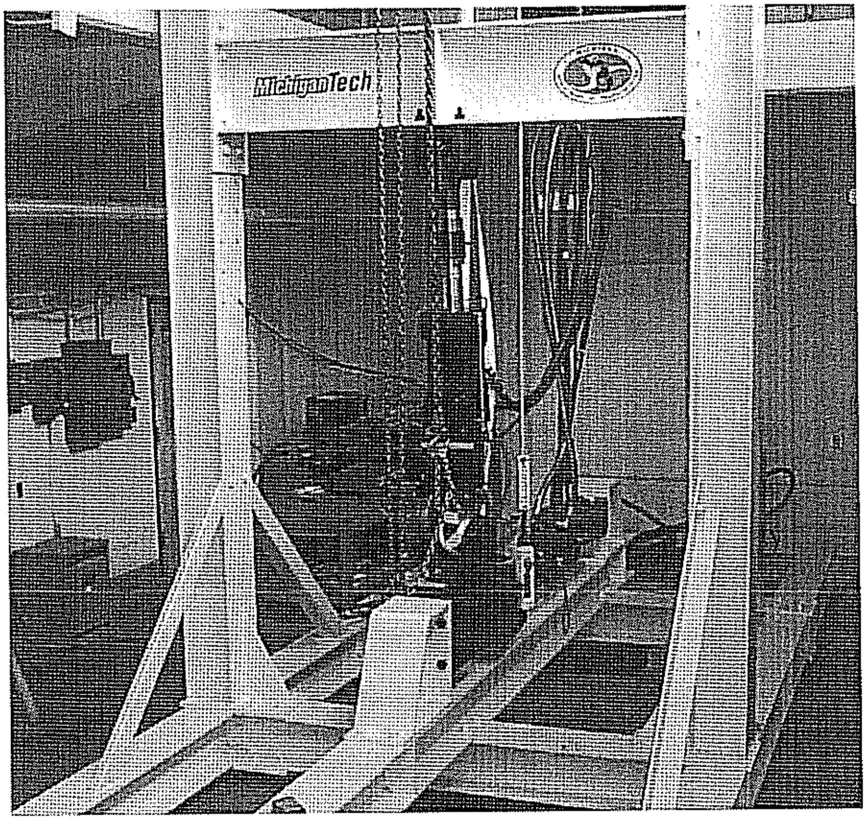


Figure 2.3. Structural testing frame after construction.

apply the shear loading. By looking at the forces on the sample the logical position of the acting force should be placed right along the crack plane. This created a problem in attaching the actuator to the moveable half, which will be referred to as the load-bearing holder, so that it would not get in the way of the fixed half, or the fixed-end holder. Therefore, the actuator needed to be offset from the load-bearing holder so that its line of action would coincide with the plane of the crack. The result was a flanged box connection, which was permanently mounted on the vertical actuator as shown in Figure 2.4a and 2.4b. The flanged box was designed to provide a maximum shear displacement of one inch. Since most pavements have been considered to fail before this point, no need of a greater displacement is foreseen.

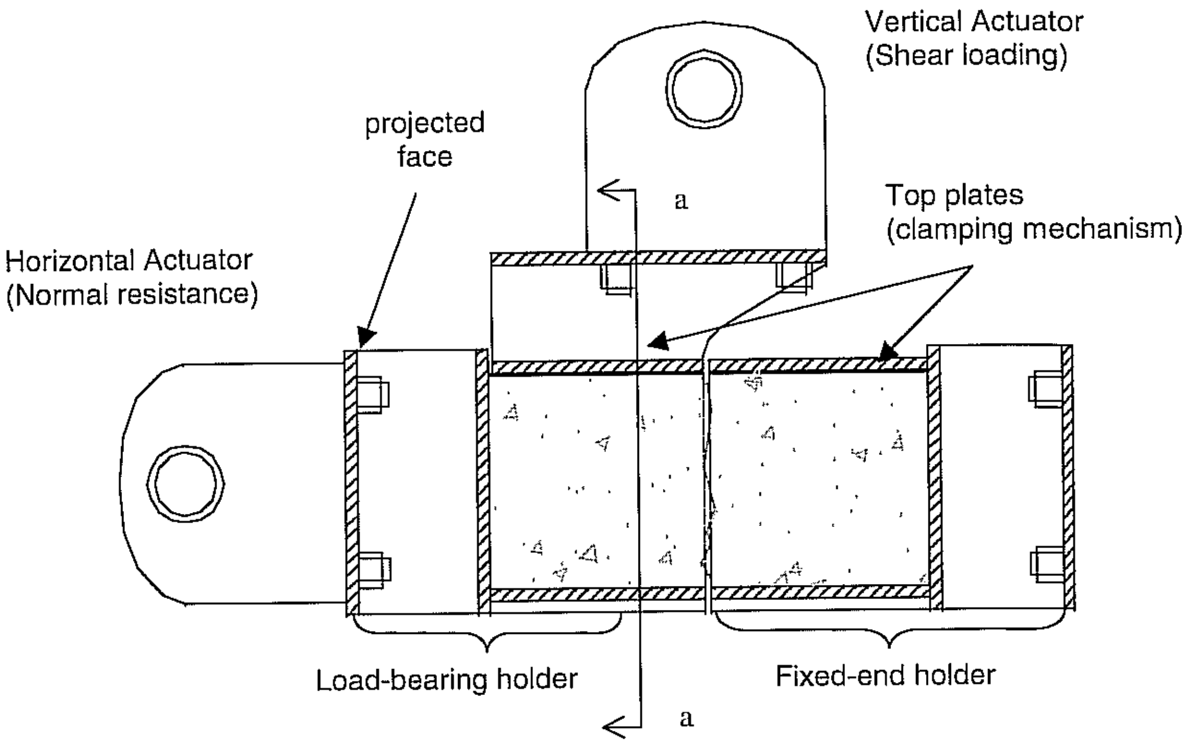


Figure 2.4a. Cross-section view of sample holder.

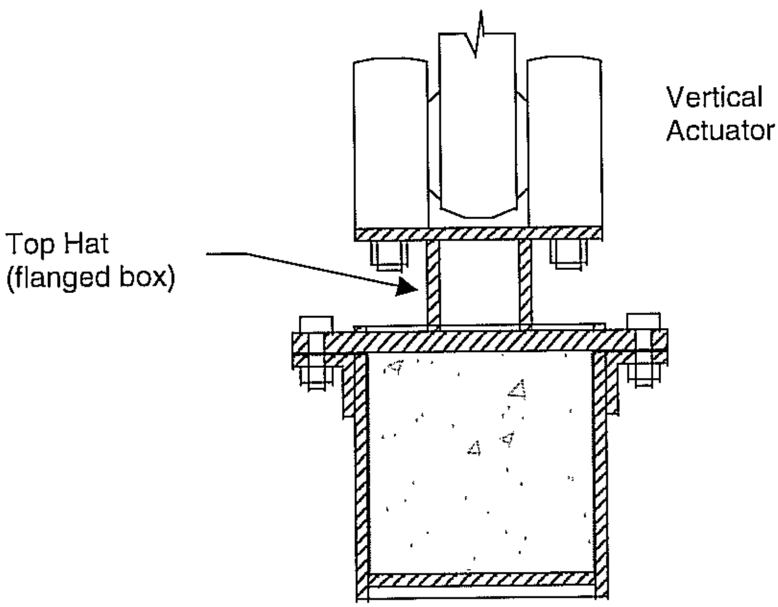


Figure 2.4b. Cross-section view of sample holder (section a-a).

There were some minor problems with the fabrication of the sample holders however. As designed, the two halves were supposed to be identical except for the lids. Due to some recalculations the plate involving the attachment of the sample holder to the thrust box was found to be inadequate for the specified factored loads. The fabricator, Royal Fabrication, Kearsarge, Michigan, was then contacted and notified of the change in dimensioning. Once in place in the lab it was also noticed that alignment of the frame and sample holders were off ¾ of an inch. This was due to the absence of adequate dimensioning on the drawings as well as some incorrect assumptions made on behalf of the fabricator. The offset put the fixed sample ¾ of an inch too far under the vertical actuator thus would create both a x and y force applied to the sample if used as fabricated instead of a pure shear force. The fixed sample holder was then taken back to the fabricator for the end plate to be moved, which made the one discrepancy in the two sample halve-holders. As-built drawings are provided in Appendix 6-A.

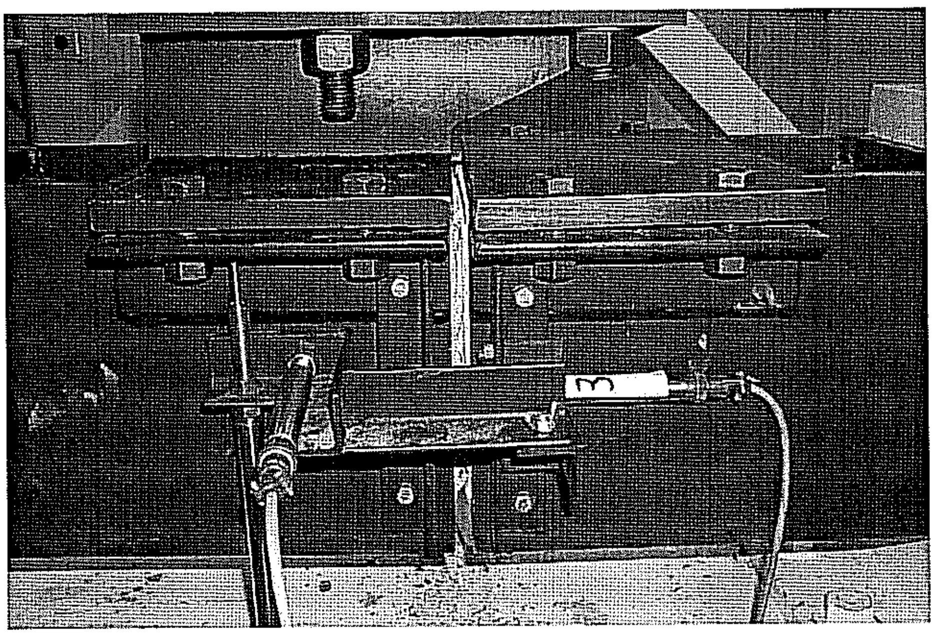


Figure 2.5. Sample holders after construction.

2.1.2.1 Platform

Aligning the two concrete surfaces together was an important consideration in the design of the test system. In addition, the design required that space be available below the sample holders so that a pan could be placed underneath the sample during a test to capture the debris created from the shearing action. To accommodate these requirements, a movable platform was constructed to support the horizontal actuator and the load-bearing sample holder prior to attaching the vertical actuator. This would also ease loading of the sample as well. Since both actuators were attached to the load-bearing holder it relied on both of them to position the sample with respect to the fixed half. The platform was used to align the sample both in the vertical and transverse directions (directions parallel to crack plane). Two guide rails were placed on the platform to keep the movable half inline with the fixed half so that the concrete sample could be positioned back together for testing. The platform was also designed so that it could be moved out of the way during a test to leave room for placement of the debris pan. The platform consisted of a steel frame mounted to a couple of screw type jacks with a piece of ¾ inch plywood fixed to the top. The platform used the weight of the actuator and the reaction of the fixed half to help position it with the two jacks at the center point as seen in Figure 2.6.

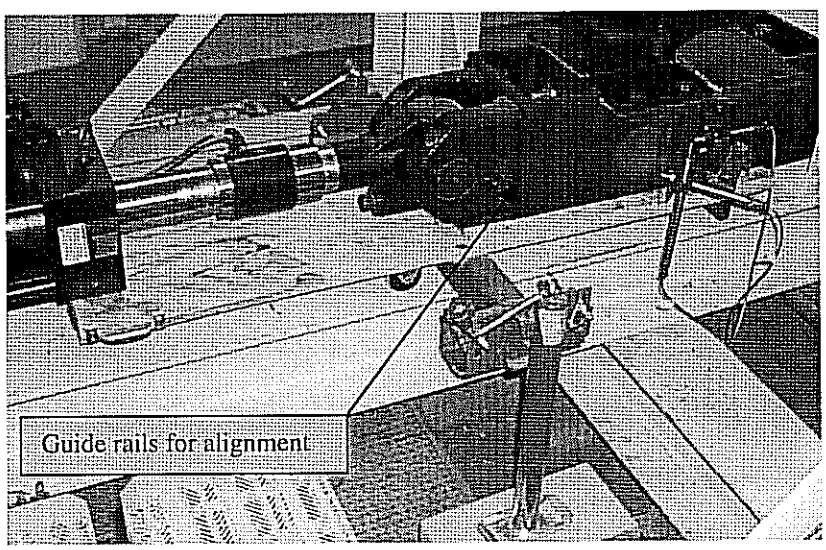


Figure 2.6. Platform for leveling and loading sample.

2.1.3 *Sample Molds*

The size of the concrete sample was based on three criteria. First, the sample weight had to be small enough so that it could be easily handled. Second, the cross-sectional area of the sample had to be a particular size to accommodate the maximum loading capacity of the actuator. Third, the sample size was required to be large enough so that results could be related to field conditions. Based on these criteria, the cross-sectional dimensions are 9 x 9 inches (22.9 x 22.9 cm), with a length of 18.25 in. (46.4 cm) for the sample were selected. While the cross-sectional area was based on the last two criteria, the length was based on the requirement to fracture the sample into two pieces of equal length. An obvious consideration was that it had to be cubic so that the sample could be securely fastened in the sample holders.

The mold for casting the concrete sample was made of steel so that it would last throughout the life of the project. The design of the molds was based on the standard MOR beam mold, but with the modified dimensions. The mold consisted of a base plate with two C-channels for the sides and two stock plates for the ends. The sample mold is shown in Figure 2.7.

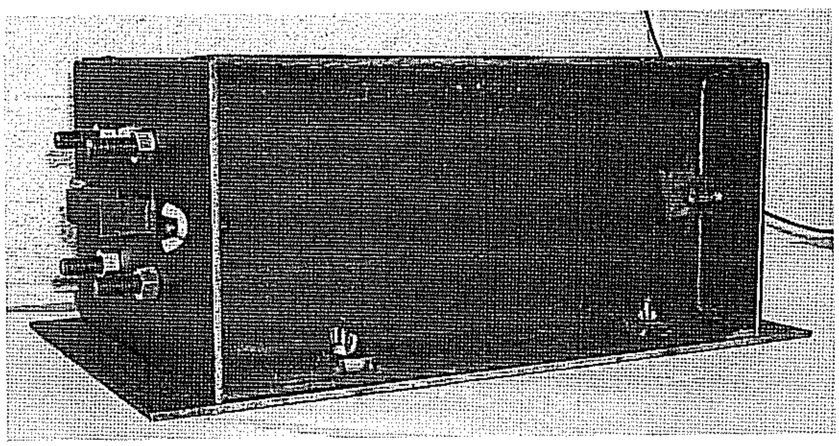


Figure 2.7. Molds for casting test sample.

In addition to the different size, the mold had to be constructed to allow the placement of eight threaded rods, which were to be cast into the sample ends to assist in pulling the sample apart. Therefore, the end plates had four holes drilled to allow for rod placement as shown in Figure 2.8. Four threaded rods were used on each end of the sample. The threaded rods were all cut to an equal length and placed so that six inches are embedded in the concrete sample after casting, having a total length of eight inches. A single nut is placed on each rod, three inches from the mold end plate to help resist in pullout. Two molds were constructed for this study.

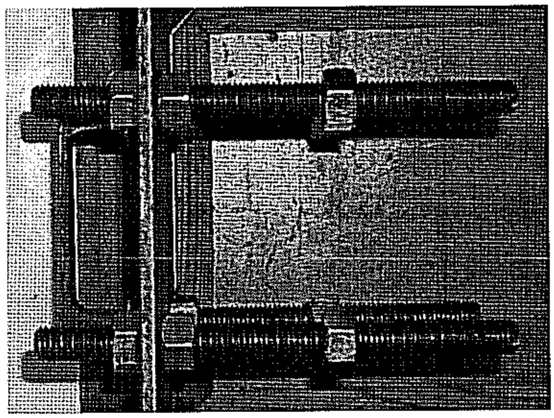


Figure 2.8. Threaded rod location on mold end plate.

An additional modification to the mold was made so that the concrete sample could fit into the sample holders. This was due to the size of the welds needed on the sample holders. To accommodate these welds, the concrete sample had to have beveled edges. To accomplish this, triangular pine strips were used in the corners of the mold as shown in Figure 2.9.

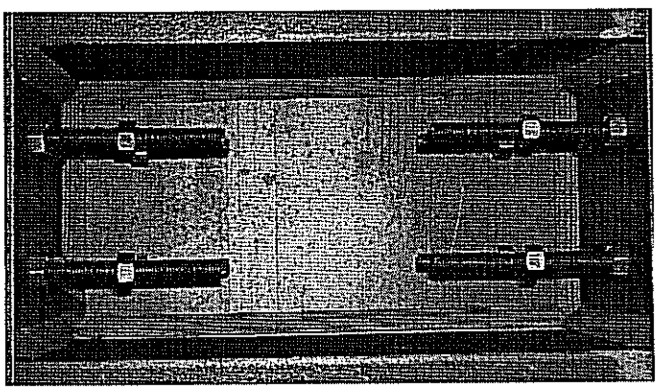


Figure 2.9. Pine stripping placement in molds.

2.1.4 Sample-Fracturing Device

To simulate field conditions as closely as possible it was decided to fracture the concrete sample in tension. In past studies the crack interface of a concrete sample was developed by three point bending or with knife-edges. In this study a fracture by a method that utilizes more of a direct tension was used since the research findings at Illinois on the crack interface indicate that this method produces a more representative crack, and that a pure tension force would better replicate the stresses out in the field. It was found at Illinois that if the sample could be pulled at both ends with equal pressure then the best available surface could be obtained. When both sides of the concrete sample were pulled apart, the crack surface tended to be more planer rather than concave. It is believed that this would better replicate the cracked surface that forms in the concrete slabs in the field during the shrinkage period.

To initiate the crack, a groove was cut around the sample center, reducing its cross-sectional area allowing the crack to be controlled and initiate at the groove. Unlike the method used by Illinois where the groove was created by the placement of steel strips laid into the concrete while it was cast, the method chosen is to cut a groove after the concrete has been allowed to cure. This would allow the coarse aggregate placement within the mix to not be controlled by the steel strips.

A hydraulic powered device was constructed to fracture the sample. In this device two hydraulic cylinders were used to apply a tension load at each end of the sample. The gripping capabilities of the threaded rods within sample made this possible. By connecting the hydraulic lines from the two cylinders together, resulting in equal pressure in each line, both sides could be pulled at the same time. Two ENERPAC cylinders, each with a 10-ton loading capacity, were set to act opposite of each other to apply the tension load. To develop the reaction to the hydraulic cylinders, two plates were welded to the face of a 9" C-channel, spaced so that a sample could be placed between them and some additional length for the threaded rods within the sample. The sample would then rest on this channel during the splitting action. The two plates were drilled with oversized holes to allow threaded rod extensions, that would attach to the sample threaded rods, to pass through and be attached to a plate that bore against the cylinder ends. This device can be viewed in Figure 2.10. A ball bearing was attached to the end of the cylinders to allow the plate to rotate and aid in equalizing pressure among the threaded rods. When hydraulic pressure was applied to the cylinders they pushed against the bearing plates, which in turn pulled on the threaded rods attached to the sample. With adequate curing time the sample could then be fractured into two blocks creating the cracked surface to be tested. Since the sample is to rest on the C-channel during fracture, friction between the concrete surface on the steel platform was of concern. To solve this, pieces of Teflon stripping were placed between the sample and the channel to reduce friction.

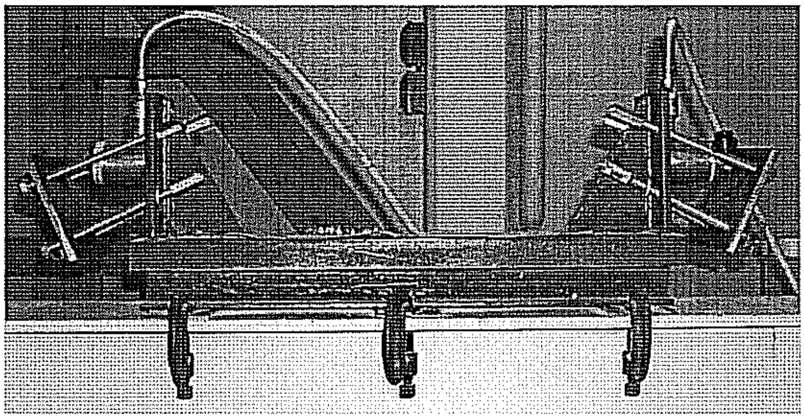


Figure 2.10. Sample-fracturing device after construction.

2.1.5 Control Systems

An important control function of the test was having the vertical actuator operate in load control, while the horizontal actuator operated in displacement control. Thus, the vertical actuator simulated vehicle wheel loading while the horizontal actuator maintained a constant crack width during the vertical loading. As the vertical load is applied to the interface, dilation occurs with a tendency to separate the surfaces outward. Since a concrete pavement remains rigid, with virtually no movement, the dilation develops a normal force at the interface. To simulate this situation the horizontal actuator must generate a normal reaction to the dilation effect of the interface in maintaining a constant crack width.

Due to the nature of this experimental design, both accurate control and high-speed data acquisition and processing was required. The loading of the concrete was accomplished using two MTS 55 kip hydraulic actuators, with each actuator controlled by a MTS 407 digital controller. A control signal was fed into the controllers from a data acquisition and control system interfaced into a PC computer.

The networking of the MTS 407 controllers and data acquisition was accomplished using the software package DASYLab version 5.0. DASYLab has 16 data acquisition channels with two channels of control. The vertical actuator was controlled by the 407 controller by a control signal produced by DASYLab, while the horizontal actuator was maintained in displacement control also by a 407 controller. Thus, the horizontal actuator was controlled in displacement control to maintain a constant crack width, while the vertical actuator was controlled in load control. The convenience of having two actuators gives the capability of controlling the crack width with greater ease, which also supplies the necessary normal loads it takes to control that crack width. Limit levels were set in each controller according to test failure criteria. A failure criterion is discussed in detail in Chapter V. The specific settings of the controllers during the preparation period of a test will also be discussed further in Chapter IV.

Each actuator was equipped with an internal LVDT (Linear Variable Differential Transformer) and a load cell. Additional displacement measurements were made using external LVDTs, which were placed on the sample to capture the sample movement. The LVDTs were rigidly attached to the fixed half of the sample holder to measure the relative displacement between the two sample holders. The external LVDTs were spring-loaded DC sensors (model GHSD), manufactured by Macro Sensors (Pennsauken, NJ). They were placed in all three directions to measure movement during testing, with two half-inch sensors measuring vertical (shearing) displacement, two quarter-inch sensors measuring crack width displacement, and two quarter-inch sensors measuring traverse movement of the sample as shown in Figure 2.11. A Tektronic dual power supply was used to supply a ±15 volts (30 volt) DC excitation source to the LVDTs. The DASYLab

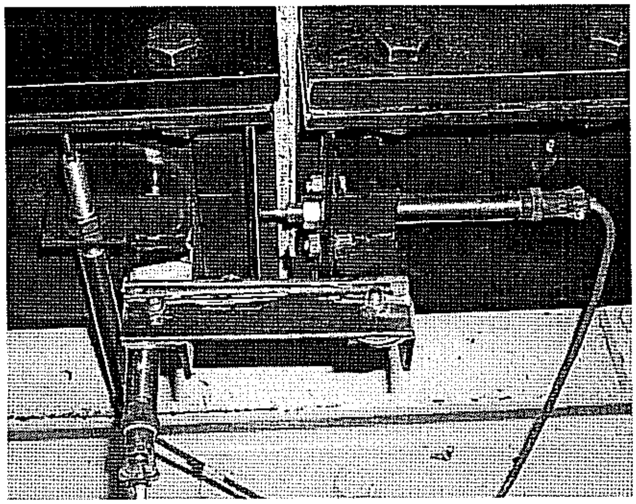


Figure 2.11. LVDT placement on one side of sample holder.

software program was used to collect data from the LVDTs accessing a high-speed 16-bit data acquisition and control board made by Mircostar Laboratories (Bellevue, WA). The Microstar board is a DAP1216a/b board with an onboard Intel 80c186XL processor. The onboard processor is used to both control and collect data independent of the main computers CPU. The DASYLab software and Microstar board were installed on a 100 MHz Pentium lunchbox style computer.

The DASYLab software is programmed to produce a loading signal for controlling the vertical actuator. The load function is a 10 Hz haversine waveform as shown in Figure 2.12. This waveform was sent to the 407 controller as an analog signal via external hook-up of the controller. The loading sequence was conducted at 1 Hz, with nine-tenths of a second at zero loading and the last tenth of a second ramping up similar to the first 180 degrees of a sine wave. The max loading or peak of the load signal is set near nine kips, while a small 100 pound load was maintained on the actuator to simulate zero loading condition to retain the stability of the actuator in load control. This loading waveform was selected to replicate field conditions when a vehicle crosses a joint.

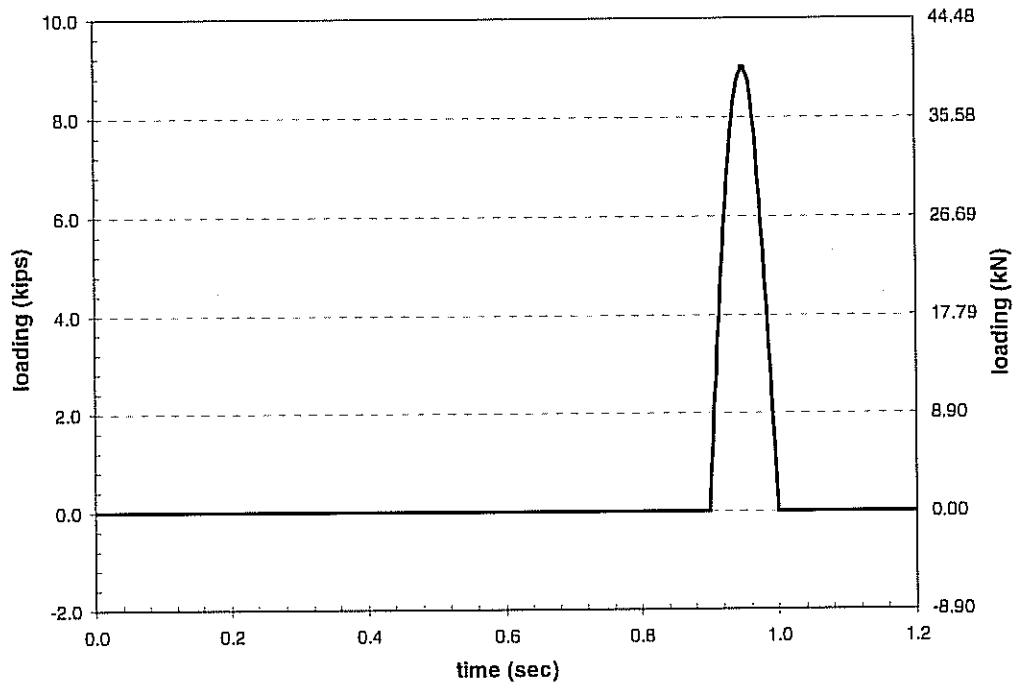
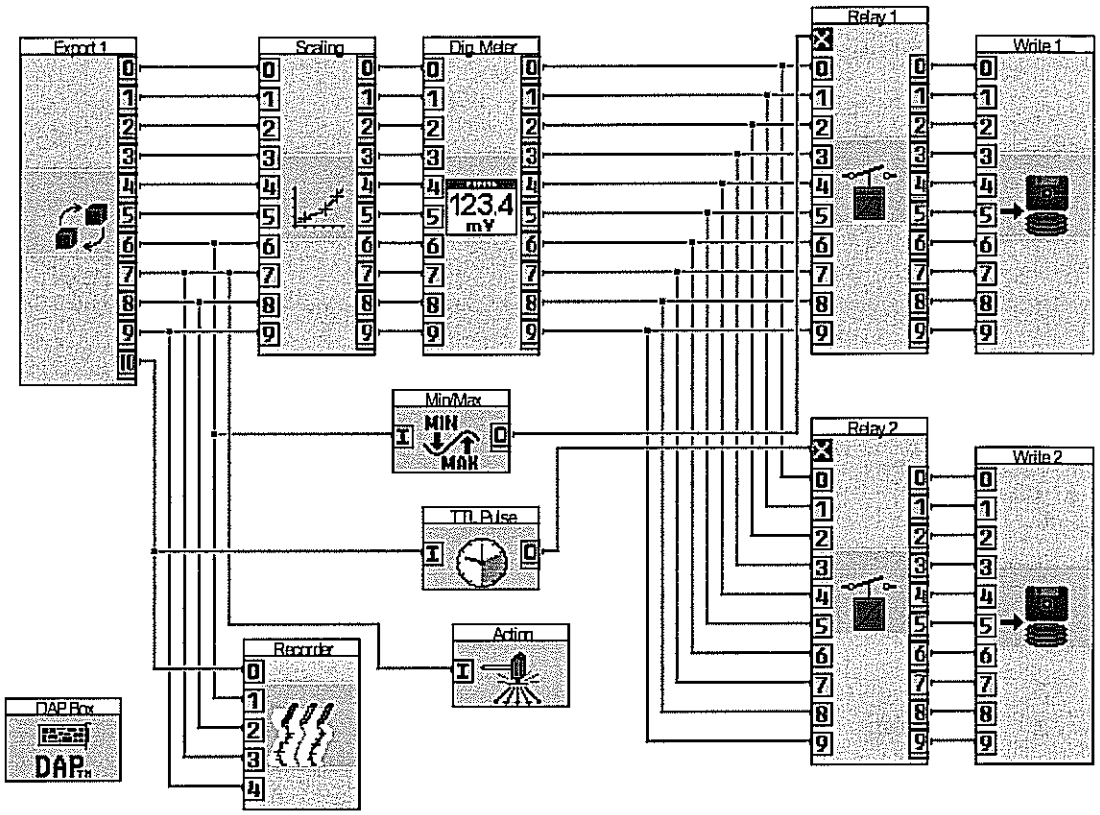


Figure 2.12. Loading control wave for vertical actuator (one cycle).

A total of ten channels of data are collected during testing. Six of these channels were the external LVDTs, while the remaining four are the internal load and displacement readings from each actuator. All ten channels are collected as analog voltage signals by the Microstar board, digitized and inputted to DASYLab. DASYLab then performs a number of functions on the data. First, the LVDT data is scaled into engineering units, from volts to inches. Second, the data is sent to a digital meter module so that the data can be observed during testing. The data acquisition rate is set at 2000 samples per second. However, data is then extracted from this data stream at two time periods. Third, one data set is collected using a maximum function module, which selects the maximum shear load for each cycle (time period 1). When the module determines that a maximum shear load has been obtained, the data is collected from all sensors and saved to a file. Therefore, if the frequency is set at 1 Hz, one set of data is taken each second. Fourth, in addition to this maximum data, every half-hour the data from a complete cycle is saved (time period 2). That is, one second of data, representing 2000 data points, is collected and stored to a file every half-hour the test is running. The full

DASYLab data acquisition worksheet is illustrated in Figure 2.13. All data is collected and stored as ASCII text. However, these two data collection methods create fairly large files, which are manipulated in a spreadsheet program. Consequently, a data-reducing program was developed to reduce these files further. This procedure will be discussed in more detail in Chapter 5.



Action – Stops program when displacement reaches 0.5 in.
DAP Box – Toggle window from the Data Acquisition Processor to DASYLab
Digital Meter – Digital Meter to display displacement readings
Export 1 – Export window to import data from DAP
Min/Max – Reading maximum value from load wave to switch relay for time period 1

Recorder – Displays ongoing measurements in graphic form
Relay 1 – Switches data for time period 1
Relay 2 – Switches data for time period 2
Scaling – Scales displacement data from volts to inches
TTL Pulse – Opens relay at specific time intervals for time period 2
Write 1 – Saves data from time period 1 as an ASCII file
Write 2 – Saves data from time period 2 as an ASCII file

Channel listing

External LVDTs
0 – West side vertical displacement
1 – West side transverse displacement
2 – West side crack width displacement
3 – East side vertical displacement
4 – East side transverse displacement
5 – East side crack width displacement

Internal readings from actuator
6 – Shear load
7 – Shear displacement
8 – Normal load
9 – Crack width displacement

Figure 2.13. DASYLab program ("cyclemax").

3 Materials and Casting Methods

3.1 General

This aggregate interlock research used the same materials and concrete mixing procedures were used as in the previous dynamic fracture research where five different coarse aggregates were investigated. Since this phase of the aggregate interlock deals with the development of the aggregate interlock testing system only two of the five coarse aggregate types were used while also maintaining the fine aggregate constant. The concrete mix design was based on MDOT's Mortar Voids Method. The details of this procedure are described in Section 4 of this report (Hopkinson, 1998). All aggregate preparation, concrete mixing, and casting conformed to ASTM standards with exceptions noted.

3.1.1 Materials

The aggregate types used in the dynamic fracture research were crushed basalt, glacial gravel, two crushed limestones, and blast furnace slag. Of these aggregates the basalt and limestone were used in this study. The limestone was obtained from the Presque Isle quarry while the igneous basalt aggregate was obtained from Bruce Mines, Ontario.

3.1.2 Casting Methods

All concrete was mixed and cured in the concrete lab in the Civil Engineering Department at Michigan Tech. Two beam samples and three cylinders were cast per batch, with a batch size of 2.75 ft^3 (0.078 m^3). In addition, unit weight testing, slump, and air content were conducted.

The beam size was 9 x 9 x 18.25 in. (22.9 x 22.9 x 46.4 cm) for casting of the concrete samples. Standard sized 6 x 12 in. (15.2 x 30.5 cm) plastic cylinder molds were used to form the concrete cylinders. All molds were oiled prior to casting.

Before the beam molds are oiled the threaded rods were placed in the predrilled holes of the mold end plates, as shown in Figure 2.7. The rods were held in place by two nuts on either side of the end plates. Precaution was taken so as to keep oil from being applied to the threaded rods when preparing the molds.

3.1.3 Curing and Stripping

All samples were placed on a cart at the time of casting and then moved into the curing room, which was at 100 % humidity. Cylinders were stripped at 24 hrs of the time of casting. All cylinders were capped prior to testing for 28-day strength. The two beam molds were removed from the curing room prior to fracturing into two blocks. This time ranged from 8 to 12 hours.

4 Experimental Procedures

4.1 General Procedure

An important consideration in this project was the consistency and repeatability of the tests. A significant amount of effort, therefore, was expended in developing the experimental procedures used in this study. These procedures include (1) fracturing the concrete sample to create the crack interface, (2) placing the concrete blocks within the sample holders and repositioning the blocks for correct alignment, (3) initiation of the computer program, data acquisition and zeroing of LVDTs, (4) sample tear down after completion of testing, and (5) reducing the data for analysis.

4.1.1 Concrete Fracturing

After the concrete has been cast and cured for 12 hrs the sample was removed from the curing room and prepared to be fractured. After stripping the molds from the samples, the surfaces of the samples were allowed to dry for a short time for handling purposes. A groove was cut around the center of the sample using a skillsaw equipped with a masonry blade. After one side of the sample was cut the sample was rolled to the adjacent side to make another cut and so on until all four sides were grooved. The depth of the groove was 0.5 inches, which then left a reduced cross-sectional area of 8 x 8 in. at midsection of the sample. A L-shaped straight edge was constructed to produce a straight line and allow the four cuts to match up. Once the sample was cut, it was placed in the sample-fracturing device to produce the crack interface. To eliminate friction between the concrete sample and the steel base of the fracturing device, Teflon strips were placed between them. To attach the sample to the device, the "coupled end extensions" of the device are attached to the threaded rods of the sample as shown in Figure 4.1. The nuts located at the end plates were used to make final adjustments. By hand tightening the nuts at the ends, equal pressure was applied to each of the threaded rods. After insuring that all rods have equal stress on both sides of the sample, the sample was then ready to be tensioned (fractured). Using a hand hydraulic pump, pressure was increased at a constant rate until fracture occurred. The two blocks were then removed and placed back into the curing room to allow further curing until shear testing began. A fractured sample, now referred to as blocks, can be seen in Figure 4.1.

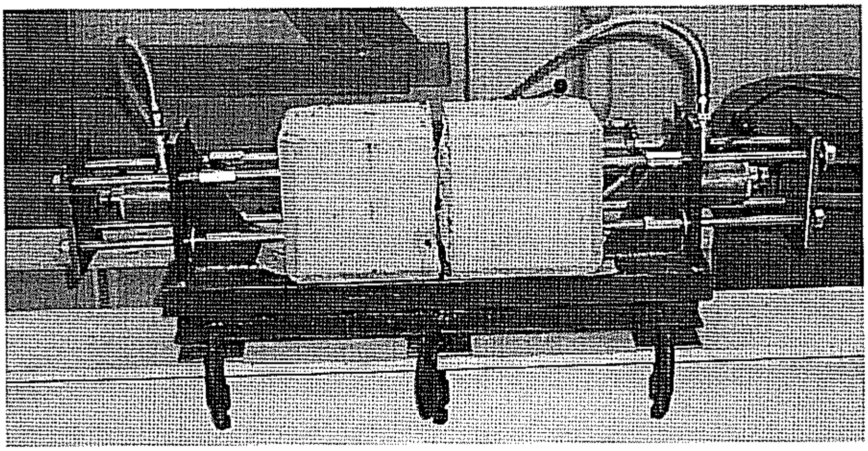


Figure 4.1. Sample after fracture creating two halves.

4.1.2 Test Preparation Methods

4.1.2.1 Installation of concrete blocks

Once the blocks were cured to a designated time, they were ready for testing. Prior to testing, the blocks were removed from the curing room early enough to allow for the sample to dry so that they would be tested in a dry condition. For trial testing the samples were set out a week before testing commenced. When preparing the blocks for testing caution was taken at all times to not disturb the fractured surface. Between the time of fracturing and testing the blocks are kept apart and are not placed back together until the time of testing.

When the blocks were ready for testing the following general steps were performed. First, the moveable platform was placed under the sample holders and horizontal actuator as shown in Figure 2.6. This allows the vertical actuator to be disconnected from the moveable half and tied off to the side for placement of the blocks. Second, each block is then placed in a holder and pushed together by hand, seating the fractured surfaces together in an attempt to minimize the damage to the interface. Third, once the fracture surfaces are together, pressure is applied by the horizontal actuator to close the remaining gap between the two surfaces. To do this an initial load of approximately 500 pounds is applied to the interface. A rubber mallet is then used to vibrate the holders allowing better seating of the interface. As the vibration causes the two surfaces to move closer together the applied load is lost since the actuator is in displacement control. A higher load is then applied in addition to vibrating the holders with the mallet. Again, if additional seating occurs load will be lost. This procedure is repeated until the interface can maintain a 1.5 kip loading without load loss due to vibration. Fourth, the vertical actuator and top plates are then connected to the sample holders. Fifth, nuts are placed on the threaded rods, which were cast into the sample ends, to secure the blocks against the sample holders. Sixth, once this is completed the platform is removed out from under the sample holders. At this point the control and data acquisition system is ready to be set up. Table 4.1 presents a detailed procedure for experimental set up of the concrete blocks.

4.1.2.2 Preparation of control and acquisition systems

After the concrete blocks have been properly installed, aligned and seated, the measurement instruments, computer control, and data acquisition are set up to control and monitor the test. Each LVDT is placed in position and adjusted to zero readings using an adjusting program in DASYLab. Two vertical displacement and transverse displacement sensors are positioned and adjusted so that a positive reading will be achieved throughout the test. At this time the vertical actuator, which applies the shear load, is then switched from displacement control to load control. This is a crucial step in the procedure, since a mistake can damage the sample before it is tested. The next crucial step is setting the crack width, which is done by controlling the horizontal actuator. After this, the crack width displacement sensors are adjusted to obtain positive readings throughout the test. At this point all readings are recorded to establish initial values, which will be used to adjust the data collected to actual displacement readings (adjusting initial value to zero). The main control and testing worksheet is then opened and initialized for testing. A pan is placed under the sample holder to retain any debris that falls during the test. Finally, the test is ready to begin. In Table 4.2 is the detailed procedure for setting up the data receiving equipment

Table 4.1. Sample placement and alignment.

1) Place the moveable platform under the horizontal actuator and adjust to bring actuator in leveled position
2) Pull the vertical actuator away from its hanging position to provide clearance for placement of concrete blocks into holders (SetPnt to -5.0 in.)[B].
3) Retract the horizontal actuator (SetPnt to -5.0 in.)[A].
4) Place blocks in sample holders.
5) Move horizontal actuator forward (SetPnt to -0.30 in.)[A].
6) Align blocks by hand to ensure a closed gap.
7) Change the scale on the horizontal controller to one inch (SETUP #2)[A].
8) Move horizontal actuator forward (SetPnt to 0.5 kips)[A].
9) Tap holders with mallet, then readjust controller (SetPnt to 0.5 kips)[A].
10) Continue seating after tapping sample until a steady 1.5 kips is obtained.
11) Release vertical actuator and reset (SetPnt to -0.2 in.)[B].
12) Insert 5/8-inch bolts to ensure alignment of top hat.
13) Adjust the vertical actuator so that the top hat is sitting directly on the concrete sample. Make sure that only a small load is applied to the sample (SetPnt w/in 1.5 kips)[B].
14) Turn nuts on bolts and tighten down by alternating sides.
15) Place the top plate on other half of sample and tighten bolts in similar manner.
16) Bolt up sample ends with washers.
17) Remove platform from below sample holders.

[A] refers to controller for horizontal actuator
[B] refers to controller for vertical actuator

Table 4.2. Preparing data acquisition and LVDTs to start testing.

1) Open up the program file 'alignment' from DASYLab and start experiment.
2) Set the vertical and side LVDT's to correct positions.
   - 2 Vertical LVDT's – metered reading roughly to 0.0 in.
   - 2 Side LVDT's – metered reading roughly to 0.125 in.
3) Record displacement from vertical actuator (SetPnt @ startvalue)^B
4) Switch vertical controller from displacement control to load control.
   - Adjust loading before applying pressure (SetPnt to -0.60 kips)^B.
5) Adjust pressure on horizontal actuator (SetPnt to 0.0 kips)^A.
6) Set desired crack width (SetPnt to XX in.)^A.
7) Set the crack width LVDT's to correct positions.
   - 2 Crack LVDT's – metered reading roughly to 0.125 in.
8) Stop program and take initial readings of all channels.
9) Open up the program file 'cyclemax' from DASYLab.
10) Place pan under sample to retain debris.
11) Make sure everything is clear to run test safely.
12) Make sure that the 'write module' is saving the data under the correct file names.
13) Begin testing by starting DASYLab program.

^{A} refers to controller for horizontal actuator
^{B} refers to controller for vertical actuator

4.1.2.3 Shut down of control and acquisition systems and removal of blocks

Once the test was complete, the vertical and transverse displacement sensors were removed by swinging the bracket arm away. The bracket arm can be seen in Figure 2.11. The debris from the test was then saved for future analysis. The vertical actuator was then switched back to displacement control so that the sample can be pulled apart without additional damage. The debris that develops within the crack during the test is then collected and saved. The vertical actuator was then set to the beginning displacement value from the start of the test so the platform can be placed back under the sample holders. The vertical actuator and top plates are disconnected and removed. After the nuts on the threaded rods are taken off the sample can be removed. The concrete blocks are removed as carefully as possible minimizing damage to the interface. The fracture surfaces are inspected to observe the nature of degradation, whether it was mostly the pulverizing of the paste or the fracturing of the aggregates. Table 4.3 is a detailed procedure for removing the sample from the test frame.

Table 4.3. Test clean-up.

1) After the test has reached its failure point, which will be defined in the following section, both the actuators and DASYLab should be in a paused mode.
2) Remove LVDT's by swinging arm away.
3) Place debris from pan into labeled bag.
4) Record displacement of vertical actuator (SetPnt @ endvalue)[B].
5) Switch vertical actuator from load control to displacement control
    - Reset controller at proper position before applying pressure (SetPnt to endvalue)[A].
6) Retract horizontal actuator after placing debris pan back under sample (SetPnt to –1.0 in.)[A]
7) Change the scale on the horizontal actuator to full scale (SETUP #1)[A].
8) Brush loose debris off of sample crack interface and place in labeled bag.
9) Extract horizontal actuator to close gap (SetPnt to 0.3 in.)[A].
10) Retract vertical actuator back to start value (SetPnt to startvalue)[B].
11) Place platform back underneath actuator and sample holders.
12) The vertical actuator is shut down so that the top hat can be disconnected. The top plate can also be disconnected at this time.
13) Once the top hat is disconnected turn the vertical actuator back on and retract (SetPnt to -5.0 in.)[B].
14) Strap actuator off to side to clear for unloading.
15) Retract horizontal actuator (SetPnt to -5.0 in.)[A].
16) Unbolt sample ends and remove sample.

[A] refers to controller for horizontal actuator
[B] refers to controller for vertical actuator

If a second sample is ready to be tested, start on step #4 of Table 4.1. If a sample is not ready then the vertical actuator can be released and hydraulic pumps shut down. Note that it is necessary that the platform be left in place under the horizontal actuator when the pumps are not on.

4.1.3 Data Reduction

The data collected for the test was taken at two different time periods. The first time period is triggering the data for collection from all ten channels at every maximum shear load. Since the test is running at a frequency of 1 Hz, a data set is collected every second. The second time period is triggering the data to be collected continually for one second at a specific time duration throughout the test. Due to the large amount of data being collected, the data is reduced for analysis. Using the same software that runs the test (DASYLab 5.0) the data can be sent through a module called "Separate" that will select specific data that is desired and discards the rest. It was selected for the initial trials that the data would simply be reduced by a factor of 10, taking every tenth value while letting nine pass. Both sets of data (full and reduced) are saved on zip disks for future reference. In Table 4.4 is the procedure used in reducing the data using the DASYLab software.

Table 4.4. Data reduction.

1) Once the test has stopped, open program file 'datareduce' from DASYLab.
2) Determine that the 'read modules' are opening the correct files and that the 'write modules' are saving the data under the correct file name.
3) When the data has been completely reduced, the read modules will both indicate EOF (end of file).
4) The DASYLab program can then be stopped and data transferred to a zip disk for analysis.

5 System Performance Evaluation

The system performance evaluation consisted of three main elements. First, failure criteria had to be established such that the test could be conducted in a reasonable amount of time, but yet allow adequate evaluation of the system. Second, actual aggregate interlock tests needed to be conducted and analyzed. Finally, the overall system performance, including the test frame, sample holders, control and data acquisition, and the sample-fracturing device had to be assessed in relation to the test results. The following sections discuss these main evaluation elements.

5.1 Failure Criteria

The general failure criterion for roadway faulting across transverse joints varies from state to state, although a commonly accepted value is 0.5 inches. In this study a failure criteria was also set such that sufficient loading cycles were applied to evaluate the efficiency of the aggregate interlock. Therefore, the test needed the capability of running unattended for a relatively long period of time. One consideration was that no equipment damage would occur if the shear displacement were allowed to increase continually without set boundaries. A second consideration was the time it would take to bring a sample to reach the failure criteria. For the first criteria, a 0.5 inch max shear displacement, was set due to the restrictions of the LVDTs (as well as 0.5 inch being a generally accepted maximum displacement for faulting). The 0.5 inch criterion was programmed into the DASYLab program and the MTS 407 controllers so that the test would automatically stop if the 0.5 inch displacement was reached. Not knowing how long this would take, it was decided to run the tests for a time period of 24 hours to observe how the displacements progressed. In addition, the testing had to be within a time period that allowed for the two samples from a batch to be tested without the influence of additional curing time of the second sample. After running the first sample for 24 hours it was realized that only limited shear displacements resulted and that it would take an exceptionally long time to reach 0.5 inches. Therefore, it was decided to limit the testing to 24 hours, which represented approximately 80,000 cycles, and then to compare the shear displacement at 24 hours of the samples tested.

5.2 Aggregate Interlock Results

To evaluate the aggregate interlock tests, concrete samples were made using the two coarse aggregate types previously described (Bruce Mines and Presque Isle). After casting the samples they were later fractured in tension and cured further until testing. After being placed in the holders and seated, a crack width opening was set. This is an extremely important parameter in the testing. A small joint opening has been determined to be very effective in transferring shear and load through aggregate interlock. However, as the crack opening increases, the aggregate interlock reduces its effectiveness thus resulting in faulting.

Information from the Federal Highway Administration (1989) noted that aggregate interlock is "ineffective at crack widths greater than 0.035 inch." Furthermore, FHWA added, "a smaller crack width, generally 0.025 inch, is considered necessary for satisfactory long-term performance of undoweled pavements." Below is a table from the WSDOT Pavement Guide of seasonal joint openings (1995). To evaluate the system the joint opening or crack width was based on the measure seasonal joint opening for the state of Michigan value as reported in the WSDOT report and shown in Table 5.1. However, in reviewing the technical data from Sutherland (1956), where this information was obtained it should be noted that the 0.024 inch was a minimum value measured in contraction joints. From Table 14 in Sutherland's report the average contraction joint opening varied from 0.024 inch to 0.252 inches for a variety of joints e.g., doweled and non-doweled. These openings are also consistent with calculated joint openings using the WSDOT formulas, which can be up to 0.1 inches. In light of the range of joint openings, it was decided to conduct all the tests at 0.024 to fully evaluate the performance of the system, since this should provide good aggregate interlock efficiency. For example, if the results of the test and evaluation confirmed the effectiveness of the interface at 0.024 inch, then the performance of the test system can be better evaluated.

Table 5.1. Measured seasonal joint openings

State	Contraction Joint Spacing, ft (m)	Expansion Joint Spacing, ft (m)	Measured Seasonal Joint Opening, in. (mm)
• Oregon	15 (4.6)	5280 (1609)	0.034 (0.86)
• Michigan	10 (3.0)	2700 (823)	0.024 (0.61)
• California	15 (4.6)	5280 (1609)	0.025 (0.64)
• Minnesota	15 (4.6)	5260 (1603)	0.043 (1.09)

Three complete tests were conducted in evaluating the performance of the system. These three tests consisted of two concrete samples made from Bruce Mines coarse aggregate and one concrete sample made with Presque Isle coarse aggregate. The reason for only one Presque Isle test is due to the difficulty in producing the crack interface, which is discussed in more detail at the end of this chapter.

General Analysis

In general, all three aggregate interlock tests produced very similar results. As expected none of the tests failed completely due to shear displacement within the 24-hour period at the set crack opening of 0.024 inch. Two samples were allowed to run for 96 hours with only limited shear displacement. Analyzing the shear displacement curve (Figure 5.1), shear failure would not occur until $2.0x10^6$ cycles had elapsed to reach a displacement of 0.5 inch, which would take approximately 23 days of testing. However, it is not known whether by continuing the test the displacement would have developed at the same rate, or would possibly at some time increase its rate to reach the half-inch displacement sooner.

As mentioned previously the results of the three tests were all relatively similar. Therefore, only the results from one of the tests, the Presque Isle aggregate concrete, are discussed in detail. However, the Bruce Mine aggregate concrete results are provided in

Appendix C. The results from the Presque Isle aggregate tests are shown in Figures 5.1 to 5.6. In general, it appears that the shear and transverse displacement have three stages. The first stage appears to be a seating period. The second stage develops as the interface stiffness increases. Finally, the third stage appears to develop as the interface stiffness becomes relatively constant. The different stages are believed to result from the initial seating of the two concrete surfaces under shear load, as well as loose material in the crack interface breaking down initially to allow for shear displacement. During this period the sample appeared to slightly twist as the interface attempts to find the path of least resistance, which is seen mostly in the transverse displacement curves. The movement appears to be more three-dimensional rather than the single one-dimensional movement in the vertical direction, which is generally assumed. As the loose material wears down and the stiffness of the system begins to increase the shear displacement becomes relatively constant but does continue to increase slightly as the concrete interface starts to break down and aggregate interlock becomes less effective. In light of this initial three-dimensional movement the results suggest that the test performed closely to what was expected, i.e., an increase in shear displacement with increase in number of cycles at a constant shear load. In addition, normal stress develops at the interface due to dilation of the interface and the restraint of the horizontal actuator maintaining a constant crack width, i.e., the interface is restrained from movement thus generating normal stresses. However, at the initiation of the shear loading this vertical load is resisted by both shear and vertical normal resistance of the concrete surfaces. As shear loading continues the interface breaks down, thus reducing the vertical normal resistance. This then results in an increase in the horizontal normal stress, which is monitored by the load cell.

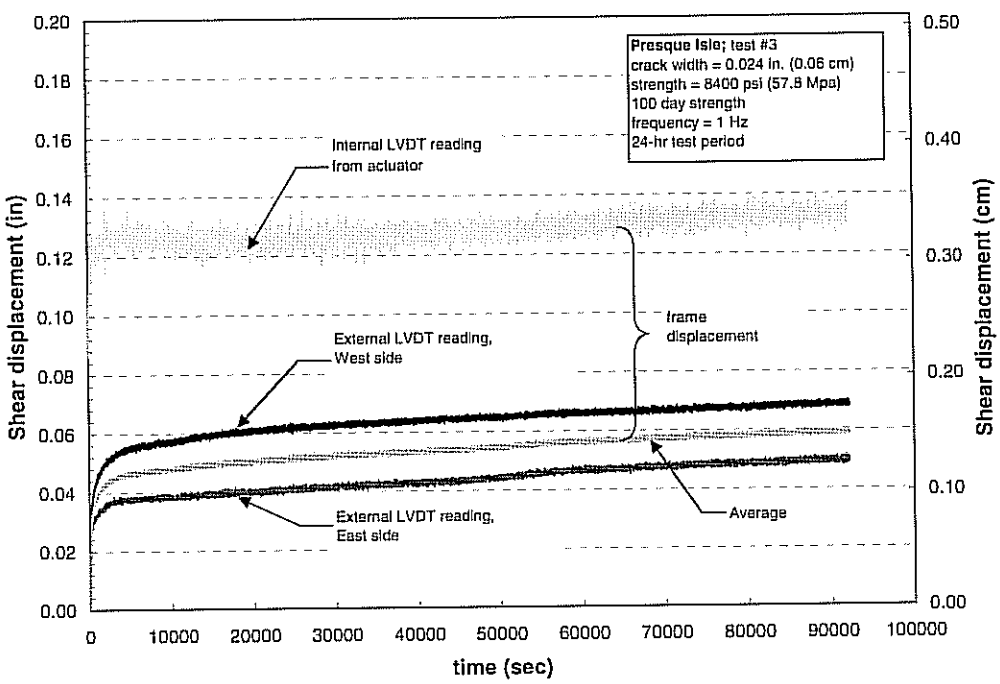


Figure 5.1. Shear displacement from aggregate interlock test (test #3).

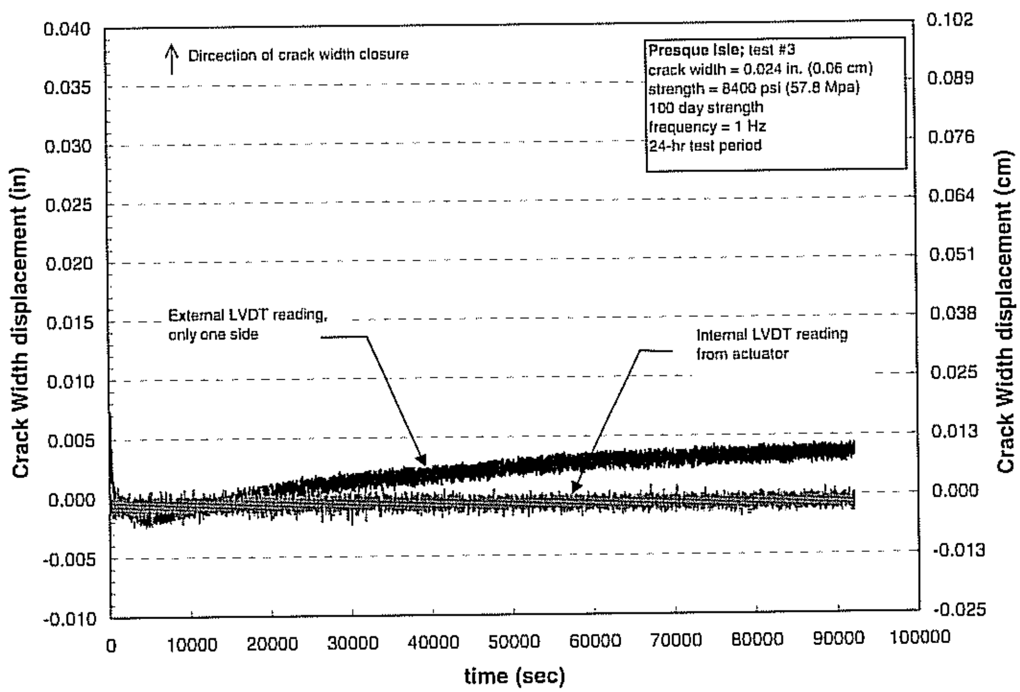


Figure 5.2. Crack width displacement from aggregate interlock test (test #3).

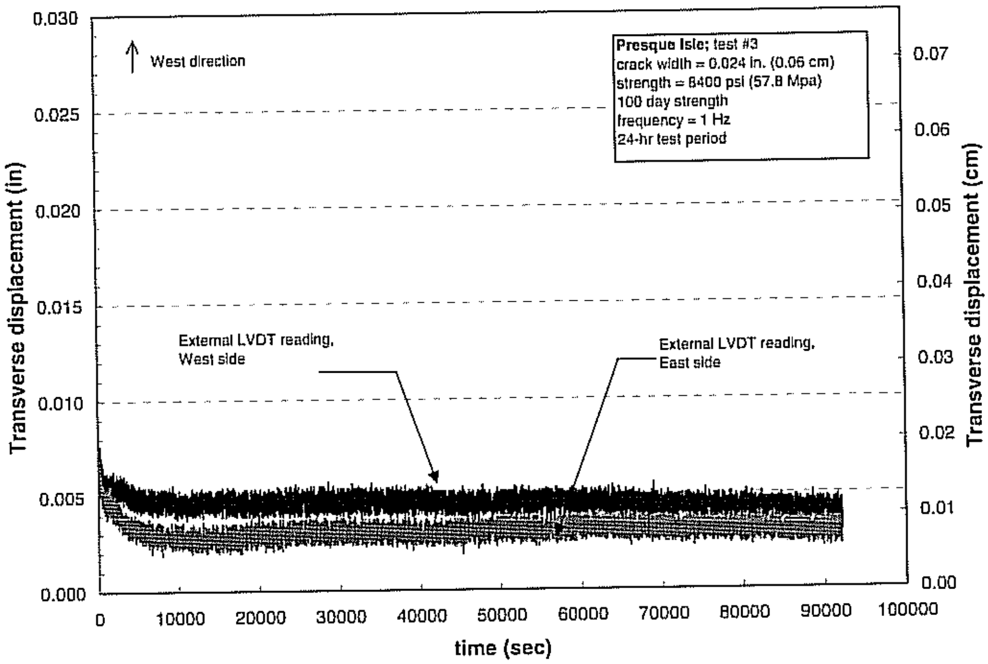


Figure 5.3. Transverse displacement from aggregate interlock test (test #3).

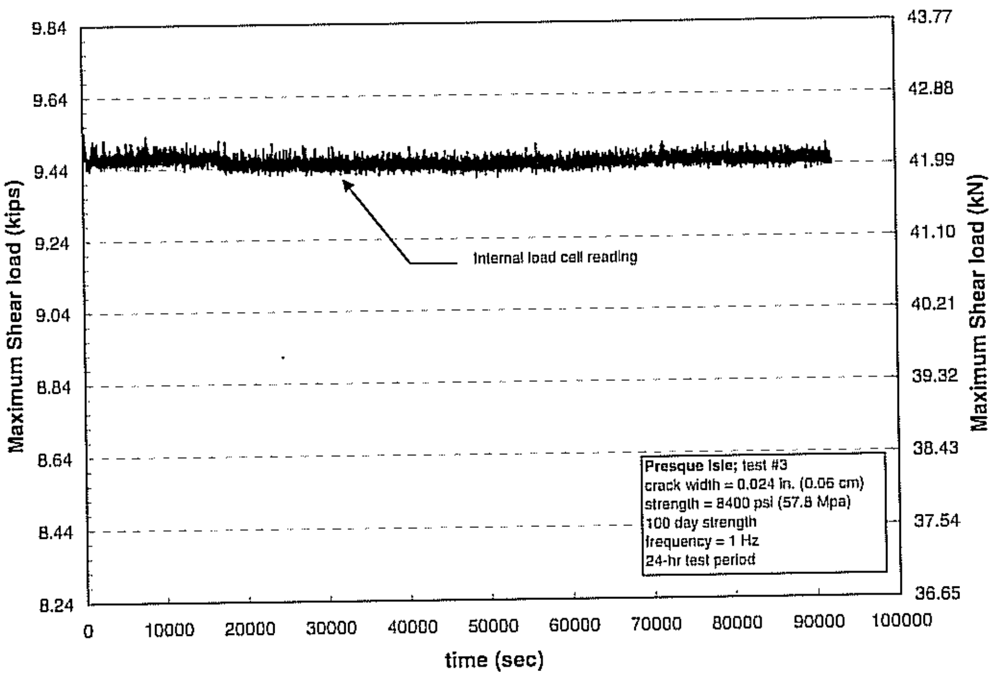


Figure 5.4. Maximum shear load from aggregate interlock test (test #3).

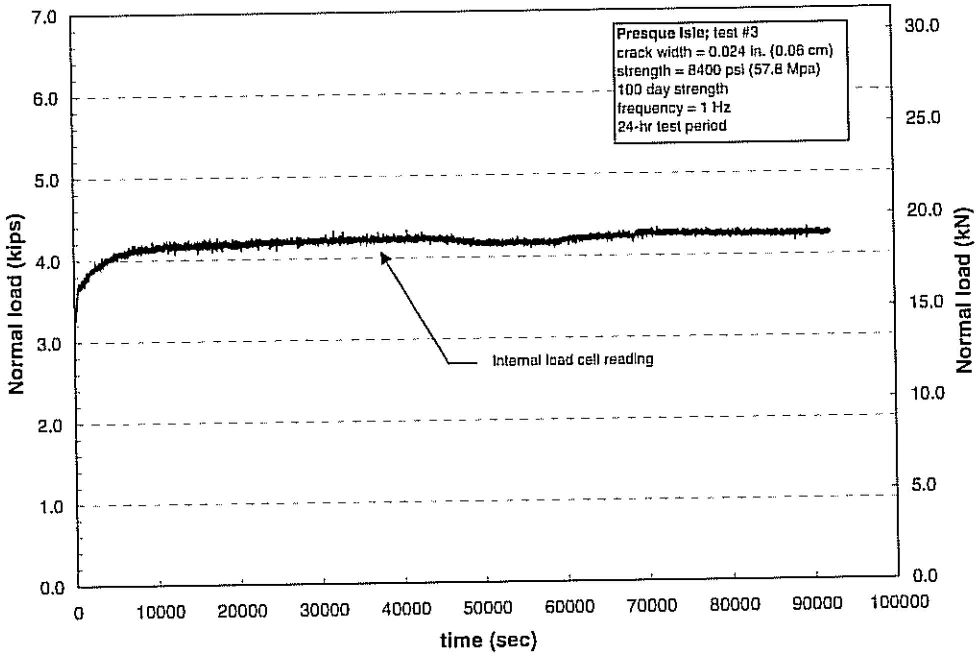


Figure 5.5. Normal load from aggregate interlock test (test #3).

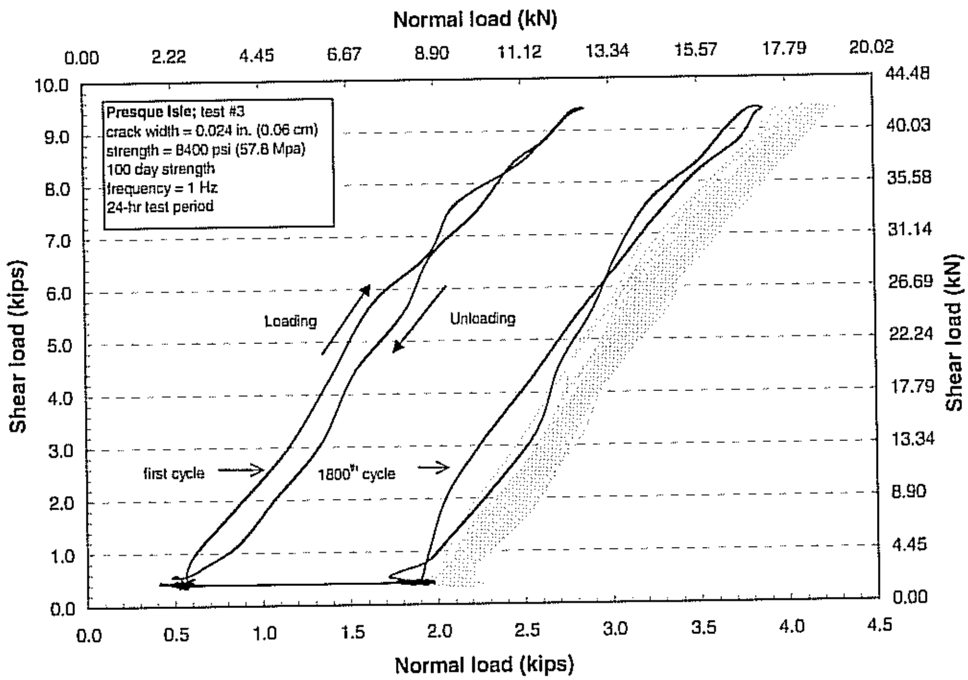


Figure 5.6. Shear vs. normal load for aggregate interlock test (test #3).

5.2.2 *Detailed Analysis*

The data collected from each test included vertical loading, shear displacement, crack width displacement (longitudinal), transverse displacement, and normal load. As mentioned, the results for the Presque Isle are presented in Figures 5.1 through 5.6. A detailed discussion on each of these figures is provided below.

The shear displacements, i.e. relative vertical movement between concrete blocks, were monitored during the test with two external LVDTs and the internal LVDT from the vertical actuator. All three of these readings have been plotted and are presented in Figure 5.1. The difference between the external and internal readings is due to the deflection of the frame. A big part of this deflection is due to the cantilevered position of the fixed sample holder as shown in Figure 5.7. Observation of the displacement of the fixed-end sample holder during the tests drew concern as to whether this holder should be stiffened, thus preventing vertical displacement during testing. A more detailed discussion concerning this matter is provided later in the next section. The difference between the two external LVDT readings was found to be the fault of a slight malfunction of the braces that mounted the sensors to the fixed half of the sample holder. This was later solved and was not a problem in additional testing. For this discussion, the two external LVDT readings have been averaged for comparisons.

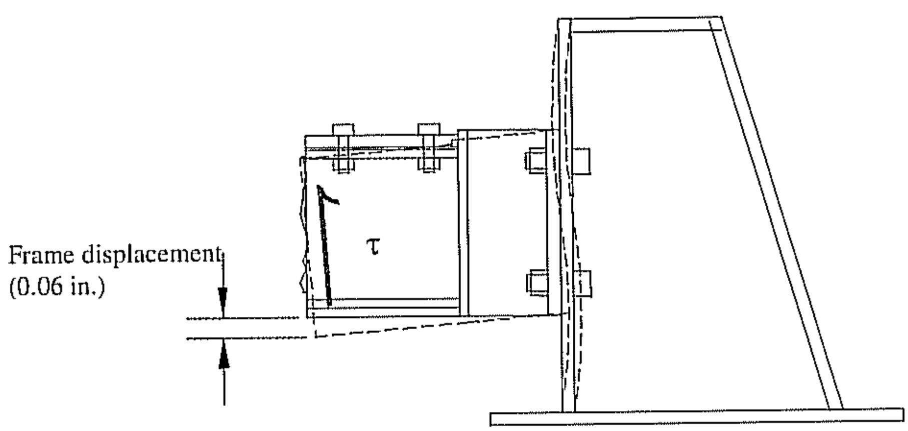


Figure 5.7. Movement of fixed-end holder.

The crack width displacement or joint movement during the test was monitored with a single external LVDT and the internal LVDT from the horizontal actuator. These readings are provided in Figure 5.2. It was planned to use two external LVDTs, but at the time of testing the second LVDT was malfunctioning and had to be sent back to the manufacture. In Figure 5.2 the crack width setting of 0.024 in. represents the origin. Also, note the indicated direction of crack closure on the figure. While the external reading shows that the sample had a rapid change in crack width during the first few cycles, followed by a slight change throughout the test, the internal reading suggests that it stayed constant. An explanation of this effect is due to the sample twisting slightly at the beginning of the test, but with the movement averaging out throughout the area of the crack interface as shown by the internal reading. That is, while on one side the crack width is opening on the other side is closing and the two together average to no movement. The actuators are equipped with swivels heads to allow rotation. While the vertical actuator swivel head is tightened at the end of the sample holder to restrict the motion, the top end is free to move, as are both ends of the horizontal actuator. This allows the sample to twist, but still maintain an average constant crack width. For future testing, four LVDTs will be used to monitor the movement at each corner, thus better describing the movement of the crack width.

Figure 5.3 plots the transverse displacement of the load-bearing block during testing. As discussed above the transverse movement of a sample under a shearing force should not occur due to the cancellation of forces in this direction. The transverse displacement for the tests were monitored with two external LVDTs as illustrated in Fig. 2.11. According to the readings the sample did move in one direction approximately 13 percent of the initial setting of the crack width opening of 0.024 inches. This would seem reasonable because there is no restraint in the transverse direction and the sample was seating itself under the initial 9,000 lb. load. Therefore, the two concrete surfaces moved until they made contact, in which the movement could be in both the vertical and transverse directions. However, it should be noted that the total transverse movement was approximately 0.003 inches while the vertical displacement was approximately 0.06 inches and therefore only represented 5% of the vertical movement. At the point when contact was made the transverse displacement remained constant, proving the validity of that the transverse shear forces cancel out.

The shear load was monitored with the load cell within the vertical actuator. A plot of this data can be found in Fig. 5.4. As noted the maximum shear load for the Presque Isle sample was close to 9.5 kips. The control signal was designed for a maximum load of 9.0 kips, however a calibration offset increased it an additional 400 lb. To be consistent, the other tests were all kept with the same control signal, although the actual load reading varied from 9.2 to 9.4 kips between all tests. As can be observed in Figure 5.4 the loading remained constant throughout the remainder of the test. However, the remaining two tests, which can be viewed in the appendix, appeared to develop some irregularity at the beginning of the test. This irregularity was believed to be due to the PID control settings of the MTS controllers not matching the initial stiffness of the interface. Although, soon after the initial cycles the loading became constant when the stiffness of the interface better matched the set PID control parameters. It should be noted that the control of a closed loop system is dependent on the PID control setting, which is set based on the stiffness of the material being tested. For example, at the beginning of a test when the shearing action is breaking down the loose material the stiffness of the system is changing dramatically. Once the stiffness increases the controllers can then adjust and keep up with the changes. For all three tests the shear load did remain constant after the first 5000 cycles.

The normal load was monitored by the load cell within the horizontal actuator and is shown in Figure 5.5. The data shows that the normal load increases with the initial cycles, indicating that in the early shear cycles the crack interface is breaking down and wearing the surfaces of the concrete so that dilation begins to increase which results in an increase in horizontal normal loading. The normal load did increase slightly over the duration of testing, again similar to what was expected.

All of the data presented in Figures 5.1 to 5.5 represents the peak values from each loading cycle. A plot of the shear vs. normal load data for the complete cycles is shown in Fig. 5.6, where every 1800th cycle is presented starting with cycle one. For the first Bruce Mines and Presque Isle samples the increment was set at 30 min, i.e., DASYLab collected a full cycle of data every 30 minutes or 1800 seconds (or 1800th

cycle). The second Bruce Mines sample had a time increment of 1 hour. The first two cycles, which were recorded (cycle 1 and cycle 1800), are indicated by a darker shade in Figure 5.6 than the following cycles. This graph shows the normal load stabilizing through this time period of testing for a given shear load, which would be indicative of an efficient interface. That is, one that transfers the shear load without inducing an increase in dilation. Again, this would be consistent with a crack width of 0.024 inches, which was determined to be an effective crack width for aggregate interlock regardless of coarse aggregate type.

5.3 Overall System Performance

The overall functionality of the structural frame and holders appeared to work well. Structurally the frame was found to be sound. Both actuators were loaded to 2/3 of the maximum load (55 kips) with no signs of distress or noticeable deformations. One of the major design features of the test system was in utilization and ease of operation. The system for placing the concrete blocks into the holders worked well, even though the concrete blocks were lifted into place by hand, the design was such that they could be put into place and secured within a reasonable amount of time. However, an exception is that the operator had to have the capability of lifting and inserting the concrete block into the holders, which weighed approximately 65 pounds.

The most important part in successfully operating the system is the operator's working knowledge of the MTS 407 controllers and DASYLab program. The procedures that have been included in chapter four rely on the ability of the operator to understand those two systems. The estimated time spent on testing a sample was an overall design criterion from the beginning. The time required for testing a sample is divided into a number of steps and presented in Table 5.2. Overall the time to prepare and test a sample based on a 24-hour test period is 32 hours, while testing took on average 8.25 hours.

Table 5.2. Time commitment for testing a single sample

	time commitment
aggregate preparation	4.00 hr.
mixing	1.00 hr.
stripping and block splitting	1.25 hr.
test set-up	1.50 hr.
duration of test	24.00 hr.
test clean-up	0.50 hr.
Total	32.25 hr.

While the structural integrity of the frame and holders functioned well, it was noticed that the fixed-end holder had noticeable deflections during testing, as discussed in the previous section. This deformation had a maximum vertical displacement of approximately 0.06 inches. In addition to the maximum displacement during the loading cycle there also appeared to be a vibrational response of the system. The effects of this response can be seen in the normal load measured by the horizontal actuator during testing and is shown in Figure 5.8. These responses, the 0.06 inch displacement of the fixed-end holder and the vibrational response immediately following the loading cycle, present concerns regarding system performance. An additional concern can be observed in Figure 5.6 and 5.8 in which the normal load does not return to zero during the 0.9 second no load period of the loading cycle. Although, a 100-pound load is maintained on the vertical actuator (to maintain stability in load control), the corresponding normal should be similar or less than 100 lb. However, from Figure 5.6 and 5.8 it can be seen that at the minimum shear load the normal load is approximately 2,000 lb.

In investigating these concerns it became apparent that the main reason for this occurrence was that the two concrete blocks were coming into contact with each other, as opposed to contact being initiated by aggregate interlock. This became obvious when calculating the geometry of the interface at maximum deflection. From Figure 5.7 it can be seen that the fixed-end holder rotates during testing to a maximum displacement of 0.06 inches. In addition the load-bearing end holder also rotates but at a greater radius of curvature due to the length of the horizontal actuator. Consequently, there will be closure of the crack width at the top of the concrete blocks while the bottom will open up, thus not maintaining parallel surfaces. While it was recognized that there would be some minor rotation of the load-bearing holder, the rotation of the fixed-end holder was not considered and assumed to be stiff enough to prevent substantial deformation. A conservative estimate of the amount of closing and opening can be made assuming the following criteria: 1) a maximum deflection of the fixed-end holder of 0.06 inches, 2) a crack width of 0.024 inches, 3) a rotational arm of the fix-end holder of 15 inches, 4) a rotational arm of the load-bearing holder of 84 inches and 5) that the rotation is along the centerline of the test system. Given these assumptions it was calculated that the fixed-end holder will rotate a maximum 0.03 inches, while the load-bearing holder will rotate a maximum of 0.006 inches due to its longer rotation arm. At maximum deflection the bottom crack width will be at an opening of 0.07 inches while the top is closed at -0.012 inches indicating that the concrete is in contact.

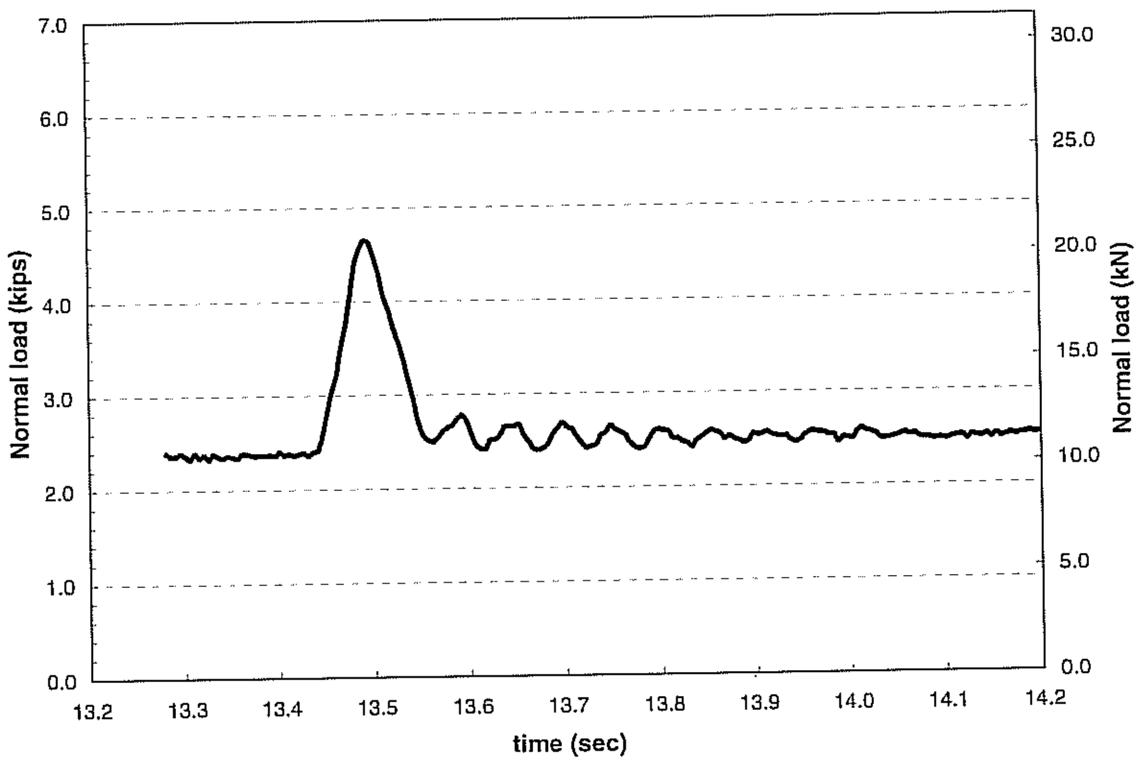


Figure 5.8. Typical cycle of normal load.

Two possible explanations for the normal load not reducing to zero are as follows: 1) a wedging effect of the concrete in contact and 2) the actual seating condition from the initial cycles relatively closes the gap. Although, it is not known exactly why the wedging effect continues after the shear load has reduced to zero within the cycle, but it may be due to conducting the loading in load control as well as the stiffness of the interface, e.g., it doesn't take much of a displacement to relieve the 9 kip vertical load, thus keeping the interface in a weighed condition. Therefore, being in a wedge condition with the vertical actuator in load control allows for the normal load (approx. 2,000 lb) to be maintained at the interface in addition to transferring the vibrational response as shown in Figure 5.8 of the fixed-end holder. However, this may be somewhat realistic for field conditions especially with narrow crack widths such as 0.024 inches. For example, Huang (1993) illustrates a field distress in Figure 5.9, where deflection of the pavement indicates the same situation. It is unknown at this point how this situation may or may not relate to our test situation. Obviously, both the rigidity of the pavement and stiffness of the base material play an important role in the development of this situation.

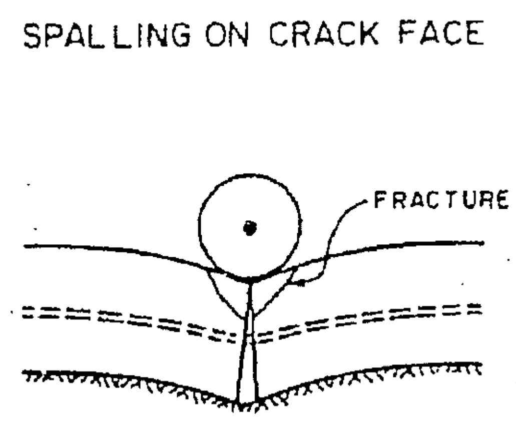


Figure 5.9. Deflections of pavement under vehicle loading

The second possibility is that the shear loading simply places the two surfaces in contact. Once in contact the vertical shearing load generates a normal load across the interface. However, upon releasing the shearing load, which is in load control and requires only an insignificant amount of displacement to release, a portion of the normal is maintained at the interface due to friction. In turn, the surfaces remain in contact.

5.4 Sample-Fracturing Performance

An essential aspect of studying aggregate interlock is producing fracture surfaces in test samples similar to those in field contraction joints, which required the development of the sample-fracturing device described in chapter two. The main task of this device was to fracture the concrete in tension, simulating shrinkage conditions. As discussed in chapter two a major consideration was the time at which to fracture the freshly cast concrete. According to personnel at the University of Illinois, they fractured their concrete at eight hours due primarily to the load limitations of their device. Following this example, eight hours was selected to fracture the initial test sample in this study. The first sample tested in the device was a gravel aggregate PCC, but it did not fracture along the inscribed groove. Instead, failure occurred at one end of the sample as shown in Figure 5.10. The next sand and gravel sample was then tested at ten hours, which did fracture at the correct position. In fracturing the two Bruce Mines concrete samples at the ten-hour period, both had fractured at the inscribed groove. However, this was not the case for the first Presque Isle sample, which fractured in the same manner as shown in Figure 5.10. To avoid this occurring with the second sample, it was fractured at 12 hours, at which it fractured correctly. Two possibilities were reviewed to account for this incorrect fracture. One possibility is that the concrete cross-sectional area at the intended fracture surface is larger than the concrete cross-sectional area at the nut location. The second possibility is that the hydraulic cylinders were not seated against the bearing plates correctly causing additional forces to react on the sample other than tension. The first possibility is ruled out with some simple calculations, proving that the concrete cross-sectional area at the groove was less than the cross-sectional area at the location of the nuts. To eliminate the second possibility, the holes on the bearing plates that allow the extended rods to pass through and attach to the concrete sample were enlarged so that the system can be better adjusted and aligned with the cylinders. If at all possible, the fracture tests should be conducted at the same time. The later the fracture tests are conducted the stronger the mortar becomes. This then may cause more fracture through the aggregate as opposed to pullout failure around the aggregate particles. An additional consideration is that it is likely that the later the fracture test, the less irregular the fracture surface will become. That is, later fracture tests may be straighter, resulting in less aggregate interlock.

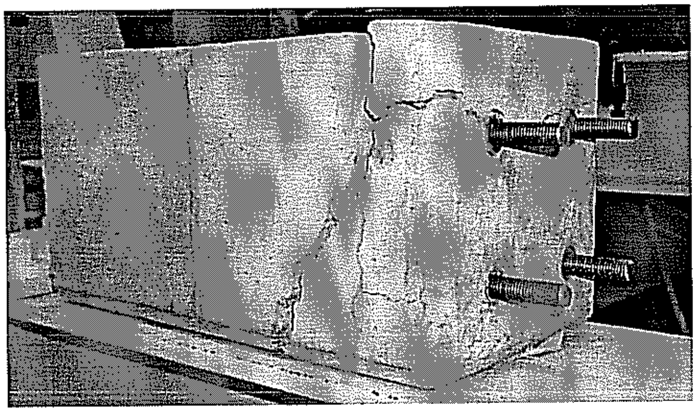


Figure 5.10. Incorrect fracture of sample.

6 Conclusions and Recommendations

The objective of this study was the design, construction, and evaluation of a test system for investigating the mechanics of aggregate interlock,. concentrating on the effect of different types of coarse aggregate. The result of this work was a functional test system. In addition, during the evaluation of the test system a number of learning experiences have occurred that will be used to modify the test system for future research. This chapter provides the conclusions developed in this work along with recommendations for future studies.

6.1 Conclusions

Based on the research conducted in this study, the following conclusions have been reached:

1) The design, fabrication and construction of the structural frame and sample holders functioned well for their intended purpose, providing a frame that was structurally sound and adaptable for other research requirements. In addition, the sample holders performed very well securely restraining the concrete blocks in place during testing. It was also found that the utilization of the holders, such as inserting the concrete blocks and alignment could be accomplished in an efficient manner.

2) The concrete fracturing device performed moderately well in developing the fractured surfaces used in this study. It is believed that the method of fracturing replicates field conditions better than by other methods used by other researchers.

3) The control and data acquisition system performed well together. The control signal was easily transferred to the 407 controller and proved to be an adequate control system.

4) Procedures that were developed allowed for minimal differences to occur between the testing of multiple samples. For example, each sample that is tested would undergo the same methods of handling from the beginning, where the concrete is cast in the mold, to the end, where the sample is removed from the sample holders at the end of testing.

5) The operation of the entire system required minimal operator time although this required a working knowledge of DASYLab and the MTS controllers.

6) It is believed that the test system appears to closely replicate aggregate interlock at contraction joints. However, some concern exists concerning the stiffness of the fixed-end holder, which may generate too much deflection and may need to be stiffened. In addition the twisting of the vertical actuator unit is of some concern as whether the surfaces remain parallel throughout the time of testing.

6.2 Recommendations for Future Work

A number of improvement areas have been mentioned in this paper. In the continuation of the work certain areas will have to be modified. A few of the most important recommendations will be discussed in this section.

An important need in this research is to better understand when contraction joint cracks are formed in the field, i.e., at what time do the stresses in the pavements overcome the concrete strength capacity? Once this is known, a better way of producing the sample in the lab can be achieved. For example, if the stress is continually gaining throughout the life of the concrete in the field then maybe the sample should be set up so that a scaled increase in stress is replicated in the lab until the crack forms. It's believed that both the strength and level of stresses generated in a pavement are increasing during the first few days of curing, but at some point the stresses generated are higher than the bonding force that keeps the concrete panels together and a crack is formed. Instead of a quick break to form the crack in the sample, a slower, more controlled break may be needed. Another benefit would be to replace the hand pump used in the splitting of the sample with an automated pump to achieve a constant loading rate.

It is important in the future work to stay focused on developing better ways in testing the samples. The direction of which the research takes is very critical in the usefulness of the data obtained. The following is the suggested direction that might want to be considered:

1. **Accelerated test conditions:** For a larger testing range and possibly the ability capture how a sample fails, an accelerated test should be considered. Increasing the frequency of loading or increase the amplitude of the applied loading can accomplish this. Both would result in obtaining a greater life history of the sample being tested by staying with the set 24-hour period. The time duration of the test could be altered as well, but

is suggest to stay within a time period so that testing of the second sample within a single batch is not introduced to additional curing time to effect the comparison of the two.

2. **Multiple tests at various constant crack widths:** For a better understanding of the mechanism of shear transfer, various crack width settings should be tested. With the current setting at 0.024 inches the rates of degradation between the tests seems to be constant. Changing the number of contact points by increasing the crack width opening might trigger different characteristics of the aggregate within the concrete to become more visible in the reaction of restraining the applied loading. Testing samples at different crack widths would provide a measure of efficiency of aggregate interlock for each type of coarse aggregate tested.

3. **Changing crack width during test:** A specific joint in the pavement does not sustain a constant crack width throughout it's life. During the course of a year the pavement goes through expansion and shrinkage cycles due to temperature change. Therefore there are times that the pavement joint opening is at 0.024 in. and other times, most likely in the winter months, that the crack width is much larger. The life span of a joint in a pavement depends on this cyclic movement. With the capabilities of the dual actuators in the system a scaled version of this cycle could be replicated. While the vertical actuator is administering the shear loading the horizontal actuator could in fact be cycling at the same time. For instance, if the frequency of the shear loading was kept at 1 Hz then the crack width could open and close with a range of 0.024-0.06 in. within a time period of 12 hours. This would better replicate the conditions in the field.

Appendix A

Detailed sketches for construction

Self Reaction Frame

design based on 50 ksi on rolled steel, 36 ksi on plates

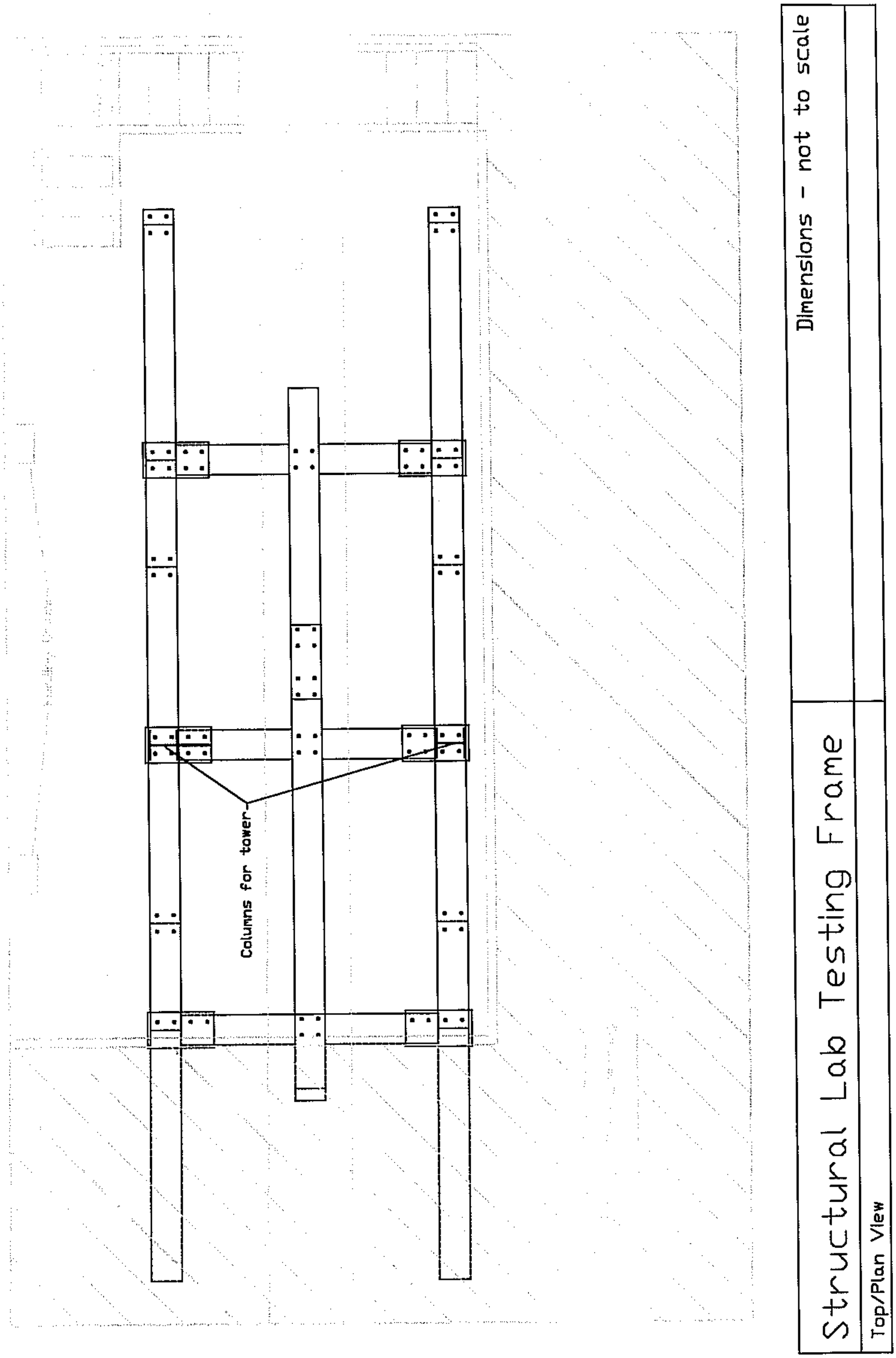
Column and Beam Schedule

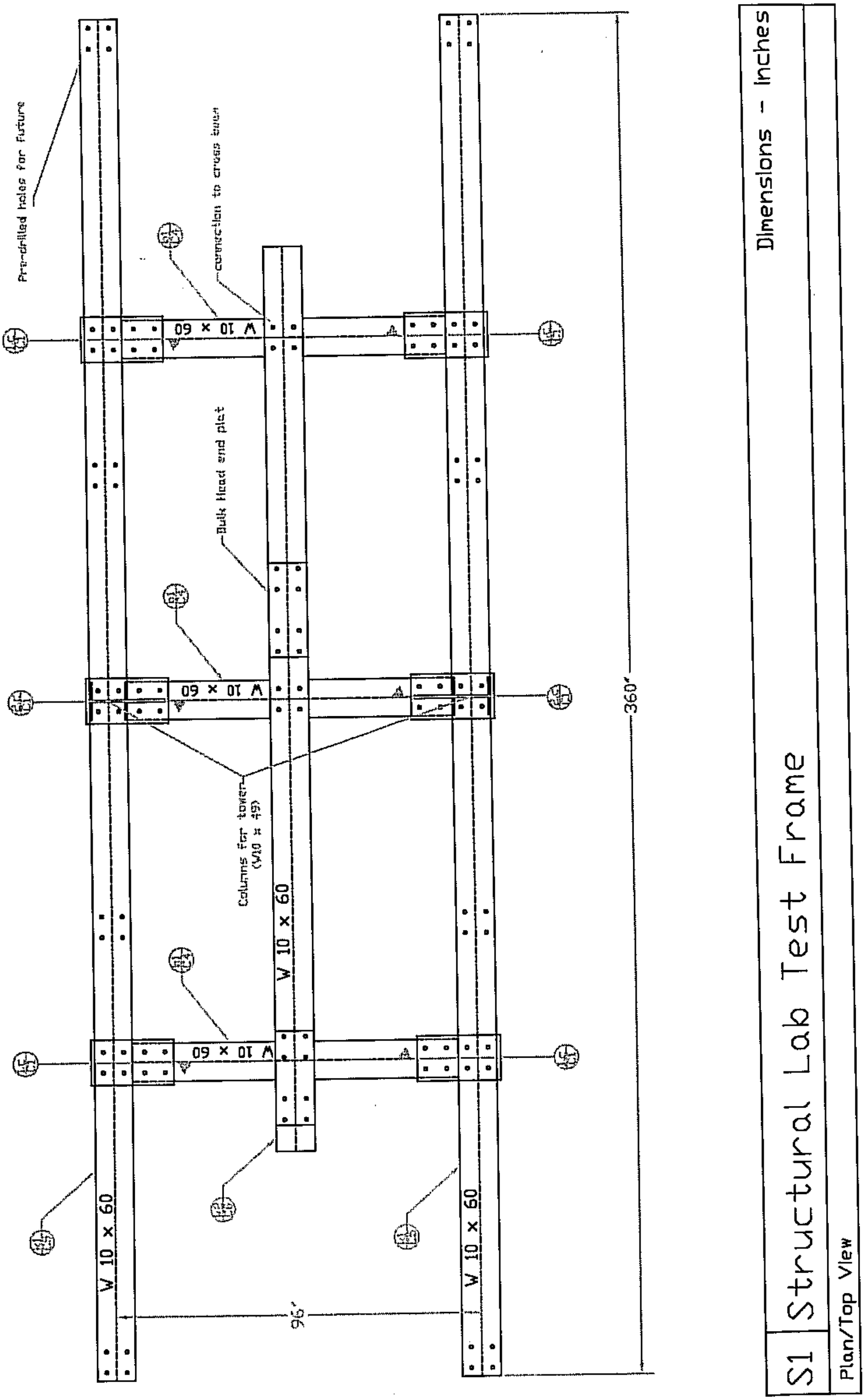
Description	Drawing Detail	Size	Length (in)	no.	bolt holes drilled (/each)	Remark
Column	C7	W10 x 49	240	2	54	
Long. Beams	C5	W10 x 60	360	2	62	
Sample Beam	C6	W10 x 60	240	1	28	
Cross Beams	C4	W10 x 60	94	3	26	
Mid. Cross Beam	C8	W16 x 67	84.5	1	12	
Top Cross Beam	C9	W12 x 26	84.5	1		
Diagonal Brace	C10	L4 x 4 x 3/8	67	4	1	

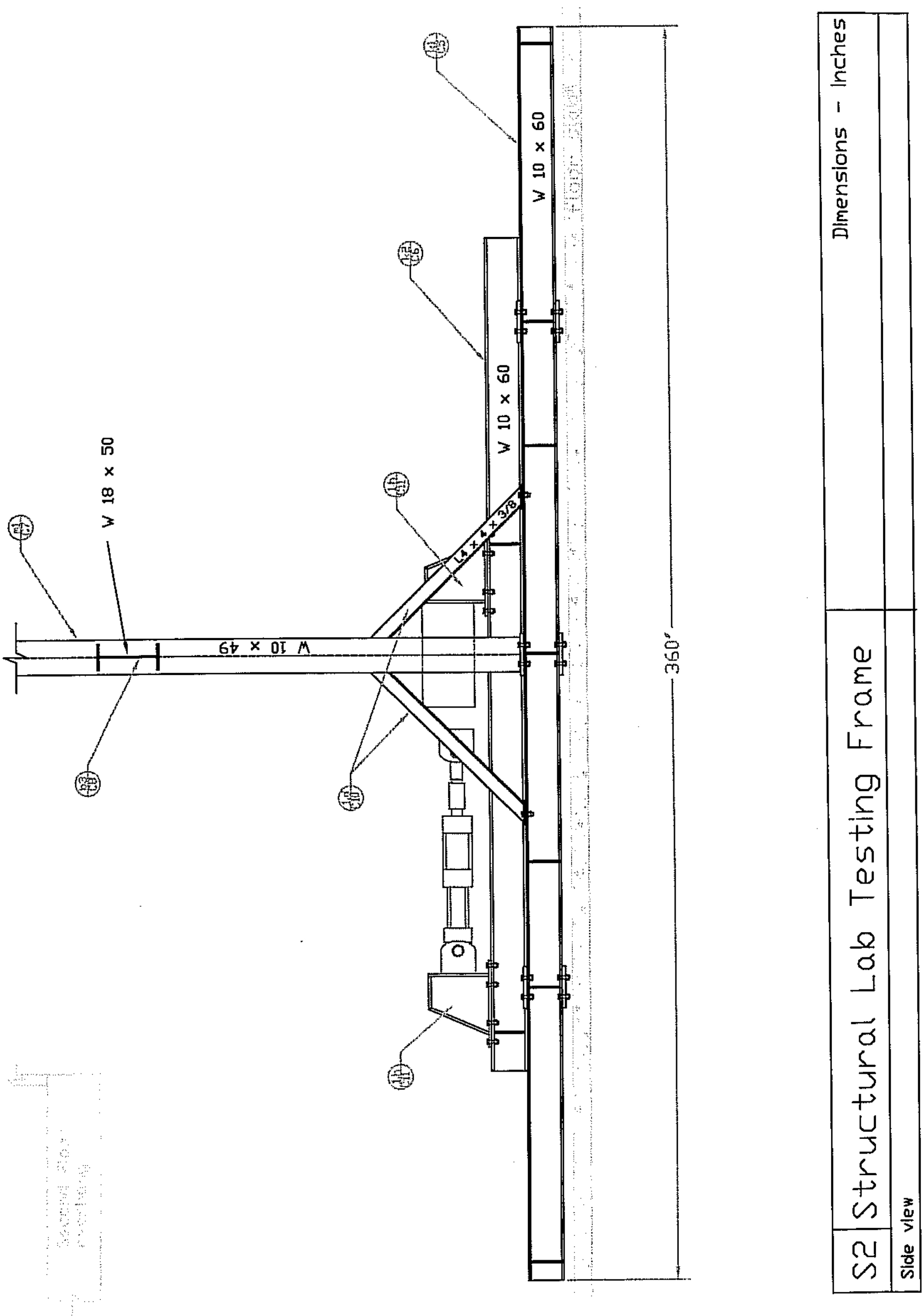
Plate Steel Schedule

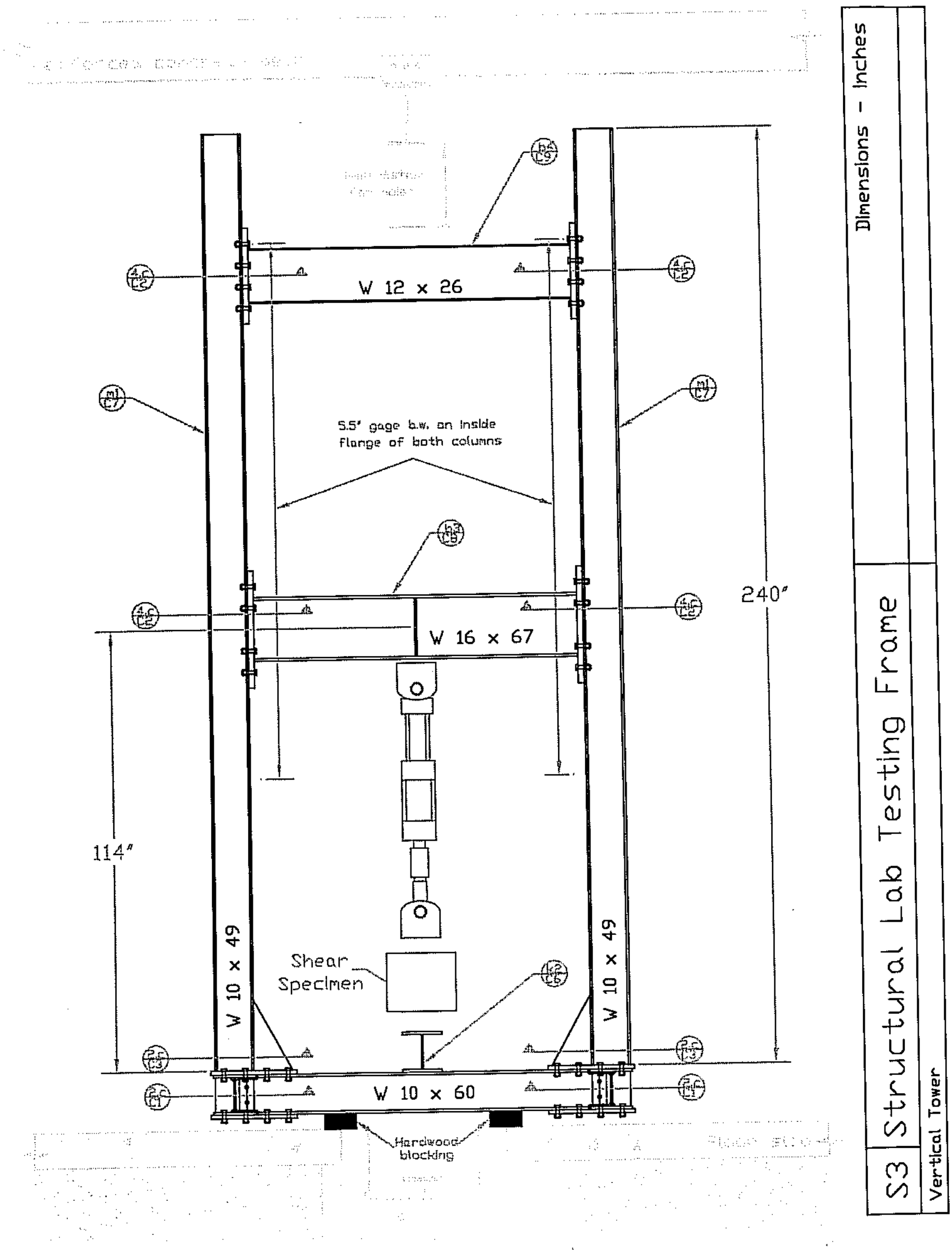
Description	Drawing Detail	Size (in)	no.	bolt holes drilled (/each)	Remark
Col. end plates	C3	12 x 22 x 0.75	2	8	
Cross beam plates	C1	12 x 22 x 0.75	10	8	
Mid. Beam end plates	C2	11.5 x 30 x 0.75	2	8	
Top Beam end plates	C2	9.5 x 24.5 x 0.75	2	8	
Diagonal brace end plates	C10	7 x 5 x 0.5	4	1	
Bulk Head			*2*		*Total*
Act. plate	C11	17.25 x 10 x 0.75	2	4	
End plate	C11	25 x 10 x 0.75	2	8	
Web plate	C11	16.5 x 16 x 0.75	2		See sketch, Top is cut down to 9"
Top plate	C11	10 x 10 x 0.75	2		See sketch, length is approx.
Back plate	C11	18.75 X 10 x 0.75	2		See sketch, length is approx.
Stiffeners					
Col. end plate stiffeners	C3	18 x 10 x 0.75	2		Rt. Triangle
Long. Beam stiffeners	C5	8.86* x 4 x 0.5	28	3	* through depth of web on W10 x 60
Mid. Beam stiffener	C8	15* x 4.5 x 0.5	2		* through depth of web on W16 x 67
Sample Beam stiffeners	C6	8.86* x 4 x 0.5	4		* through depth of web on W10 x 60

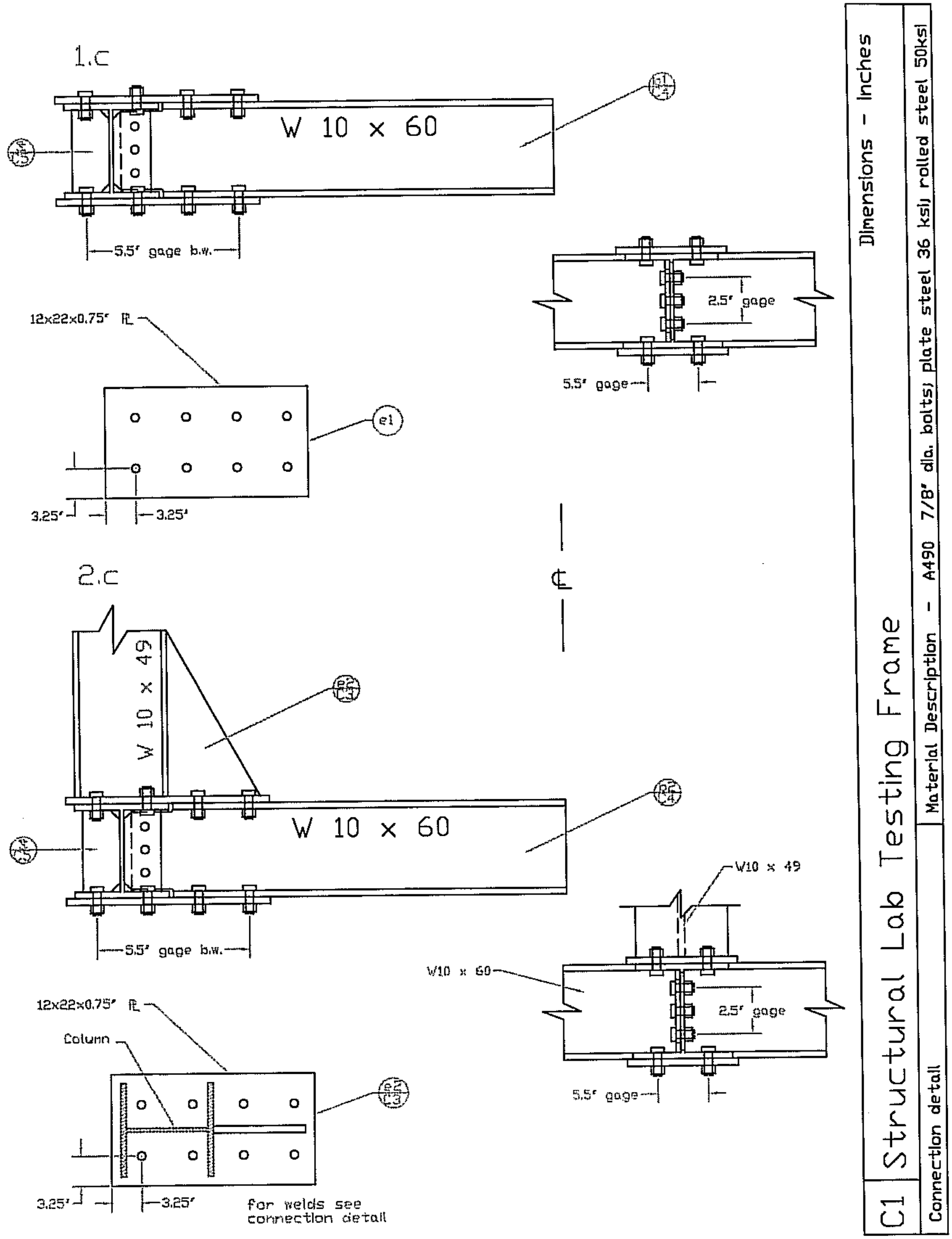
Total number of holes in all pieces		548
Total number of bolts (A490 7/8" dia. - various length)		182
	lengths (in)	
	2.0	22
	2.25	48
	2.5	112
	Total	182

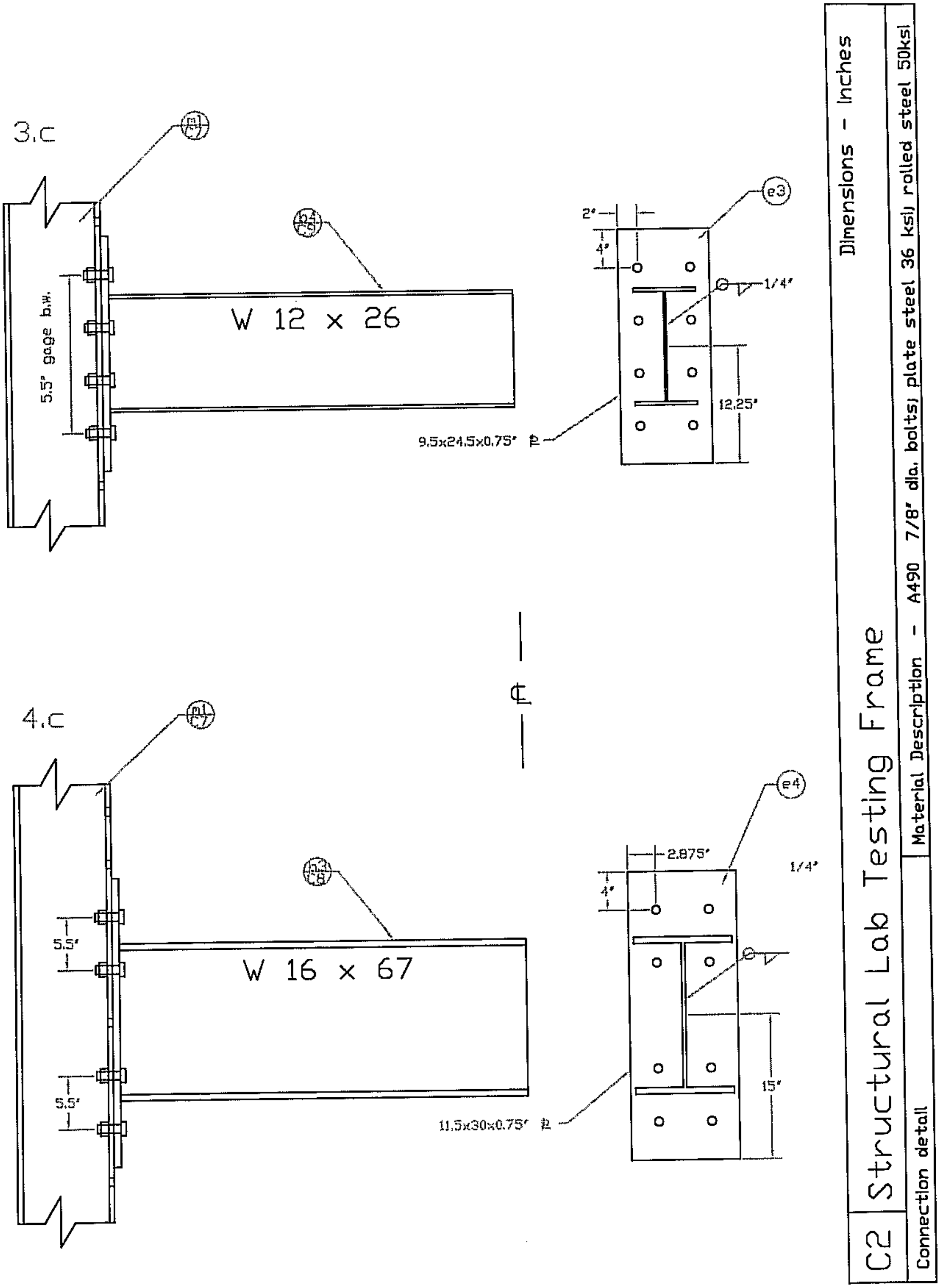

Dimensions - not to scale
Structural Lab Testing Frame
Top/Plan View
Columns for tower


S1 Structural Lab Test Frame
Plan/Top View
Dimensions - Inches
W 10 x 60
360"
96"
Pre-drilled holes for future
connection to cross beam
Columns for tower (W10 x 49)

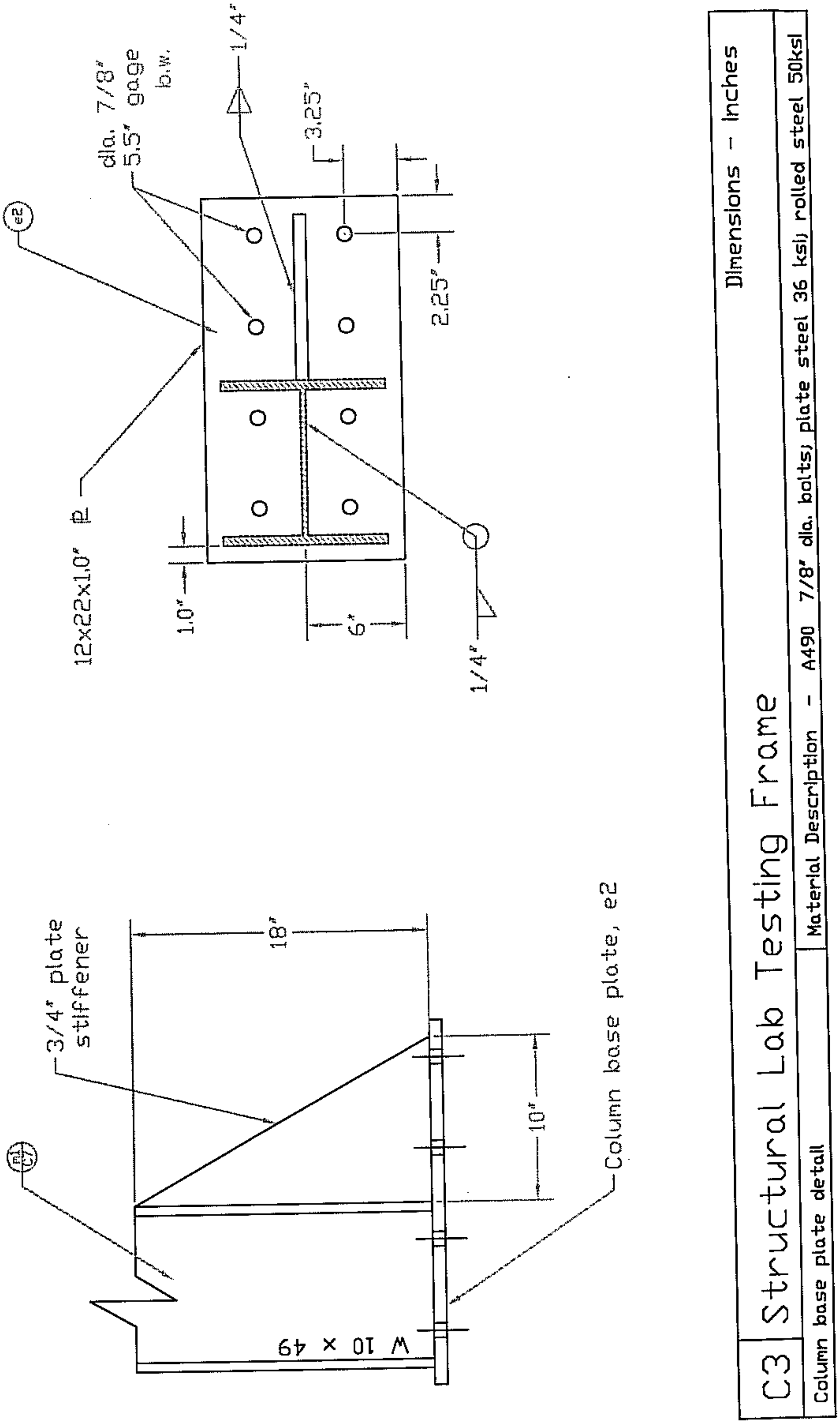

W 18 x 50
W 10 x 49
W 10 x 60
W 10 x 60
L4 x 4 x 3/8
360"
Dimensions - Inches
S2 Structural Lab Testing Frame
Side view


W 12 x 26
5.5" gage b.w. on inside flange of both columns
W 16 x 67
240"
114"
W 10 x 49
W 10 x 49
Shear Specimen
W 10 x 60
Hardwood blocking
Dimensions - Inches
S3 Structural Lab Testing Frame
Vertical Tower

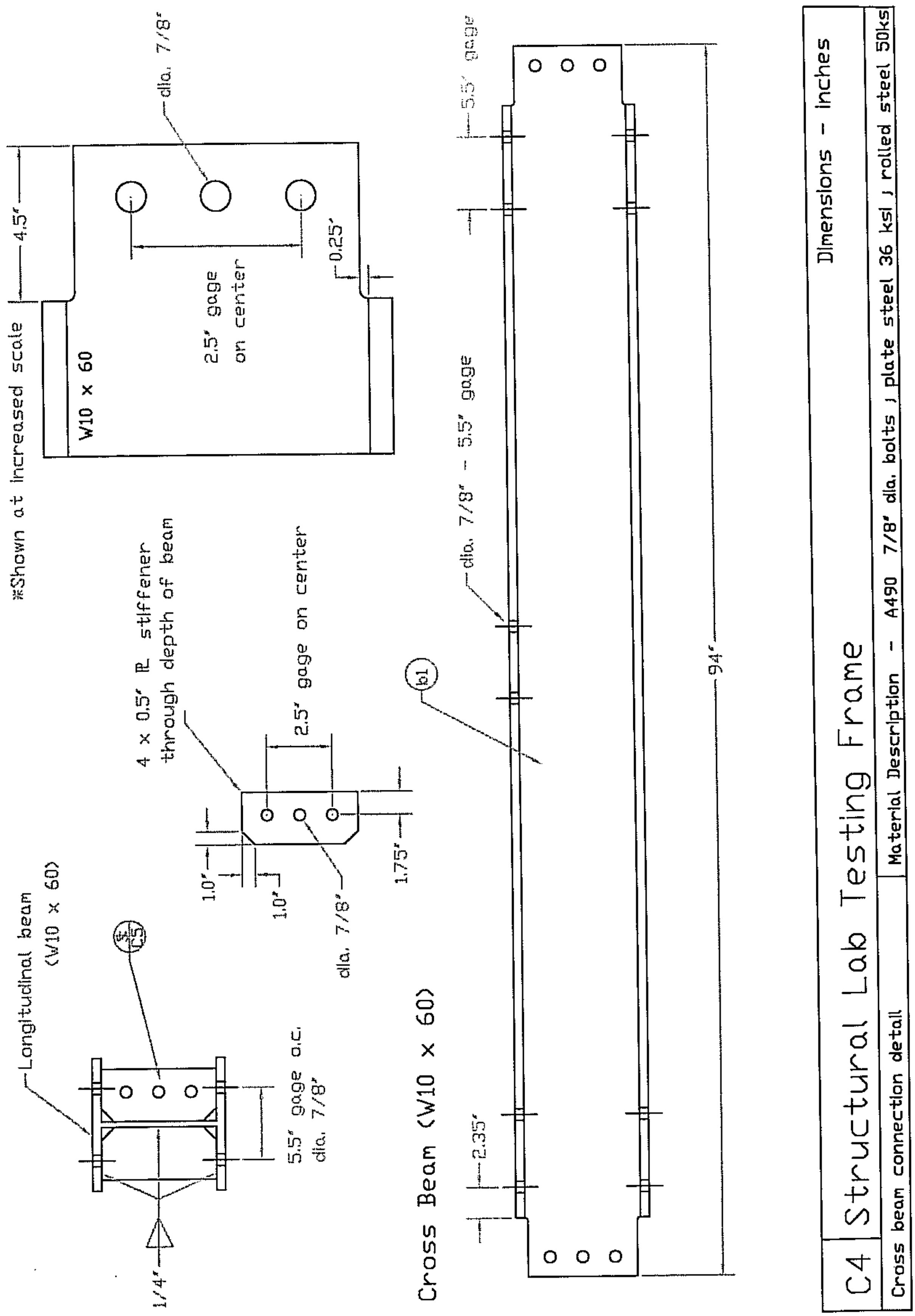

1.c
W 10 x 60
5.5" gage b.w.
12x22x0.75" PL
3.25"
3.25"
2.5" gage
5.5" gage
2.c
W 10 x 49
W 10 x 60
5.5" gage b.w.
12x22x0.75" PL
Column
3.25"
3.25"
For welds see connection detail
W10 x 49
W10 x 60
2.5" gage
5.5" gage
Dimensions - Inches
C1 Structural Lab Testing Frame
Material Description - A490 7/8" dia. bolts; plate steel 36 ksi; rolled steel 50ksi
Connection detail

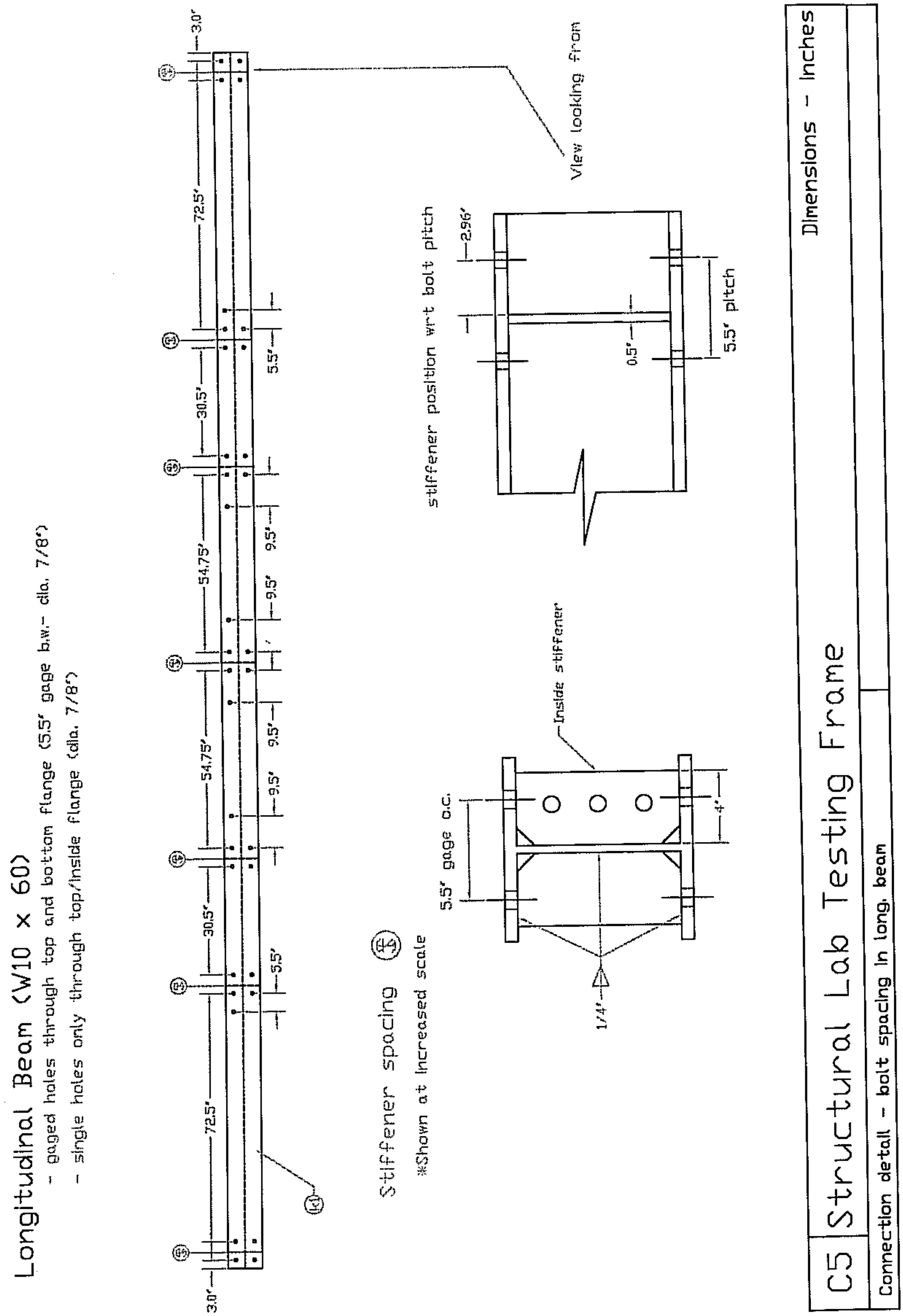


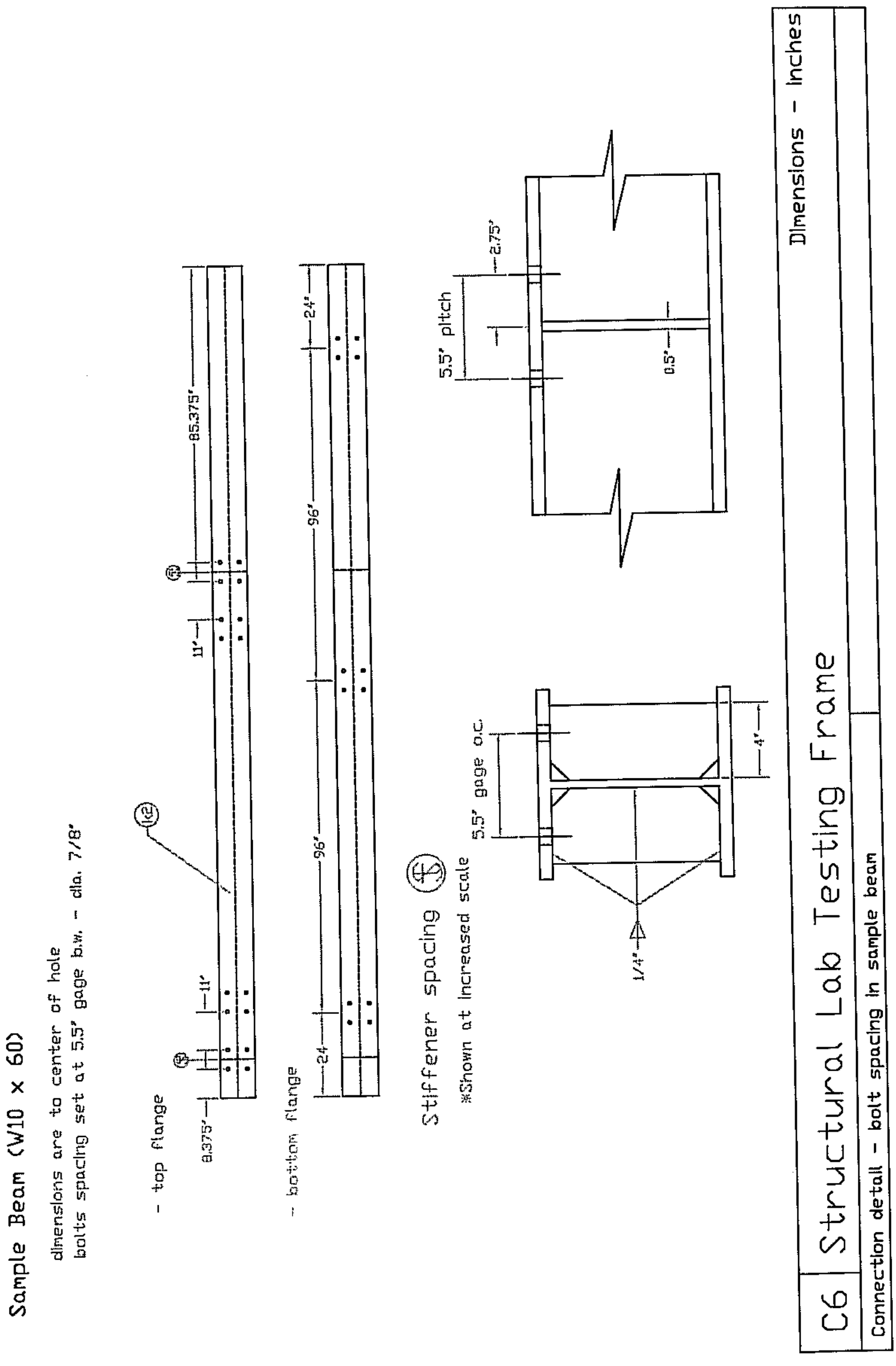
3.c
W 12 x 26
5.5" gage b.w.
2"
4"
1/4"
12.25"
9.5x24.5x0.75" PL
e3
4.c
W 16 x 67
5.5"
5.5"
2.875"
4"
1/4"
15"
11.5x30x0.75" PL
e4
Dimensions - Inches
C2 Structural Lab Testing Frame
Material Description - A490 7/8" dia. bolts; plate steel 36 ksi; rolled steel 50ksi
Connection detail

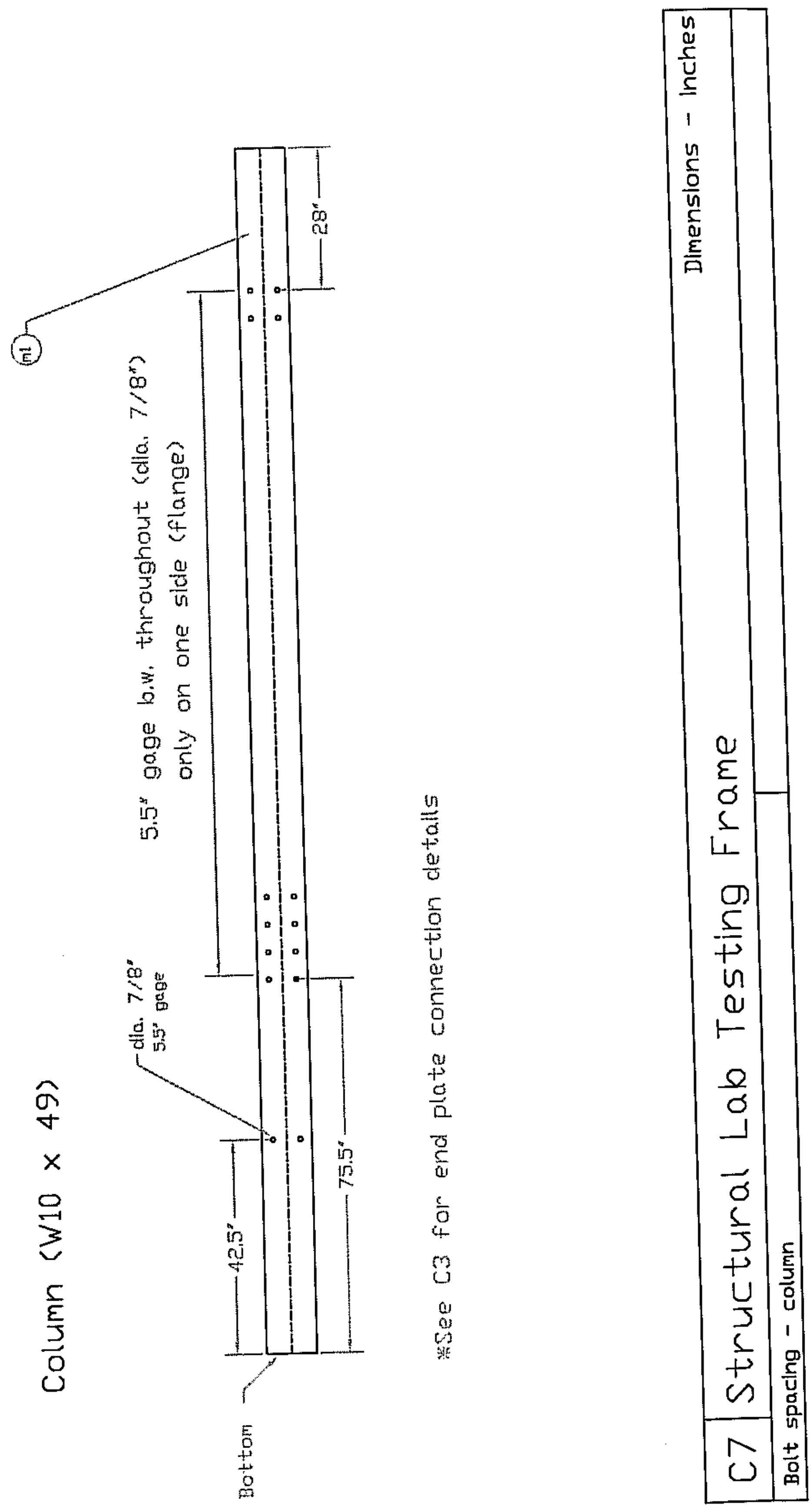


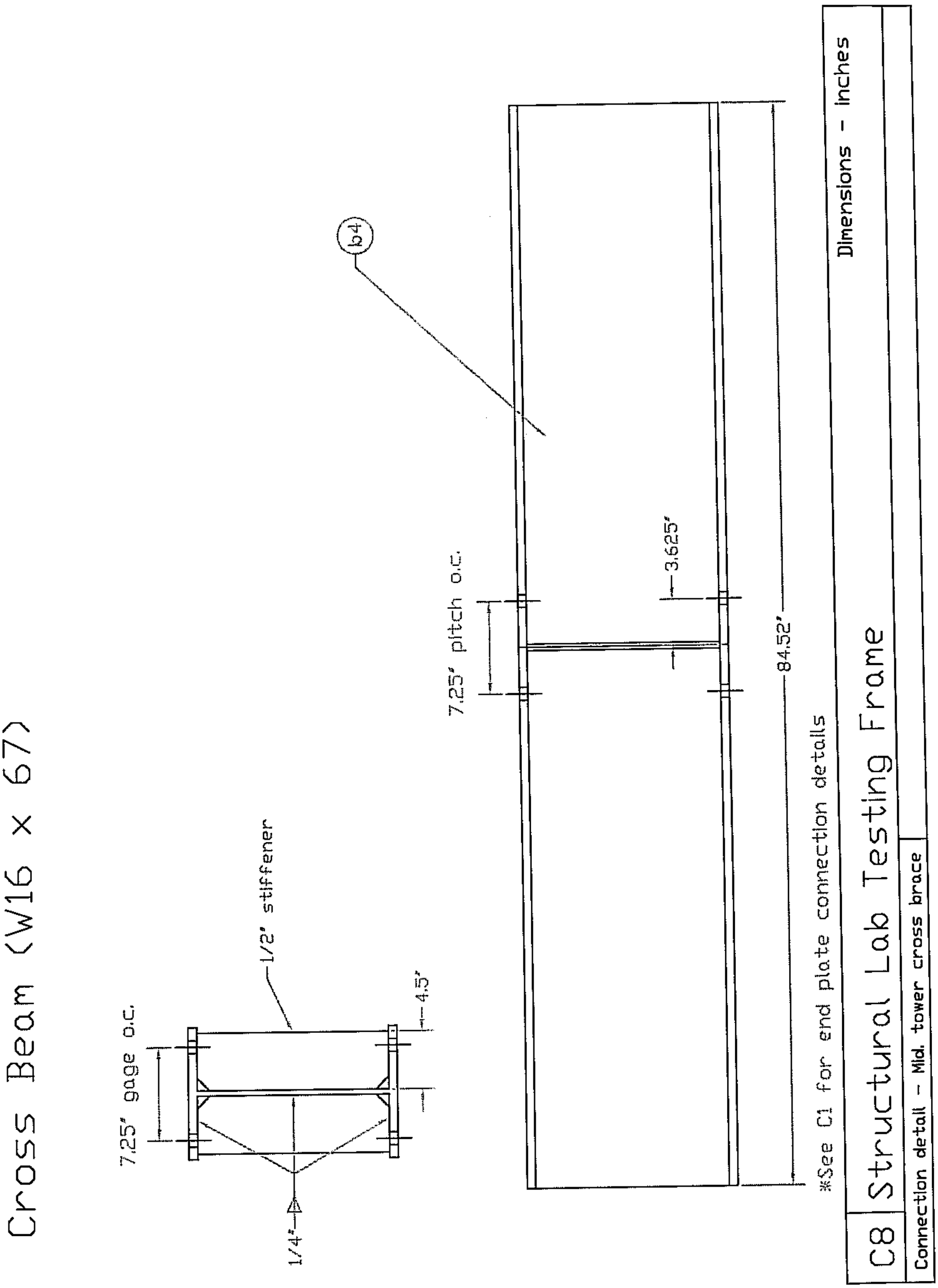
dia. 7/8"
5.5" gage
b.w.
1/4"
3.25"
2.25"
e2
12x22x1.0" PL
1.0"
6"
1/4"
m1
C7
3/4" plate
stiffener
18"
10"
Column base plate, e2
W 10 x 49
C3
Structural Lab Testing Frame
Dimensions - Inches
Column base plate detail
Material Description - A490 7/8" dia. bolts; plate steel 36 ksi; rolled steel 50ksi


*Shown at increased scale
W10 x 60
4.5"
dia. 7/8"
2.5" gage on center
0.25"
4 x 0.5" PL stiffener through depth of beam
2.5" gage on center
1.0"
1.0"
dia. 7/8"
1.75"
Longitudinal beam (W10 x 60)
5.5" gage o.c. dia. 7/8"
1/4"
Cross Beam (W10 x 60)
dia. 7/8" - 5.5" gage
5.5" gage
94"
2.35"
b1
Dimensions - Inches
C4 Structural Lab Testing Frame
Material Description - A490 7/8" dia. bolts ; plate steel 36 ksi ; rolled steel 50ksi
Cross beam connection detail

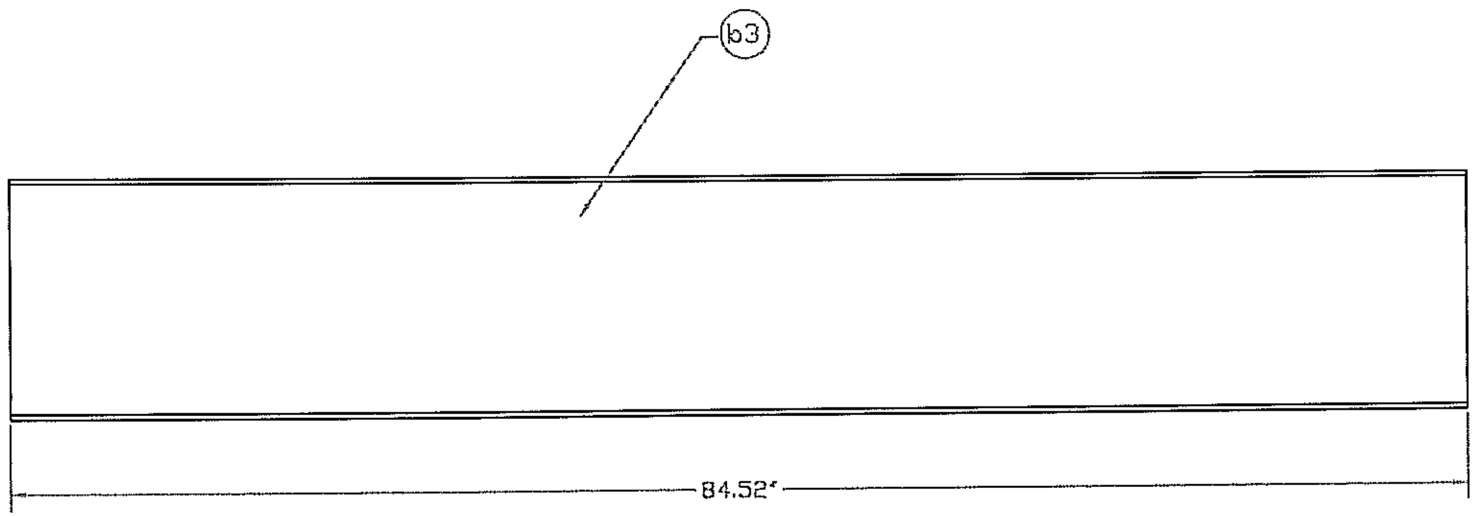

Longitudinal Beam (W10 x 60)
- gaged holes through top and bottom flange (5.5" gage b.w.- dia. 7/8")
- single holes only through top/inside flange (dia. 7/8")
3.0"
72.5"
30.5"
54.75"
54.75"
30.5"
72.5"
3.0"
5.5"
9.5"
9.5"
9.5"
9.5"
5.5"
Stiffener spacing
*Shown at increased scale
5.5" gage o.c.
Inside stiffener
4"
1/4"
stiffener position wrt bolt pitch
2.96"
0.5"
5.5" pitch
View looking from
Dimensions - Inches
C5 Structural Lab Testing Frame
Connection detail - bolt spacing in long. beam


Sample Beam (W10 x 60)
dimensions are to center of hole
bolts spacing set at 5.5" gage b.w. - dia. 7/8"
- top flange
- bottom flange
8.375"
11"
85.375"
24"
96"
96"
24"
2.75"
5.5" pitch
0.5"
5.5" gage o.c.
4"
1/4"
Stiffener spacing
*Shown at increased scale
Dimensions - Inches
C6 Structural Lab Testing Frame
Connection detail - bolt spacing in sample beam


Column (W10 x 49)
5.5" gage b.w. throughout (dia. 7/8")
only on one side (flange)
dia. 7/8"
5.5" gage
m1
28"
42.5"
75.5"
Bottom
*See C3 for end plate connection details
C7
Structural Lab Testing Frame
Bolt spacing - column
Dimensions - Inches

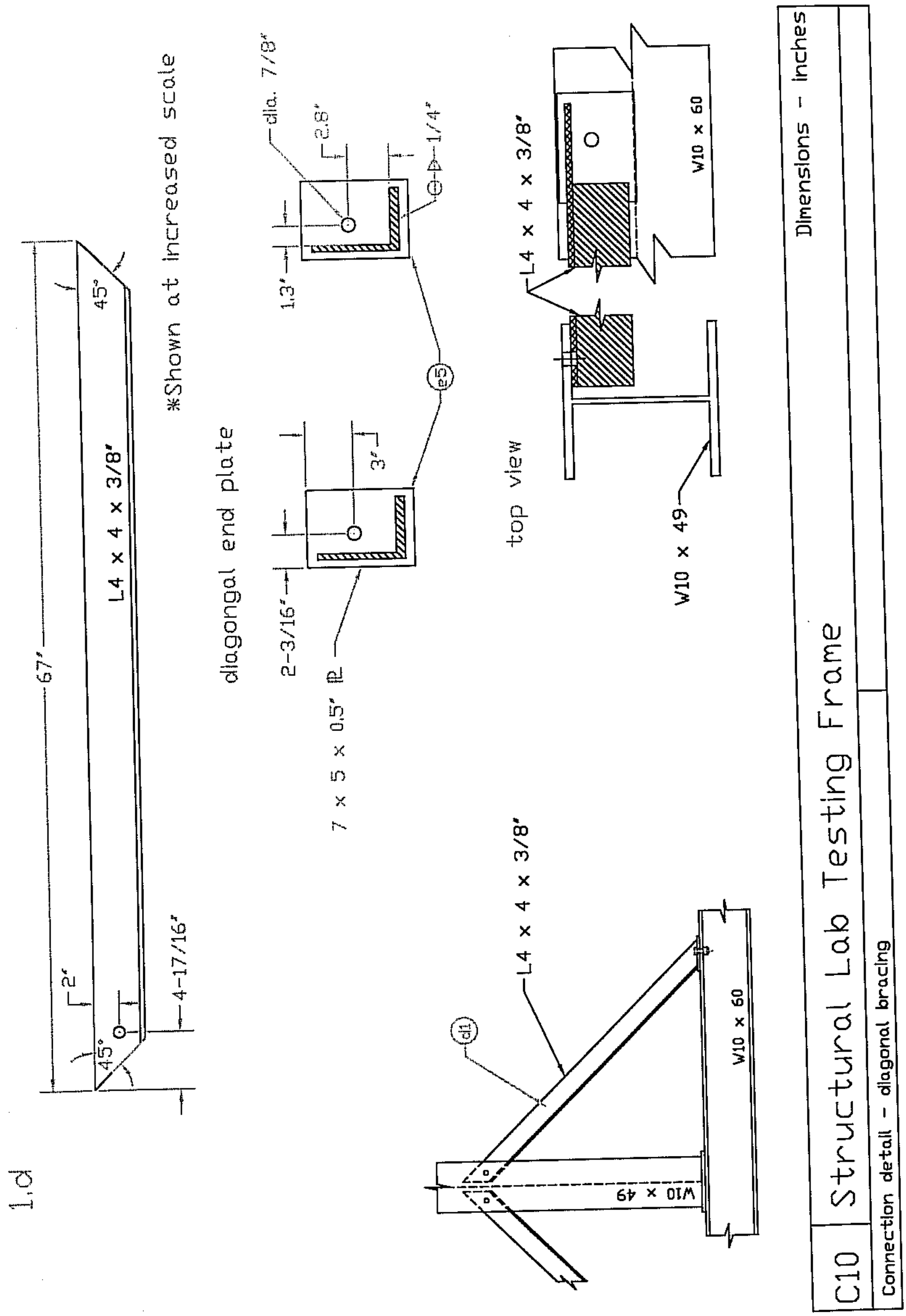

Cross Beam (W16 x 67)
7.25" gage o.c.
1/2" stiffener
4.5"
1/4"
b4
7.25" pitch o.c.
3.625"
84.52"
*See C1 for end plate connection details
Dimensions - inches
C8 Structural Lab Testing Frame
Connection detail - Mid. tower cross brace

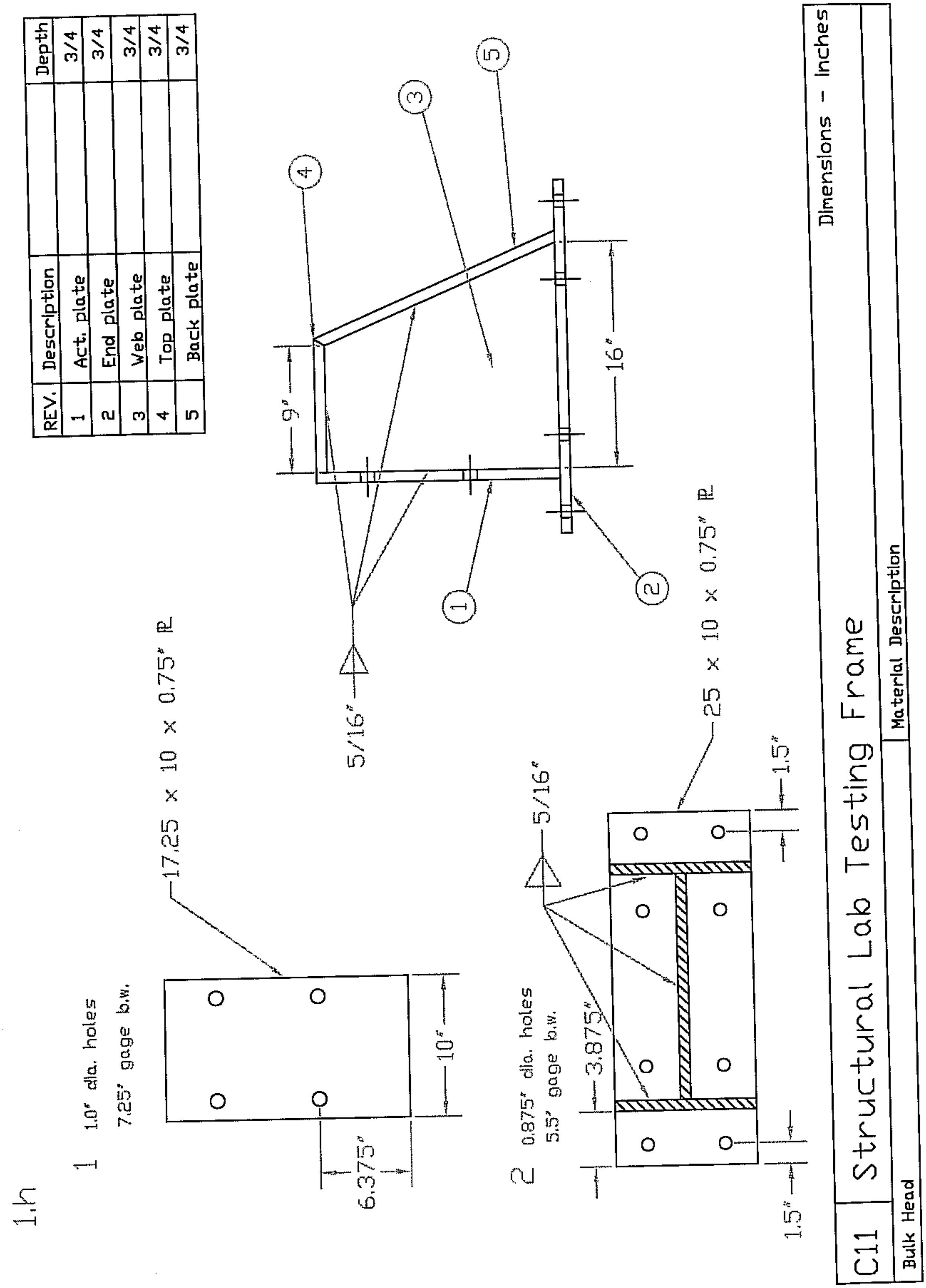
Top Cross Beam (W12 x 26)



*See C2 for end plate connection details

C9	Structural Lab Testing Frame	Dimensions - Inches
Connection detail - Top tower cross beam		


1.d
67"
L4 x 4 x 3/8"
45°
2"
4-17/16"
*Shown at increased scale
diagonal end plate
7 x 5 x 0.5" PL
2-3/16"
3"
1.3"
2.8"
dia. 7/8"
1/4"
e5
top view
L4 x 4 x 3/8"
W10 x 60
W10 x 49
d1
Dimensions - inches
C10
Structural Lab Testing Frame
Connection detail - diagonal bracing


REV. | Description | Depth
1 | Act. plate | 3/4
2 | End plate | 3/4
3 | Web plate | 3/4
4 | Top plate | 3/4
5 | Back plate | 3/4
9"
16"
5/16"
25 x 10 x 0.75" PL
1 1.0" dia. holes
7.25" gage b.w.
17.25 x 10 x 0.75" PL
10"
6.375"
2 0.875" dia. holes
5.5" gage b.w.
3.875"
5/16"
1.5"
1.5"
Dimensions - Inches
C11 Structural Lab Testing Frame
Material Description
Bulk Head

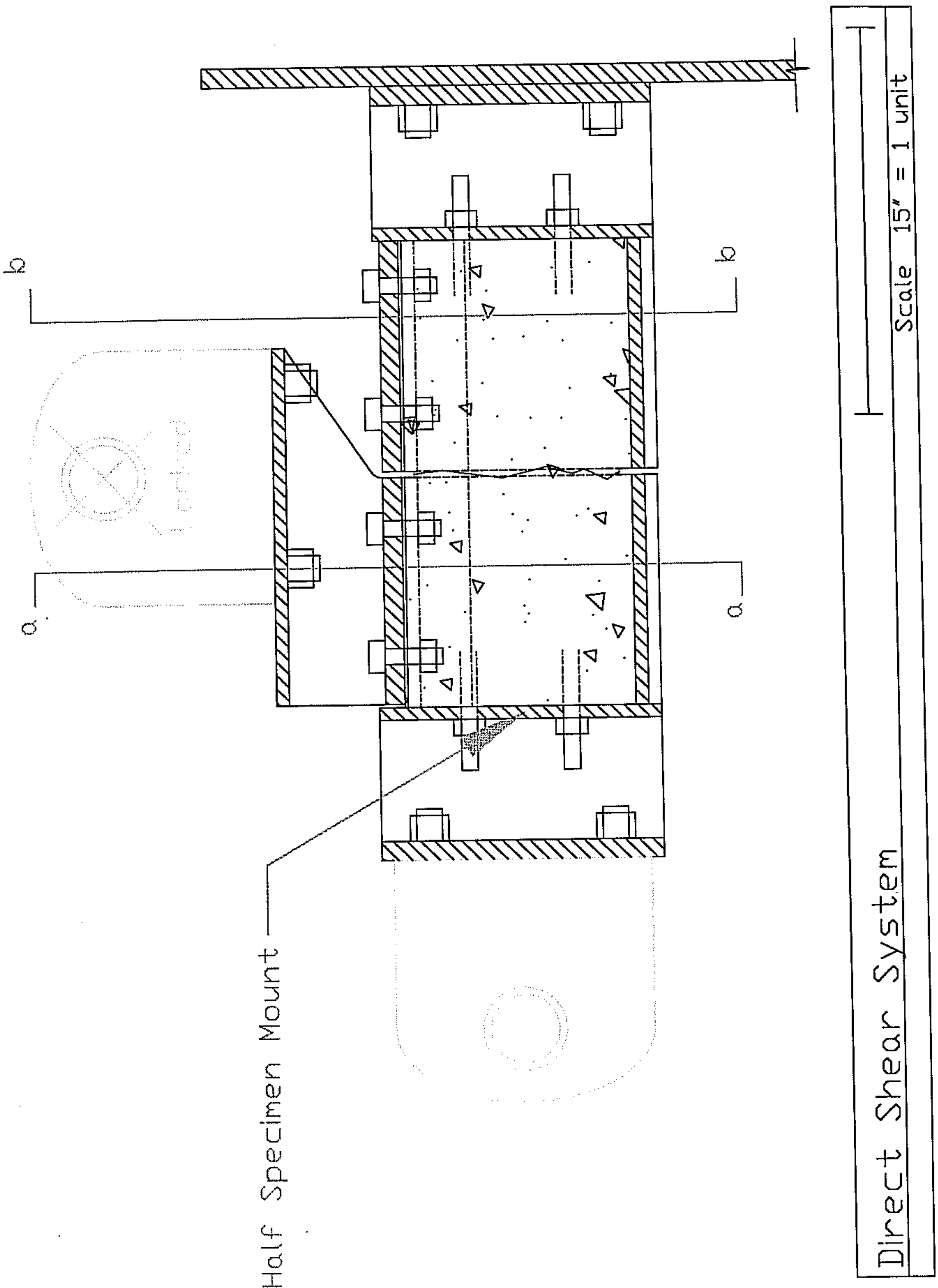
Direct Shear System

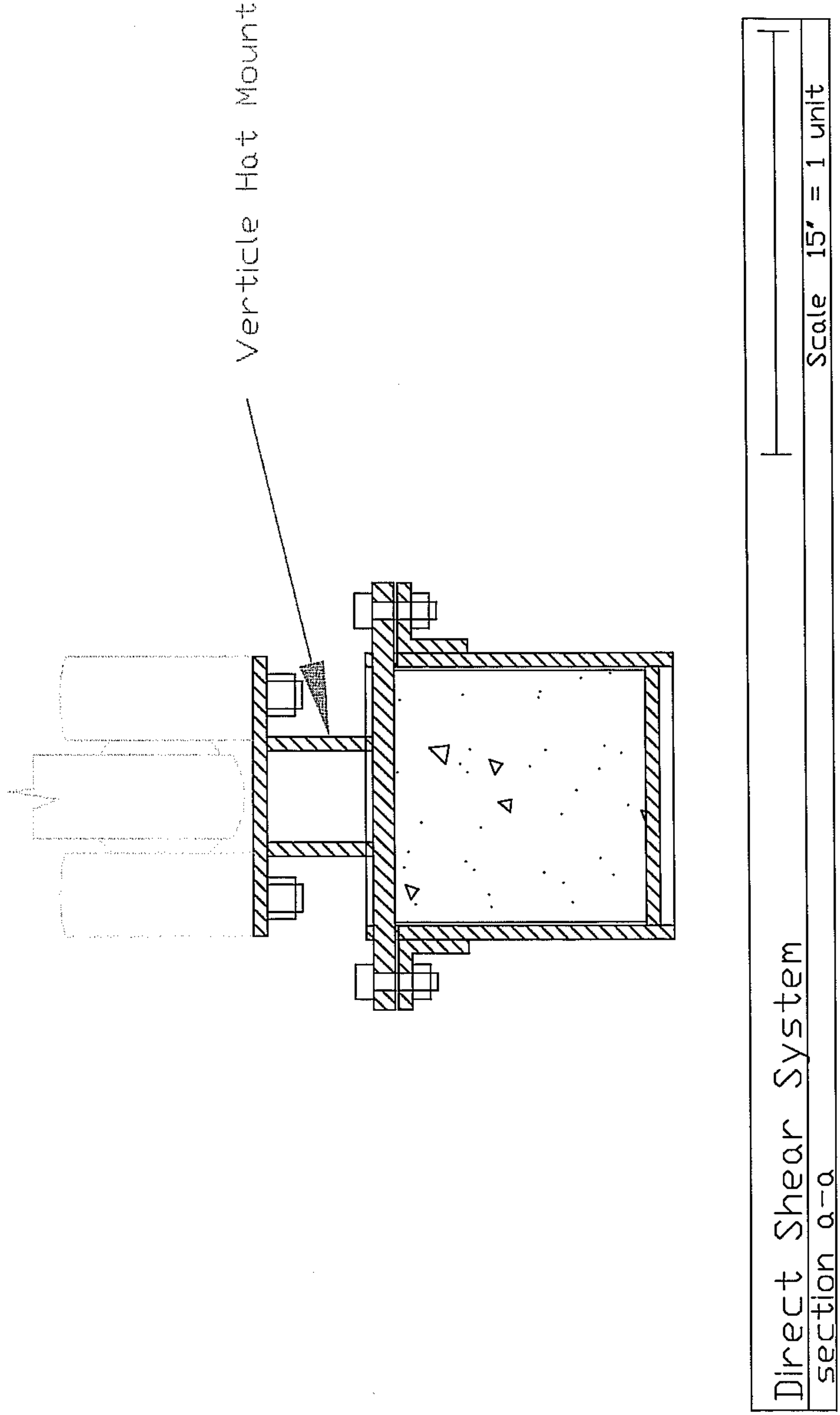
design based on 50 ksi yeild strength for all plates

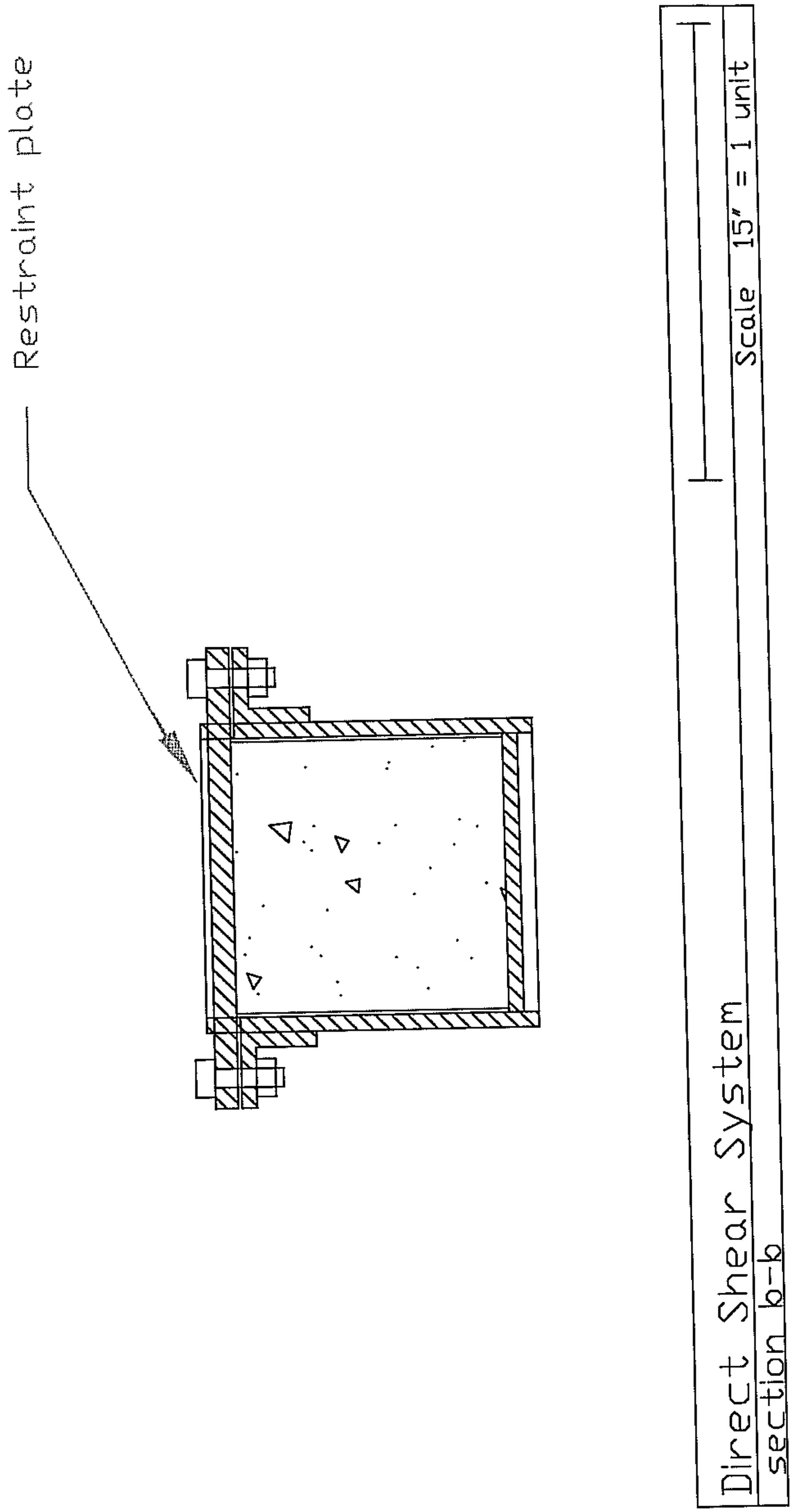
Plate Steel Schedule

Discription	Size (in)	no.	bolt holes drilled (/each)	Remark
Half Specimen Mount		*2*		*Total*
Actuator plate	11 x 10.25 x 0.75	2	4	
Specimen plate	11 x 10.25 x 0.5	2	4	
Side plate	15 x 11* x 0.5	4		*length varies, see sketch
Floor plate	9 x 9.25 x 0.5	2		
Angle	L 2-1/2 x 2-1/2 x 1/2	4	2	
Verticle Hat Mount				
Actuator plate	15 x 10 x 0.5	1	4	
Specimen plate	15.25 x 8.875 x 0.75	1	4	
Side plate	15* x 3.75 x 0.5	2		*length varies, see sketch
Restraint Plate				
Top plate	15.25 x 8.875 x 0.75	1	4	

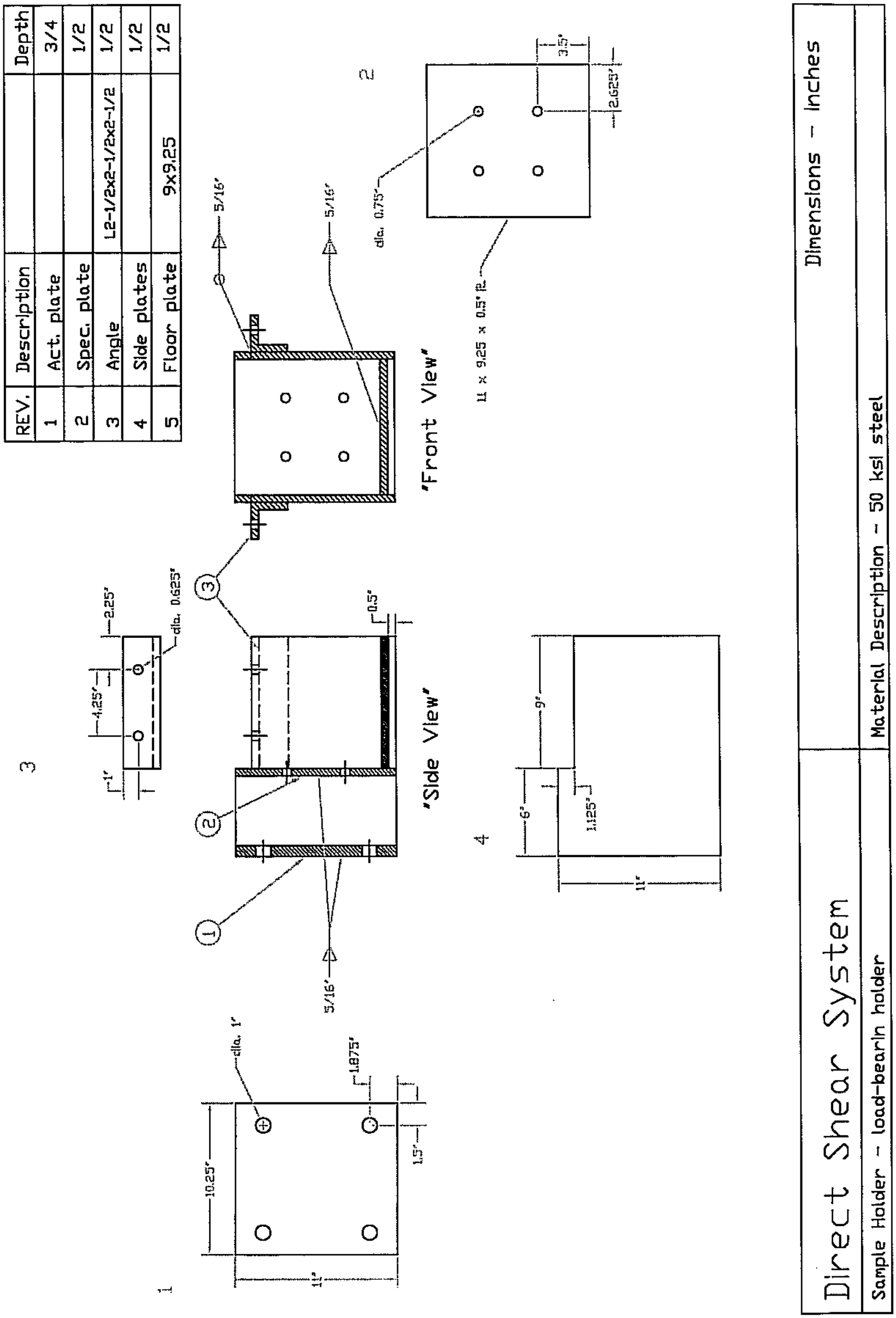
Total number of holes in all pieces	36
Total number of bolts (5/8" dia. - length=2.25")	8
Total number of bolts (1.0" dia. - ? length)	2


b
b
a
a
Half Specimen Mount
Scale 15" = 1 unit
Direct Shear System

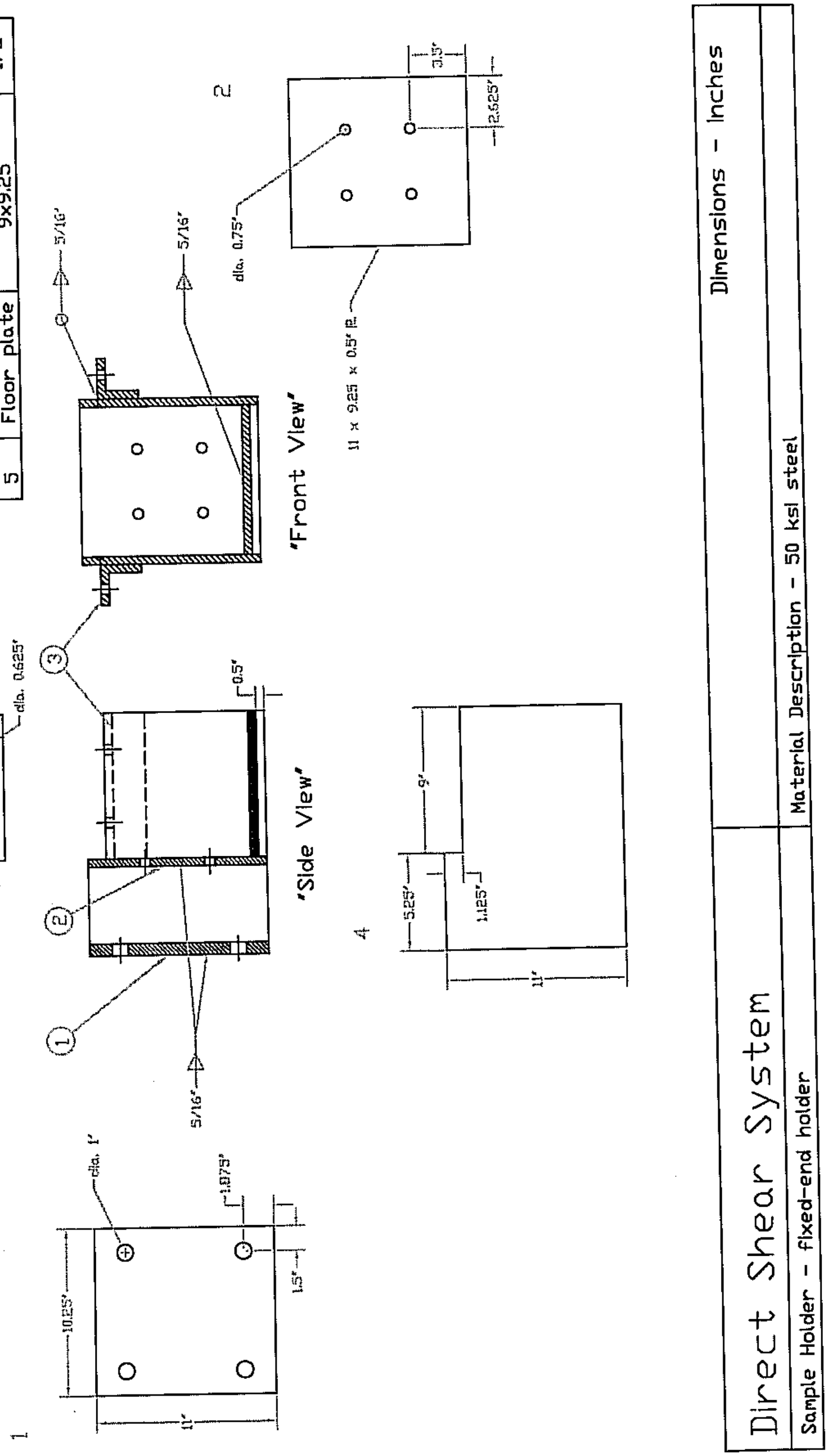

Verticle Hat Mount
Direct Shear System
section a-a
Scale 15' = 1 unit

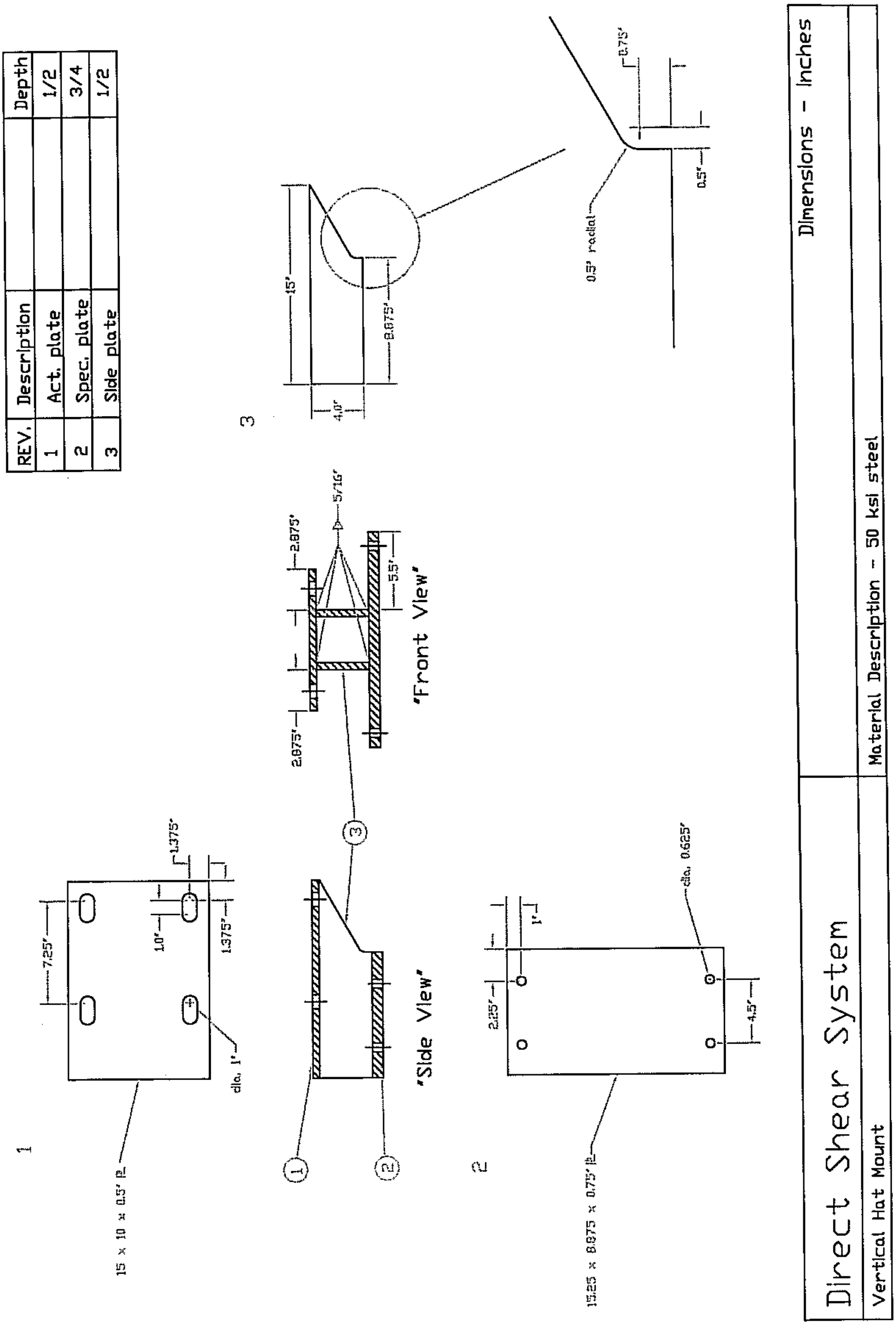

Restraint plate
Direct Shear System
section b-b
Scale 15" = 1 unit

REV.	Description		Depth
1	Act. plate		3/4
2	Spec. plate		1/2
3	Angle	L2-1/2x2-1/2x2-1/2	1/2
4	Side plates		1/2
5	Floor plate	9x9.25	1/2

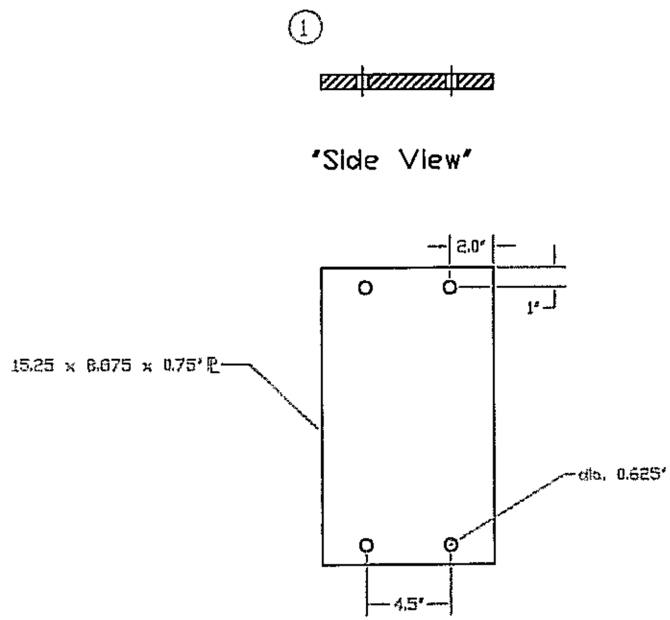


REV.	Description		Depth
1	Act. plate		3/4
2	Spec. plate		1/2
3	Angle	L2-1/2x2-1/2x2-1/2	1/2
4	Side plates		1/2
5	Floor plate	9x9.25	1/2

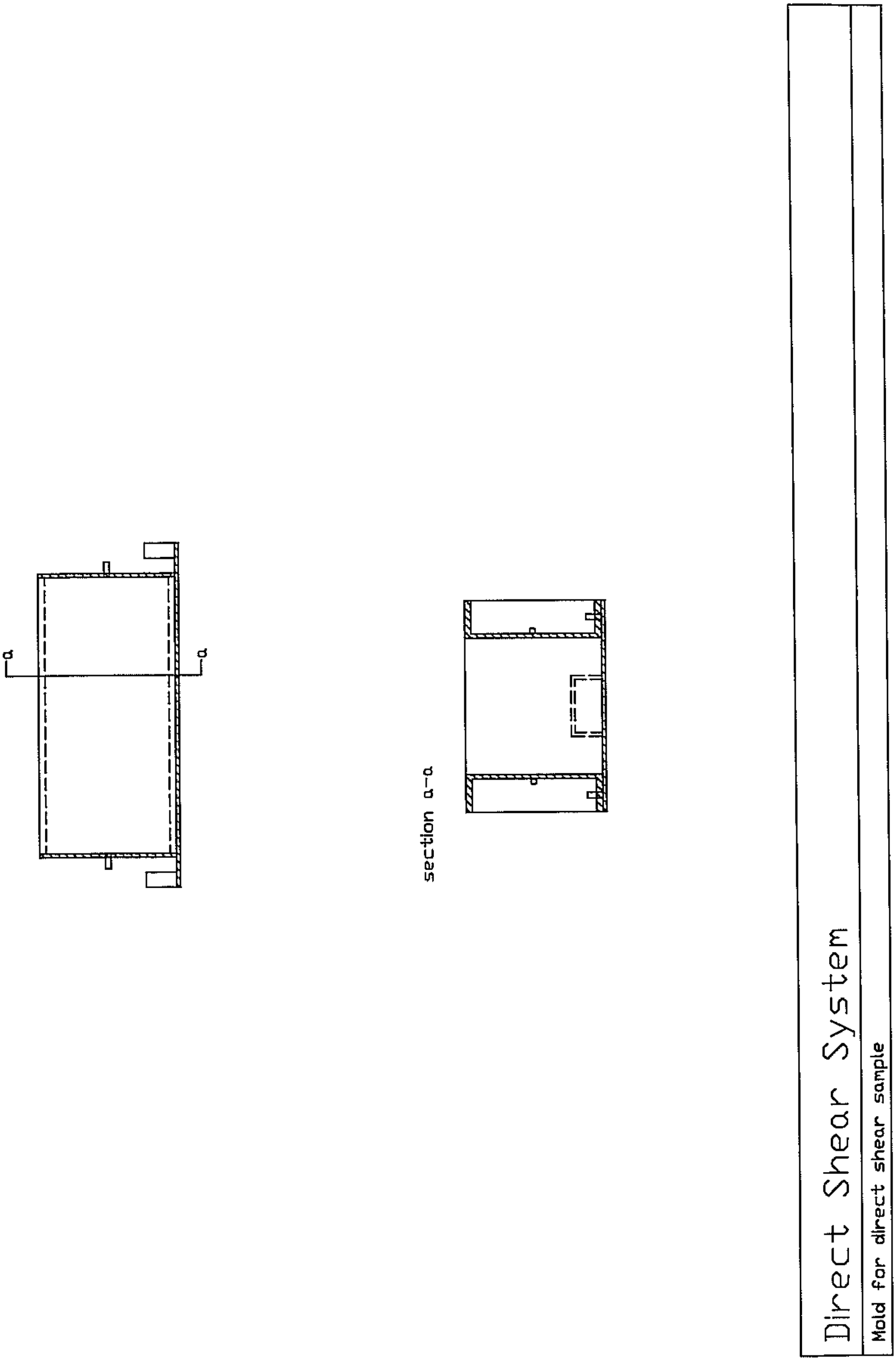


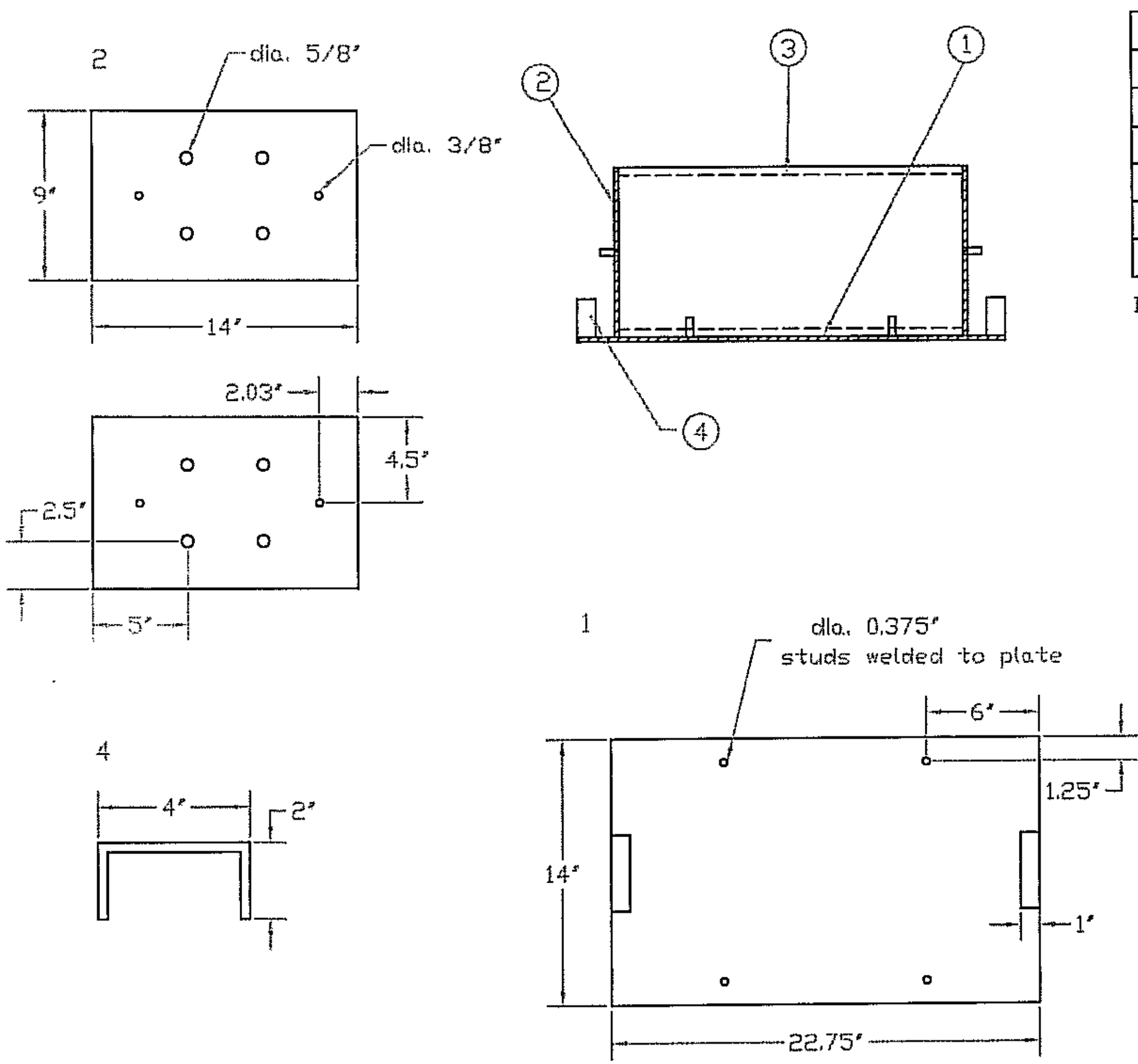

REV. | Description | Depth
1 | Act. plate | 1/2
2 | Spec. plate | 3/4
3 | Side plate | 1/2
15 x 10 x 0.5' PL
7.25'
1.375'
1.0'
dia. 1'
"Side View"
"Front View"
2.875'
5/16'
5.5'
15'
8.875'
4.0'
0.75'
0.5'
0.5' radial
dia. 0.625'
2.25'
4.5'
15.25 x 8.875 x 0.75' PL
Direct Shear System
Vertical Hat Mount
Material Description - 50 ksi steel
Dimensions - Inches

REV.	Description		Depth
1	R. plate		3/4



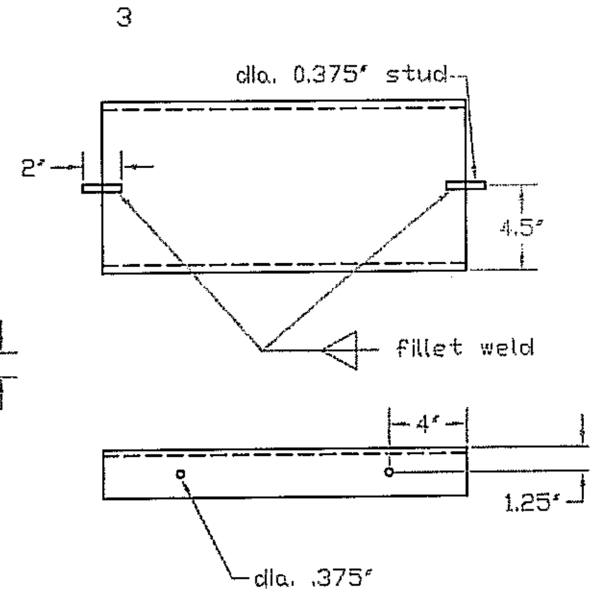
Direct Shear System	Dimensions - Inches
Restraint plate	Material Description - 50 ksi steel


a
a
section a-a
Direct Shear System
Mold for direct shear sample



REV.	Description		Depth
1	Base plate		1/4
2	Side plate		1/4
3	C-channels	C9 x 15	
4	Handle		1/4
	Studs	dia. 3/8"	

Interior dimensions need to be: 9 x 9 x 18.25"

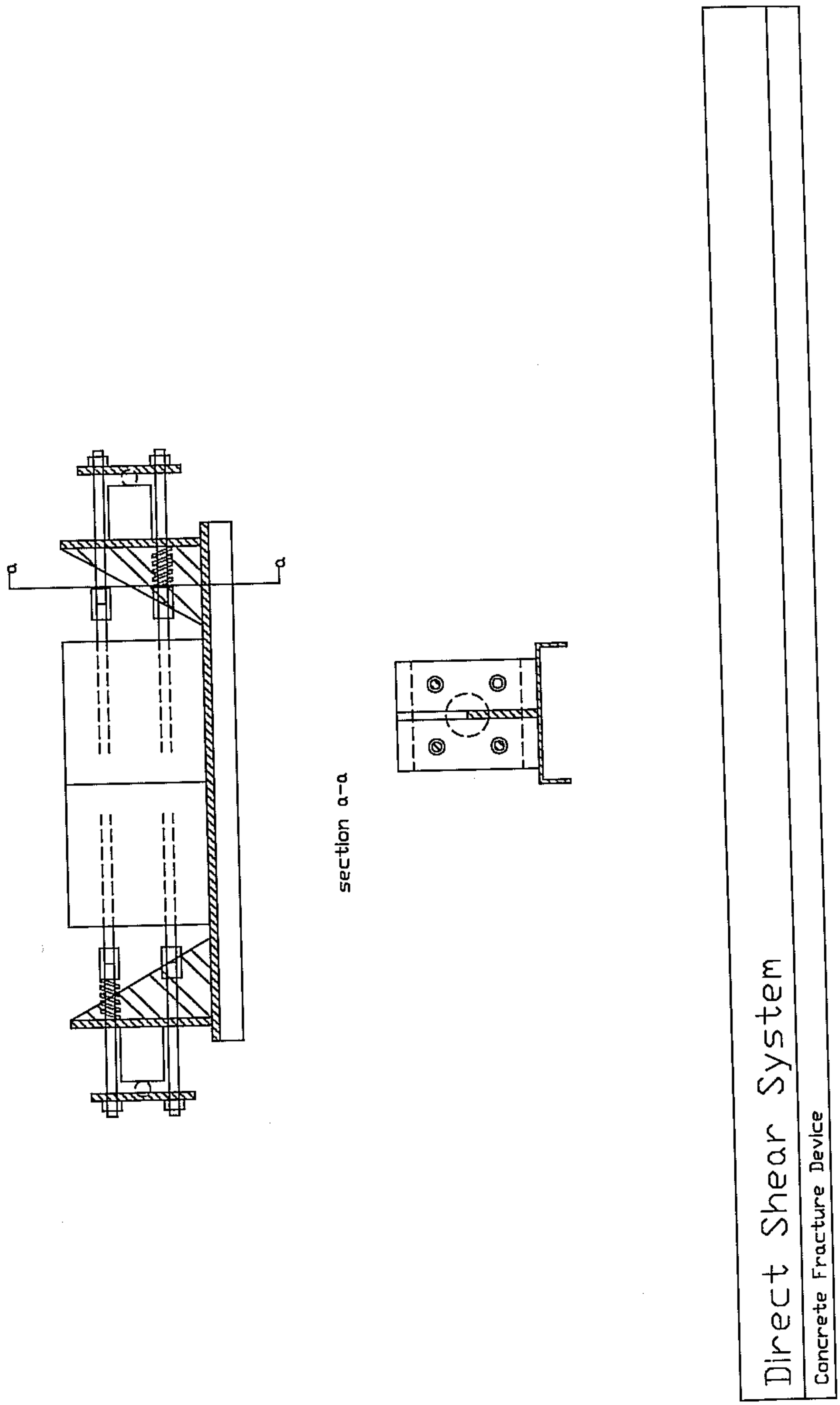


Direct Shear System

Dimensions - Inches

Mold for direct shear sample

Material Description


a
a
section a-a
Direct Shear System
Concrete Fracture Device

Appendix B

Mix design worksheets

MICHIGAN DEPARTMENT OF TRANSPORTATION

FORM 1830

CONCRETE PROPORTIONING DATA

FILE 300

CONTROL SECTION ID: RESEARCH
JOB NUMBER: MI. TECH.
LAB NUMBER: 99C-1028
GRADE OF CONCRETE: P1
INTENDED USE OF CONCRETE: Pavement (Conv. Form)

DATE: 7/01/1999
SPECIFICATION: 1996 STD SPECS
MIX DESIGN NUMBER: 99-1038

CONCRETE MATERIALS

MATERIAL	SOURCE	SOURCE NUMBER	CLASS	SPECIFIC GRAVITY	ABSORPTION PERCENT
CEMENT	(SEE REMARKS)		1/1A	3.15	
FINE AGG.	Superior S & G	31-45	2NS	2.66	1.14
COARSE AGG.	Bruce Mines	95-10	6AA	2.88	0.36
FLY ASH					

CEMENT CONTENT, kg/m^3 335
AIR CONTENT (DESIGN): 6.5% (SPECIFIED): 6.5%
R.W.C: 1.15
FLY ASH CONTENT, kg/m^3: 0

B/Bo : 0.72
SPECIFICATION TOLERANCE (±): 1.5%
THEORETICAL YIELD: 100.00%

WEIGHT OF COARSE AGG. (DRY/LOOSE) kg/m^3	AGGREGATE AND WATER PROPORTIONS QUANTITIES, kg/m^3 OF CONCRETE		
	FINE AGG (OVEN DRY)	COARSE AGG (OVEN DRY)	TOTAL WATER
1456	847	1048	160
1466	841	1056	159
1476	836	1063	159
1486	830	1070	158
1496	824	1077	158
1506	818	1084	158
1516	812	1092	157
1526	807	1099	157
1536	801	1106	157
1546	795	1113	156
1556	789	1120	156

REMARKS:
THIS CHART FOR USE WITH CEMENTS OF THE CLASS SHOWN FROM APPROVED SOURCES.

TYPICAL UNIT WEIGHT (DRY, LOOSE) OF COARSE AGGREGATE AS DESCRIBED ABOVE IS 1506 kg/m^3

SPECIAL MESSAGES: Dynamic Fracture Research Project

CC:
S. Vitton-Mi. Tech.
T. Woodhouse-MDOT

JOHN F. STATON
MATERIALS RESEARCH ENGINEER

MIX PROPORTIONS WORKSHEET

	Laboratory No	Bulk Dry Specific Gravity	% Absorption
Cement: Lafarge (Alpena) Type 1		3.15	-
Coarse Aggregate: CA-B (Bruce M.) Source No. 95-10 Specification 6AA	MTU	2.88 ★	0.36 ★
Fine Aggregate: FA-Y Source No. 31-45 Specification 2NS		2.66 ★	1.14 ★

Batch size 0.0779779 m^3

Material	Weight, kg/m^3	Batch Proportions kg					
Cement	335 ★	26.12	Total cement (C)				
Coarse Aggregate (DRY)	1084 ★			Pass	Ret	%	
		21.132	21.13	25.0mm	19.0mm	25	
		21.132	21.14	19.0mm	12.5mm	25	
		21.132	21.13	12.5mm	9.5mm	25	
		21.132	21.13	9.5mm	4.75mm	25	
			84.53				
		84.53	Total Coarse Agg. (a)				
Fine Aggregate (DRY)	818 ★	63.79	Total Fine Agg. (b)				
Total Water	158 ★	12.32	Total Water per batch (d)				

Absorbed Water	agg*absorption =		absorbed h_2O		
Coarse Agg	1084	0.0036	3.90	13.23	Absorbed water (W) kg/m^3
Fine Agg	818	0.0114	9.33		
			13.23		

Total Aggregate Contains 43.0 % Fine Aggregate

Note: ★ Provided by MDOT (Form 1830, File 300) and listed in Table 2.3

BATCH COMPUTATIONS WORKSHEET **WEIGHT IN kg**

Coarse Aggregate

84.53 Coarse Agg (a)

Pail tare	1.71	1.73	3.44 + pails
			87.97 = total
25.0 - 19.0mm	21.13	0.00	
19.0 - 12.5mm	0.00	21.14	
12.5 - 9.5mm	0.00	21.13	
9.5 - 4.75mm	21.13	0.00	
Sub total	43.97	44.00	87.97 Total

BATCH NO. BM-T1

COARSE AGG CA-B (Bruce M.)

DATE: 12/1/99

Batch Made Wed. @ 4:30

Fine Aggregate

63.79 Fine Agg (b)

Moisture content

wet	dry
121.15	116.1
0.0435	MC

0.0435 **MC**

2.77 **Moisture**

Dry weight 63.79

\+

Moisture 2.77

Total **66.56**

WATER MEASUREMENT

Coarse Agg +pail 43.97

Coarse Agg +pail 44.00

Total 87.97

\+ Total Batch Water 11.32 (d) 11.32

\- Reserve Water 3.00 3.00

= **Pails, Agg&Water** **96.29** H_2O 8.32

RESERVE WATER

Res water	3.00	1.45 surplus & Tare
+ Tare	0.29	0.29 - tare
= Total	3.29	1.16 = surplus

Reserve Water 3.00

\- Surplus Water 1.16

= 1.84 H_2O + 8.32

Subtotal of water in batch = 10.16

\+ Moisture in Fine Aggregate + 2.77

Total Water in Batch **(D) = 12.93**

Cement

26.12 **Cement (C)**

Pail ID A', B'

Tare weight 0.85 1.70 tare

Tare weight 0.85 27.82 Pail + cement

Total tare 1.70

UNIT WEIGHT

Weight of Concrete & Bucket 42.20

\- Weight of Bucket 8.15

= Weight of Concrete in Bucket **34.05** **(f)**

Air Entraining Admixture 29 ml

Batch Summary

(a)	Coarse Aggregate as Designed	84.53	kg
(b)	Fine Aggregate as Designed	63.79	kg
(c)	Cement as Designed	26.12	kg
(D)	Total Water of Batch	12.93	kg
(e)	**Total Weight of Batch**	**187.37**	kg

SLUMP = 0.5 " **12.7** mm

AIR CONTENT

\- Factor of Aggregate Porosity

= Percent Air 4

CONCRETE TEMPERATURE, C 19

Note: a,b,C,d come from mix proportions worksheet

MICHIGAN DEPARTMENT OF TRANSPORTATION

FORM 1830 CONCRETE PROPORTIONING DATA FILE 300

CONTROL SECTION ID: RESEARCH
JOB NUMBER: MI. TECH.
LAB NUMBER: 99C-1030
GRADE OF CONCRETE: P1
INTENDED USE OF CONCRETE: Pavement (Conv. Form)

DATE: 7/01/1999
SPECIFICATION: 1996 STD SPECS
MIX DESIGN NUMBER: 99-1040

CONCRETE MATERIALS

MATERIAL	SOURCE	SOURCE NUMBER	CLASS	SPECIFIC GRAVITY	ABSORPTION PERCENT
CEMENT	(SEE REMARKS)		I/IA	3.15	
FINE AGG.	Superior S & G	31-45	2NS	2.66	1.14
COARSE AGG.	Presque Isle Stone	71-47	6AA	2.55	1.35
FLY ASH					

CEMENT CONTENT, kg/m^3 335
AIR CONTENT (DESIGN): 6.5% (SPECIFIED): 6.5%
R.W.C: 1.15
FLY ASH CONTENT, kg/m^3: 0

B/Bo : 0.72
SPECIFICATION TOLERANCE (±): 1.5%
THEORETICAL YIELD: 100.00%

WEIGHT OF COARSE AGG. (DRY/LOOSE) kg/m^3	AGGREGATE AND WATER PROPORTIONS QUANTITIES, kg/m^3 OF CONCRETE		
	FINE AGG (OVEN DRY)	COARSE AGG (OVEN DRY)	TOTAL WATER
1328	822	956	167
1338	815	963	167
1348	809	971	166
1358	802	978	166
1368	796	985	166
1378	789	992	165
1388	783	999	165
1398	776	1007	165
1408	769	1014	164
1418	763	1021	164
1428	756	1028	164

REMARKS:
THIS CHART FOR USE WITH CEMENTS OF THE CLASS SHOWN FROM APPROVED SOURCES.

TYPICAL UNIT WEIGHT (DRY, LOOSE) OF COARSE AGGREGATE AS DESCRIBED ABOVE IS 1378 kg/m^3

SPECIAL MESSAGES: Dynamic Fracture Research Project

CC:
S. Vitton-Mi. Tech.
T. Woodhouse-MDOT

JOHN F. STATON
MATERIALS RESEARCH ENGINEER

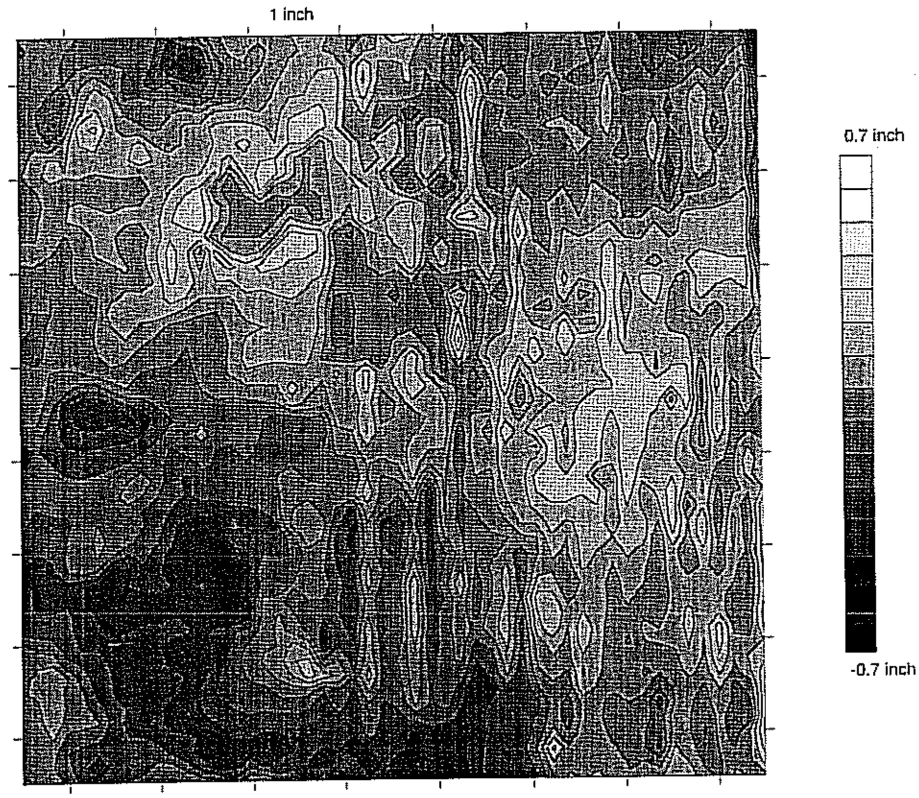


Figure 4.7 (a) Bruce Mines #3 Before testing

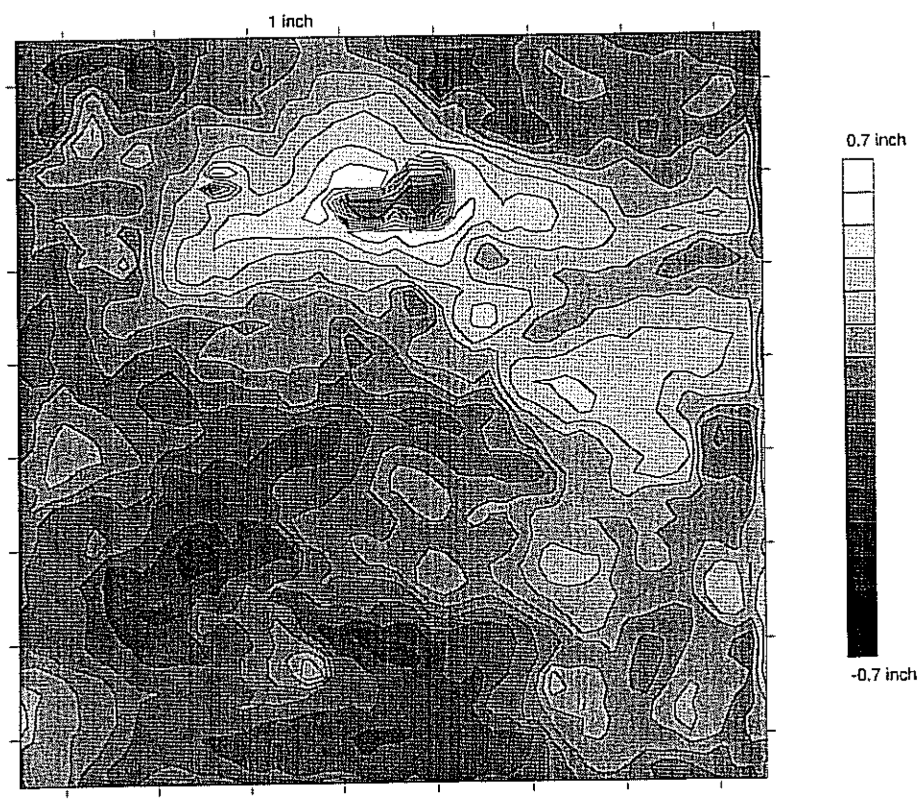


Figure 4.7 (b) Bruce Mines #3 After testing

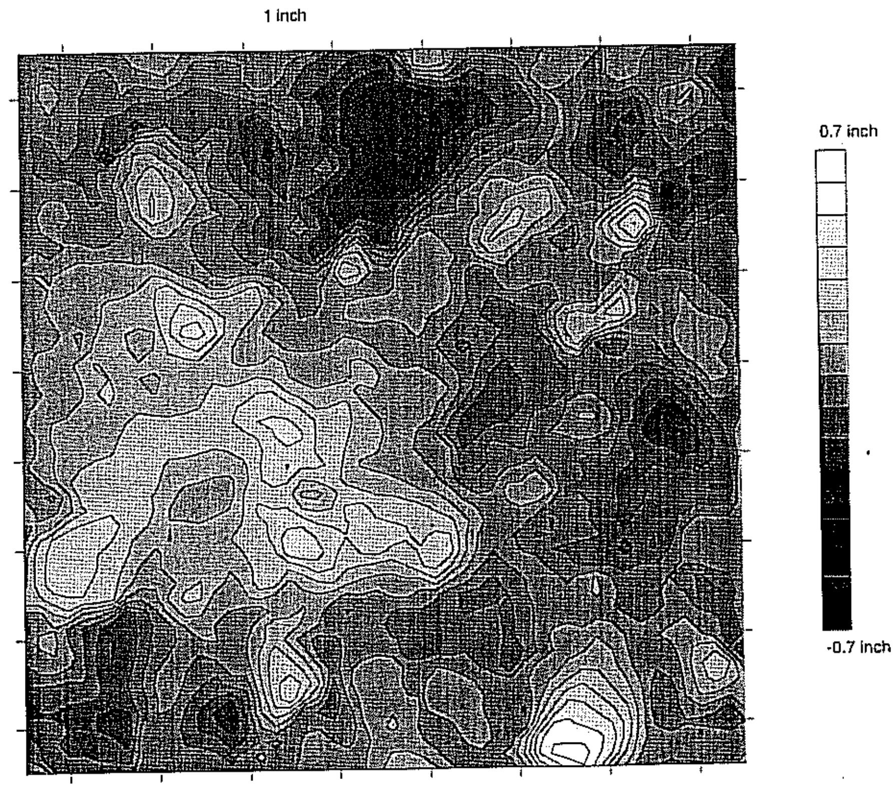


Figure 4.8 (a) Bruce Mines #4 Before testing

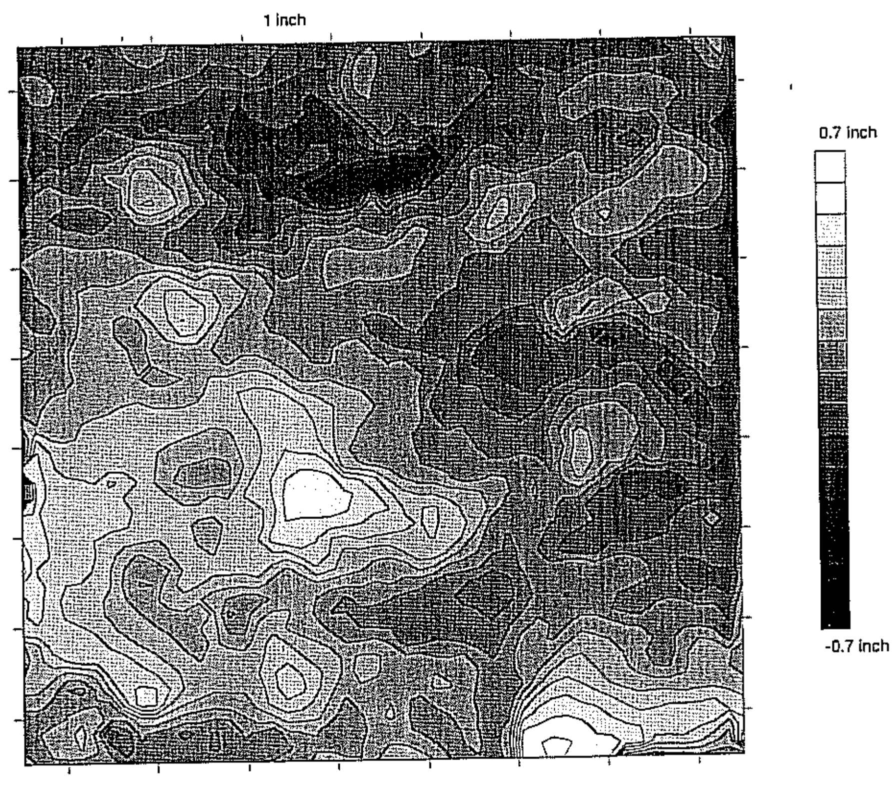


Figure 4.8 (b) Bruce Mines #4 After testing

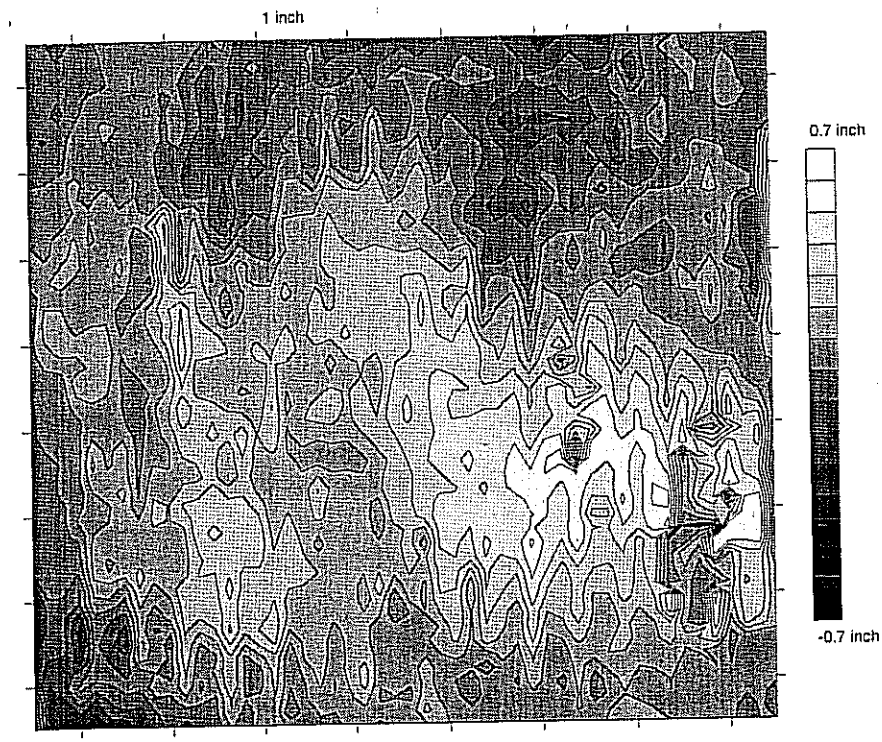


Figure 4.9 (a) Presque Isle #2 Before testing

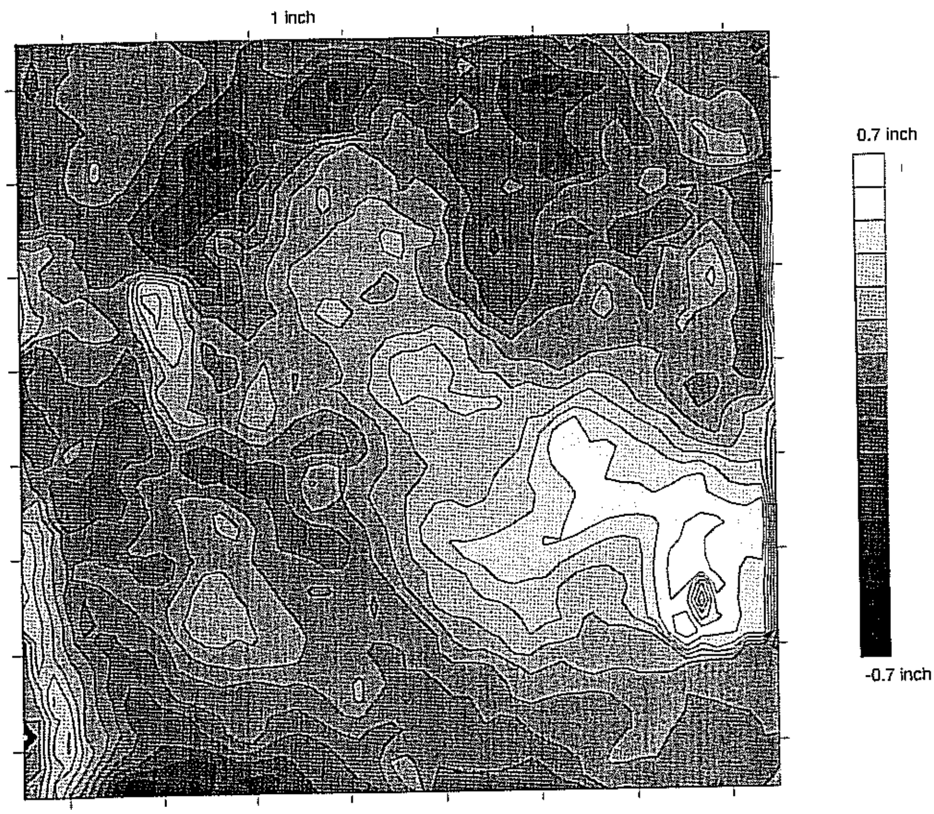


Figure 4.9 (b) Presque Isle #2 After testing

Three observations were made from the topographic results presented in Figures 4.6 through 4.9. First, the black regions on the plots indicate a low region, while the white represent high region on the samples. A general view of the two Bruce Mines, blocks #3 and #4 show a greater black and white contrast than do the Levy #5 and Presque Isle #2. This is confirmed by the data presented in Table 4.2, showing that the maximum distance between the high and low points on the surfaces are greater for the Bruce Mines surfaces than the Levy and Presque Isle concrete surfaces. However, it is recognized that this is not conclusive, but it does suggest that the Bruce Mines surfaces are rougher than the Levy and Presque Isle. A second and somewhat interesting observation, is the linear features that appear in the Levy #5, Bruce Mines #4, and Presque Isle #2 surfaces, prior to aggregate interlock testing, but not present after interlock testing. One possible explanation is that the linear features are due to the effects of rodding during the casting process. Then during aggregate interlock testing, they were eliminated. A second possible explanation is that the linear features are an artifact of the CNC mill measuring system or the software itself, but one would expect the linear feature to be present on all of the surfaces, both before and after.

The third observation is that the topographic plots show a smoothing of the surfaces after interlock testing. This is seen by large areas of more equal elevation. This is also seen in Table 4.2, where the max distances for the before and after results show a reduction in three of the four surfaces, after aggregate interlock testing

While some useful data was obtained from the surface roughness measurements, there were a number of problems with this system. These problems include: (1) the length of time required to record the data (in excess of 4 hours per sample), (2) data acquisition failures (no data recorded mid-stream), and (3) analysis time (from 0.5 to 8 hours per sample).

4.4 Aggregate Interlock Test Results

Sixteen aggregate interlock tests were planned for testing. However, 18 concrete sample blocks were cast since one batch of Levy concrete had low slump (1 batch produced two concrete test samples) an additional batch was made generating two additional Levy samples. Due to system set up and evaluation requirements as well as difficulty with the hydraulic pump system, not all of the 18 samples were successfully tested. While the procedure for setting up the system parameters consumed some of the test samples, the hydraulic pump's problems during sample testing consumed significantly more. The difficulty in the testing, when the hydraulic pump shut down prematurely, was that both the vertical and horizontal loads were lost, making it almost impossible to resume the test where it had stopped. Table 4.3 lists the results of all 18 tests. Of these 18 tests, eight were considered useful tests. Although the number of tests is significantly lower than expected, some useful trends were observed, as well as comparisons with previous aggregate interlock research.

The first test that was considered useful was an initial test that was conducted at a crack width of 0.024 inches and a shear load of 3 kips. This test was conducted on the Levy #1 concrete specimen.

The next two tests that are useable and were tested under the same parameters at a 0.05-inch crack width and a 4.5 kip load are as follows:

Levy specimen #3
Presque Isle specimen #3

The next five useable tests that are directly comparable are the following tests:

Levy specimen #5,
Presque Isle specimen #2
Presque Isle specimen #4,
Bruce Mines specimens #3
Bruce Mines specimen #4

These five tests all had the same crack width of 0.035 inches; a continuous loading cycle at 2 Hz, and amplitude of 3.0 kips. All of the load and displacement versus time plots are provided in Appendix A through D.

The first test conducted was on the Levy #1 specimen, which had a low slump and high compressive strength and was considered expendable, was tested with a 0.0240-inch crack width, 3.0 kip load at 2.5 Hz. The sample lasted for 48 hours without reaching the failure criteria. After 48 hours, it was decided to discontinue the test since it was planned to run the tests so that they would not exceed 48 hours. However, to complete the Levy #1 test, the crack width was increased to 0.050 inches, and the test continued. It failed after 240 cycles or approximately two hours at the 0.050 inches crack width. It was determined, at this point; that the 0.024 inch crack width was too tight (too efficient) to conduct testing in a reasonable time frame, at 3.0 kips.

Following this test, the Presque Isle specimen #3 was placed in the apparatus and tested at a crack width of 0.050 inches and a load of 4.5 kips. This test lasted 41 hours, and reached failure at 147,000 cycles. Following this test, the Levy #3 block was tested under the same conditions as the Presque Isle #3. The results of this test were surprising since the sample failed after 50 minutes, with only 2900 cycles. Based on these test results, it was decided to use a 0.035 inch crack width with a 3.0 kip load at 2 Hz for the remaining test specimens, which now numbered 14. However, as stated previously, trouble with the hydraulic pump system started after these initial tests were completed.

4.4.1 Analysis of Aggregate Interlock Tests

The primary parameter investigated is the amount of degradation that occurred over a given number of cycles for a fully reversing sine wave load. Since the interface degradation results in greater vertical (shear) displacement, which is required to resist the shear load, the aggregate interlock degradation can be given in terms of displacement versus the number of loading cycles.

The first two tests that are comparable are the Levy #3 and Presque Isle #3, which were conducted in the test evaluation phase with a crack width at 0.050 inches and a load of 4.5 kips. The shear displacement versus loading cycles for these tests is shown in Figure 4.10. Note that the loading cycles are plotted on a log scale. In these tests, as noted above, the Levy sample only lasted 2,900 cycles, while the Presque Isle sample lasted 147,000 cycles, or 50 times the cycles to failure for the Levy sample.

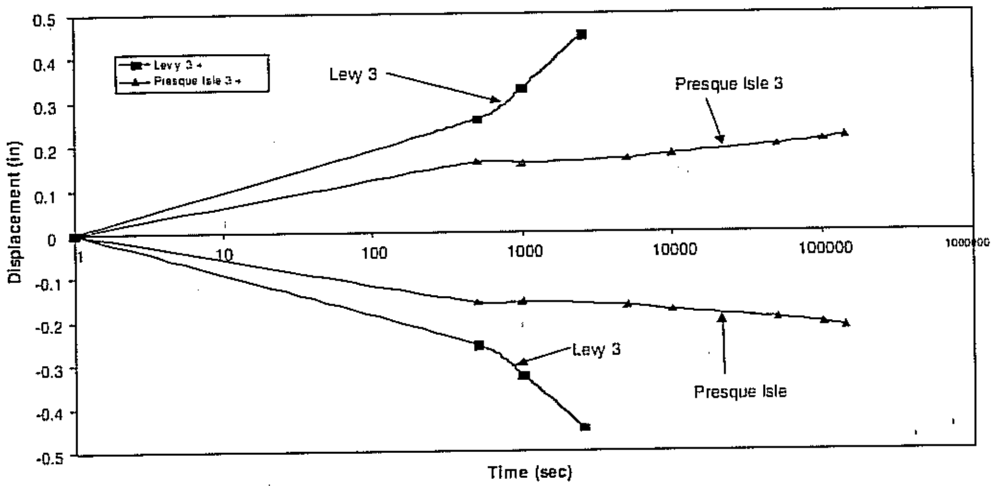


Figure 4.10 Aggregate interlock failure at 0.050 inches, 2.5 Hz, 4.5 kips, 0.4 sec sine wave, 0.6 sec rest, and the load wave produced by DasyLab.

The five tests that are comparable are shown in Figures 4.11 through 4.13, with the Levy #5 test is shown in Figure 4.11, the Presque Isle tests #2 and #4 shown in Figure 4.12 and the Bruce Mines tests #3 and #4 in Figure 4.13. Since two tests were performed for the Presque Isle and Bruce Mines concrete, a dashed line has been placed on the figures to illustrate the average of the two tests. It can be noticed in Figure 4.13, the Bruce Mines concrete specimens, that the two tests are very consistent. The two tests for the Presque Isle specimens in Figure 4.12 are not as close but have the same pattern of interface degradation. Unfortunately, there was only one Levy test for this test parameter so no comparison can be made.

Figure 4.14 presents the combined results of the aggregate interlock test at a crack width of 0.0350 inches and a shear load of 3 kips. Based on previous research it was anticipated that the stronger coarse aggregate concrete would provide better aggregate interlock and degrade slower than the weaker coarse aggregate concrete, as suggested by Colley and Humphrey (1967) and Abdel-maksoud, (2000). However, the opposite happened as seen in Figure 4.15. Under the same loading condition, crack width and failure limit, which was set at 0.5 inches overall; the Bruce Mines concrete reached the failure limit in approximately, 21,000 cycles for block #3 and 43,000 cycles for block #4, an average of 32,000 cycles; the two Presque Isle samples (#2 and #4), reached 345,000 and 350,000 cycles respectively; the one Levy slag concrete sample reached 500,000 cycles and was stopped prior to reaching the failure limit. As stated earlier, these results were opposite of what was expected.

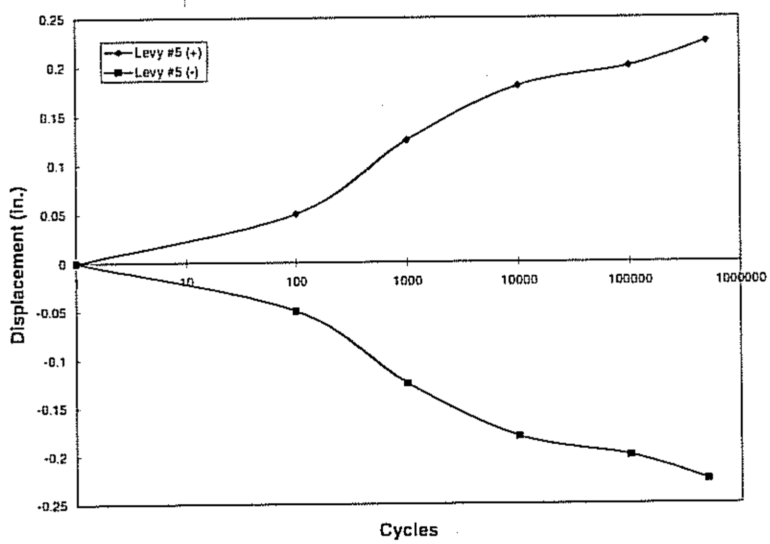


Figure 4.11 Vertical displacement versus loading cycles for the Levy concrete specimen illustrating concrete interface degradation at 0.035 inch crack width.

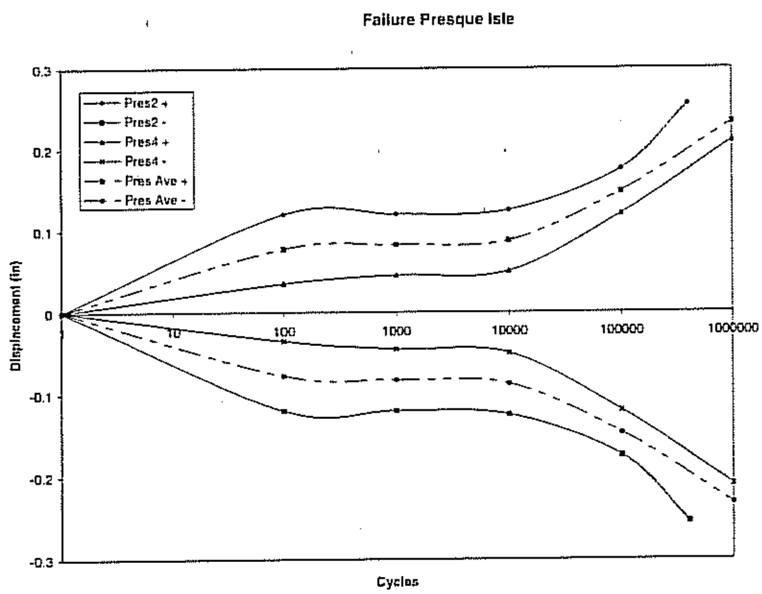


Figure 4.12 Vertical displacement versus loading cycles for the Presque Isle specimens #2 and #4 illustrating concrete interface degradation at a crack width of 0.035 inches.

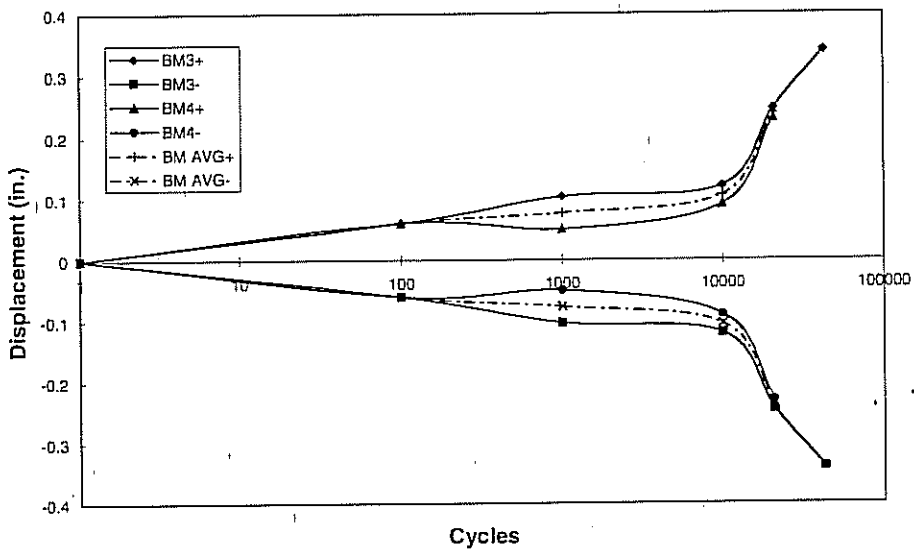


Figure 4.13 Vertical displacement versus loading cycles for Bruce Mines specimens #3 and #4 showing interface degradation at a crack width of 0.035 inches.

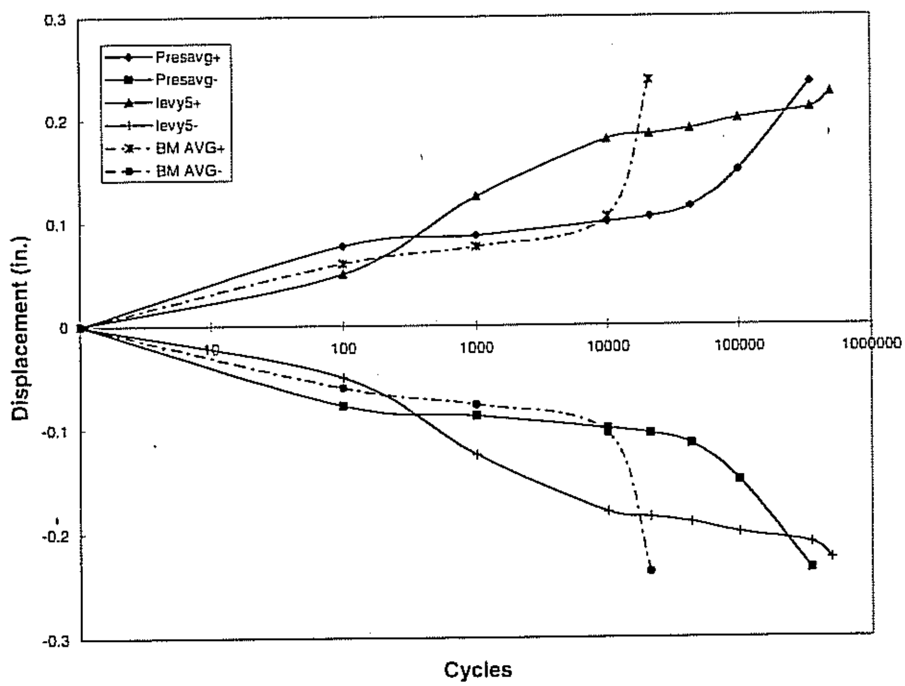


Figure 4.14 Aggregate Interlock sample Failure at 0.0350 inch crack width, 2 Hz, 3.0 kips, continuous sine wave, and load wave produced by MTS 407 controller.

An additional measurement made during the test was to collect the material that was generated below the crack interface. As expected, the majority of this material was well pulverized, with some larger pieces, which fell off the concrete faces when the samples were removed from the sample holders. This material was weighed for each test and the measurements presented in Figure 4.15. From this data, it can be seen that the Bruce Mines samples produced the greatest amount of debris while the Levy samples produced the least amount. This is in light of the Levy concrete being under the same test conditions, with 15 times more loading cycles. Again, it is important to note that these tests were at a crack width of 0.0350 inches.

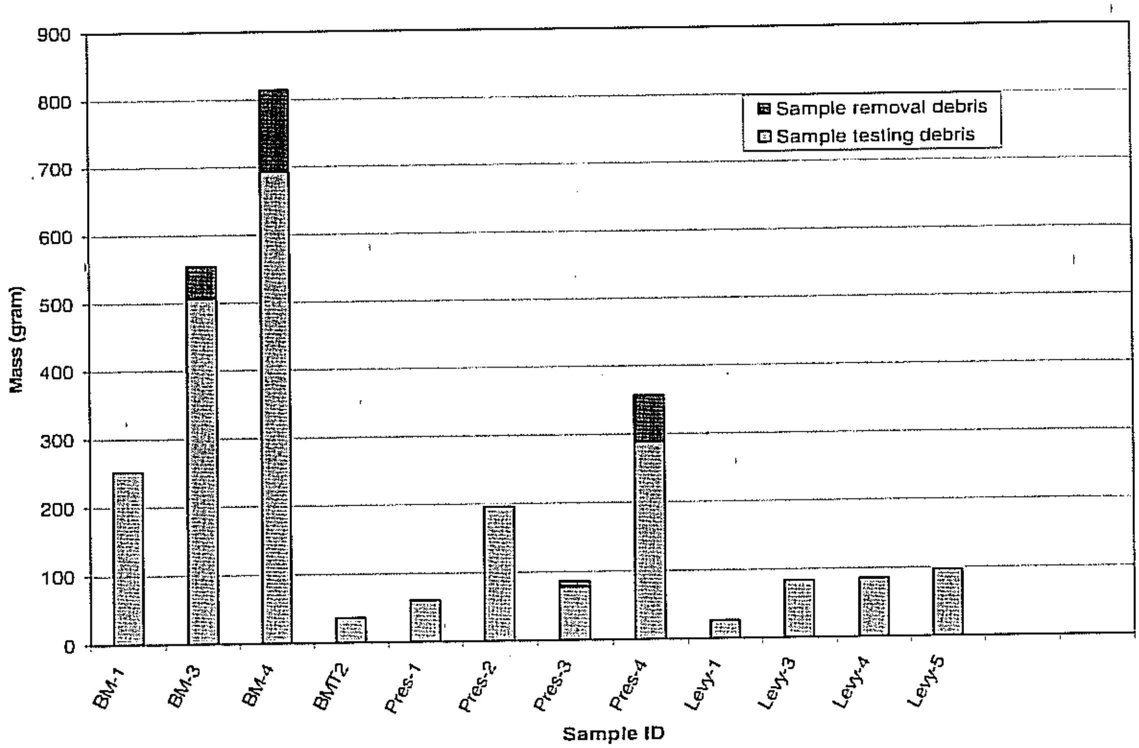


Figure 4.15 Joint interface debris collected after interlock testing.

Table 4.3 Aggregate interlock tests data.

Bold face indicates usable test:

Sample ID	Crack width (in)	Load signal (per sec)	Load Amplitude (kips)	Load Frequency (Hz)	Time test ran	Cycles	Date Tested	Comments
Levy 1	0.024	0.40 sec sine, 0.60 sec rest.	3	2.5	Over 48 hours		8/2/00	Had 2 hrs of operation but failed
Levy 2								Sample had low slump: not used
Levy 3	**0.05**	**0.40 sec sine, 0.60 sec rest.**	**4.5**	**2.5**	**50 minutes**	**2900**	**8/7/00**	Full displacement of 0.5 inches
Levy 4	0.035	continuos	3-5	5	1.5 hours	450	9/20/00	Pump shut down 3 time: Failed
Levy 5	**0.035**	**continuos**	**3**	**2**	**69.5 hours**	**500000**	**10/13/00**	**Good test.**
Levy 6	0.035	continuos	3	2	1 hour	120	10/19/00	Pump shut down 3 time: Failed
Presque 1	0.024	continuos	3	5	3 - 4 hours		9/14/00	Used for system setup.
Presque 2	**0.035**	**continuos**	**3**	**2**	**48 hours**	**345600**	**10/8/00**	**Good test.**
Presque 3	**0.05**	**0.40 sec sine, 0.60 sec rest.**	**4.5**	**2.5**	**41 hours**	**147000**	**8/5/00**	**Good test.**
Presque 4	**0.035**	**continuos**	**3**	**2**	**48.5 hours**	**350000**	**10/16/00**	**Good test.**
Port In 1	**0.024**	**0.1 sec sine, 0.9 sec rest**	**3**	**2**	**5.75 hours**	**20750**	**9/11/00**	**Adjust p-gain after 1 hour Reached 1.5 inches**
Port In 2	0.035	continuos	3-5	5	30 minutes		9/22/00	System failure, bad Accumulator?
Port In 3	0.024	5X0.2 sec sine	8	5	5 minutes	300	9/13/00	Bad data, too high of loads. used for system setup. Failed
Port In 4	0.035	continuos	5-3	5	220 sec	1100	10/8/00	System could not keep up. Failed
Bruce Mines 1	0.024	3X0.1 sec sine, 0.2 sec rest		10	800 sec	2400	9/9/00	New chip, new wave. Failed
Bruce Mines 2	0.024						9/5/00	Failed, new e-prom chip
Bruce Mines 3	**0.035**	**continuos**	**3**	**3, then 2 at 5116**	**2.25 hours**	**21348**	**10/12/00**	**Good test.**
Bruce Mines 4	**0.035**	**continuos**	**3**	**2**	**6 hours**	**43000**	**10/18/00**	**Good test.**

4.5 Aggregate Interlock Results Discussion

Although fewer tests were successful than anticipated, the results are useful. Basically, the aggregate interlock test results at a 3 kip load and a crack width of 0.035 inches showed that the weaker coarse aggregate concrete handled more loading cycles than the stronger coarse aggregate concrete. This result was surprising based on the research results of Colley and Humphrey (1967) and the University of Illinois (2000),. which resulted in the opposite findings. However, in evaluating the test conditions and results as well as examining previous literature, other factors may have contributed to this result. To discuss the results the following six factors will be examined: (1) stress level, (2) crack width, (3) joint surface morphology, initial shear loading degradation, (5) effect of concrete strength, and (6) hydraulic system performance.

4.5.1 Stress Level

The first factor examined concerns the stress level applied to the concrete specimens. The original load applied to the specimens was 9.0 kips, which was based on a wheel load of 9.0 kips and previous research using a 9.0 kip load. After initial testing, this load was found to cause excessive damage to the concrete interface and was subsequently reduced to 4.5 kips and later to 3.0 kips, based on a simplified 2:1 slope stress distribution at mid-depth as discussed previously. This would be consistent with other researchers who also used lower shear stress levels when conducting aggregate interlock testing. Although a relatively simplified analysis, the average shear stress acting on the concrete interface in this research was estimated by dividing the load applied to the interface by the cross-sectional area of the concrete specimen. For a 3.0 kip load[1] the average shear stress was 47 psi at the concrete interface. As a comparison, the University of Illinois's research on studying airport pavements used 72 psi in their original testing, then after experiencing excessive damage to the concrete interface, reduced the loading to 49 psi. The research by Colley and Humphrey (1967) used an average shear stress that ranged from 12 psi to 28 psi. This would suggest that the 47 psi

[1] The cross-sectional area of the concrete specimens was 64 inches.

average shear stress used in this research was high in comparison to the Colley and Humphrey research on road pavements and the Illinois research on airport pavements. For example, an 18 kip single axial load, on a 12-foot wide, 10-inch thick slab, would place an average shear stress of 12.5 psi on the joint interface, assuming of course that the wheel loads were equally distributed across the joint interface.

Interestingly, Colley and Humphrey's research indicated that there appears to be a critical stress level below which the effectiveness of the joint does not appear to degrade. That is, there is a stress level at which, once below this stress level, limited degradation to the interface will occur. On the other hand, Illinois's research suggests that there is both a lower limiting stress at which when exceeded limited joint interface degradation begins and a higher stress level that when exceeded significant interface damage starts. Based on this observation, Illinois suggests that the "allowable stresses (on a joint interface) should be designed in a manner that protects joint interfaces from significant damage rather than designing it as a function of concrete strength." That is, Illinois is suggesting that the joint should be design based on its maximum expected stress level as opposed to specifying a given concrete strength for the PCC pavement.

4.5.2 Crack Width

The second factor is the crack width used during testing. In all of the tests conducted the crack width remained relatively constant during the shear loading as shown in Table 2.1. This indicates that the horizontal hydraulic actuator functioned well.

From field evidence and previous research, the crack width of 0.024 inch is considered an efficient crack width. Consequently, testing at this width for any concrete type, i.e., for different coarse aggregates, should prove effective, as long as excessive loads are not placed on the interface. This was seen in the first slag concrete specimen tested, which ran for 48 hours, with minimal damage and in which the 0.5-inch failure limit was not reached. However, joint openings in Michigan have been measured at 0.060 inches and larger. The Illinois research tested crack width ranging from 0.030 to 0.100 inches. A crack width of 0.060 inches would be considered mid-range, while the joint opening used in this research, 0.0350 inches, would be on the lower end of the

Illinois research. Even at the smaller crack widths, though, the Illinois research indicated that stronger coarse aggregate concrete provides better aggregate interlock than weaker coarse aggregate concrete as seen in Figure 1.5 of this section. Nowland (1968) also suggests that stronger coarse aggregate concrete should provide stronger aggregate interlock. However, the tests conducted at a crack width of 0.0350 inches crack showed that the slag and limestone aggregates, which are weaker aggregates, performed better than the stronger igneous aggregate, as shown in Figure 4.14. That is, the number of cycles to failure was greater for the slag and limestone concrete than for the traprock (basalt) concrete. At a crack width of 0.05 inches for the slag and limestone concrete, however, the stronger limestone performed significantly better than the weaker slag. This finding is consistent with the Illinois research. That is, as the crack width increases the strength of the aggregate becomes more important for aggregate interlock.

A possible explanation for test results at the crack width of 0.035 inch can be provided using the University of Illinois research findings as well as observations made in this research. The Illinois aggregate interlock research proposed two mechanisms that contribute to the mobilization of friction at a PCC joint and which are directly related to crack width. The first mechanism occurs in joints with larger crack widths in which the surface-to-surface contact is largely by coarse aggregate contact. During shearing the interface surfaces will have a greater tendency to dilate, or override the roughness of the surfaces. When dilation occurs, normal stresses develop at the interface due to the restraining action of the concrete slabs. Conversely, the development of normal stresses indicates that the joint is mobilizing shear resistance by dilation. It is the development of normal stresses that also cause significant damage (degradation) to the interface by the crushing and wearing of the aggregate-to-aggregate contacts. The second mechanism occurs in joints at smaller crack widths and with smoother surfaces. At a smaller crack width, joints mobile shear resistance by shearing through the interface roughness and developing frictional resistance and to a lesser degree by dilation, i.e., developing normal stresses at the interface. A result of mobilizing frictional resistance as opposed to dilation is that smaller normal stresses develop at the interface. This concept is illustrated in Figure 4.16 where the smoother surface (block A) has a smaller normal force developed as opposed to the rougher surface (block B) with a larger normal force developing.

Again, the development of the normal forces is a result of the two slabs being restrained from movement as the surfaces attempt to push past each other apart during shear loading.

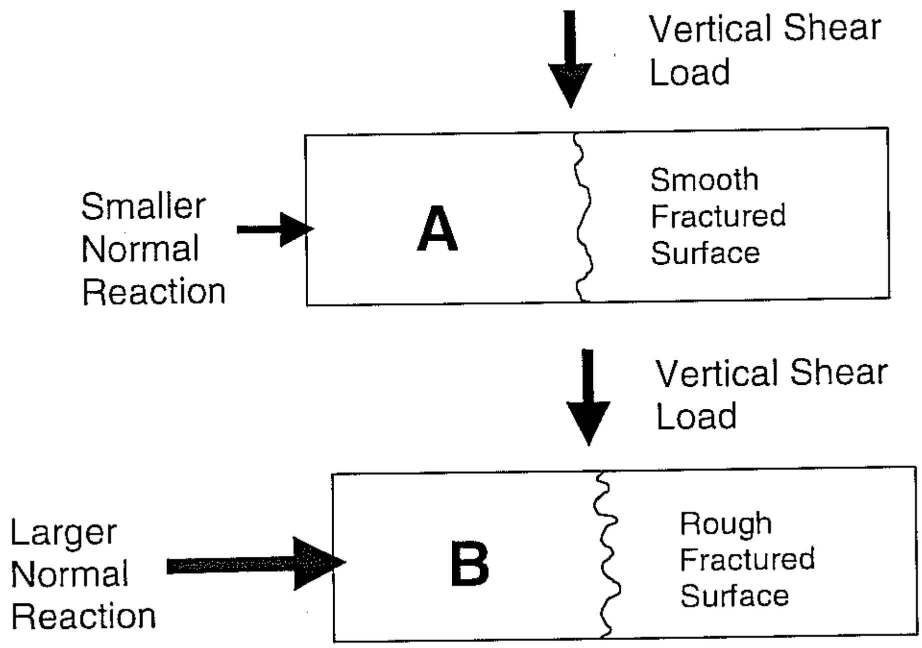


Figure 4.16 Rough versus smooth fracture surface and the development of normal stresses.

In considering these two mechanisms coupled with the higher shear stress levels used in this research, it is possible that the shear stresses were too high. At the 0.0350 inch crack width, coupled with a high interface shear stress of approximately 49 psi, the stronger coarse aggregates may have sheared through the weaker cement paste as well as dislodging embedded coarse aggregate. This would explain the large amount of material generated from the interface of the igneous samples. This was also seen in the Illinois research where the shear stress level on basalt coarse aggregate concrete (traprock) was increased by a factor of 1.6 with the resulting interface degradation increasing by 3.7. As shown in Figure 4.2, the difference in strength for the Bruce Mines concrete between the coarse aggregate (both dynamic and static strength) and the concrete is significantly large. On the other hand, for the slag concrete, the strength of the concrete is somewhat greater than the strength of the slag coarse aggregate. When the difference in strength between the coarse aggregate and concrete is large, such as for the Bruce Mines concrete, the possibility of the stronger coarse aggregate gouging the cement paste appears plausible for higher shear stresses. When the strength of coarse aggregate and the cement paste are similar, however, it is equally plausible that this would diminish the damaging effects of protruding stronger aggregates and would enhance the mobilization of friction with smaller normal stresses developing at the interface and consequently less interface damage. Thus, at a crack width of 0.0350 inches and the shear mobilization through friction as opposed to dilation the lower strength coarse aggregate may produce better aggregate interlock at high shear stresses. However, a number of other factors must also be considered in evaluating the results of these tests.

4.5.3 Joint Surface Morphology

Another significant factor involved in joint efficiency is the morphology or roughness of the concrete surfaces as noted above. In general, for any shape concrete surface, i.e., smooth or rough, the smaller the crack opening the greater number of surface-to-surface contacts there will be as opposed to larger crack widths. The larger number of surface-to-surface contacts during shear loading would then reduce the shear stress level at the contact points for a given surface load. The roughness or shape of the concrete surface also plays an important role in the shear stress transfer. For surfaces that are relatively flat or smooth the contact area will dramatically reduce as the crack opening increases. For rough surfaces the displacement that the surface may have to undergo to achieve shear resistance may be larger but at some point the surfaces will come into contact.

The difference in the morphology of the surfaces can be seen in Table 4.2 in the measurement of maximum roughness. Maximum roughness was defined as the maximum distance from the lowest point on the concrete surface (as defined by the four corners of the concrete specimen) to the highest point as illustrated in Figure 4.17. While visually the Bruce Mines concrete had the roughest surface and the slag the least rough, the measurements in Table 4.2 confirm this observation with a maximum roughness of 1.35 and 1.40 for the Bruce Mines concrete, 1.20 for the Presque Isle concrete and 1.05 for the slag concrete. The topographic mapping of the surfaces also suggest that the Bruce Mines had the roughest surface followed by the Presque Isle then the slag concrete as seen in Figures 4.6 through 4.9.

Although only two tests were conducted at a crack width of 0.05 inches, there was a large drop off of loading cycles for the slag concrete compared to the Presque Isle concrete as shown in Figure 4.10. This result would suggest that for smoother surfaces a larger crack width would result in a reduced interlocking capability. This can be seen in Figure 4.18 where the loading cycles for the slag concrete, at a crack width of 0.024, 0.035 and 0.05 inches, are compared. It should also be noted that a 4.5 kip load was used in the 0.024 crack width tests. Although insufficient tests were conducted to confirm that aggregate type plays a role in joint efficiency, the data does suggest that if aggregate type influences the shape of a joint's fracture surface (morphology), then the crack width opening can strongly influence joint efficiency with rougher surfaces performing better at larger crack widths. Conversely, the smoother surfaces will perform poorly at larger crack width compared to the rougher surfaces. In contrast to this, the data also suggest, although again not enough tests were conducted to confirm this, that smoother joint surface may perform better at smaller crack widths than the rougher surfaces, depending on the level of stress induced at the interface.

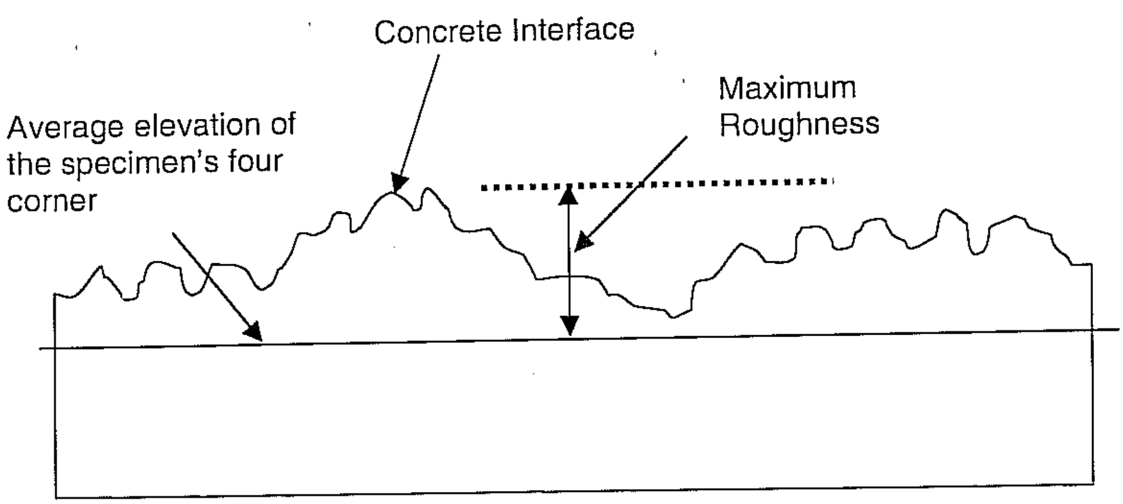


Figure 4.17 Definition of maximum roughness.

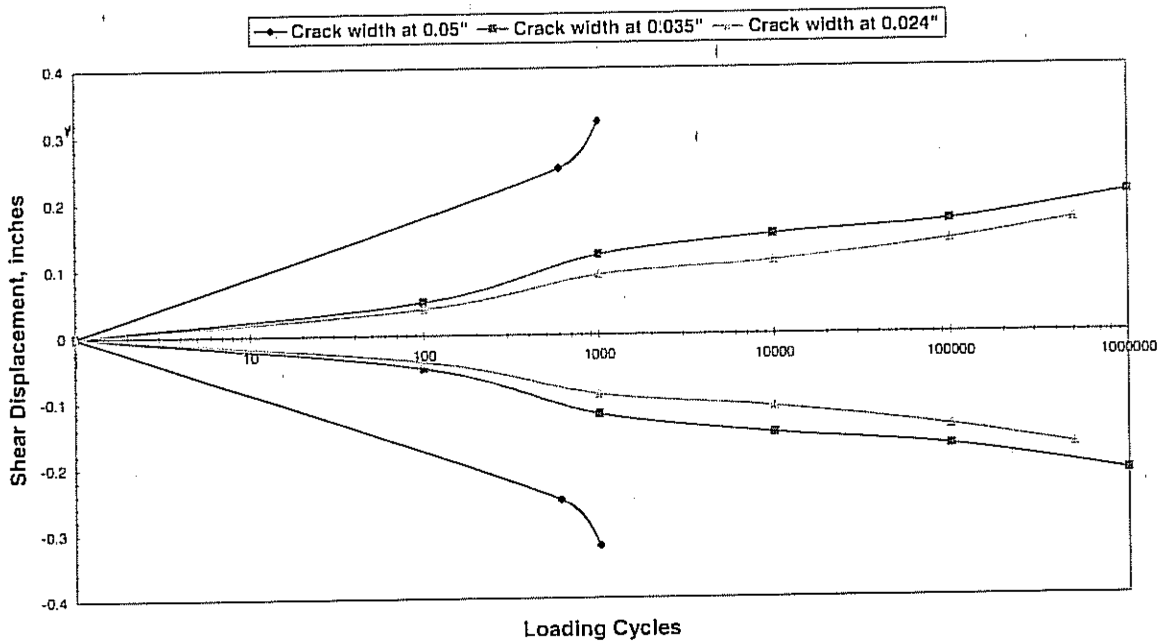


Figure 4.18 Comparison of shear displacement versus loading cycles for slag concrete at cracks widths of 0.024, 0.035, and 0.050 inches.

4.5.4 Initial Shear Loading Degradation

In all of the aggregate interlock tests conducted in this research as well as the research reported by Illinois, the majority of interface degradation occurs in the in the first 1000 loading cycles followed by degradation between 1000 and 10,000 cycles. As noted previously, all of the degradation plots in this chapter have the loading cycles plotted in a log scale. In viewing these plots, then, the change in degradation up to 1000 and between 1000 and 10,000 loading cycles is relatively apparent. Interestingly, the degradation in the Bruce Mines concrete has a significant change in the rate of degradation at 1000 cycles, the slag concrete also at 1000 cycles while the Presque Isle concrete has a change at 10,000 loading cycles. In the Illinois research, the change in rate of degradation was noted to be a function of crack width. For example, for crack widths at 0.01 inches (0.254 mm) the change in rate of degradation was at 1000 loading cycles while for a crack width of 0.030 inches (0.76 mm) the number of loading cycles increases to 10,000 cycles. A reason for this change in degradation behavior between

1000 and 10,000 was not discussed but appears to be relatively consistent throughout the aggregate interlock research.

An additional consideration concerning interface degradation is the initial crack width. At small crack widths the shear displacement on the first cycle should be less than for larger crack widths. That is, as the crack opening increases it should take more shear displacement to resist the shear loading. This means that at a fixed shear displacement limit of 0.5 inches, the larger crack openings should reach this limit at a lower number of cycles than at smaller crack widths. However, this was not evaluated in this research but should be a factor in considering the amount of displacement between the two concrete blocks during future aggregate interlock testing.

4.5.5 Effect of Concrete Strength

As reported earlier research on the effect of concrete strength on aggregate interlock was inconclusive with some research indicating that it is important while others indicated that it had a minimal effect. While not enough tests were successfully conducted in this research, it is interesting to compare the strength of the concrete for the specimens tested at 0.035 inches. As shown in Figure 4.14 the slag concrete, which had the largest number of loading cycles, had a 28-day unconfined compressive strength of 4,704 psi, followed by the Presque Isle concrete at strength of 6,838 psi and the Bruce Mines concrete with strength of 4,456 psi. While the Bruce Mines concrete had the lowest concrete strength and load cycles to failure, the slag concrete had the largest but its strength was significantly closer to the Bruce Mine concrete strength. Consequently, this data did not support the research that shows that concrete strength is an important parameter in aggregate interlock. This collaborates the Illinois research that state "The strength of intact concrete alone, as measured in a 28-day unconfined concrete compression test, does not have a significant impact on joint performance. Other factors such as aggregate size, aggregate quality, and roughness have a more dominant role on joint performance under cyclic shear than concrete strength." However, more testing would have to be conducted before this issue can be resolved.

4.5.6 Hydraulic System Performance

A significant factor in the dynamic testing of materials is the ability of the servo-hydraulic closed loop control system to conduct the required test. That is, can the testing system perform what the control system is requiring? Since the aggregate interlock tests were conducted in load control at a 2 Hz frequency, the ability of the system to consistently provide this load must be considered. In addition to these factors, a more critical factor is the stiffness of the material being tested; in this case the aggregate interlock stiffness. This issue was addressed in chapter three of this section, which discussed the PID control settings of the MTS 407 controller (for the vertical actuator) and the adjustment of the PID to account for the stiffness of a given test material. In general, the stiffness of the material being tested determines how much displacement will occur for a given load. For highly deformable materials there will be significant displacement at a given load. In dynamic testing this becomes a problem since the hydraulic actuators will need to travel further to obtain the desired load. This in turn requires a higher flow rate of hydraulic fluid from the pump. For many dynamic tests the size of the hydraulic pump limits the speed at which a test can be performed. A major consideration of testing aggregate interlock of concrete specimens is not only the initial stiffness of the concrete interface but, as shown by the results in this research, the change in stiffness as degradation occurs. That is, the amount of displacement (degradation) increases for each 3 kip load applied. This is a fundamental consideration in examining the results of this research as well as other research where cyclical loads are applied to a degrading interface.

The aggregate interlock research using hydraulic actuators that was similar to this research, was the Colley and Humphrey (1967) and the Illinois research (Abdel-maksoud, 2000). In the Colley and Humphrey testing two hydraulic actuators were used, one on each side of the joint. However, no discussion was provided concerning the performance of the hydraulic system. Since both actuators were in compression and were simply offset by a time delay to simulate traffic loading, though, it is highly likely that they performed adequately. The difficulty in dynamic testing is when a loading sequence requires both compressive and tension loading in the same cycle, which was required in this research and also in the Illinois work. In the Illinois work, however, the testing frequency was significantly longer. According to Abdel-maksoud (2000) the majority of tests were conducted at a loading rate of 0.05 Hz and some at 0.10 Hz. However, no mention of the waveform type used in the testing was provided. It is assumed from the experimental discussion that a triangular waveform was in fact used. This means that the tests were conducted at a 20 second and ten second loading cycle, which is longer than the 2 Hz frequency i.e., 0.5 second loading cycle, used in this research. In the Illinois work a 25 kip actuator was used with a 20 gpm hydraulic pump and an 8500 Instron controller. Based on previous research conducted by the author this system should be adequate for testing the aggregate interlock based on two assumptions. First, the 8500 Instron controller[2] is an excellent controller with the ability to use auto-adapting loop shaping to change the PID setting as the system stiffness changes during testing. However, there is no discussion in the Illinois research concerning the use of the auto-adapting capability of the Instron unit or if not available how the system response to changing stiffness was handled. However, the following statement concerning changing stiffness during aggregate interlock testing was provided: "The stiffness of the ram needed to be adjusted because of the reduction in the stiffness at the crack interface as testing progressed." This is the only statement concerning changing aggregate interlock stiffness in the Illinois research. However, the statement is somewhat confusing since the stiffness of the actuator should not change. It is the stiffness of the aggregate interlock interface that is changing and the only way to respond to the stiffness change is to adjust the control response of the system through the PID control settings. The second reason that the Illinois test system should have functioned adequately is that a 25 kip actuator with a three inch stroke and a 20 gpm pump should not have any difficulty conducting a 20 second loading cycle in both compression and tension. As a comparison, in this research two 55 kip actuators with ten-inch strokes were used with a 35-gpm pump.

In this research two MTS 407 digital controllers were used to independently control both the horizontal (to maintain a constant crack width in displacement control) and the vertical (shear loading in load control) actuators. The PID settings of the 407

[2] A 8500 Instron controller was used to test the 28-day unconfined compressive strength of the concrete test in Section Four.

controllers were set using a Levy slag concrete specimen. This control setting was used throughout all of the testing, since it was believed that the Levy slag concrete would have a lower joint stiffness than the Presque Isle or Bruce Mines concrete and require the most sensitive settings, e.g., highest gain setting. Accordingly, testing concrete specimens with higher stiffness (it was believed) would not require as sensitive a control setting, i.e., if the system response works adequately for the lower stiffness material it should function equally well for the higher stiffness material.

An example of the load and displacement versus time record for the Levy slag concrete specimen #5 (for which the control setting for all of the aggregate interlock testing was set) is shown in Figure 4.19. The displacement axis in Figure 4.19 is the total movement of the hydraulic actuator and not the LVDT's, which were measuring the displacement between the two concrete specimens, i.e., the relative displacement or shear displacement (degradation). It can be seen from this figure that the 3 kip compressive (positive) and tensile (negative) load was well maintained over the test[3]. The corresponding movement of the hydraulic actuator was also relatively consistent indicating that the test functioned well.

Figure 4.20 illustrates the load and displacement versus time for the Presque Isle (limestone) #4 concrete specimen. In this figure it can be seen that the start of loading was at 3.5 kips although the controller was programmed for 3 kips. The larger shear load may have resulted from the controller overreacting to the stiffer interface. However, between 40,000 and 50,000 loading cycles a change in the stiffness of the specimen occurs resulting in a change the shear load being applied. This change, at about 45,000 cycles, can also be seen in Figure 4.12 where there is a change in the degradation rate of the specimen, i.e., an increase in shear displacement. In addition, the ability of the control system to produce equal 3 kip compression and tension loads is changing with the system producing a slightly decreasing in compressive load while the tension load decreasing rather rapidly. As speculated above, it is believed that the primary reason for this happening is that the control settings (PID) were set too high to allow the limestone concrete test to run properly. In addition, as noted in the experimental setup discussion,

[3] The Levy #5 test run for 500,000 cycle which was 250,000 seconds.

the hydraulic system was also experiencing cavitation of the hydraulic fluid in the return lines, which may have also been a factor in the poorer response.

Figure 4.21 illustrates the load and displacement versus time for the Bruce Mines (basalt) #4 concrete specimen. This response is fundamentally different than the slag or limestone concrete. First, note that the vertical displacement of the hydraulic actuator is very small at about 0.1 inches. As a comparison, the initial vertical displacement of the limestone concrete is about 0.28 inches and the slag concrete is approximately 0.30 inches. This indicates that relatively little displacement was required to reach the 3 kip load in either compression or tension for the basalt concrete. Basically, the load was relatively consistent up to approximately 11,000 loading cycles, with a gradual increase in actuator displacement (degradation). However, at approximately 11,000 cycles the system became erratic and essentially unstable. This response occurred at 11,000 loading cycles for both basalt samples tested. After 11,000 loading cycles the hydraulic system was unable to apply equal compression and tension loads on the concrete. In effect, the hydraulic system was only able to apply the 3 kip compression load and limited tension load. This resulted in the hydraulic actuator continuously pushing down on the specimen with only a small tension load being applied during each cycle to relieve the compression load. The continuous downward movement of the actuator is seen as positive movement in Figure 4.20. As with the limestone concrete the cavitation in the return line may have played a role in this erratic behavior. However, it is strongly believed that the basic reason for this response is the inability of the MTS 407 controllers to adequately control the test due to the changing interface stiffness. Although the MTS 407 controller has PID control capability, it is made for relatively straightforward testing. It appears that a different PID setting should have been used for each aggregate type tested. It is believed that this would have resulted in better test data and an elimination of the erratic behavior of the system.

In reviewing Figure 4.14 it can also be clearly seen that both the limestone and basalt concrete had far less degradation between 1000 and 10,000 cycles than did the slag concrete. If the testing system had not become unstable it is likely that the test results would have been reversed with the basalt and limestone tests producing less degradation than the slag concrete for a given number of loading cycles. It is also likely that the large amount of damage (as measured by the amount of fragments collected after the test) as seen in Figure 4.15 can be explained by the hydraulic system becoming unstable. Therefore, it is strongly believed that the inability of the 407 controller to control the Bruce Mines test played a significant role in the low number of loading cycles to failure. In future aggregate interlock testing it is recommended that either an individual test should be conducted to set the PID control setting tests for each aggregate type tested or a more advanced controller such as the MTS TestStar controller, which can better handle changing system stiffness, should be used.

4.6 Aggregate Interlock Summary

Based on the research conducted in this report as well as existing aggregate interlock research the following summary is provided:

1. In reviewing aggregate interlock test systems used by other researchers, it is believed that the aggregate interlock test system developed and used in this research can be an effective system for testing aggregate interlock provided the following changes are made. First, the cavitation problem with the hydraulic return line must be eliminated. This can be accomplished by reducing the length of the supply and return hoses as well as slowing down the test frequency to one hertz (as opposed to two hertz). While changing the frequency will double the time for testing, it should also improve the test results. Second, a different controller must be used to control the vertical (shear loading) actuator. A MTS TestStar controller (or equivalent) is recommended to accomplish this task. Third, even with a better controller the PID control setting should be set based on the aggregate type concrete being tested. That is, one concrete specimen per coarse aggregate type should be used to determine the system response and PID control settings. It should be noted, however, that this specimen once used for control setting would probably not be useful for further testing. The main reason for this is that significant degradation occurs in the early stages of loading.

Consequently, it is likely that during the control setting that erratic behavior may occur, which will render the specimens as unusable.

2. It was apparent in the test results that aggregate interlock stiffness varies significantly between aggregate concrete types. The variation in stiffness is a function of the degradation of the interface.

3. The aggregate interlock test set is relatively easier to use and can be set up in a short period of time. It has been estimated that a test specimen (cured and ready for testing) can be set up in approximately two hours. As a comparison, in personal communications with the personnel at the University of Illinois it was stated that it took them approximately two days to set up a test sample.

4. Another feature that significantly improves aggregate interlock testing is the ability to maintain a constant crack width during testing. Since the horizontal actuator is used to accomplish this, it is relatively straightforward to adjust the crack width for any width desired. In addition, it would be possible to vary the crack width to simulate warm (small crack width) and cold weather (large crack widths) effects during a single test if desired.

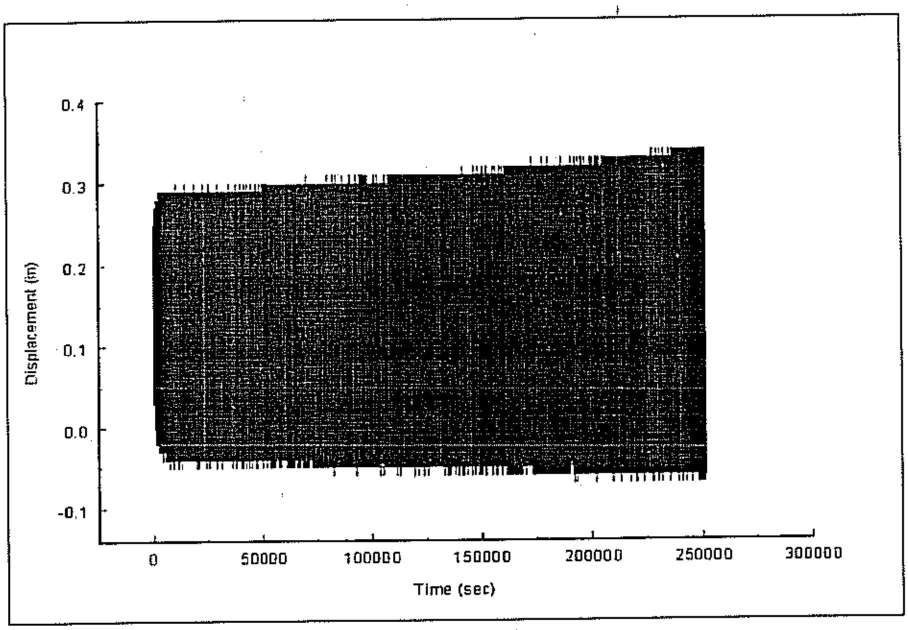
5. While not enough successful aggregate interlock tests were conducted, the data that was obtained suggests that for small crack widths the coarse aggregate type may not significantly affect aggregate interlock. This is especially true for crack widths at 0.024 inches. However, for large crack widths the strength of the coarse aggregate may play an important role. This was demonstrated in the Illinois research and in the two tests conducted at 0.05 inches in this research. However, it is unclear as to whether the concrete surface morphology plays a more critical role or the strength and deformation properties of the coarse aggregate in maintaining aggregate interlock.

6. However, it appears that the coarse aggregate type does affect the morphology of the concrete fracture surface. While the method used to quantify surface morphology was not as successful as anticipated, it did reveal to some extent that the stronger aggregate (Bruce Mines) generated a rougher surface than the weaker aggregate (Levy slag). However, additional testing will have to be conducted to verify this observation. Newer technologies are now available that may be

provide a better means of quantifying the surface morphology, such as the use of digital imaging systems.

7. The sample fracture device worked well producing consistent concrete specimens. It is recommended that future testing monitor the strength of fracture as well as the deformation to failure. In fracturing the concrete it was apparent that coarse aggregate types play a role in the strength at fracture. For example, the Levy slag concrete always required higher pressure to fracture the concrete. It is possible that this higher pressure required for fracture might generate smoother fractured surfaces with a high percent of coarse aggregate fractures. Lower pressure fracture appeared to produce rougher surfaces with less coarse aggregate fracture.
8. Concrete strength does not appear to be a significant factor in aggregate interlock performance at large crack widths. This may be an important fact for future testing since it suggests that testing specimens at significantly different times in relation to their 28-day strength may not be important. That is, concrete may be tested anytime after a 28-day cure.
9. The stress level used in aggregate interlock testing should be no greater than 49 psi and preferably less.
10. The aggregate interlock test was conducted under pure aggregate interlock since no base reaction was provided.

Levy #5 Displacement



Levy #5 Load

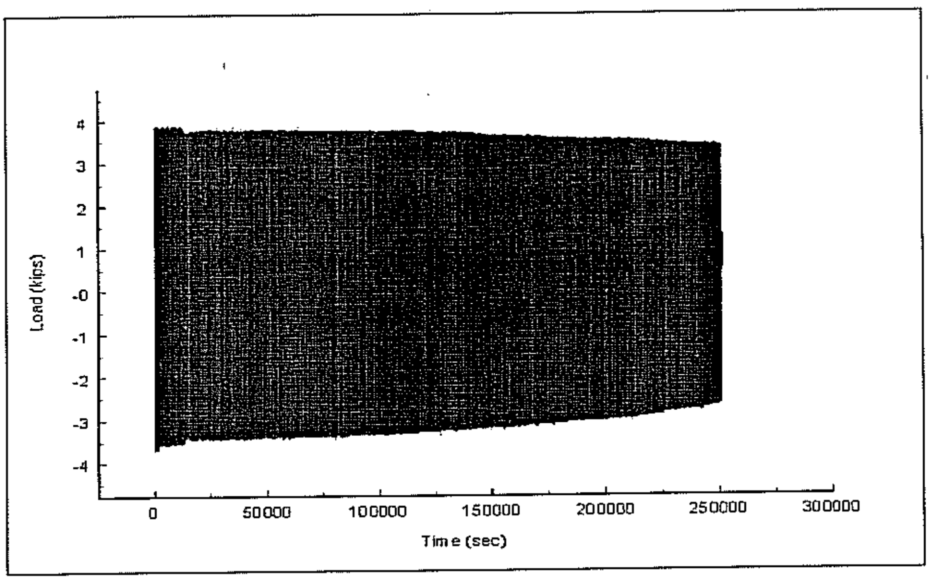
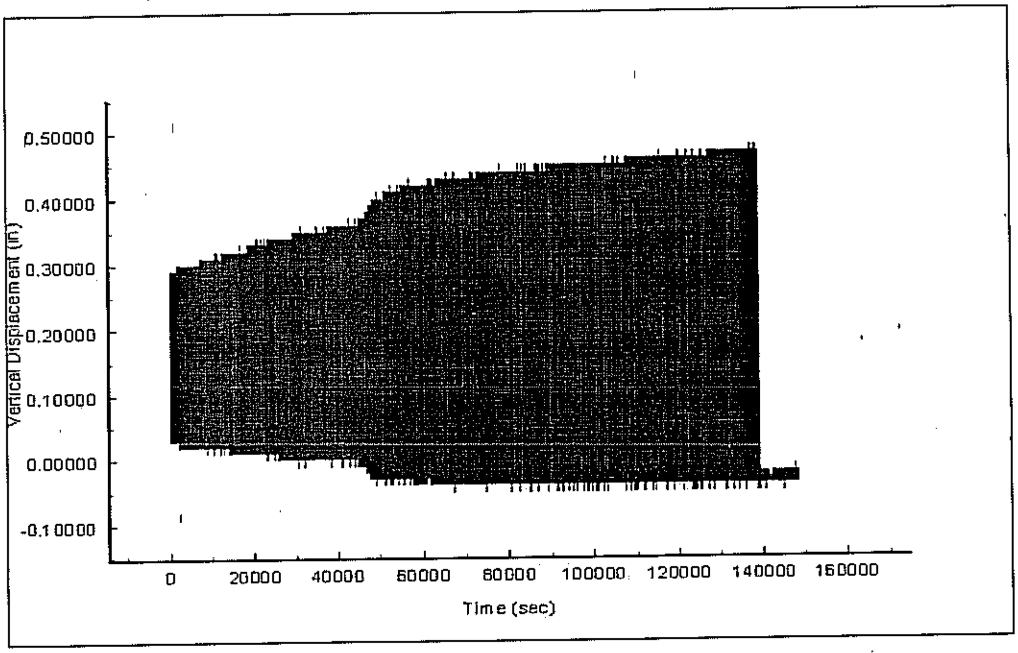


Figure 4.19 Load and displacement response versus time for slag concrete.

Presque Isle #2 Displacement



Presque Isle #2 load

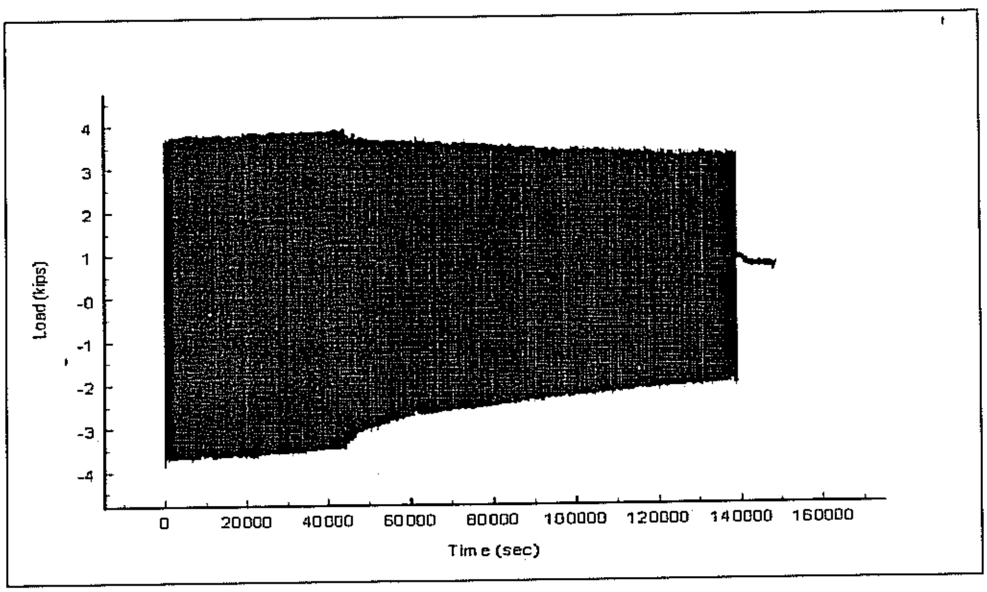
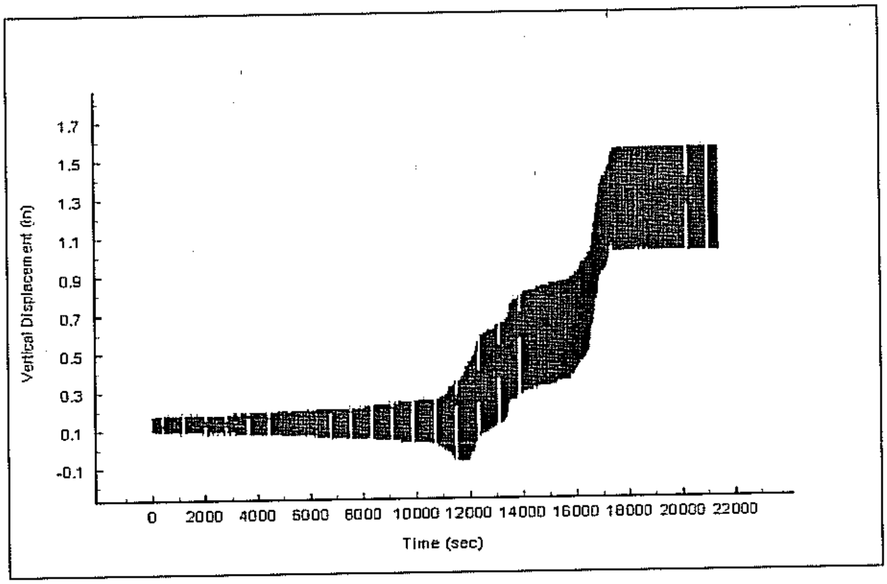


Figure 4.20 Load and displacement response versus time for Presque Isle concrete.

Bruce Mines #4 Displacement



Bruce Mines #4 Load

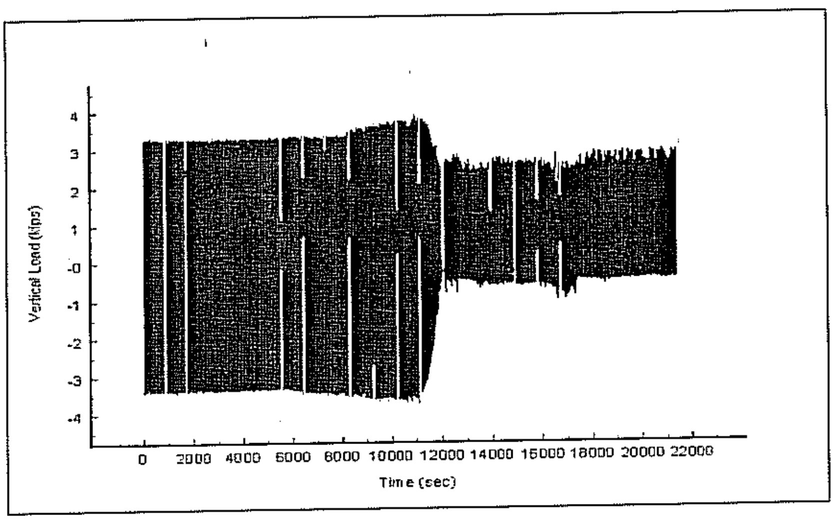


Figure 4.20 Load and displacement response versus time for Bruce Mine concrete.

5 Conclusions and Recommendations

There were five objectives for the research presented in this section. The first objective was to make improvements in the aggregate interlock test system to better replicate field conditions. The second objective was to prepare consistent concrete samples for testing with only the coarse aggregate as a variable. The third objective was to improve the ability of the sample fracture device to better split the concrete test blocks. The fourth objective was to develop a method to characterize the fracture surface of the concrete. The fifth and final objective was to conduct aggregate interlock tests on concrete made from different coarse aggregate types. This chapter provides the conclusions and recommendations from this aggregate interlock research.

1) The test and control system used was reprogrammed from a purely compressive loading to both a compressive and tensile load on the concrete sample interface with a 0.9 second constant minimum load. However, due to system limitations, the system was not able to produce a cyclical wave followed by a rest period. The main reasons for this was that firstly the hydraulic system was not able to maintain the loading throughout the 48 hour test period due to caviation in the hydraulic return lines. Secondly, the system control unit (MTS 407) was unable to change its PID control settings to adequately handle the changing stiffness of the interface as degradation occurred.

2) The same concrete mixing procedures as in prior projects at Michigan Tech were used, but less consistency in the concrete mixes occurred based on the 28-day compressive strength testing. One possible explanation may be in the mixing and rodding operations.

3) A torque wrench was used to place a constant load on all of the threaded rod secured in the concrete, prior and during sample fracture. In addition, anchor nuts were repositioned to the ends of the embedded threaded rods to provide better anchorage.

These changes improved the performance of the sample fracture device, produced accurate data, and effectively produced cracks in all of the test blocks except but one.

4) With the improvements of the sample fracture device and more consistent fracture surface development, the concrete with stronger coarse aggregate had rougher surfaces, with more aggregate pullout, while the weaker coarse aggregate, had less rough surfaces, with more aggregate fracture.

5) The method of using a CNC mill with an LVDT displacement gauge, only proved to be partly successful. The main obstacle was in collection and analysis of the data. However, surfaces analyzed indicated, both visually and numerically, the difference in roughness, but not in an overly convincing way.

6) The aggregate interlock tests were only partly successful. Both system and hydraulic pump failures prevented the majority of the concrete samples from being tested. The samples that were tested indicated that at a 0.035 inch crack width, the weaker coarse aggregate concrete appeared more efficient than the stronger coarse aggregate concrete. However, it is believed that due to the higher stiffness of the stronger aggregate concrete coupled with problems with the control system, the loading system become unstable resulting in early failure of the stiffer aggregate concrete. The research suggests that future aggregate interlock testing should be conducted for crack width openings from 0.035 to 0.06 inches to study the effectiveness of different types of coarse aggregate.

7) The aggregate interlock tests at a crack width of 0.024 inches indicated that this is an effective crack width regardless of the aggregate type. However, it is possible that stronger coarse aggregate concrete under very high vehicle loading may experience more degradation due to the gouging of the cement paste by the coarse aggregate.

The following recommendations are presented for future research an aggregate interlock testing:

1) Due to a continuous, non-linear change in the stiffness of the interface of the concrete samples, a MTS Test-Star controller (or equivalent) should be used to generate the desired load wave and control ability. The advantage of the Test-Star controller (or an equivalent) is that it can continuously adjust the PID (Proportional gain, Integral gain, and Derivative gain) to compensate for the ever-changing interface stiffness during testing.

2) The effect of aggregate base and subgrade support for PCC pavements in aggregate interlock was not studied in this research. However, further research should be conducted to investigate the interaction of base and the aggregate interlock load transfer mechanism. This has been anticipated with the addition of the threaded rod to the fixed end holder. Within the range of the fixed end holder stiffness, an additional set of plate steel may be fabricated that will add stiffness at a given level. It is possible that the test system can be modified to produce a load transfer mechanism that represents the sub-base material load transfer capabilities.

3) Surface morphology of the concrete interface surfaces should be measured using stereographic imagery, which would allow a better characterization of the concrete surfaces to be investigated. This would be very important in investigating the change in surface morphology with variations in crack width. This information could be entered into a mathematical model to determine a relationship of surface texture to crack width.. This is important to show that on larger crack widths, there is less surface area to generate the required load transfer. It could also be used to develop guidelines for crack width in the field.

4) The MTS 30 gpm pump used for testing did not perform as expected. It was assumed that the system could keep up with most dynamic loads since it was designed for earthquake loading. Initially, the PID was adjusted every 10 minutes, or if the load dropped below 80% of the design load. This would turn into continuous adjustment and eventual destruction of the sample as the rate of loading continued to increase. Comparison of tests would be impossible as would

running a test for 48 hours. After testing and failing samples without the proper loads being maintained, it was determined that the PID control had to be manually changed for the test wave to keep up. This was a trial and error approach, since it was assured by MTS that the system could keep up, the test wave was adjusted until it was working well without excessive need for PID adjustment. The system could be adjusted in size as required per the following equation;

$$g = \frac{2 * d * A}{f} \qquad \textbf{5.1}$$

Where: g = gallon per minute pump required for test. (gpm)
d = total displacement of test in one direction (required) (in)
A = cross sectional area of actuator piston (19.6 in^2 for our 55 kip actuator.) (in^2)
f = length of full sine wave (sec)

For example, calculations show that the system needs a 102 gpm or larger pump in order to be capable of the one inch total displacement.

5) The concrete made and tested in this test was not subjected to any sort of environmental conditions. In-situ concrete has to undergo many conditions that can be replicated in the lab to more fully understand the effects that coarse aggregate type has on this load transfer durability.

6) During pull-apart, debris fell from most samples. The material, which was of concern, was the material larger than ¼ inch cube. Since the crack width was decided upon as 0.035 inches, a piece of debris this large could be significant. This is being suggested since the in-situ concrete does not open 4-6 inches when the crack forms like the pull-apart test does, thus keeping this debris contained within the crack surface.

7) As shown on the "Bruce Mines #3 Before Testing" Plot, there is a possibility for the aggregate in the concrete to assume a parallel nature that is most likely not

present in the field. It is believed that this may be due to the rodding conducted on the samples during preparations. A concrete vibrating screen should be used in place of rodding to better represent field conditions.

Appendix 7A

Levy Slag
(82-019)

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

68.07 Coarse Agg (a)

Pail tare	1.74	1.74	3.48 + pails
			71.55 = total
25.0 - 19.0mm	17.01	0.00	
19.0 - 12.5mm	0.00	17.02	
12.5 - 9.5mm	0.00	17.02	
9.5 - 4.75mm	17.02	0.00	
Sub total	35.77	35.78	71.55 Total

BATCH NO. LS

COARSE AGG CA-S (Slag)

DATE: 3/9/00

Batch Made Thurs. @ 3:00

Fine Aggregate

62.62 Fine Agg (b)

Moisture content

wet	dry
345.94	334.84
0.0332	MC

0.0332 MC

2.08 Moisture

Dry weight 62.62
+
Moisture 2.08
Total 64.69

WATER MEASUREMENT

Coarse Agg +pail	35.77		
Coarse Agg +pail	35.78		
Total	71.55		
+ Total Batch Water	14.11	(d)	14.11
- Reserve Water	3.00		3.00
= Pails, Agg&Water	82.66	H_2O	11.11

Cement

26.12 Cement (C)

Pail ID A', B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

RESERVE WATER

Res water	3.00	1.33 surplus & Tare
+ Tare	0.29	0.29 - tare
= Total	3.29	1.04 = surplus

Reserve Water	3.00	
- Surplus Water	1.04	
=	1.96	H_2O + 11.11
Subtotal of water in batch		= 13.07
+ Moisture in Fine Aggregate		+ 2.08
Total Water in Batch		**(D) = 15.15**

Air Entraining Admixture

22 ml

UNIT WEIGHT

Weight of Concrete & Bucket	39.44	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	31.29	(f)

Batch Summary

(a)	Coarse Aggregate as Designed	68.07 kg
(b)	Fine Aggregate as Designed	62.62 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	15.15 kg
(e)	**Total Weight of Batch**	**171.96 kg**

SLUMP = 2 " 50.8 mm

AIR CONTENT

- Factor of Aggregate Porosity
= Percent Air 4

CONCRETE TEMPERATURE, C 19

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

68.07 Coarse Agg (a)

Pail tare	1.72	1.72	3.44 + pails
			71.51 = total
25.0 - 19.0mm	17.01	0.00	
19.0 - 12.5mm	0.00	17.02	
12.5 - 9.5mm	0.00	17.02	
9.5 - 4.75mm	17.02	0.00	
Sub total	35.75	35.76	71.51 Total

Fine Aggregate

62.62 Fine Agg (b)

Moisture content

wet	dry
236.36	227.48
0.0390	MC

0.0390 MC

2.44 Moisture

Dry weight 62.62

\+ Moisture 2.44

Total 65.06

Cement

26.12 Cement (C)

Pail ID A', B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

Air Entraining Admixture

20 ml

Batch Summary

(a) Coarse Aggregate as Designed	68.07	kg
(b) Fine Aggregate as Designed	62.62	kg
(c) Cement as Designed	26.12	kg
(D) Total Water of Batch	16.38	kg
(e) Total Weight of Batch	**173.19**	kg

BATCH NO. LS

COARSE AGG CA-S (Slag)

DATE: 6/29/00

Batch Made Thurs. @ 3:00

WATER MEASUREMENT

Coarse Agg +pail	35.75		
Coarse Agg +pail	35.76		
Total	71.51		
+ Total Batch Water	14.11	(d)	14.11
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**82.62**	H_2O	11.11

RESERVE WATER

Res water	3.00	1.70	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	1.41	= surplus

Reserve Water	3.00		
- Surplus Water	1.41		
=	2.82	H_2O +	11.11
Subtotal of water in batch		=	13.93
+ Moisture in Fine Aggregate		+	2.44
Total Water in Batch		**(D) =**	**16.38**

UNIT WEIGHT

Weight of Concrete & Bucket	38.80	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	30.65	(f)

SLUMP = 2.5 " 63.5 mm

AIR CONTENT

- Factor of Aggregate Porosity

= Percent Air 5.5

CONCRETE TEMPERATURE, C 22

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

68.07 Coarse Agg (a)

Pail tare	1.72	1.72	3.44 + pails
			71.51 = total
25.0 - 19.0mm	17.01	0.00	
19.0 - 12.5mm	0.00	17.02	
12.5 - 9.5mm	0.00	17.02	
9.5 - 4.75mm	17.02	0.00	
Sub total	35.75	35.76	71.51 Total

BATCH NO. LS

COARSE AGG CA-S (Slag)

DATE: 7/7/00

Batch Made Tues. @ 3:00

Fine Aggregate

62.62 Fine Agg (b)

Moisture content

wet	dry
177.92	171.64
0.0366	MC

0.0366 MC

2.29 Moisture

Dry weight 62.62
\+
Moisture 2.29
Total 64.91

WATER MEASUREMENT

Coarse Agg +pail 35.75
Coarse Agg +pail 35.76

Total 71.51
\+ Total Batch Water 14.11 (d) 14.11
\- Reserve Water 3.00 3.00
= Pails, Agg&Water 82.62 H_2O 11.11

RESERVE WATER

Res water	3.00	1.55 surplus & Tare
+ Tare	0.29	0.29 - tare
= Total	3.29	1.26 = surplus

Reserve Water 3.00
\- Surplus Water 1.26
= 2.97 H_2O + 11.11
Subtotal of water in batch = 14.09
\+ Moisture in Fine Aggregate + 2.29
Total Water in Batch (D) = 16.38

Cement

26.12 Cement (C)

Pail ID A', B'

Tare weight 0.85 — 1.70 tare

Tare weight 0.85 — 27.82 Pail + cement

Total tare 1.70

UNIT WEIGHT

Weight of Concrete & Bucket 38.04
\- Weight of Bucket 8.15
= Weight of Concrete in Bucket 29.89 (f)

Air Entraining Admixture

20 ml

Batch Summary

(a)	Coarse Aggregate as Designed	68.07 kg
(b)	Fine Aggregate as Designed	62.62 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	16.38 kg
(e)	**Total Weight of Batch**	**173.19** kg

SLUMP = 4 " **101.6** mm

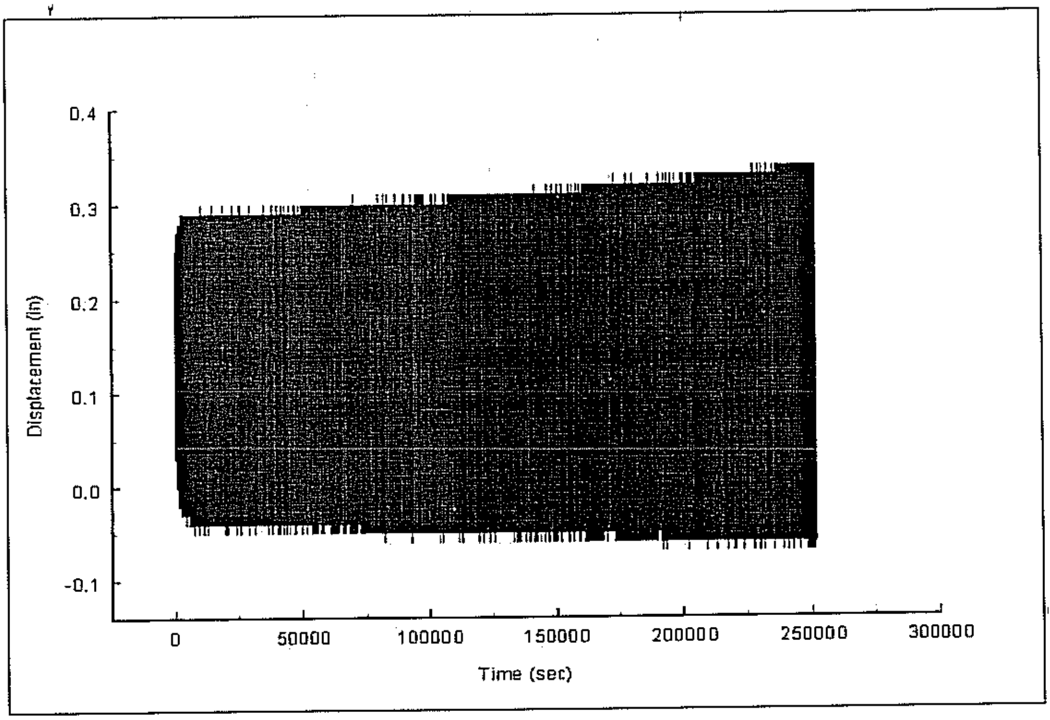
AIR CONTENT

\- Factor of Aggregate Porosity
= Percent Air 6.25

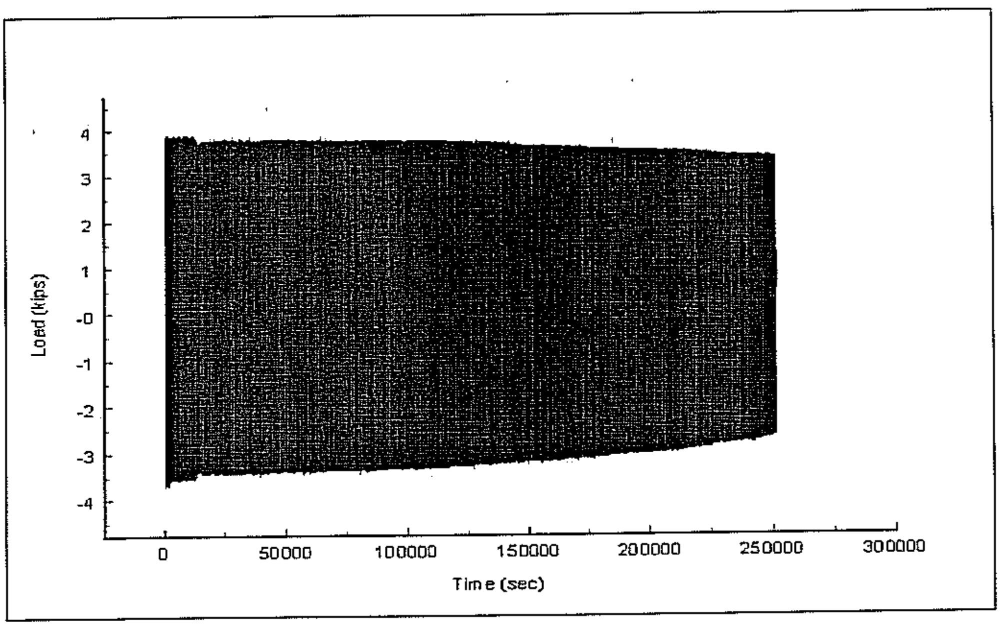
CONCRETE TEMPERATURE, C 24

Note: a,b,C,d come from mix proportions worksheet

Levy #5 Displacement



Levy #5 Load



Appendix 7B

Bruce Mines
(95-010)

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

BATCH NO. BM

COARSE AGG CA-B (Bruce M.)

DATE: 3/8/00

Batch Made Wed @ 1:00

Coarse Aggregate

84.53 Coarse Agg (a)

Pail tare	1.75	1.75	3.50 + pails
			88.03 = total
25.0 - 19.0mm	21.13	0.00	
19.0 - 12.5mm	0.00	21.14	
12.5 - 9.5mm	0.00	21.13	
9.5 - 4.75mm	21.13	0.00	
Sub total	44.01	44.02	88.03 Total

Fine Aggregate

63.79 Fine Agg (b)

Moisture content

wet	dry
302.51	295.55
0.0235	MC

0.0235 MC

1.50 Moisture

Dry weight 63.79

\+ Moisture 1.50

Total 65.29

Cement

26.12 Cement (C)

Pail ID A' , B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

Air Entraining Admixture

29 ml

Batch Summary

(a)	Coarse Aggregate as Designed	84.53 kg
(b)	Fine Aggregate as Designed	63.79 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.05 kg
(e)	**Total Weight of Batch**	**187.48** kg

WATER MEASUREMENT

Coarse Agg +pail	44.01		
Coarse Agg +pail	44.02		
Total	88.03		
+ Total Batch Water	12.32	(d)	12.32
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**97.35**	H_2O	9.32

RESERVE WATER

Res water	3.00	1.06	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	0.77	= surplus

Reserve Water	3.00		
- Surplus Water	0.77		
=	2.23	H_2O +	9.32
Subtotal of water in batch		=	11.55
+ Moisture in Fine Aggregate		+	1.50
Total Water in Batch		**(D) =**	**13.05**

UNIT WEIGHT

Weight of Concrete & Bucket	41.80	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**33.65**	**(f)**

SLUMP = 1.75 " **44.5** mm

AIR CONTENT

- Factor of Aggregate Porosity

= Percent Air **4.5**

CONCRETE TEMPERATURE, C 20

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

84.53 Coarse Agg (a)

Pail tare	1.72	1.72	3.44 + pails
			87.97 = total
25.0 - 19.0mm	21.13	0.00	
19.0 - 12.5mm	0.00	21.14	
12.5 - 9.5mm	0.00	21.13	
9.5 - 4.75mm	21.13	0.00	
Sub total	43.98	43.99	87.97 Total

Fine Aggregate

63.79 Fine Agg (b)

Moisture content

wet	dry
464.01	451.3
0.0282	MC

0.0282 MC

1.80 Moisture

Dry weight 63.79

\+

Moisture 1.80

Total 65.58

Cement

26.12 Cement (C)

Pail ID A', B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

Air Entraining Admixture

29 ml

Batch Summary

(a)	Coarse Aggregate as Designed	84.53 kg
(b)	Fine Aggregate as Designed	63.79 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.05 kg
(e)	**Total Weight of Batch**	**187.48** kg

BATCH NO. **BM**

COARSE AGG **CA-B (Bruce M.)**

DATE: **6/20/00**

Batch Made **Tues @ 3:00**

WATER MEASUREMENT

Coarse Agg +pail	43.98		
Coarse Agg +pail	43.99		
Total	87.97		
+ Total Batch Water	12.32	(d)	12.32
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**97.29**	H_2O	9.32

RESERVE WATER

Res water	3.00	1.36 surplus & Tare
+ Tare	0.29	0.29 - tare
= Total	3.29	1.07 = surplus

Reserve Water	3.00		
- Surplus Water	1.07		
=	1.93	H_2O +	9.32
Subtotal of water in batch		=	11.25
+ Moisture in Fine Aggregate		+	1.80
Total Water in Batch		**(D) =**	**13.05**

UNIT WEIGHT

Weight of Concrete & Bucket	41.31	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**33.16**	**(f)**

SLUMP = 3.25 " **82.6** mm

AIR CONTENT

- Factor of Aggregate Porosity

= Percent Air **6.0**

CONCRETE TEMPERATURE, C 23

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

84.53 Coarse Agg (a)

Pail tare	1.72	1.72	3.44	+ pails
			87.97	= total
25.0 - 19.0mm	21.13	0.00		
19.0 - 12.5mm	0.00	21.14		
12.5 - 9.5mm	0.00	21.13		
9.5 - 4.75mm	21.13	0.00		
Sub total	43.98	43.99	87.97	Total

Fine Aggregate

63.79 Fine Agg (b)

Moisture content

wet	dry
448.11	436.69
0.0262	MC

0.0262 MC

1.67 Moisture

Dry weight 63.79

\+

Moisture 1.67

Total 65.45

Cement

26.12 Cement (C)

Pail ID A' , B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

Air Entraining Admixture 24 ml

Batch Summary

(a)	Coarse Aggregate as Designed	84.53	kg
(b)	Fine Aggregate as Designed	63.79	kg
(c)	Cement as Designed	26.12	kg
(D)	Total Water of Batch	13.05	kg
(e)	**Total Weight of Batch**	187.48	kg

BATCH NO. **BM**

COARSE AGG **CA-B (Bruce M.)**

DATE: **6/21/00**

Batch Made **Wed @ 3:00**

WATER MEASUREMENT

Coarse Agg +pail	43.98		
Coarse Agg +pail	43.99		
Total	87.97		
+ Total Batch Water	12.32	(d)	12.32
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**97.29**	H_2O	9.32

RESERVE WATER

Res water	3.00	1.23	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	0.94	= surplus

Reserve Water	3.00		
- Surplus Water	0.94		
=	2.06	H_2O +	9.32
Subtotal of water in batch		=	11.38
+ Moisture in Fine Aggregate		+	1.67
Total Water in Batch		**(D) =**	**13.05**

UNIT WEIGHT

Weight of Concrete & Bucket	40.19	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**32.04**	**(f)**

SLUMP = 3 " 76.2 mm

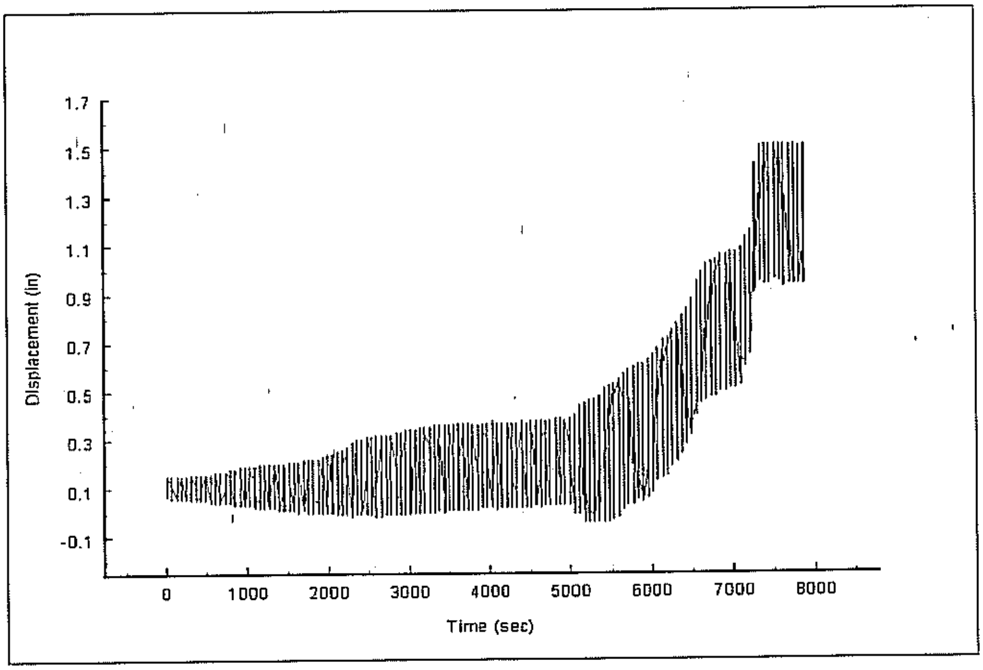
AIR CONTENT

- Factor of Aggregate Porosity
= Percent Air 6.5

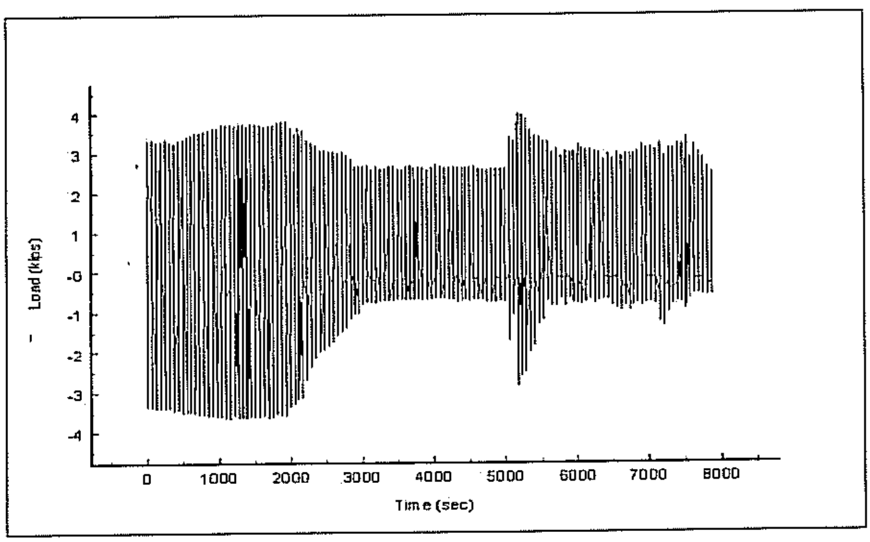
CONCRETE TEMPERATURE, C 23

Note: a,b,C,d come from mix proportions worksheet

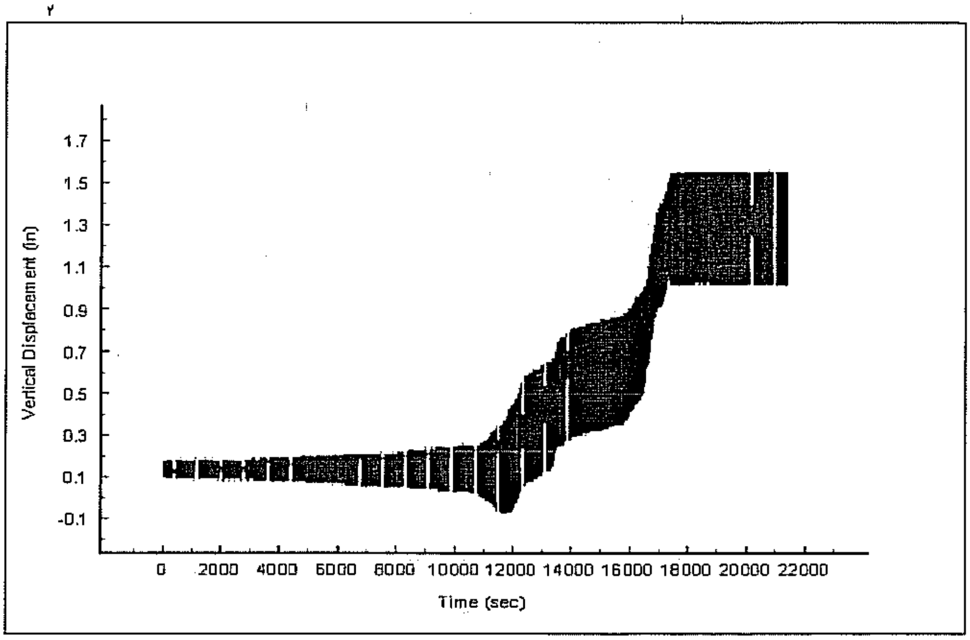
Bruce Mines #3 Displacement



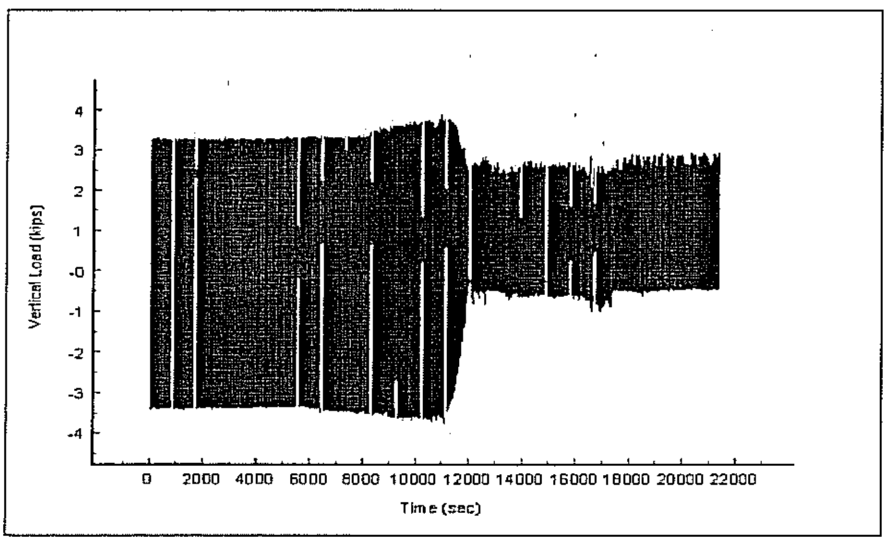
Bruce Mines #3 Load



Bruce Mines #4 Displacement



Bruce Mines #4 Load



Appendix 7C

Port Inland (75-005)

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

83.67 Coarse Agg (a)

Pail tare	1.75	1.75	3.50 + pails
			87.17 = total
25.0 - 19.0mm	20.92	0.00	
19.0 - 12.5mm	0.00	20.91	
12.5 - 9.5mm	0.00	20.92	
9.5 - 4.75mm	20.92	0.00	
Sub total	43.59	43.58	87.17 Total

BATCH NO.	Po.In.
COARSE AGG	CA-P (Port In.)
DATE:	3/10/00
Batch Made	Fri. @ 3:00

Fine Aggregate

59.26 Fine Agg (b)

Moisture content

wet	dry
357.26	345.07
0.0353	MC

0.0353 MC

2.09 Moisture

Dry weight 59.26

\+

Moisture 2.09

Total 61.36

9.16

WATER MEASUREMENT

Coarse Agg +pail	43.59		
Coarse Agg +pail	43.58		
Total	87.17		
+ Total Batch Water	12.16	(d)	12.16
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**96.33**	H_2O	9.16

Cement

26.12 Cement (C)

Pail ID	A', B'		
Tare weight	0.85	1.70	tare
Tare weight	0.85	27.82	Pail + cement
Total tare	1.70		

RESERVE WATER

Res water	3.00	1.39	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	1.10	= surplus

Reserve Water	3.00		
- Surplus Water	1.10		
=	1.90	H_2O +	9.16
Subtotal of water in batch		=	11.07
+ Moisture in Fine Aggregate		+	2.09
Total Water in Batch		**(D) =**	**13.16**

Air Entraining Admixture

25 ml

UNIT WEIGHT

Weight of Concrete & Bucket	39.82	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**31.67**	**(f)**

Batch Summary

(a)	Coarse Aggregate as Designed	83.67 kg
(b)	Fine Aggregate as Designed	59.26 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.16 kg
(e)	**Total Weight of Batch**	**182.22** kg

SLUMP = 3.25 " **82.6** mm

AIR CONTENT

- Factor of Aggregate Porosity	
= Percent Air	**5.25**

CONCRETE TEMPERATURE, C 19

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET | **WEIGHT IN kg**

Coarse Aggregate — 83.67 Coarse Agg (a)

Pail tare	1.72	1.72	3.44 + pails
			87.11 = total
25.0 - 19.0mm	20.92	0.00	
19.0 - 12.5mm	0.00	20.91	
12.5 - 9.5mm	0.00	20.92	
9.5 - 4.75mm	20.92	0.00	
Sub total	43.56	43.55	87.11 Total

Fine Aggregate — 59.26 Fine Agg (b)

Moisture content

wet	dry
457.29	446.61
0.0239	MC

0.0239 MC

1.42 Moisture

Dry weight 59.26

\+

Moisture 1.42

Total **60.68**

9.16

Cement — 26.12 Cement (C)

Pail ID A' , B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

Air Entraining Admixture 21 ml

Batch Summary

(a)	Coarse Aggregate as Designed	83.67 kg
(b)	Fine Aggregate as Designed	59.26 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.16 kg
(e)	**Total Weight of Batch**	**182.22** kg

BATCH NO. Po.In.

COARSE AGG CA-P (Port In.)

DATE: 6/23/00

Batch Made Fri. @ 3:00

WATER MEASUREMENT

Coarse Agg +pail	43.56		
Coarse Agg +pail	43.55		
Total	87.11		
+ Total Batch Water	12.16	(d)	12.16
- Reserve Water	3.00		3.00
= Pails, Agg&Water	96.27	H_2O	9.16

RESERVE WATER

Res water	3.00	0.71	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	0.42	= surplus

Reserve Water	3.00		
- Surplus Water	0.42		
=	2.58	H_2O +	9.16
Subtotal of water in batch		=	11.74
+ Moisture in Fine Aggregate		+	1.42
Total Water in Batch		(D) =	**13.16**

UNIT WEIGHT

Weight of Concrete & Bucket	38.93	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**30.78**	**(f)**

SLUMP = 3.5 " **88.9** mm

AIR CONTENT

- Factor of Aggregate Porosity ______

= Percent Air **6.25**

CONCRETE TEMPERATURE, C 24

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

83.67 Coarse Agg (a)

Pail tare	1.72	1.72	3.44 + pails
			87.11 = total
25.0 - 19.0mm	20.92	0.00	
19.0 - 12.5mm	0.00	20.91	
12.5 - 9.5mm	0.00	20.92	
9.5 - 4.75mm	20.92	0.00	
Sub total	43.56	43.55	87.11 Total

BATCH NO. Po.In.

COARSE AGG CA-P (Port In.)

DATE: 6/27/00

Batch Made Tues. @ 3:00

Fine Aggregate

59.26 Fine Agg (b)

Moisture content

wet	dry
163.62	157.81
0.0368	MC

0.0368 MC

2.18 Moisture

Dry weight 59.26

\+

Moisture 2.18

Total **61.45**

9.16

Cement

26.12 Cement (C)

Pail ID A', B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

WATER MEASUREMENT

Coarse Agg +pail	43.56		
Coarse Agg +pail	43.55		
Total	87.11		
+ Total Batch Water	12.16	(d)	12.16
- Reserve Water	3.00		3.00
= Pails, Agg&Water	96.27	H_2O	9.16

RESERVE WATER

Res water	3.00	1.48	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	1.19	= surplus

Reserve Water	3.00		
- Surplus Water	1.19		
=	1.81	H_2O +	9.16
Subtotal of water in batch		=	10.98
+ Moisture in Fine Aggregate		+	2.18
Total Water in Batch		**(D) =**	**13.16**

Air Entraining Admixture

19.75 ml

UNIT WEIGHT

Weight of Concrete & Bucket	39.75	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	31.60	(f)

Batch Summary

(a) Coarse Aggregate as Designed	83.67	kg
(b) Fine Aggregate as Designed	59.26	kg
(c) Cement as Designed	26.12	kg
(D) Total Water of Batch	13.16	kg
(e) Total Weight of Batch	**182.22**	kg

SLUMP = 2.75 " 69.9 mm

AIR CONTENT

- Factor of Aggregate Porosity

= Percent Air 5.5

CONCRETE TEMPERATURE, C 22

Note: a,b,C,d come from mix proportions worksheet

Appendix D

Presque Isle
(71-047)

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

77.35 Coarse Agg (a)

Pail tare	1.75	1.75	3.50 + pails
			80.85 = total
25.0 - 19.0mm	19.34	0.00	
19.0 - 12.5mm	0.00	19.33	
12.5 - 9.5mm	0.00	19.34	
9.5 - 4.75mm	19.34	0.00	
Sub total	40.43	40.42	80.85 Total

Fine Aggregate

61.52 Fine Agg (b)

Moisture content

wet	dry
350.81	339.42
0.0336	MC

0.0336 MC

2.06 Moisture

Dry weight 61.52

\+

Moisture 2.06

Total 63.59

Cement

26.12 Cement (C)

Pail ID A' , B'

Tare weight 0.85 — 1.70 tare

Tare weight 0.85 — 27.82 Pail + cement

Total tare 1.70

Air Entraining Admixture

21 ml

Batch Summary

(a)	Coarse Aggregate as Designed	77.35 kg
(b)	Fine Aggregate as Designed	61.52 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.76 kg
(e)	**Total Weight of Batch**	**178.76** kg

BATCH NO. **Pr.Is.**

COARSE AGG **CA-I (Pres. Isle)**

DATE: **3/15/00**

Batch Made **Wed. @ 12:30**

WATER MEASUREMENT

Coarse Agg +pail	40.43		
Coarse Agg +pail	40.42		
Total	80.85		
+ Total Batch Water	12.87	(d)	12.87
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**90.72**	H_2O	9.87

RESERVE WATER

Res water	3.00	1.46 surplus & Tare
+ Tare	0.29	0.29 - tare
= Total	3.29	1.17 = surplus

Reserve Water	3.00		
- Surplus Water	1.17		
=	1.83	H_2O +	9.87
Subtotal of water in batch		=	11.70
+ Moisture in Fine Aggregate		+	2.06
Total Water in Batch		**(D) =**	**13.76**

UNIT WEIGHT

Weight of Concrete & Bucket	39.84	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**31.69**	**(f)**

SLUMP = 3.25 " **82.6** mm

AIR CONTENT

- Factor of Aggregate Porosity

= Percent Air **5**

CONCRETE TEMPERATURE, C 18

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

77.35 Coarse Agg (a)

Pail tare	1.71	1.71	3.42 + pails
			80.77 = total
25.0 - 19.0mm	19.34	0.00	
19.0 - 12.5mm	0.00	19.33	
12.5 - 9.5mm	0.00	19.34	
9.5 - 4.75mm	19.34	0.00	
Sub total	40.39	40.38	80.77 Total

Fine Aggregate

61.52 Fine Agg (b)

Moisture content

wet	dry
322.7	316.01
0.0212	MC

0.0212 MC

1.30 Moisture

Dry weight 61.52

\+

Moisture 1.30

Total 62.83

Cement

26.12 Cement (C)

Pail ID A', B'

Tare weight 0.85

Tare weight 0.85

Total tare 1.70

1.70 tare

27.82 Pail + cement

Air Entraining Admixture

21 ml

Batch Summary

(a)	Coarse Aggregate as Designed	77.35 kg
(b)	Fine Aggregate as Designed	61.52 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.76 kg
(e)	**Total Weight of Batch**	**178.76** kg

BATCH NO. **Pr.Is.**

COARSE AGG **CA-I (Pres. Isle)**

DATE: **6/14/00**

Batch Made **Wed. @ 3:00**

WATER MEASUREMENT

Coarse Agg +pail	40.39		
Coarse Agg +pail	40.38		
Total	80.77		
+ Total Batch Water	12.87	(d)	12.87
- Reserve Water	3.00		3.00
= Pails, Agg&Water	**90.64**	H_2O	9.87

RESERVE WATER

Res water	3.00	0.70	surplus & Tare
+ Tare	0.29	0.29	- tare
= Total	3.29	0.41	= surplus

Reserve Water	3.00		
- Surplus Water	0.41		
=	2.59	H_2O +	9.87
Subtotal of water in batch		=	12.46
+ Moisture in Fine Aggregate		+	1.30
Total Water in Batch		**(D) =**	**13.76**

UNIT WEIGHT

Weight of Concrete & Bucket	40.18	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	**32.03**	**(f)**

SLUMP = 2.25 " **57.2** mm

AIR CONTENT

- Factor of Aggregate Porosity

= Percent Air **4.5**

CONCRETE TEMPERATURE, C 22

Note: a,b,C,d come from mix proportions worksheet

BATCH COMPUTATIONS WORKSHEET

WEIGHT IN kg

Coarse Aggregate

77.35 Coarse Agg (a)

Pail tare	1.71	1.71	3.42 + pails
			80.77 = total
25.0 - 19.0mm	19.34	0.00	
19.0 - 12.5mm	0.00	19.33	
12.5 - 9.5mm	0.00	19.34	
9.5 - 4.75mm	19.34	0.00	
Sub total	40.39	40.38	80.77 Total

Fine Aggregate

61.52 Fine Agg (b)

Moisture content

wet	dry
352.78	345.91
0.0199	MC

0.0199 MC

1.22 Moisture

Dry weight 61.52

\+

Moisture 1.22

Total 62.75

Cement

26.12 Cement (C)

Pail ID A' , B'

Tare weight 0.85 — 1.70 tare

Tare weight 0.85 — 27.82 Pail + cement

Total tare 1.70

Air Entraining Admixture

21 ml

Batch Summary

(a)	Coarse Aggregate as Designed	77.35 kg
(b)	Fine Aggregate as Designed	61.52 kg
(c)	Cement as Designed	26.12 kg
(D)	Total Water of Batch	13.76 kg
(e)	**Total Weight of Batch**	**178.76** kg

Note: a,b,C,d come from mix proportions worksheet

BATCH NO. Pr.Is.

COARSE AGG CA-I (Pres. Isle)

DATE: 6/15/00

Batch Made Thurs. @ 3:00

WATER MEASUREMENT

Coarse Agg +pail	40.39		
Coarse Agg +pail	40.38		
Total	80.77		
+ Total Batch Water	12.87	(d)	12.87
- Reserve Water	3.00		3.00
= Pails, Agg&Water	90.64	H_2O	9.87

RESERVE WATER

Res water	3.00	0.62 surplus & Tare
+ Tare	0.29	0.29 - tare
= Total	3.29	0.33 = surplus

Reserve Water	3.00		
- Surplus Water	0.33		
=	2.67	H_2O +	9.87
Subtotal of water in batch		=	12.54
+ Moisture in Fine Aggregate		+	1.22
Total Water in Batch		**(D) =**	**13.76**

UNIT WEIGHT

Weight of Concrete & Bucket	40.37	
- Weight of Bucket	8.15	
= Weight of Concrete in Bucket	32.22	(f)

SLUMP = 1 " — 25.4 mm

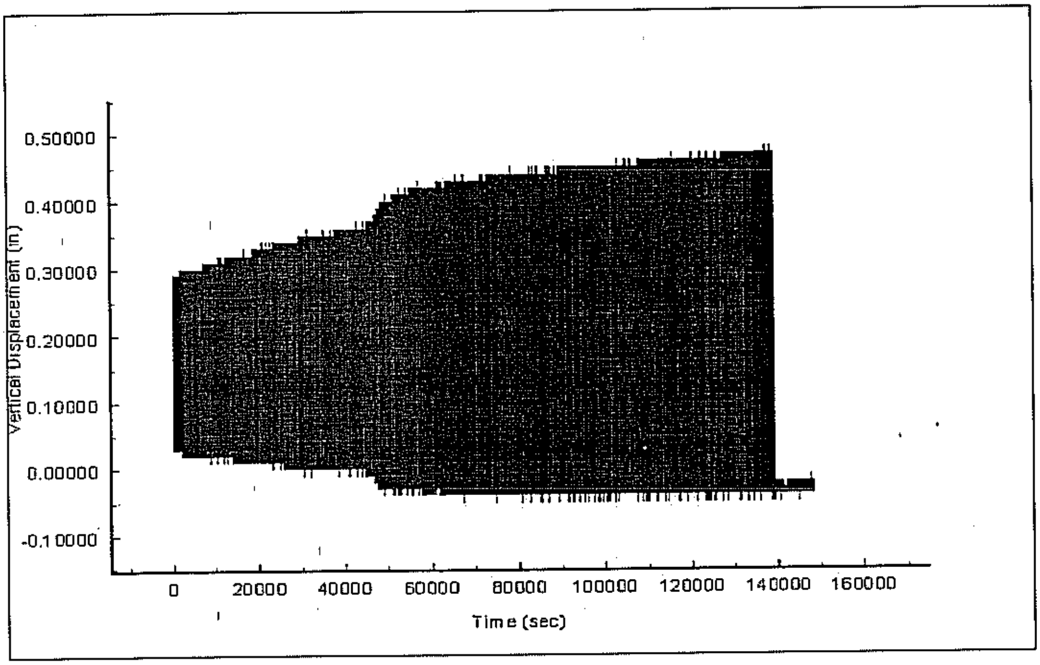
AIR CONTENT

- Factor of Aggregate Porosity

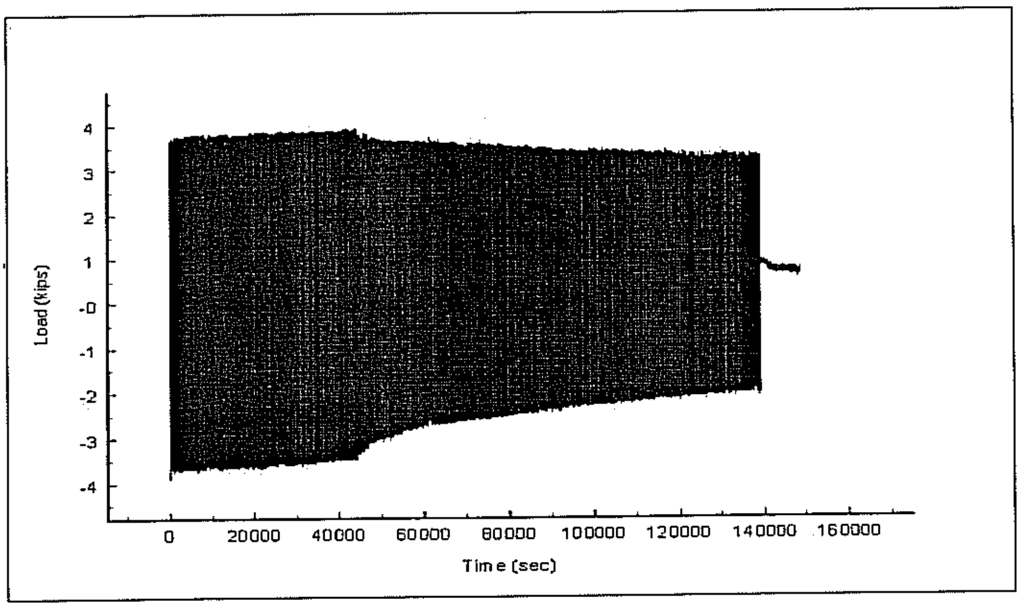
= Percent Air 4.75

CONCRETE TEMPERATURE, C 22

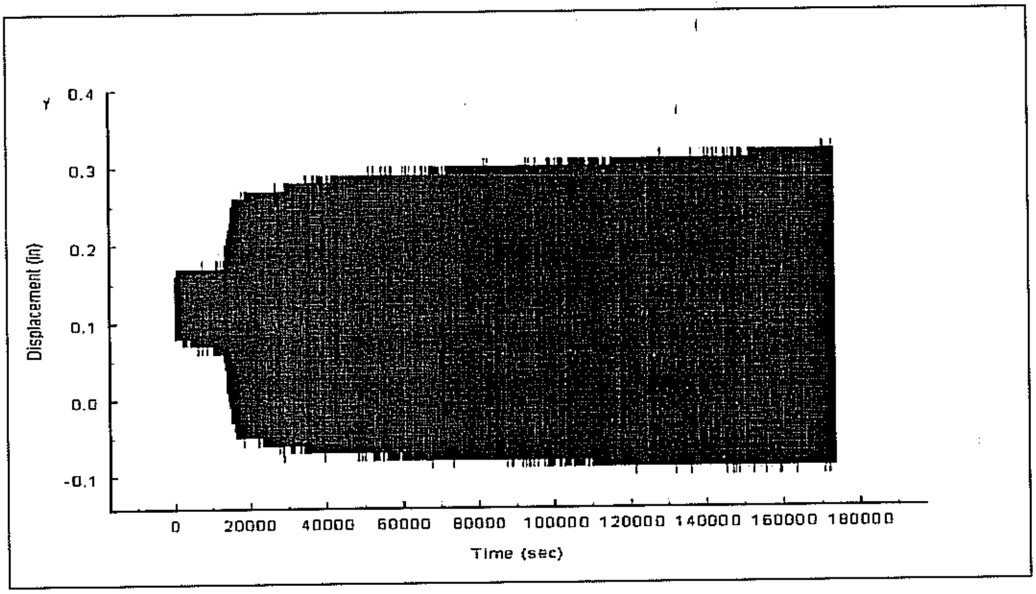
Presque Isle #2 Displacement



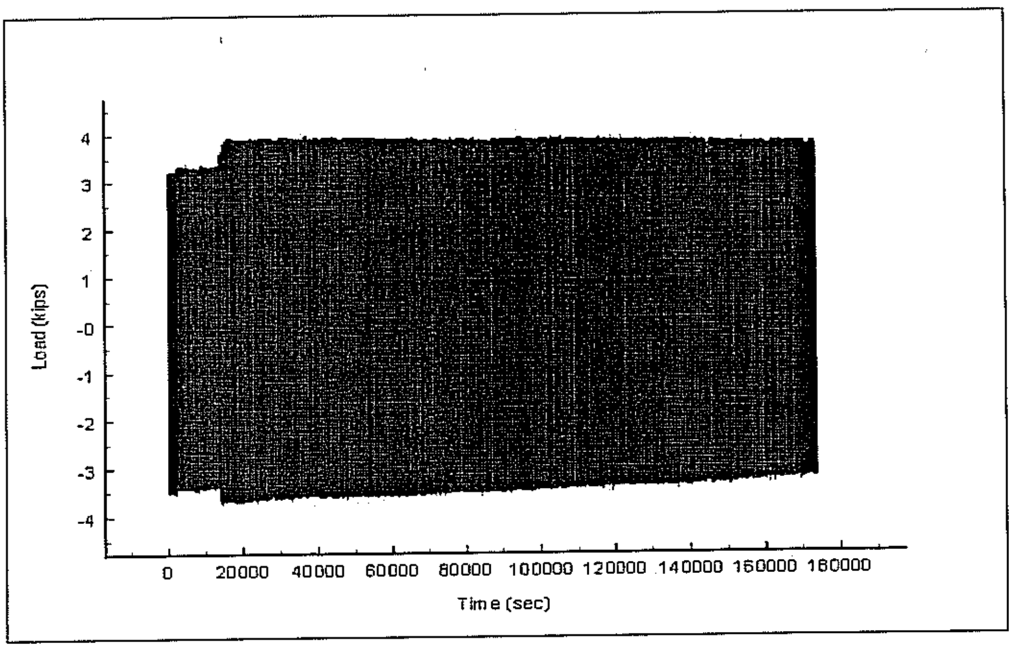
Presque Isle #2 load



Presque Isle #4 Displacement



Presque Isle #4 Load



Appendix E

Batch properties and six inch static compressive cylinders

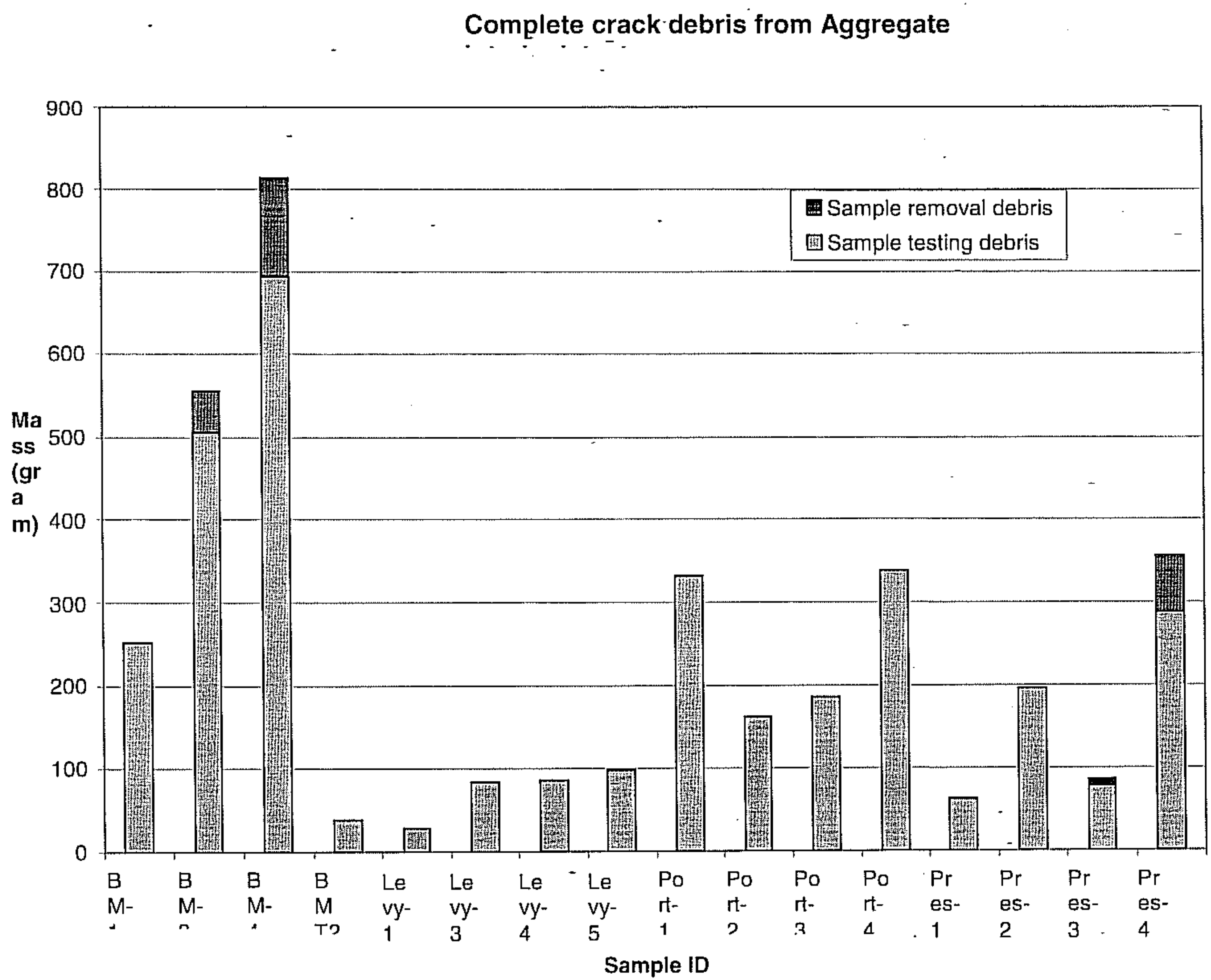
Complete sample crack test debris data

Comparison of static compressive strength of 3 X 6 inch cored cylinders

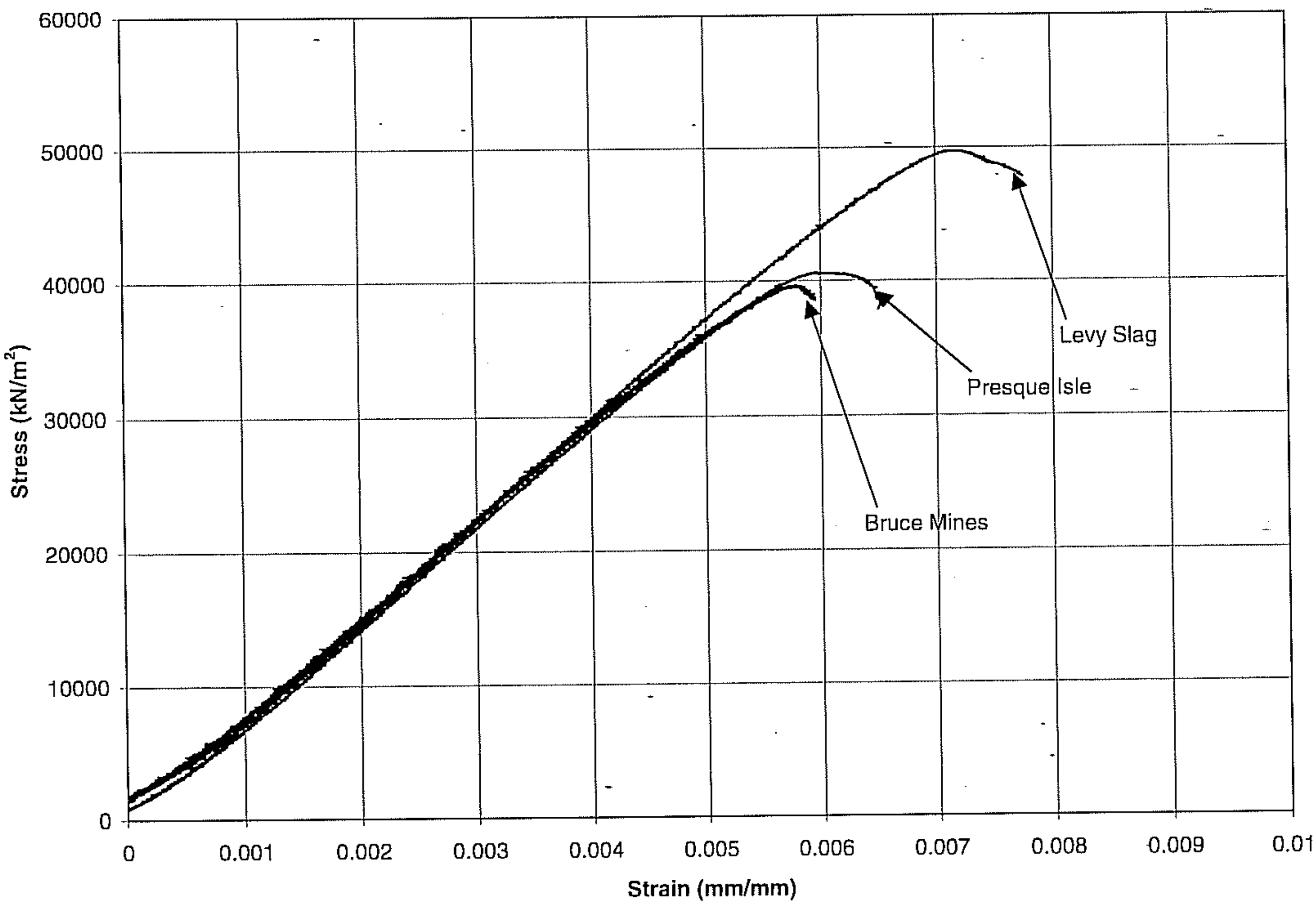
Test setup, truck configuration

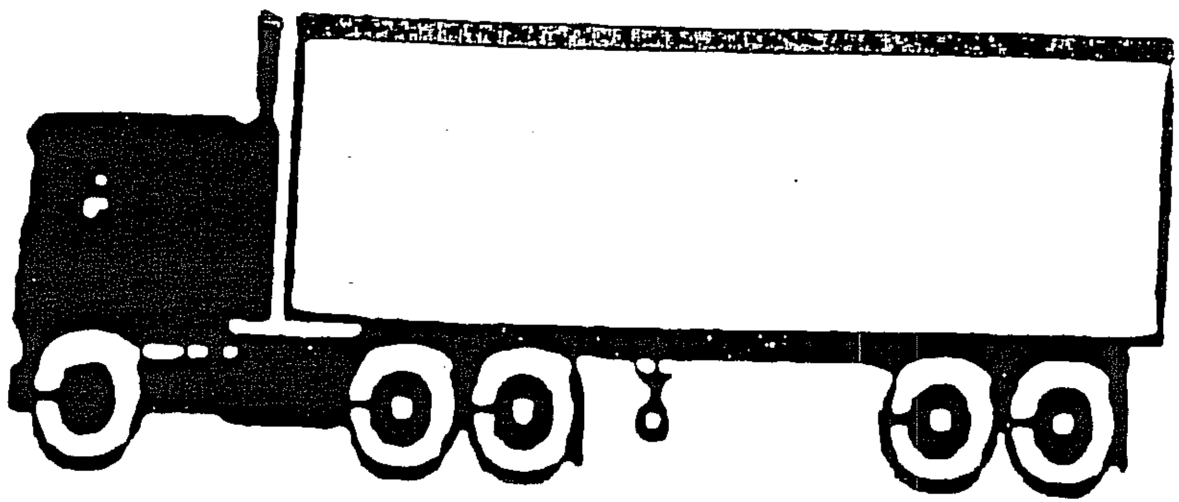
Aggregate type/sample ID #	Slump (inch)	Air (%)	Temp (C)	Strength (psi)
Levy 1/2	0.75	4.25	23	7197.34
Levy 3/4	4	6.25	24	5812.11
Levy 5/6	2.5	5.5	21	4703.92
Presque Isle 1/2	2.25	4.5	22	
Presque Isle 3/4	1	4.75	22	6837.77
Port Inland 1/2	3.5	6.25	24	4049.61
Port Inland 3/4	2.75	5.5	22	4939.7
Bruce Mines 1/2	3.25	6	23	5470.22
Bruce Mines 3/4	3	6.5	23	4456.34

Date Cast	Date tested	Sample I.D.	Break Type	Load at failure (average)	Stress at failure	Comments
6/15/00	7/18/00	Pres 3/4	Double cone	193333.3333	6837.773701	Heavy aggregate fracture
6/20/00	7/18/00	BM 1/2	Single cone/Planer	154666.6667	5470.218961	No aggregate fracture
6/21/00	7/21/00	BM 3/4	Crush	126000	4456.342171	
6/23/00	7/21/00	Port 1/2	Crush	114500	4049.612528	
6/27/00	7/28/00	Port 2/3	Plane/Crush	139666.6667	4939.702036	
6/28/00	7/28/00	Levy 1/2	Double cone	203500	7197.346284	Low Slump, heavy aggregate fracture
6/29/00	7/28/00	Levy 3/4	Cone/Planer	164333.3333	5812.107646	Partical pull-out
7/5/00	8/7/00	Levy 5/6	Crush	133000	4703.916736	


Complete crack debris from Aggregate
Sample removal debris
Sample testing debris
Mass (gram)
900
800
700
600
500
400
300
200
100
0
BM-
BM-
BM-
BMT2
Levy-1
Levy-3
Levy-4
Levy-5
Port-1
Port-2
Port-3
Port-4
Pres-1
Pres-2
Pres-3
Pres-4
Sample ID

Average of All Aggregate Types, Compression Test, Dry Specimens: (09/05/00) by RAC





Test setup, truck configuration

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